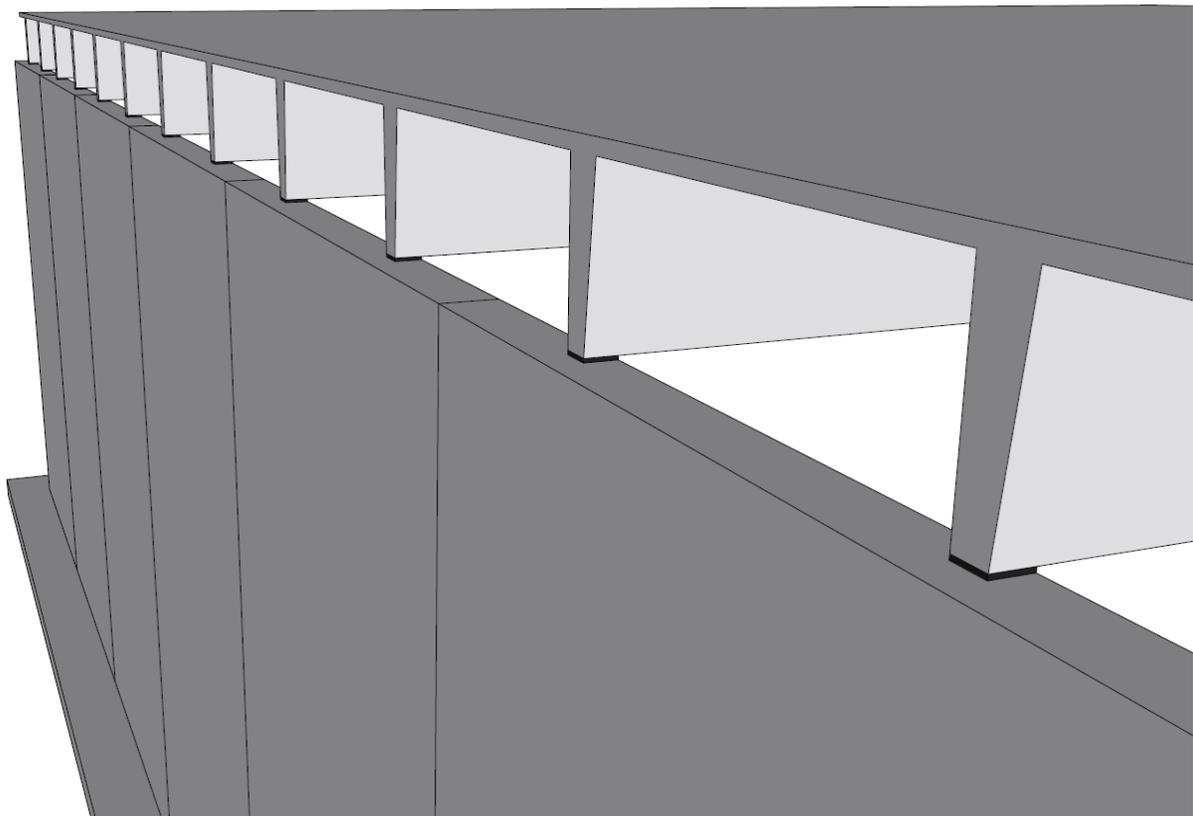
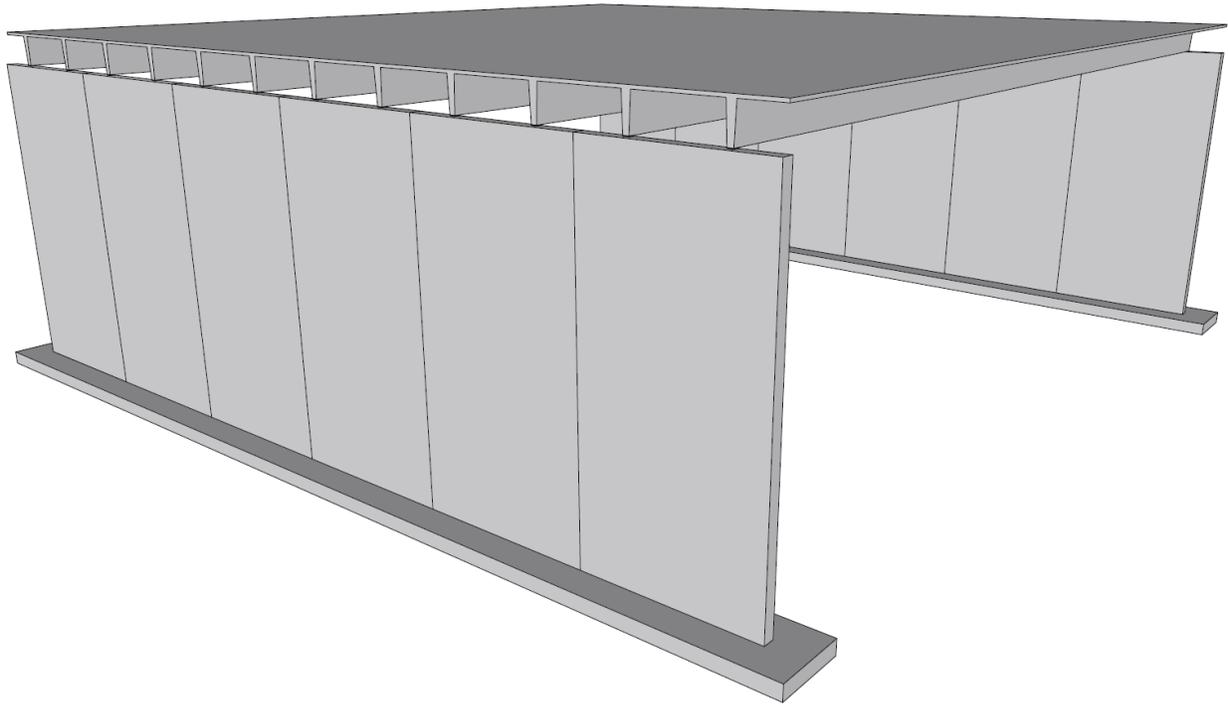


Precast Concrete Bearing Wall Panel Design (Alternative Analysis Method) (ACI 318-19)



Precast Concrete Bearing Wall Panel Design (Alternative Analysis Method) (ACI 318-19)

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 = 468.00 plf

Roof dead load = 20.00 psf

Roof live load = 30.00 psf

Wind load = 30.00 psf

The 10DT24 are spaced 5.00 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 by [StructurePoint](#).

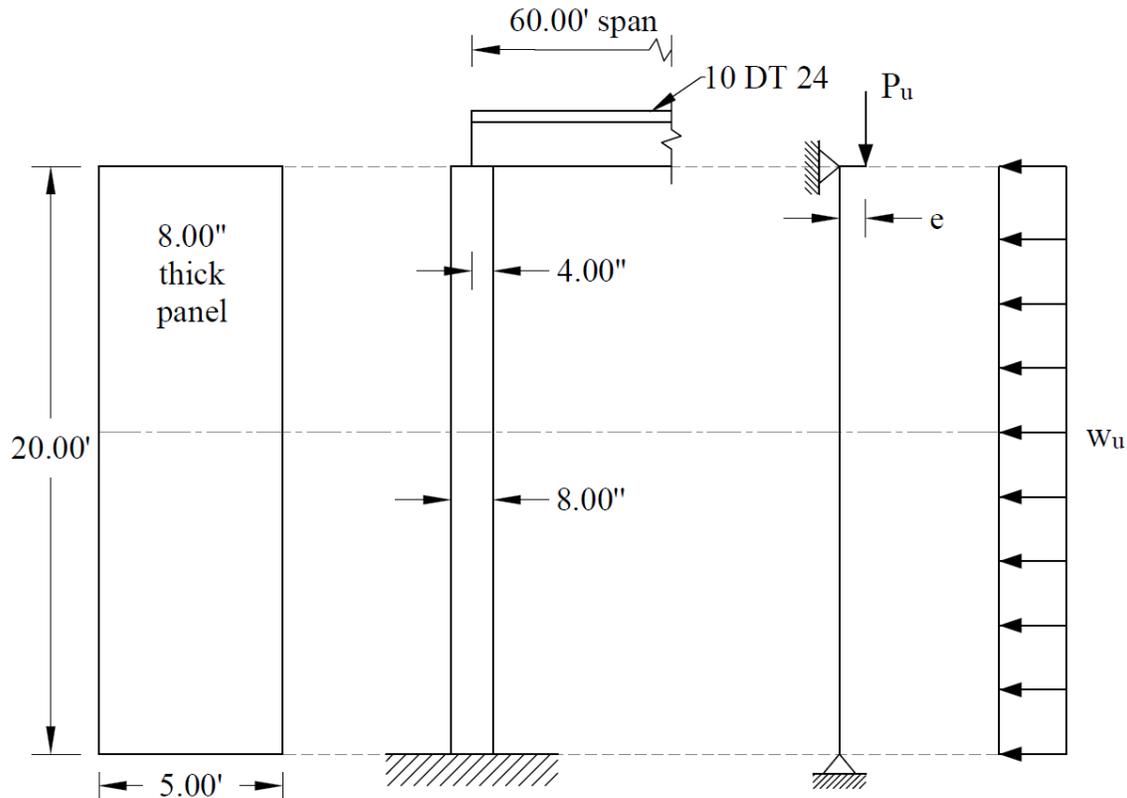


Figure 1 – Reinforced Concrete Precast Wall Panel Geometry

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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, Example 21.3
- [spWall Engineering Software Program Manual v10.50, STRUCTUREPOINT, 2026](#)
- Contact Support@StructurePoint.org to obtain supplementary materials ([spWall](#) model: DE-Bearing-Wall-ACI-19.walx)

Design Data

$f_c' = 4,000$ psi normal weight concrete ($w_c = 150.00$ pcf)

$f_y = 60,000$ psi

Wall length = 20.00 ft

Assumed wall thickness = 8.00 in.

Assumed Vertical Reinforcement: single layer of #4 bars at 9.00 in. at centerline of wall.

$$(A_{s, vertical} = 0.20 / 9.00 \text{ in.} \times 12.00 \text{ in.} = 0.27 \text{ in.}^2/\text{ft})$$

1. Notations

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

a = depth of equivalent rectangular stress block, in.

A_s = area of nonprestressed longitudinal tension reinforcement, in.²

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

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in.

E_c = modulus of elasticity of concrete, psi

E_s = modulus of elasticity of reinforcement and structural steel, excluding prestressing reinforcement, psi

f_c' = specified compressive strength of concrete, psi

f_r = modulus of rupture of concrete, psi

f_y = specified yield strength for nonprestressed reinforcement, psi

h = overall thickness, height, or depth of member, in.

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

I_{cr} = moment of inertia of cracked section transformed to concrete, in.⁴

l_c = length of compression member, measured center-to-center of the joints, in.

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

M_a = maximum moment in member due to service loads at stage deflection is calculated, in.-lb

M_{cr} = cracking moment, in.-lb

M_n = nominal flexural strength at section, in.-lb

M_{sa} = maximum moment in wall due to service loads, excluding $P\Delta$ effects, in.-lb

M_u = factored moment at section, in.-lb

M_{ua} = moment at midheight of wall due to factored lateral and eccentric vertical loads, not including $P\Delta$ effects, in.-lb

N_u = factored axial force normal to cross section occurring simultaneously with V_u or T_u ; to be taken as positive for compression and negative for tension, lb

P_n = nominal axial compressive strength of member, lb

P_s = unfactored axial load at the design, midheight section including effects of self-weight, lb

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

- $P\Delta$ = secondary moment due to lateral deflection, in.-lb
- s = center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, tendons, or anchors, in.
- w_c = density, unit weight, of normalweight concrete or equilibrium density of lightweight concrete, lb/ft³
- w_u = factored load per unit length of beam or one-way slab, lb/in.
- y_t = distance from centroidal axis of gross section, neglecting reinforcement, to tension face, in.
- β_1 = factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis
- Δ_{cr} = calculated out-of-plane deflection at midheight of wall corresponding to cracking moment M_{cr} , in.
- Δ_n = calculated out-of-plane deflection at midheight of wall corresponding to nominal flexural strength M_n , in.
- Δ_s = out-of-plane deflection due to service loads, in.
- Δ_u = calculated out-of-plane deflection at midheight of wall due to factored loads, in.
- ϵ_t = net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature
- ϵ_{ty} = value of net tensile strain in the extreme layer of longitudinal tension reinforcement used to define a compression-controlled section
- λ = modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength
- ρ_l = ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement
- ϕ = strength reduction factor

2. Minimum Vertical Reinforcement

$$\rho_l = \frac{A_{v,vertical}}{h \times s_1} = \frac{0.27}{12 \times 8} = 0.0028 \quad \text{ACI 318-19 (2.2)}$$

$$\rho_{l,min} = 0.0012 \quad \text{ACI 318-19 (Table 11.6.1)}$$

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

$$s_{l,max} = \text{smallest of } \left\{ \begin{array}{l} 3 \times h \\ 18 \text{ in.} \end{array} \right\} = \text{smallest of } \left\{ \begin{array}{l} 3 \times 8 \\ 18 \text{ in.} \end{array} \right\} = \text{smallest of } \left\{ \begin{array}{l} 24 \text{ in.} \\ 18 \text{ in.} \end{array} \right\} = 18.00 \text{ in.} \quad \text{ACI 318-19 (11.7.2.1)}$$

$$s_{l,provided} = 9.00 \text{ in.} \leq s_{l,max} = 18.00 \text{ in.} \quad \text{(o.k.)}$$

3. 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 318-19. 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 318-19 (11.8.1.1(a))
- The wall can be designed as simply supported ACI 318-19 (11.8.2.1)
- Maximum moments and deflections occurring at midspan ACI 318-19 (11.8.2.1)
- The wall must be axially loaded ACI 318-19 (11.8.2.1)
- The wall must be subjected to an out-of-plane uniform lateral load ACI 318-19 (11.8.2.1)
- The wall shall be tension-controlled ACI 318-19 (11.8.1.1(b))
- The reinforcement shall provide design strength greater than cracking strength ACI 318-19 (11.8.1.1(c))
- P_u at the midheight section does not exceed $0.06 f_c' A_g$ ACI 318-19 (11.8.1.1(d))
- Out-of-plane deflection due to service loads including PA effects does not exceed $l_c/150$ ACI 318-19 (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 318-19 (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.

$$\text{Distribution Width of Concentrated Loads} = \min \left\{ \begin{array}{l} W + \frac{l_c}{2} \\ \text{spacing of the concentrated loads} \end{array} \right\} \quad \underline{\underline{ACI 318-19 (11.8.2.2)}}$$

$$\text{Distribution Width of Concentrated Loads} = \min \left\{ \begin{array}{l} \frac{3.75}{12} + \frac{20.00}{2} = 10.31 \text{ ft} \\ 5.00 \text{ ft} \end{array} \right\} = 5.00 \text{ ft}$$

4. Wall Structural Analysis

Using 11.8 provisions, calculate factored loads as follows for each of the considered load combinations:

4.1. Roof Load per foot width of wall

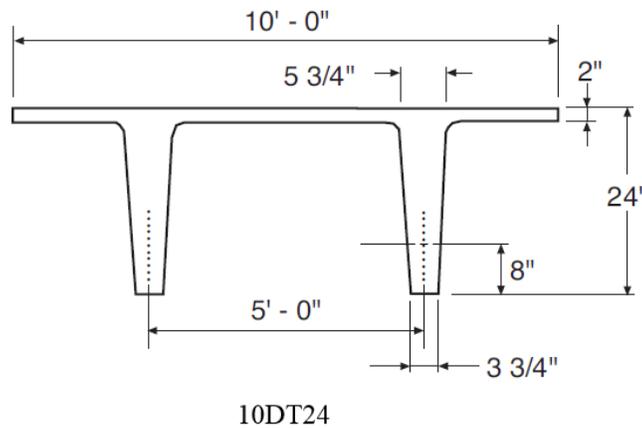


Figure 2 – Typical Geometry of 10DT24 Section

$$\text{Wall self-weight} = \frac{8.00}{12} \times 20.00 \times 150.00 = 2,000.00 \text{ plf}$$

$$D = \left[\frac{468}{2} + (20 \times 5) \right] \times \left(\frac{60}{2} \right) = 10,020 \text{ lbs / 5 ft} = 2,004.00 \text{ plf}$$

$$L = (30 \times 5) \times \left(\frac{60}{2} \right) = 4,500 \text{ lbs / 5 ft} = 900.00 \text{ plf}$$

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

4.2. Calculation of Maximum Wall Forces

The calculation of maximum factored wall forces in accordance with 11.8 is summarized in [Figure 3](#) including moment magnification due to second order (P-Δ) effects.

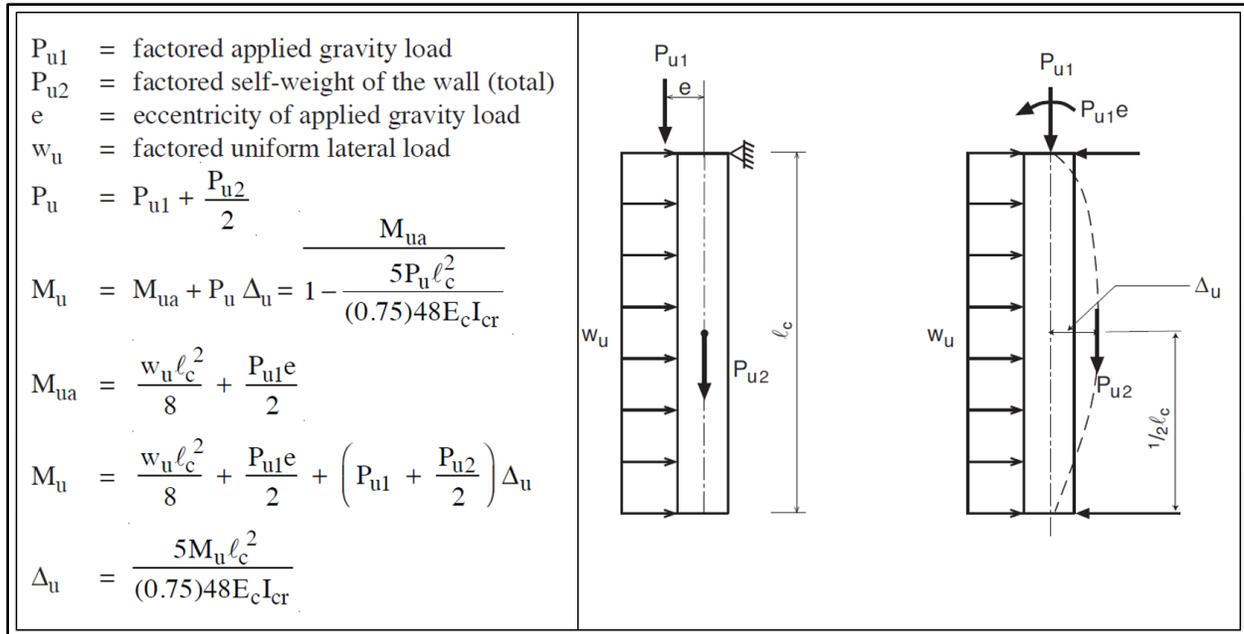


Figure 3 – Wall Structural Analysis According to the Alternative Design of Slender Walls Method (PCA Notes)

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

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

$$M_u = \frac{M_{ua}}{1 - \frac{5 \times P_u \times \ell_c^2}{0.75 \times 48 \times E_c \times I_{cr}}} \quad \text{ACI 318-19 (11.8.3.1d)}$$

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

Where M_{ua} is the maximum factored moment at midheight of wall due to lateral and eccentric vertical loads, not including $P\Delta$ effects. ACI 318-19 (11.8.3.1)

$$E_c = 57,000 \times \sqrt{f'_c} = 57,000 \times \sqrt{4,000} = 3,605,000 \text{ psi} \quad \text{ACI 318-19 (19.2.2.1.b)}$$

$$I_{cr} = n \times A_{se,w} \times (d - c)^2 + \frac{I_w \times c^3}{3} \quad \text{ACI 318-19 (11.8.3.1c)}$$

$$n = \frac{E_s}{E_c} = \frac{29,000}{3,605} = 8.04 > 6.0 \text{ (o.k.)}$$

ACI 318-19 (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 318-19 (R11.8.3.1)

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

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

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

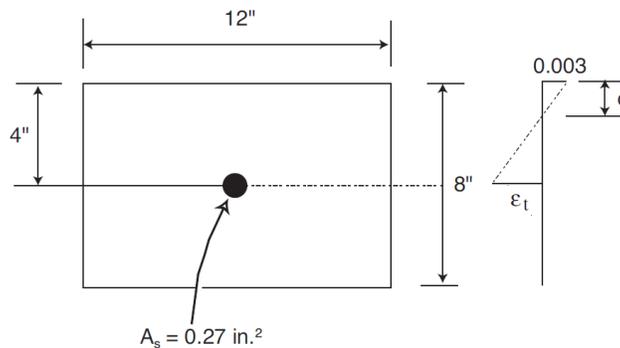


Figure 4 – 1 ft Wall Strip Cross-Section

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

ACI 318-19 (11.8.3.1c)

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

ACI 318-19 (21.2.2.1)

$$\varepsilon_t = \left(\frac{0.003}{c} \right) \times d_t - 0.003 = \left(\frac{0.003}{0.58} \right) \times 4.0 - 0.003 = 0.0176 > 0.003 + \varepsilon_{ty} = 0.00507$$

Therefore, section is tension controlled

ACI 318-19 (Table 21.2.2)

$$\phi = 0.90$$

ACI 318-19 (Table 21.2.2)

$$M_u = \frac{M_{ua}}{1 - \frac{5 \times P_u \times l_c^2}{0.75 \times 48 \times E_c \times I_{cr}}}$$

ACI 318-19 (11.8.3.1d)

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

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	31.20	3,605	8	0.31	0.46	0.54	30.7	0.0193	0.9	38.80

4.3. Tension-Controlled Verification

ACI 318-19 (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.

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

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

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

$$\epsilon_t = \left(\frac{0.003}{c} \right) \times d_t - 0.003 = \left(\frac{0.003}{0.623} \right) \times 4.0 - 0.003 = 0.0163 > 0.00507$$

Therefore, section is tension controlled.

ACI 318-19 (Table 21.2.2)

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

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

$$f_r = 7.5\lambda\sqrt{f'_c} = 7.5 \times 1.0 \times \sqrt{4,000} = 474.34 \text{ psi} \quad \text{ACI 318-19 (19.2.3.1)}$$

$$I_g = \frac{l_w h^3}{12} = \frac{12 \times 8^3}{12} = 512.00 \text{ in.}^4$$

$$y_t = \frac{h}{2} = \frac{8}{2} = 4.00 \text{ in.}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{474.34 \times 512.00}{4} \times \frac{1}{1000} = 60.72 \text{ in.-kip} \quad \text{ACI 318-19 (24.2.3.5)}$$

6. 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 $\rightarrow \phi = 0.9$

$$\phi M_n = \phi \times M_n = 0.9 \times 75.82 = 68.24 \text{ in.-kip} > M_u = 5.25 \text{ in.-kips (o.k.)} \quad \text{ACI 318-19 (11.5.1.1(b))}$$

$$\phi M_n = 68.24 \text{ in.-kip} > M_{cr} = 60.72 \text{ in.-kips (o.k.)} \quad \text{ACI 318-19 (11.8.1.1(c))}$$

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	$5.25 < \phi M_n$	o.k.	$60.72 < \phi M_n$	o.k.
1.2 D + 1.6 Lr + 0.8 W	78.61	0.9	70.75	$29.38 < \phi M_n$	o.k.	$60.72 < \phi M_n$	o.k.
1.2 D + 0.5 Lr + 1.6 W	75.29	0.9	67.76	$45.22 < \phi M_n$	o.k.	$60.72 < \phi M_n$	o.k.
0.9 D + 1.6 W	70.53	0.9	63.47	$38.80 < \phi M_n$	o.k.	$60.72 < \phi M_n$	o.k.

7. Wall Vertical Stress Check

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

$$\frac{P_u}{A_g} = \frac{5,044.8}{8 \times 12} = 52.55 \text{ psi} < 0.06 \times f'_c = 0.06 \times 4,000 = 240.00 \text{ psi (o.k.)} \quad \text{ACI 318-19 (11.8.1.1(d))}$$

8. 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 2.13 kips. The wall has a shear capacity approximately 18.12 kips and no detailed calculations are required by engineering judgement. (See [Figure 13](#) for detailed shear force diagram)

9. Wall Mid-Height Deflection (Δ_s)

The maximum out-of-plane deflection (Δ_s) due to service lateral and eccentric vertical loads, including $P\Delta$ effects, shall not exceed $l_c/150$. Where Δ_s is calculated as follows: ACI 318-19 (11.8.1.1(e))

$$\Delta_s = \left\{ \begin{array}{l} \frac{2}{3} \Delta_{cr} + \frac{M_a - \frac{2}{3} M_{cr}}{M_n - \frac{2}{3} M_{cr}} \times \left(\Delta_n - \frac{2}{3} \Delta_{cr} \right) \quad \text{When } M_a > \frac{2}{3} M_{cr} \\ \left(\frac{M_a}{M_{cr}} \right) \Delta_{cr} \quad \text{When } M_a < \frac{2}{3} M_{cr} \end{array} \right. \quad \text{ACI 318-19 (Table 11.8.4.1)}$$

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

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

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

$$P_s = P_{s1} + \frac{P_{s2}}{2} = (2.004 + 0.9) + \frac{2.0}{2} = 3.90 \text{ kips}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = 60.72 \text{ in.-kip (as calculated previously)} \quad \text{ACI 318-19 (24.2.3.5)}$$

$$\Delta_{cr} = \frac{5}{48} \times \frac{M_{cr} \times l_c^2}{E_c \times I_g} = \frac{5}{48} \times \frac{60.72 \times (20 \times 12)^2}{3,605 \times 512} = 0.197 \text{ in.} \quad \text{ACI 318-19 (11.8.4.3a)}$$

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 .

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

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

$$M_a = M_{sa} + P_s \Delta_s = 21.87 + 3.90 \times 0.071 = 22.15 \text{ in.-kips}$$

$$\Delta_s = \left(\frac{M_a}{M_{cr}} \right) \Delta_{cr} = \frac{22.15}{60.72} \times 0.197 = 0.072 \text{ in.}$$

ACI 318-19 (Table 11.8.4.1)

No further iterations are required

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

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

The wall is adequate with #4 @ 9.00 in. vertical reinforcement and 8.00 in. thickness.

10. 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 318-19 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.

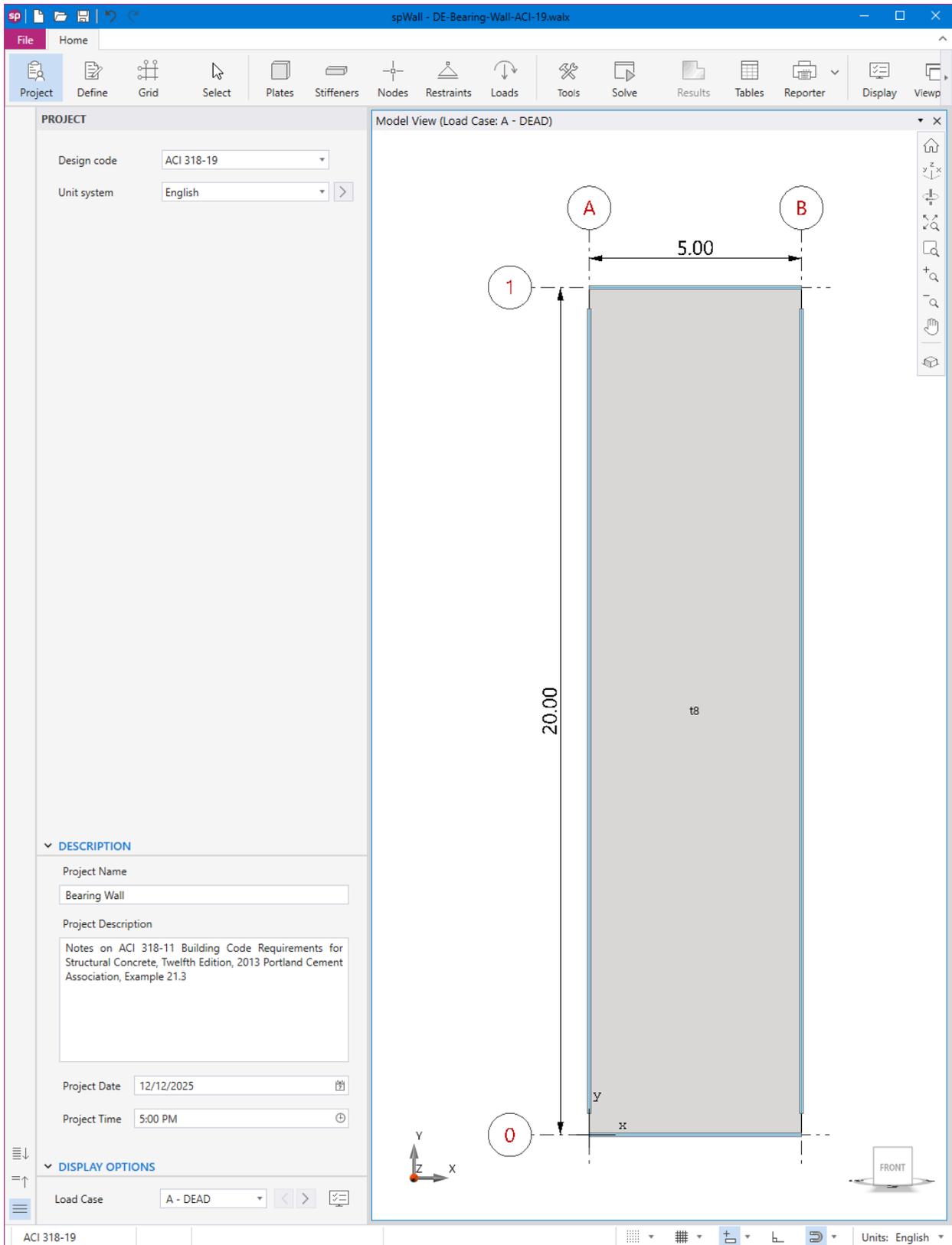


Figure 5 – spWall Interface

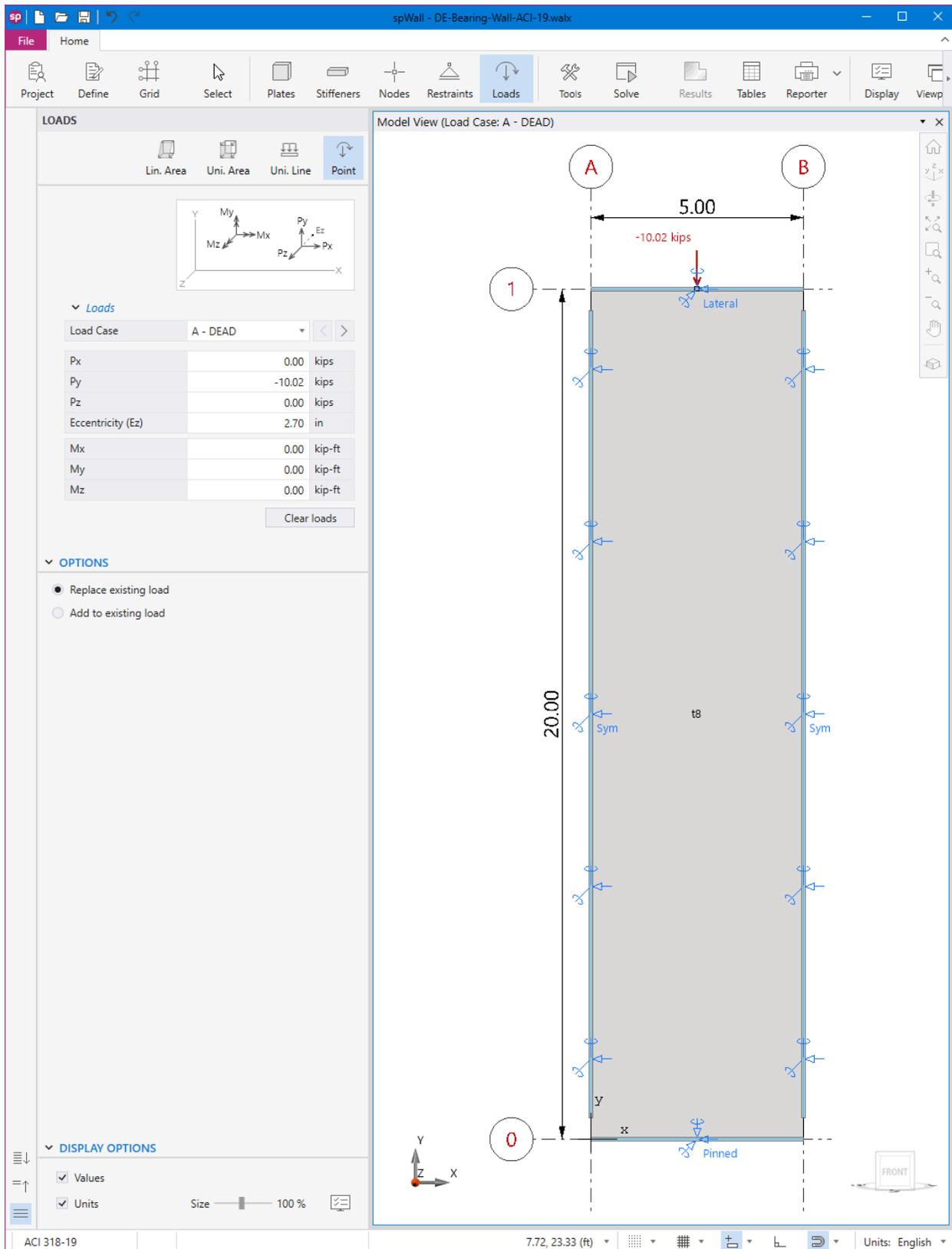


Figure 6 – Assigning Dead Loads for Bearing Wall (spWall)

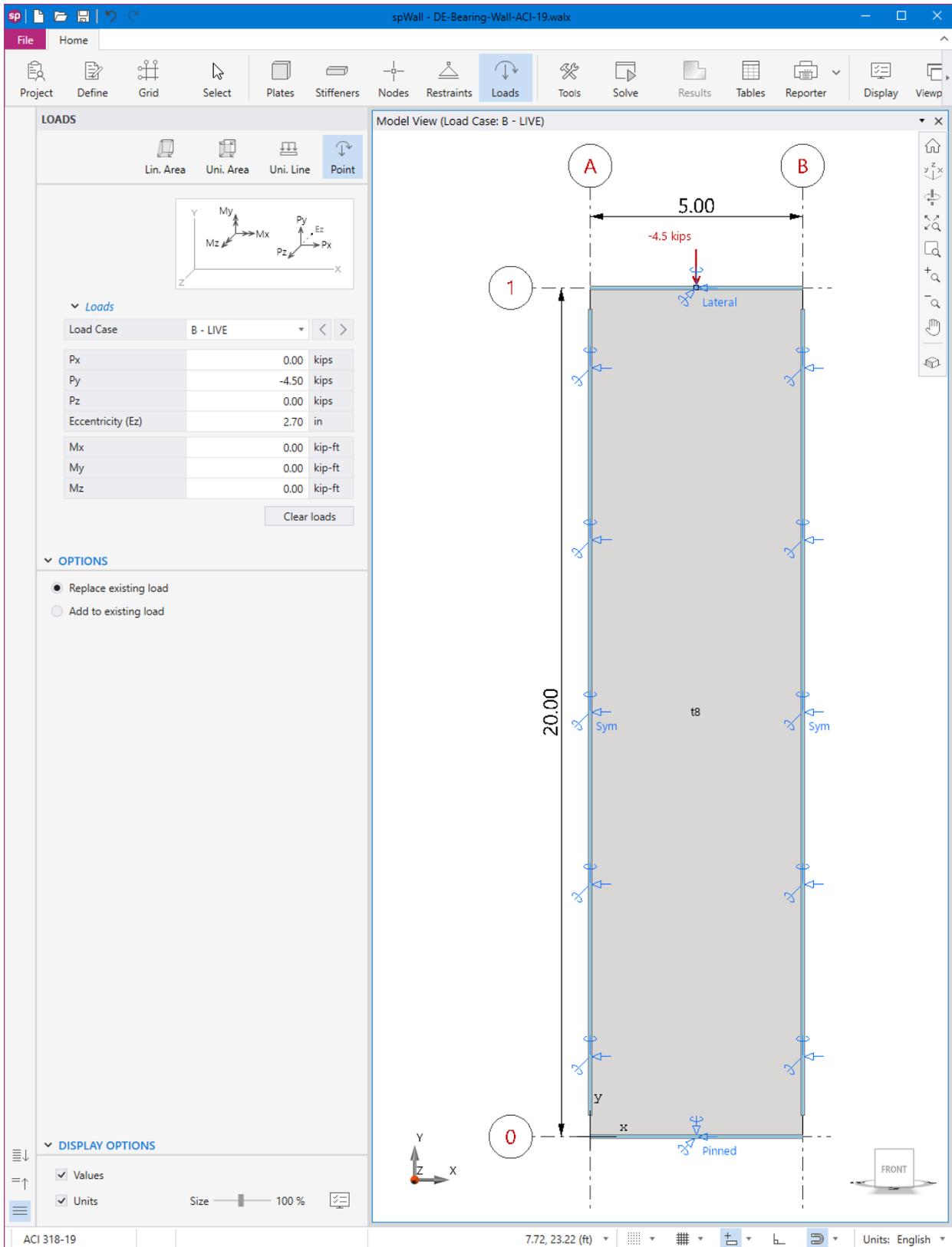


Figure 7 – Assigning Live Loads for Bearing Wall (spWall)

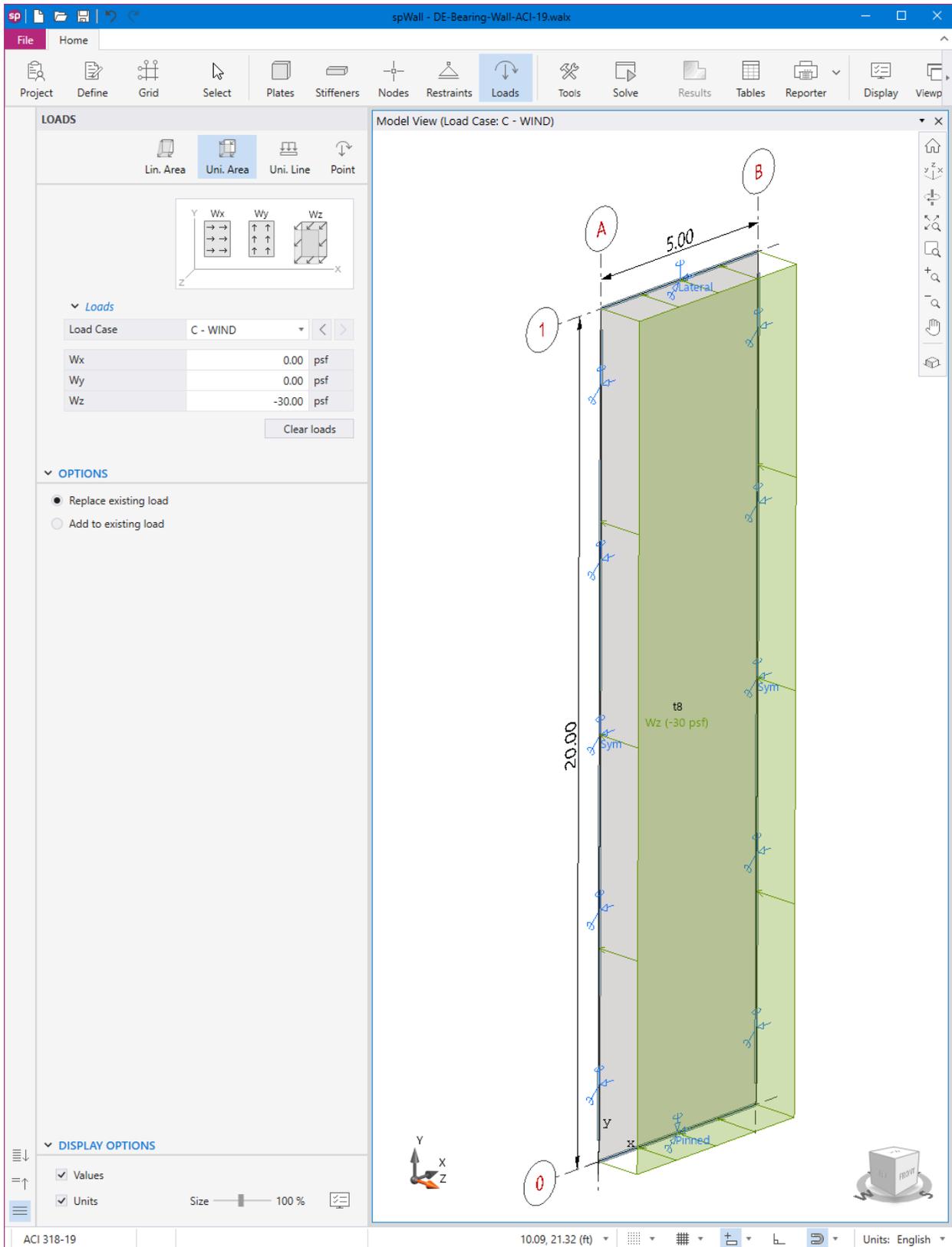


Figure 8 – Assigning Wind Loads for Bearing Wall (spWall)

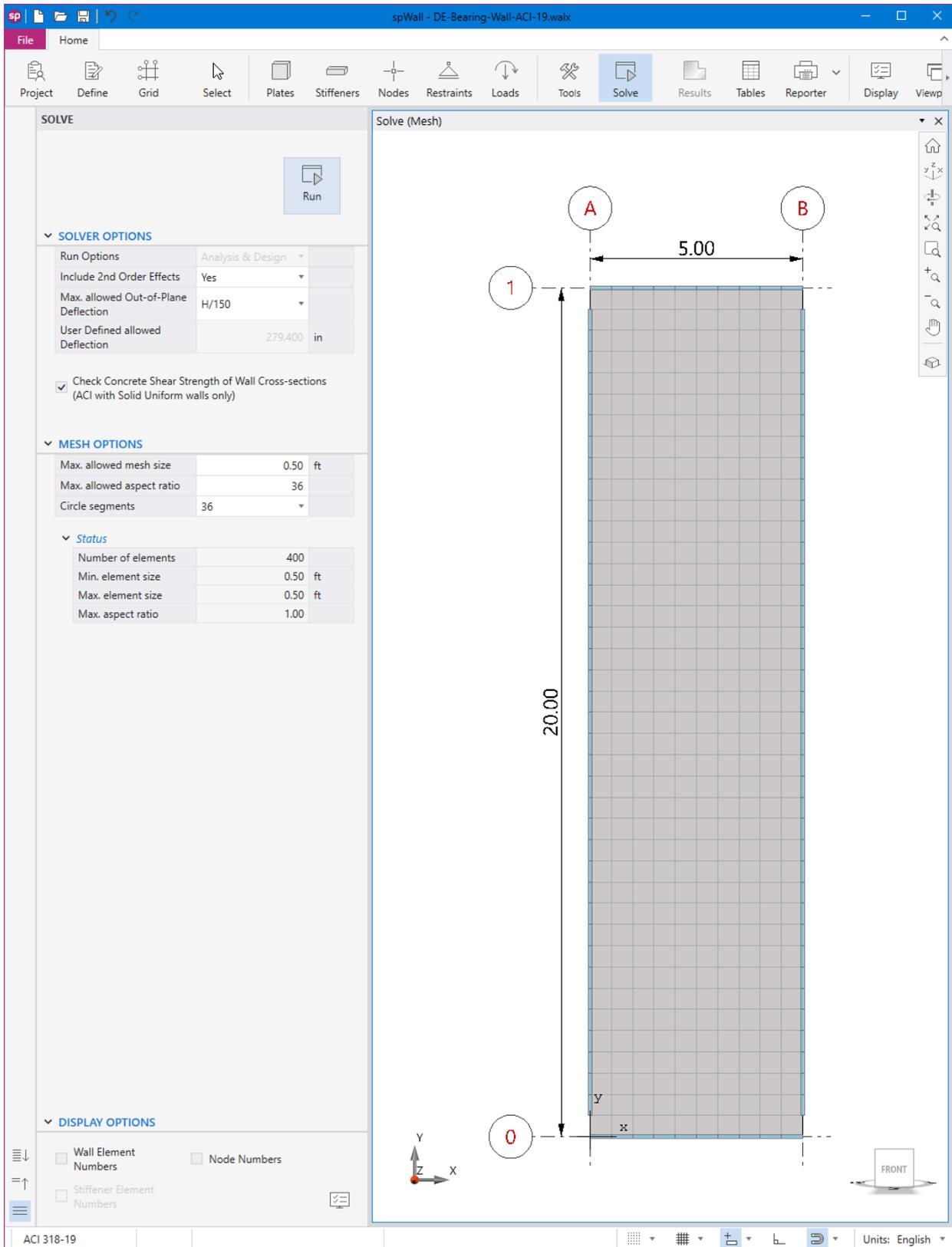


Figure 9 – Solve and Mesh Options (spWall)

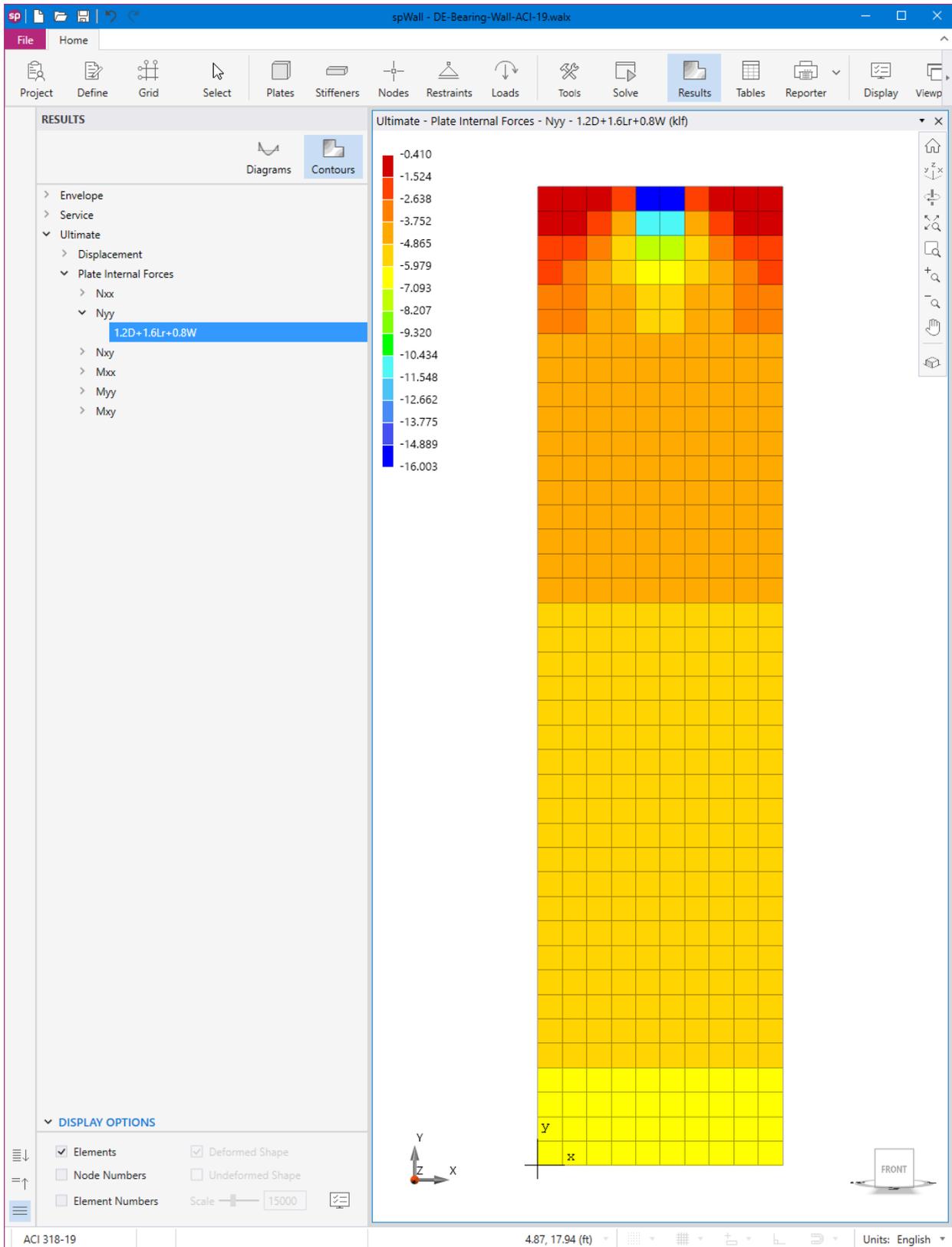


Figure 10 – Factored Axial Forces Contour Normal to Precast Wall Panel Cross-Section (spWall)

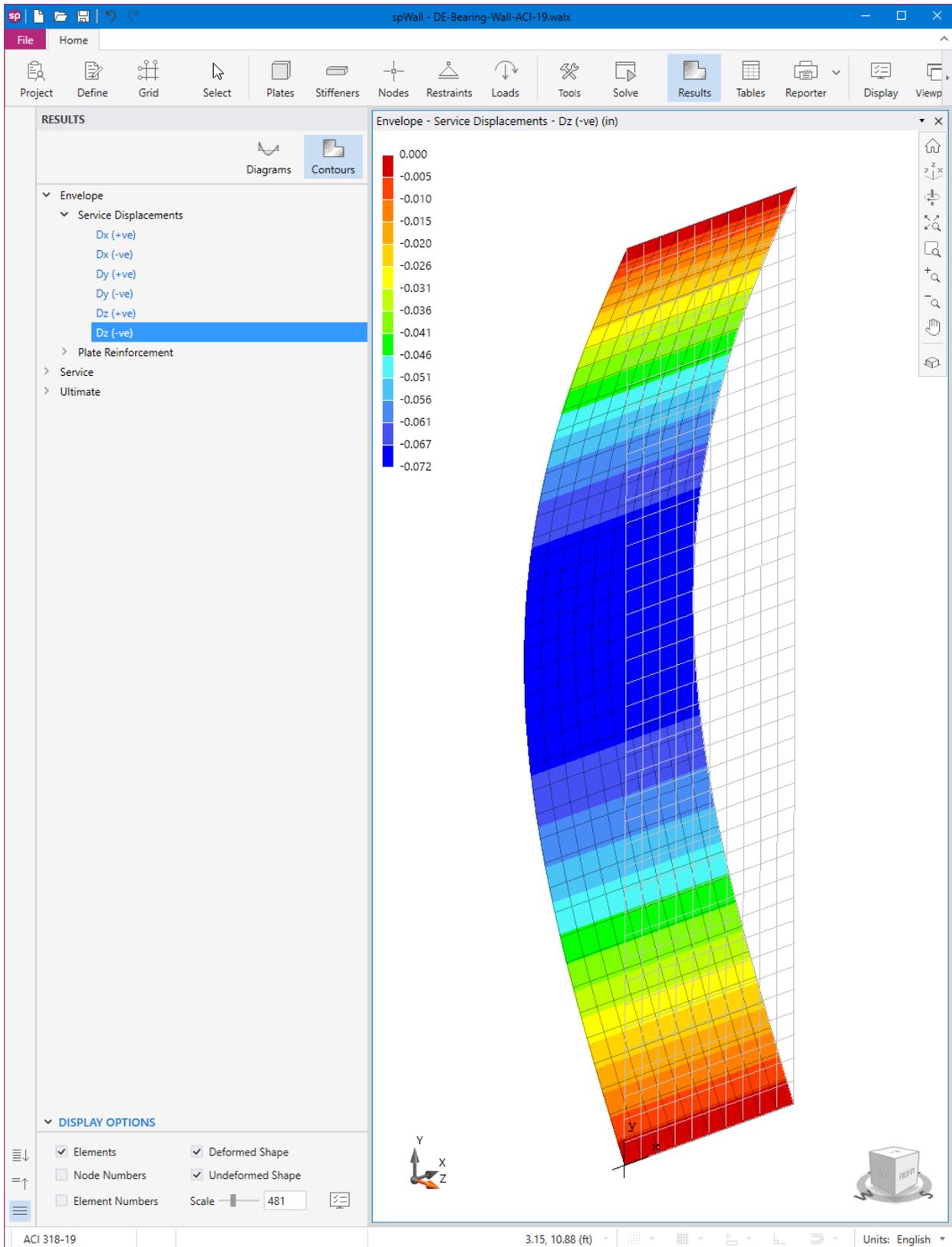


Figure 11 – Precast Wall Panel Lateral Displacement Contour (Out-of-Plane) (spWall)

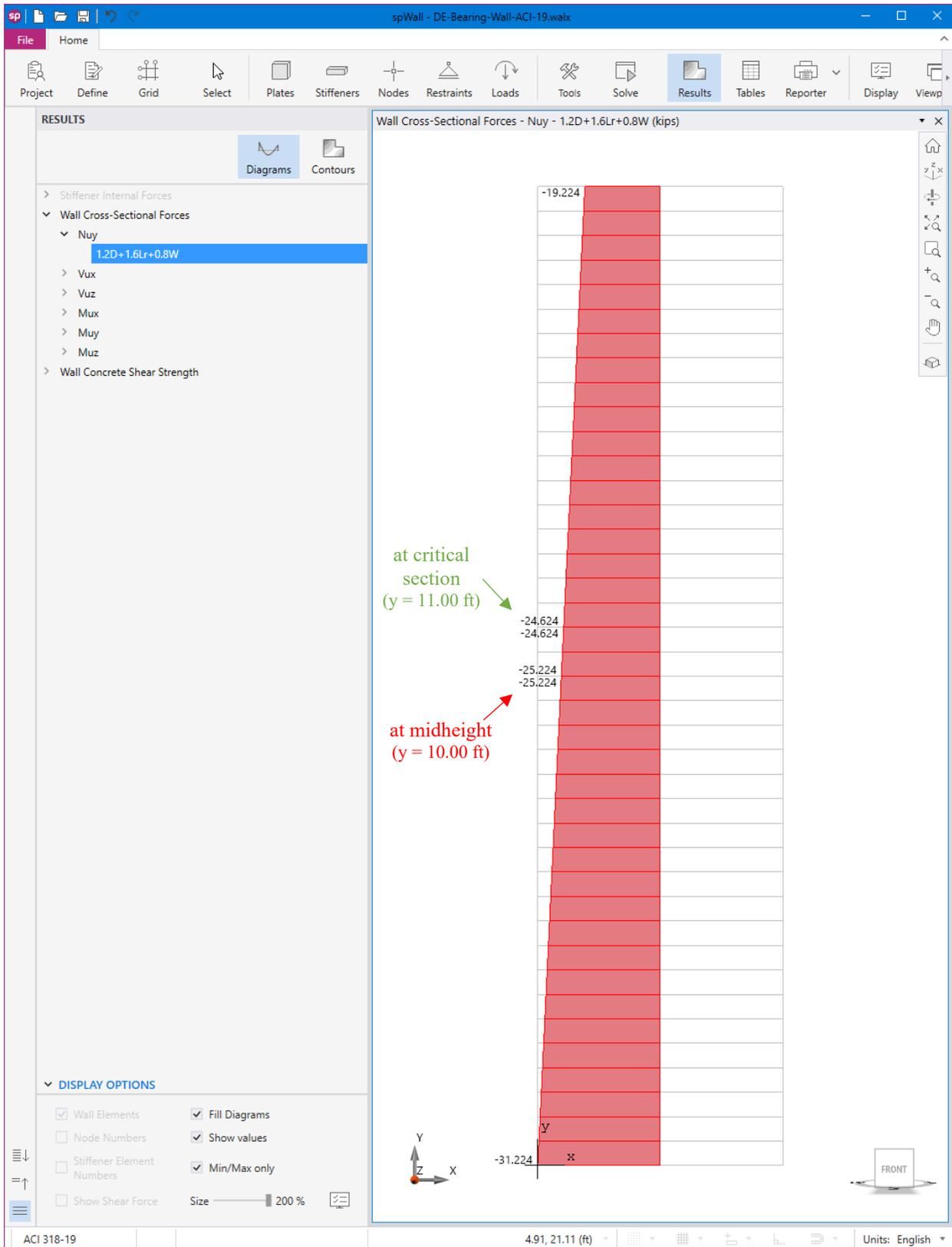


Figure 12 – Precast Wall Panel Axial Load Diagram (spWall)

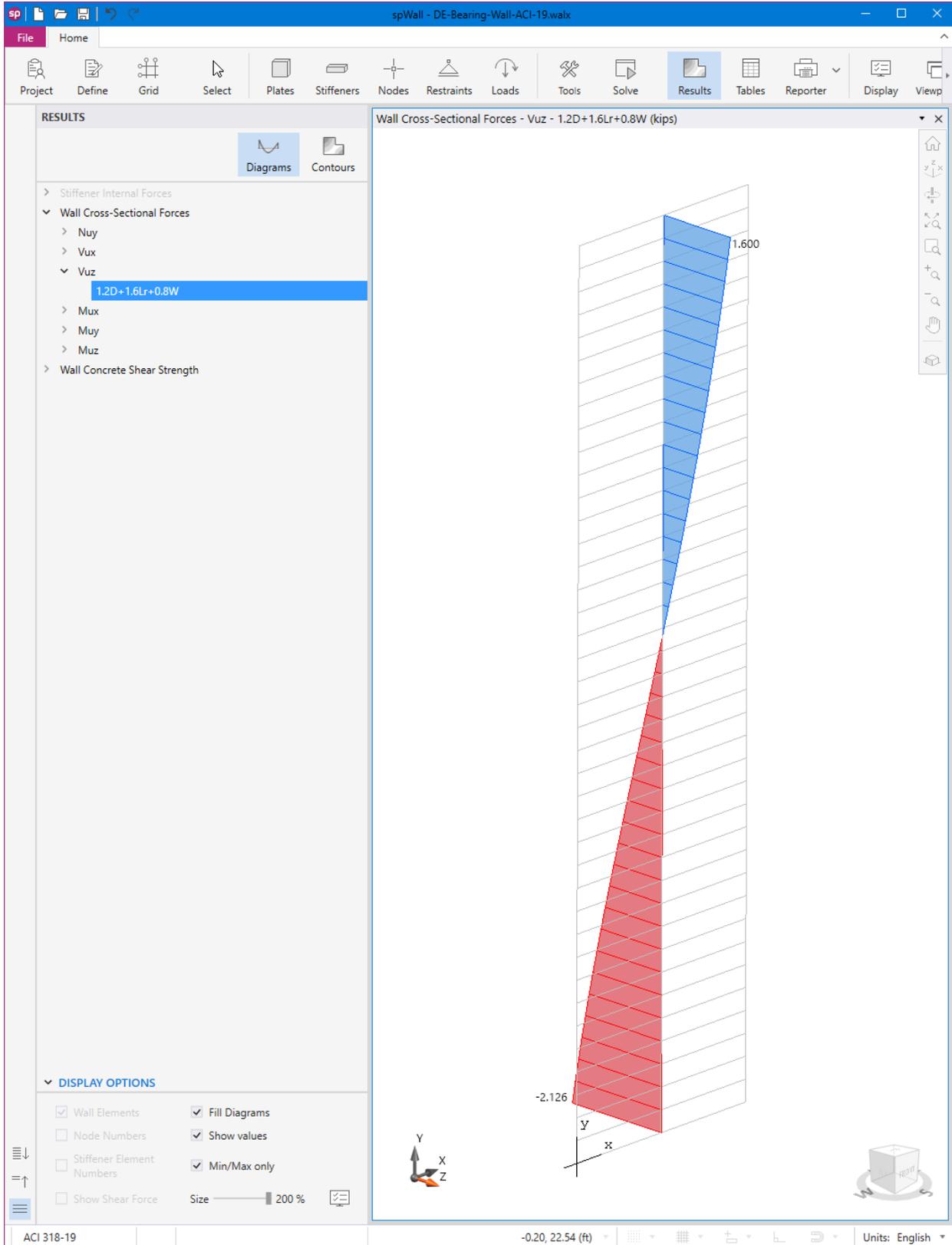


Figure 13 – Out-of-plane Shear Diagram (spWall)

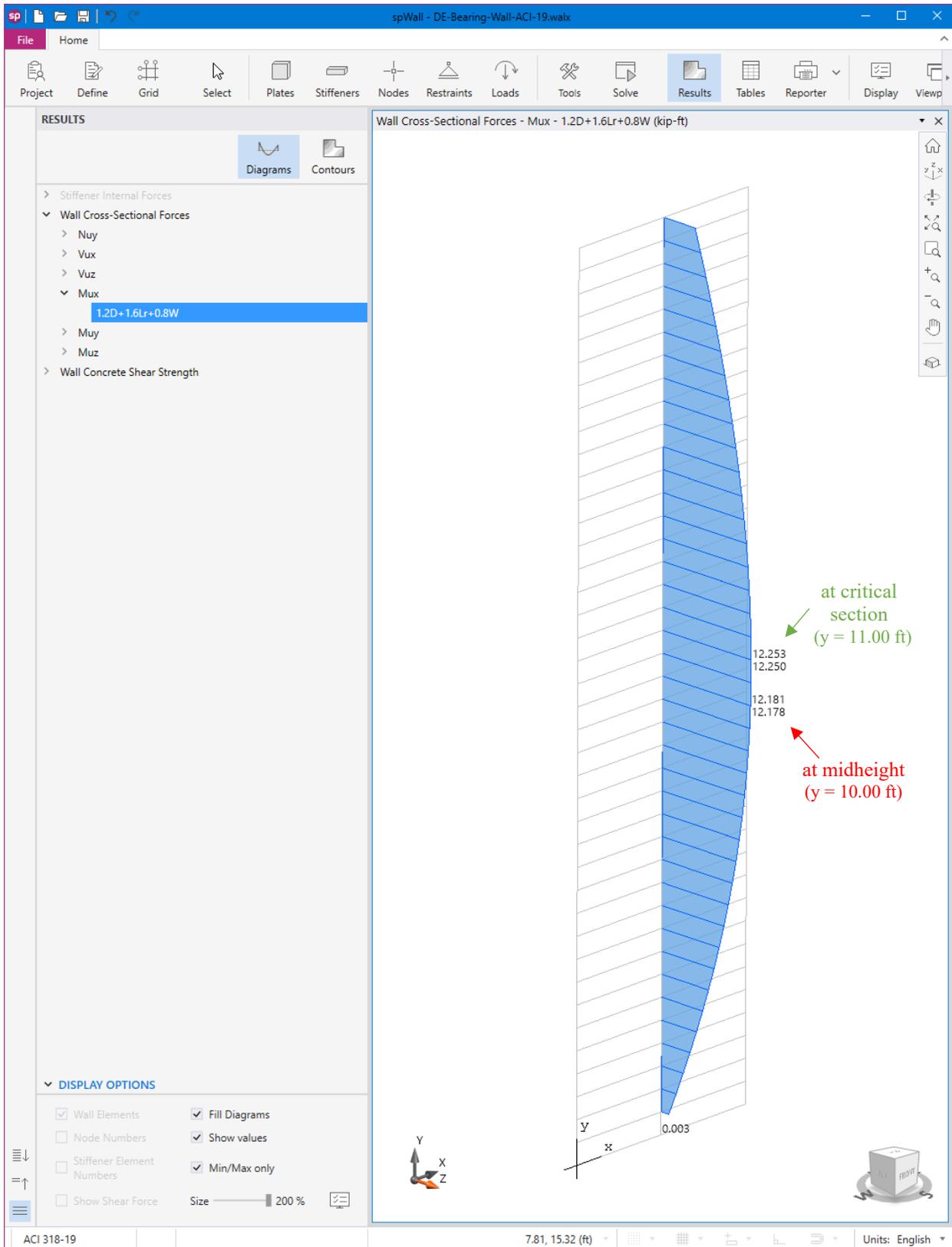


Figure 14 – Bearing Wall Moment Diagram (spWall)



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1. Project

1.1. General Information

File Name	DE-Bearing-Wall-ACI-19.walx
Project	Bearing Wall
Code	ACI 318-19
Units	English
Date	12/12/2025
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 crosssection	Yes

2. Definitions

2.1. Grid Lines

2.1.1. Vertical

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

2.1.2. Horizontal

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

2.2. Objects

2.2.1. Plates

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

2.3. Properties

2.3.1. Concrete

Label	f_c ksi	W_c pcf	E_c ksi	ν -	Precast	Used
C4	4.0000	150.00	3605.0	0.20	No	Yes

2.3.2. Reinforcement

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

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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

Label	Translations			Rotations			Used
	Dx	Dy	Dz	Rx	Ry	Rz	
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. ft	Y Coord. ft	Rigid Support	Spring Support
N1	2.50	20.00		

3.2. Plates

ID	Label	Shape	Top Left/Center X ft	Top Left/Center Y ft	Width (B) ft	Height (H)/Dia. (D) ft
P1	t8	Polygonal	0.00	20.00	5.00	20.00

3.3. Stiffeners

ID	Label	Direction	Start X	End X	Start Y	End Y	Length	Rigid Support
			ft	ft	ft	ft	ft	
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	Fy	Fz	Mx	My	Mz	Ecc.
		kips	kips	kips	kip-ft	kip-ft	kip-ft	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	Wy	Wz
		psf	psf	psf
P1	C	0.00	0.00	-30.00

4. Results

4.1. Envelope

4.1.1. Plate Reinforcement

Coordinate System: Global

at midheight
(y = 10.00 ft)

Element	Curtains	Direction	Mu (x/y)	Nu (x/y) Ld Comb.	ϵ_t	ϕ	As (x/y)	Rho Tie
			kip-ft/ft	kif	-		in ² /ft	%
191	1	Horizontal	0.48	-1.02 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
192	1	Horizontal	0.48	-1.02 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
193	1	Horizontal	0.48	-1.01 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
194	1	Horizontal	0.48	-1.01 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
195	1	Horizontal	0.48	-1.01 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
196	1	Horizontal	0.48	-1.01 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
197	1	Horizontal	0.48	-1.01 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
198	1	Horizontal	0.48	-1.01 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
199	1	Horizontal	0.48	-1.02 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
200	1	Horizontal	0.48	-1.02 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.42	-5.07 1.2D+1.6Lr+0...	0.0161	0.90	0.269	0.28
201	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.01 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
202	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.01 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
203	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.01 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
204	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20

Element	Curtains	Direction	Mu (x/y) kip-ft/ft	Nu (x/y) Ld Comb. klf	ϵ_t	ϕ	As (x/y) in ² /ft	Rho Tie %
		Vertical	2.44	-5.01 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
205	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.02 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
206	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.02 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
207	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.01 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
208	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.01 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
209	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.01 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
210	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0...	0.0299	0.90	0.192	0.20
		Vertical	2.44	-5.01 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
211	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.95 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
212	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.95 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
213	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.95 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
214	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.96 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
215	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.96 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
216	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.96 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
217	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.96 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
218	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.95 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
219	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.95 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
220	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.95 1.2D+1.6Lr+0...	0.0162	0.90	0.269	0.28
221	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.89 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
222	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.89 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
223	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.89 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
224	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.90 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
225	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.90 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
226	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.90 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
227	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.90 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
228	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.89 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
229	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.89 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28
230	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0...	0.0300	0.90	0.192	0.20
		Vertical	2.45	-4.89 1.2D+1.6Lr+0...	0.0163	0.90	0.269	0.28

at midheight
(y = 10.00 ft)

at critical
section
(y = 11.00 ft)

at midheight
(y = 10.00 ft)

at critical section
(y = 11.00 ft)

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

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

4.2. Service

4.2.1. Nodal Displacements

4.2.1.1. 1.2D+1.6Lr+0.8W

Coordinate System: Global

Node	Dx in	Dy in	Dz 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
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
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

at midheight
(y = 10.00 ft)

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

at critical
section
(y = 11.00 ft)

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

4.3. Ultimate

4.3.1. Plate Internal Forces

4.3.1.1. 1.2D+1.6Lr+0.8W

Coordinate System: Global

Element	Nxx kif	Nyy kif	Nxy kif	Mxx kip-ft/ft	Myy kip-ft/ft	Mxy 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
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

at midheight
(y = 10.00 ft)

$N_{yy,avg} = 5.04 \text{ kips}$

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

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
204	-1.00	-5.01	0.00	0.49	2.44	-0.00
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
220	-0.99	-4.95	-0.00	0.49	2.45	0.00
221	-0.98	-4.89	0.00	0.49	2.45	-0.00
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

at midheight
(y = 10.00 ft)

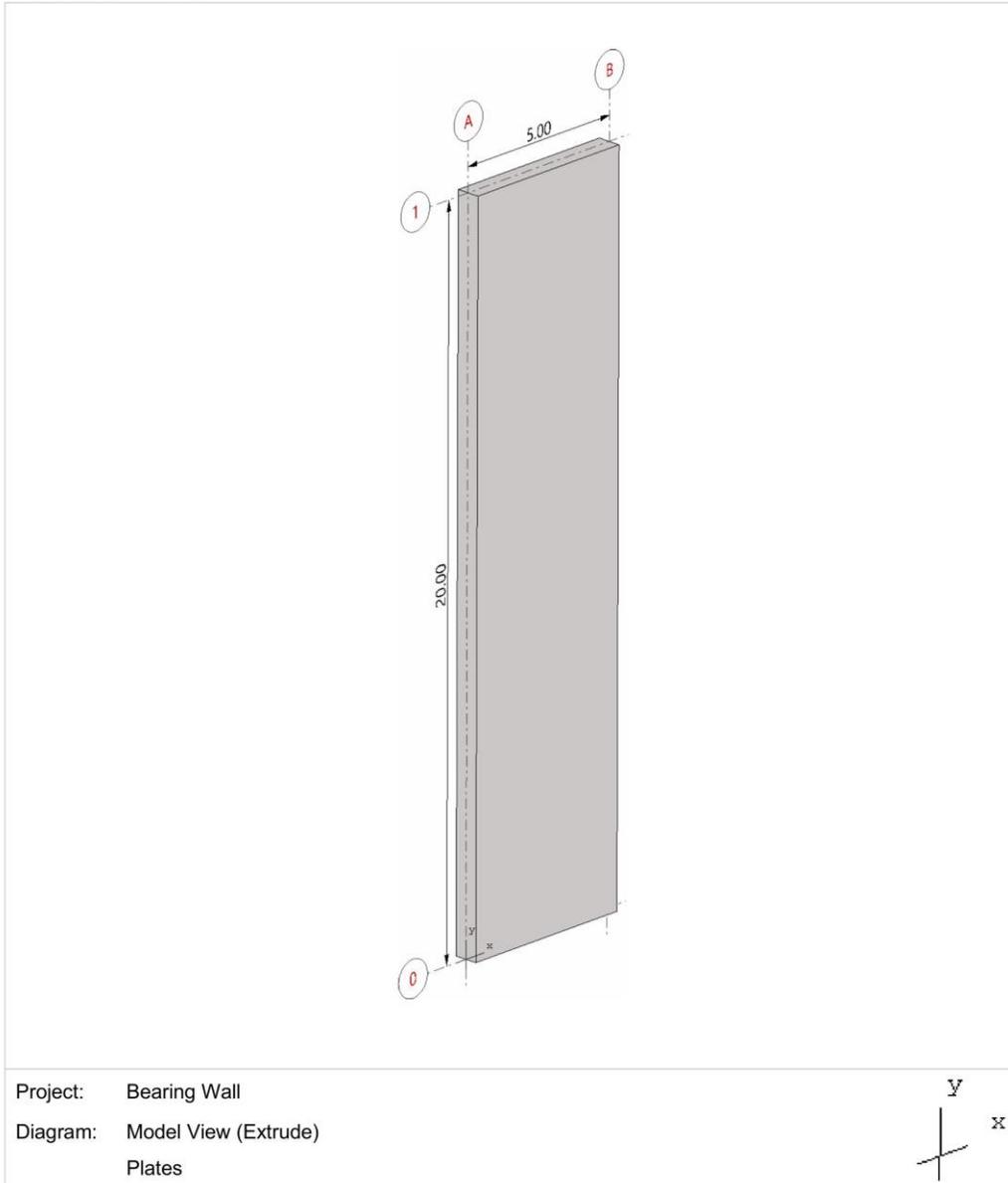
at critical
section
(y = 11.00 ft)

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

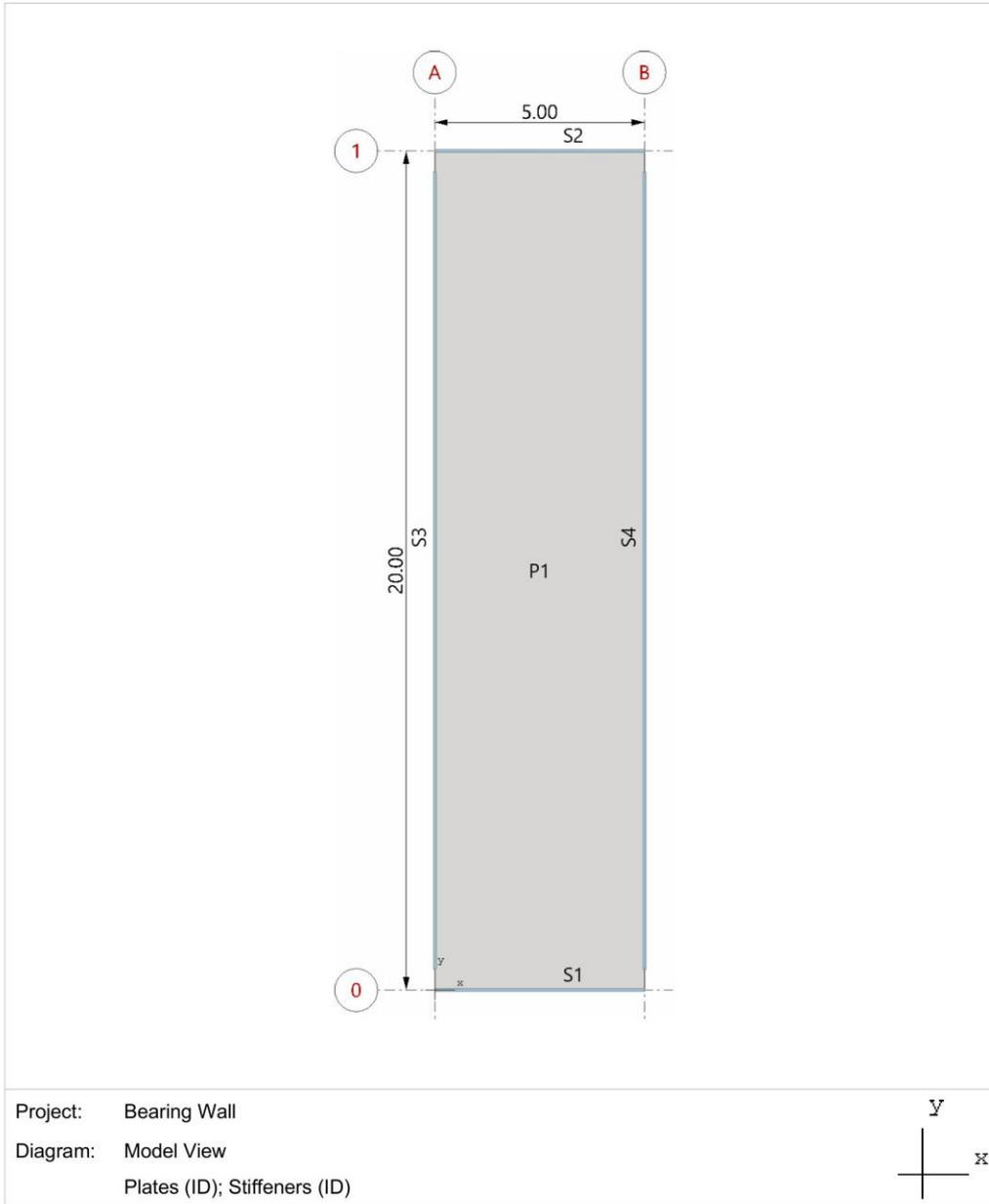
$$M_{yy,avg} = 2.45 \text{ kip-ft/ft}$$

5. Screenshots

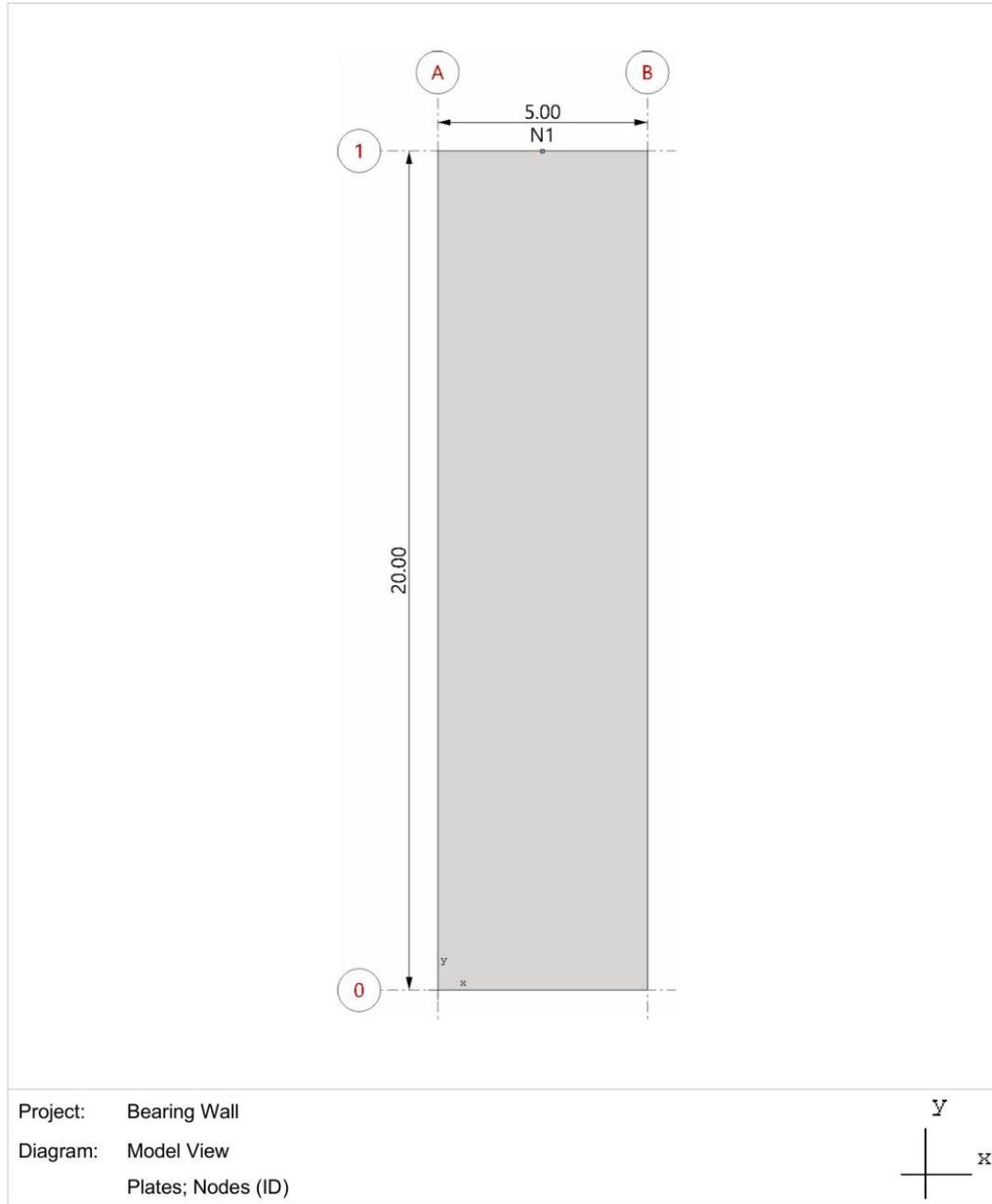
5.1. Extrude 3D view



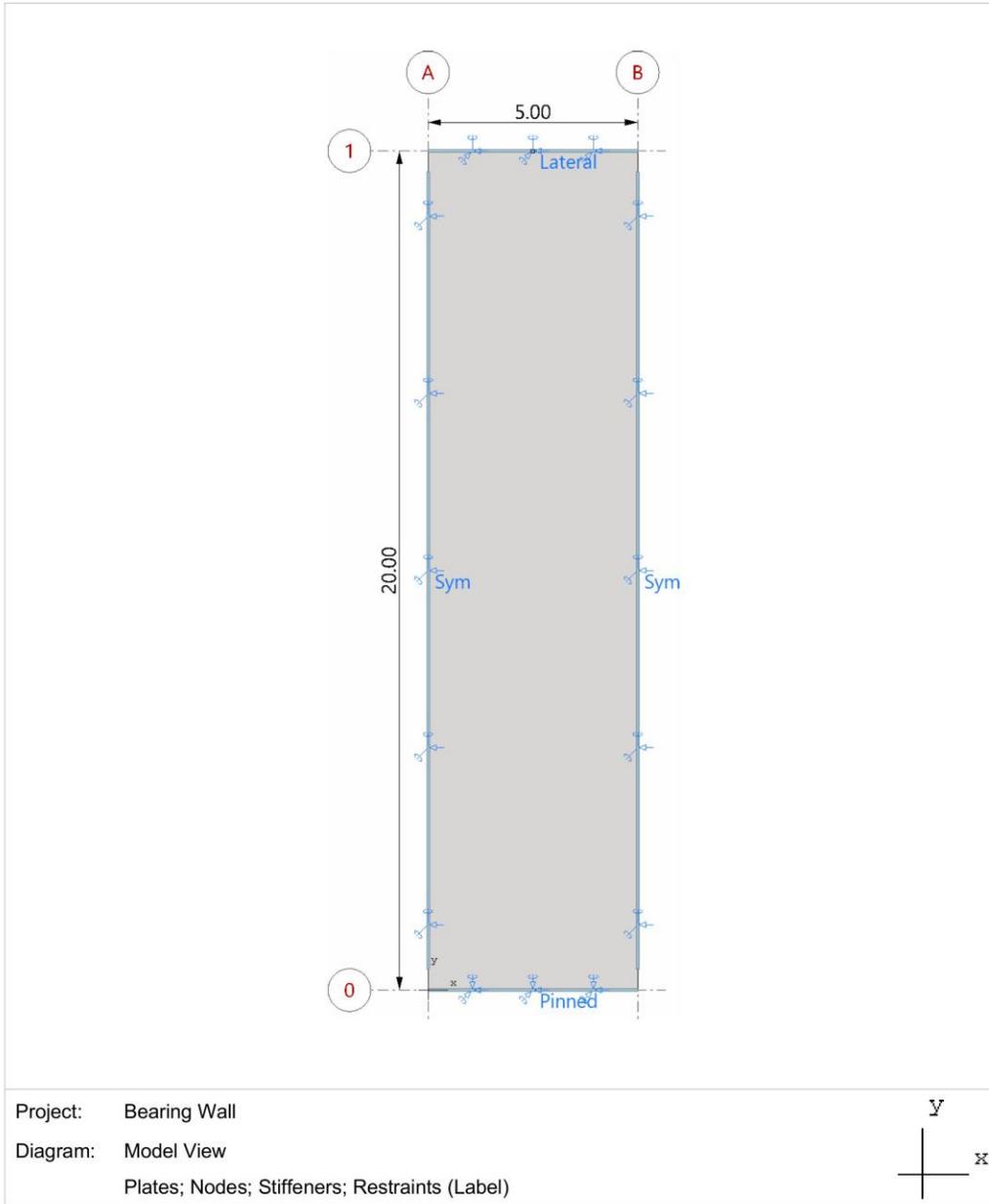
5.2. Plates & Stiffeners ID



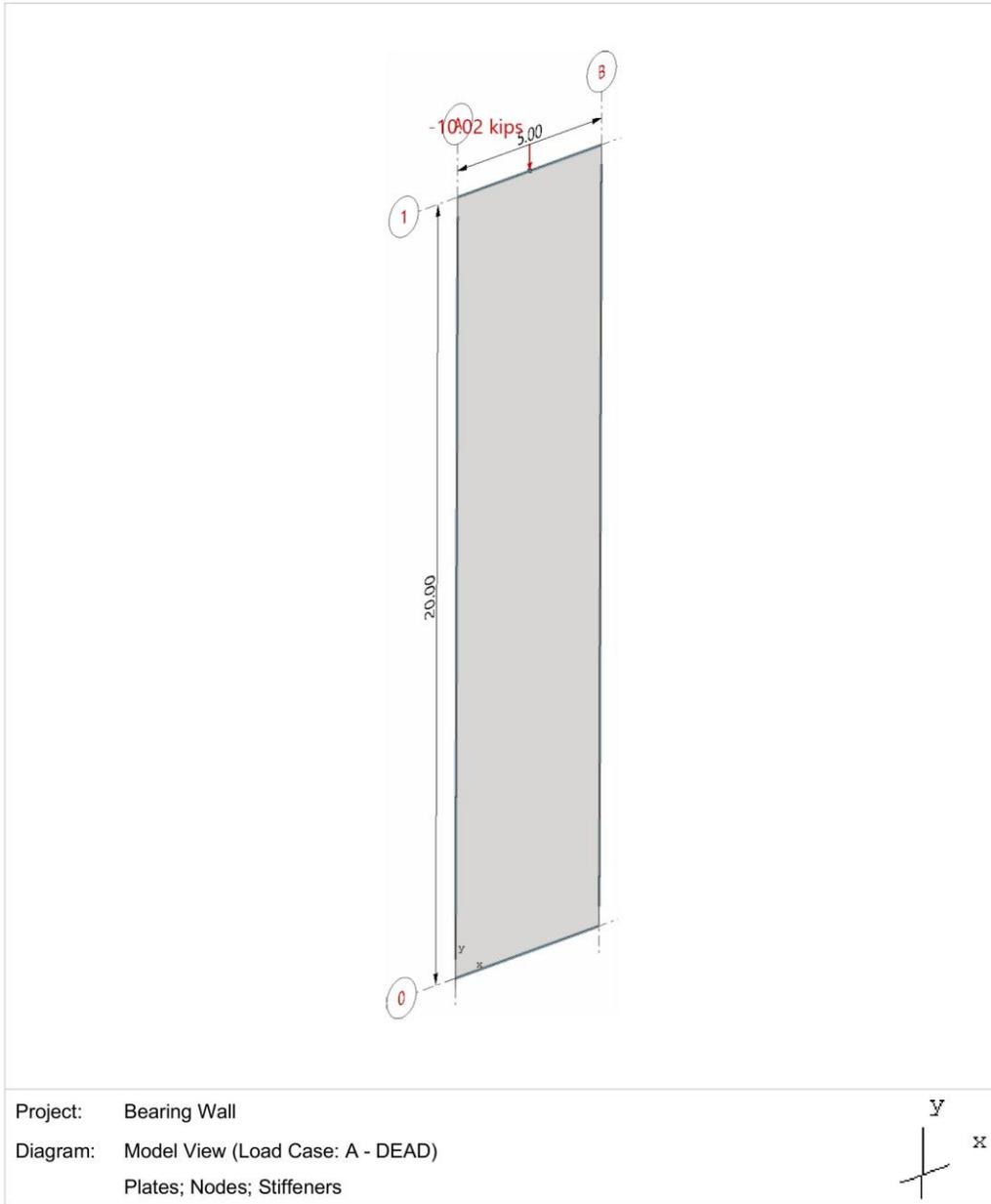
5.3. Nodes ID



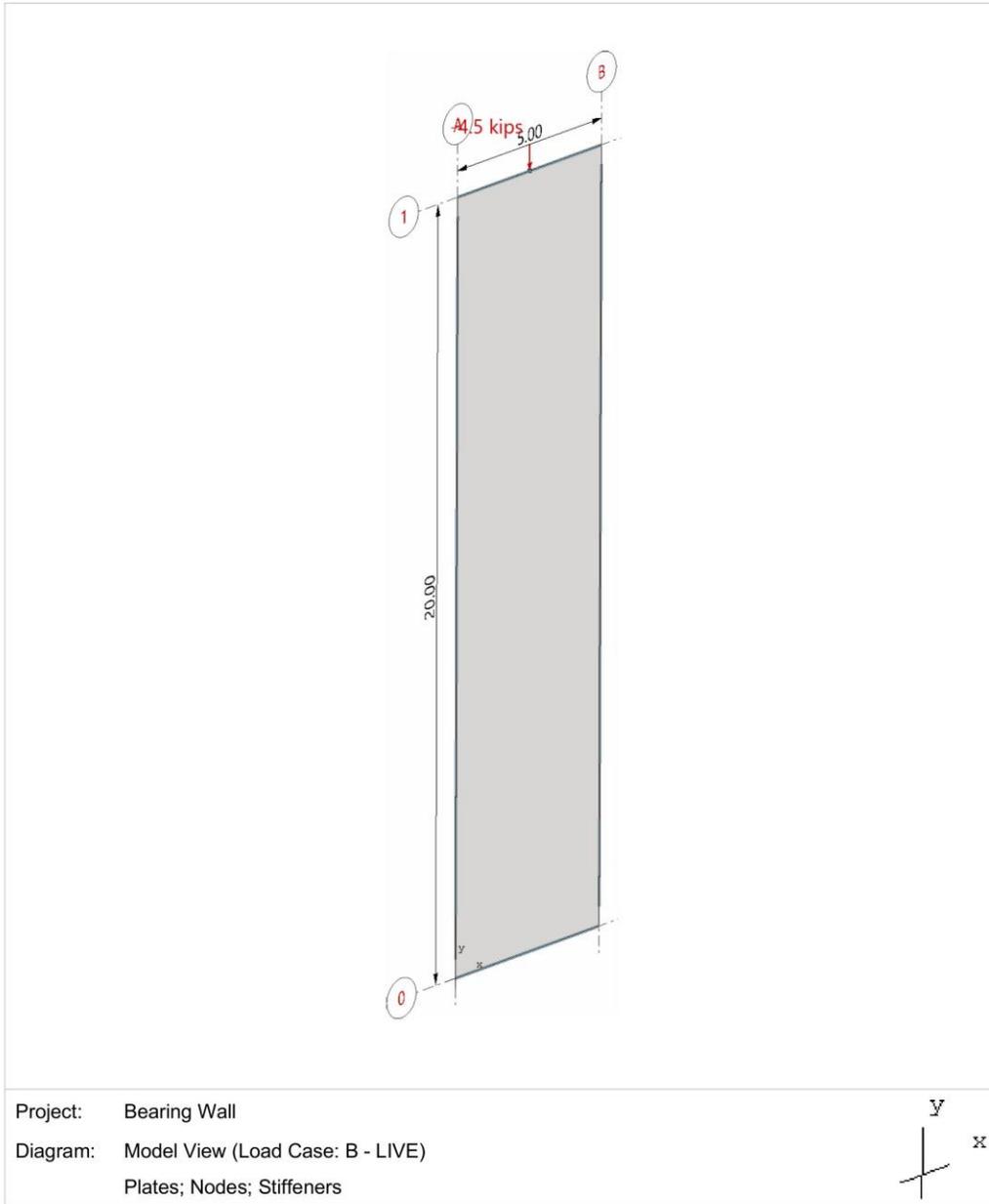
5.4. Restraints



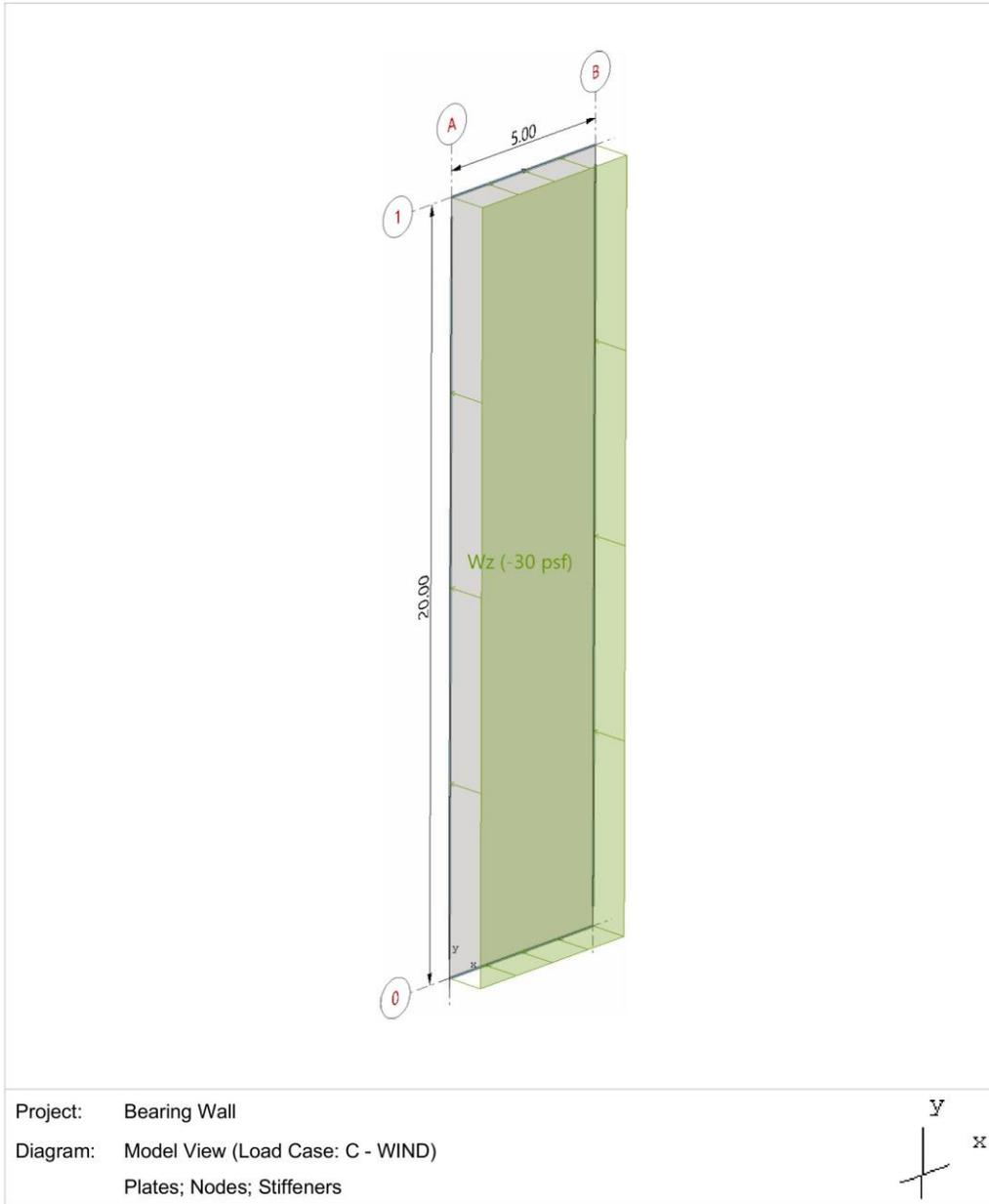
5.5. Loads - Case A - DEAD



5.6. Loads - Case B - LIVE



5.7. Loads - Case C - WIND



11. 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 $0.70I_g$ for uncracked walls and $0.35I_g$ for cracked walls. **ACI 318-19 (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 318-19 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 512.00 in.⁴ (Load Combination #2). Thus, the out-of-plane cracking coefficient for ultimate load combinations can be found as follows:

$$\alpha = \text{cracking coefficient} = \frac{0.75 \times I_{cr}}{I_g} = \frac{0.75 \times 33.38}{512.00} = 0.04890$$

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.

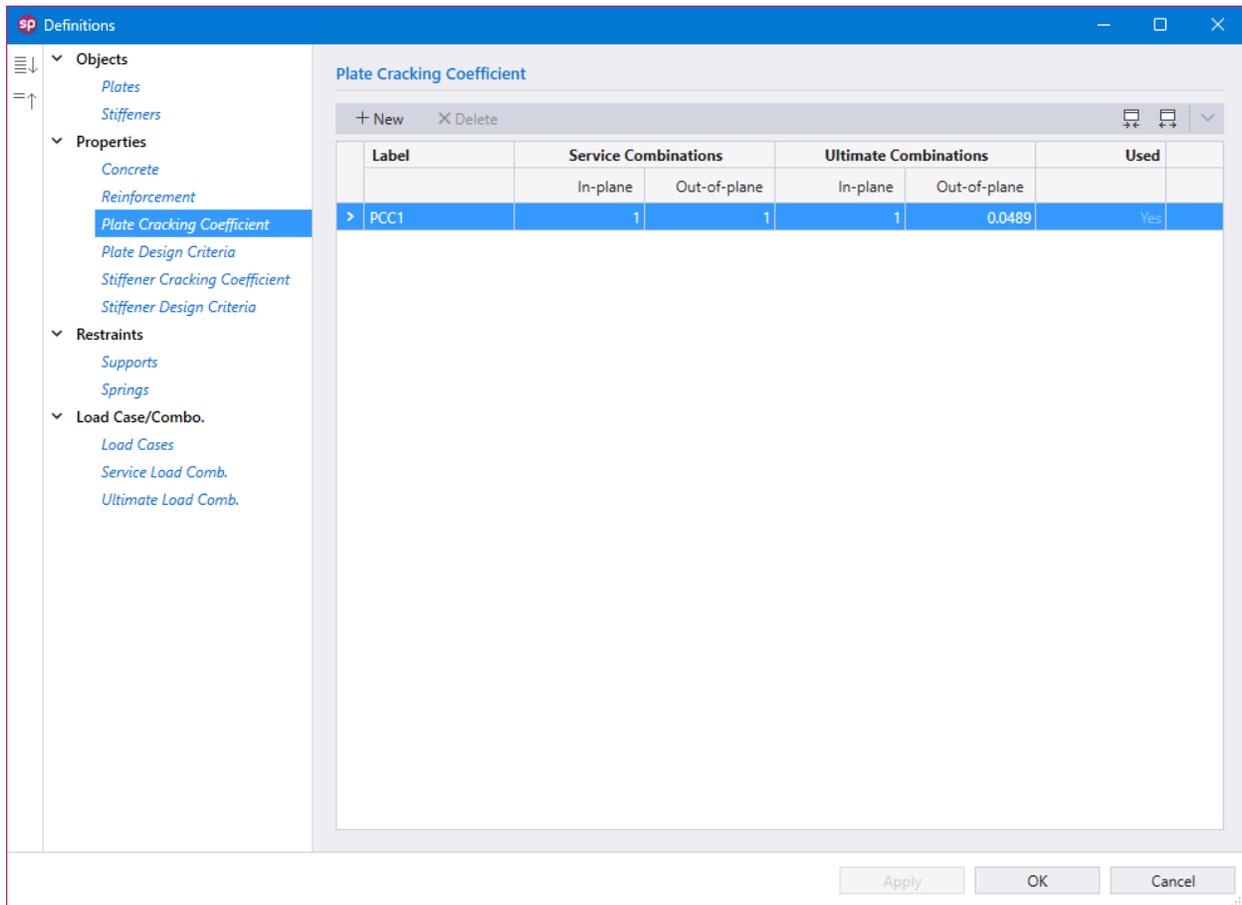


Figure 15 – Defining Cracking Coefficient (spWall)

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 $P\Delta$ 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	3.79
1.2 D + 1.6 L _r + 0.8 W	19.53	19.58
1.2 D + 0.5 L _r + 1.6 W	32.61	32.66
0.9 D + 1.6 W	31.20	31.22

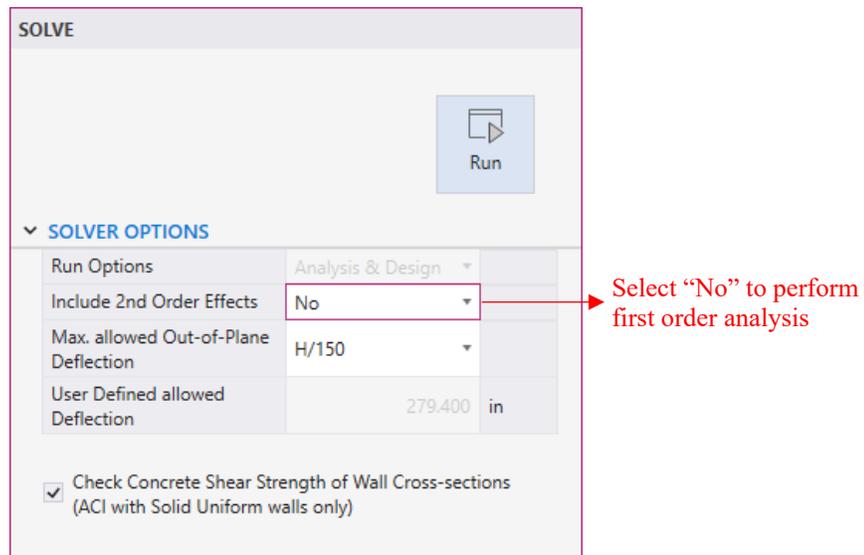


Figure 16 – Solver Module ([spWall](#))