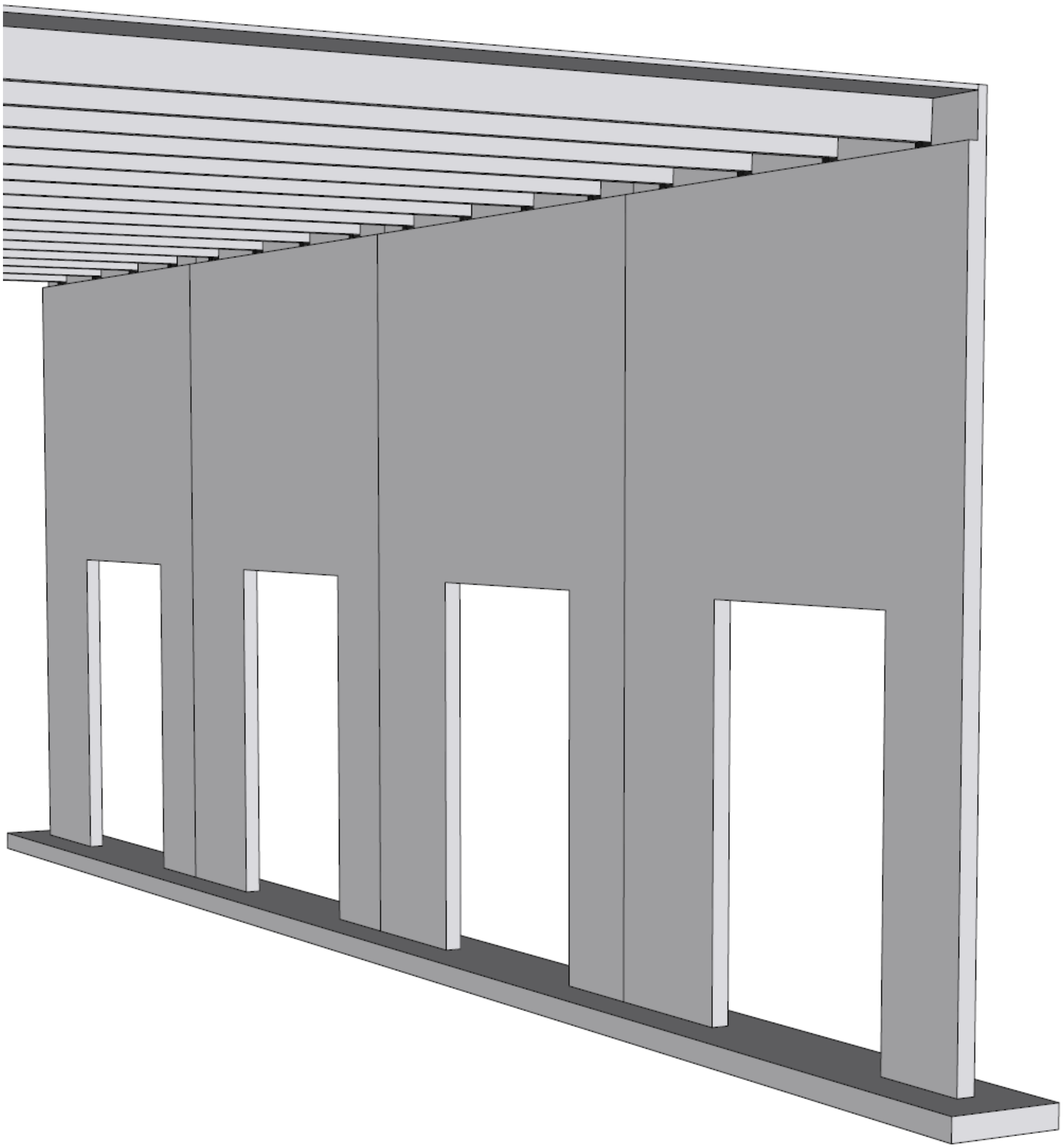
Reinforced Concrete Tilt-Up Wall Panel with Opening Analysis and Design (ACI 318-14 – ACI 551)



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Tilt-up is form of construction with increasing popularity owing to its flexibility and economics. Tilt-up concrete is essentially a precast concrete that is site cast instead of traditional factory cast concrete members. A structural reinforced concrete tilt-up wall panel with opening in a single-story warehouse (big-box) building provides gravity and lateral load resistance for the following applied loads from four roof joists bearing in wall pockets in addition to the wind:

Roof dead load	= 2.4 kip per joist
Roof live load	= 2.5 kip per joist
Wind load	= 27.2 psf (Out-of-Plane)

The assumed tilt-up 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. Additionally, different modeling and analysis techniques using spWall engineering software program to investigate and design tilt-up wall panels with openings are discussed.

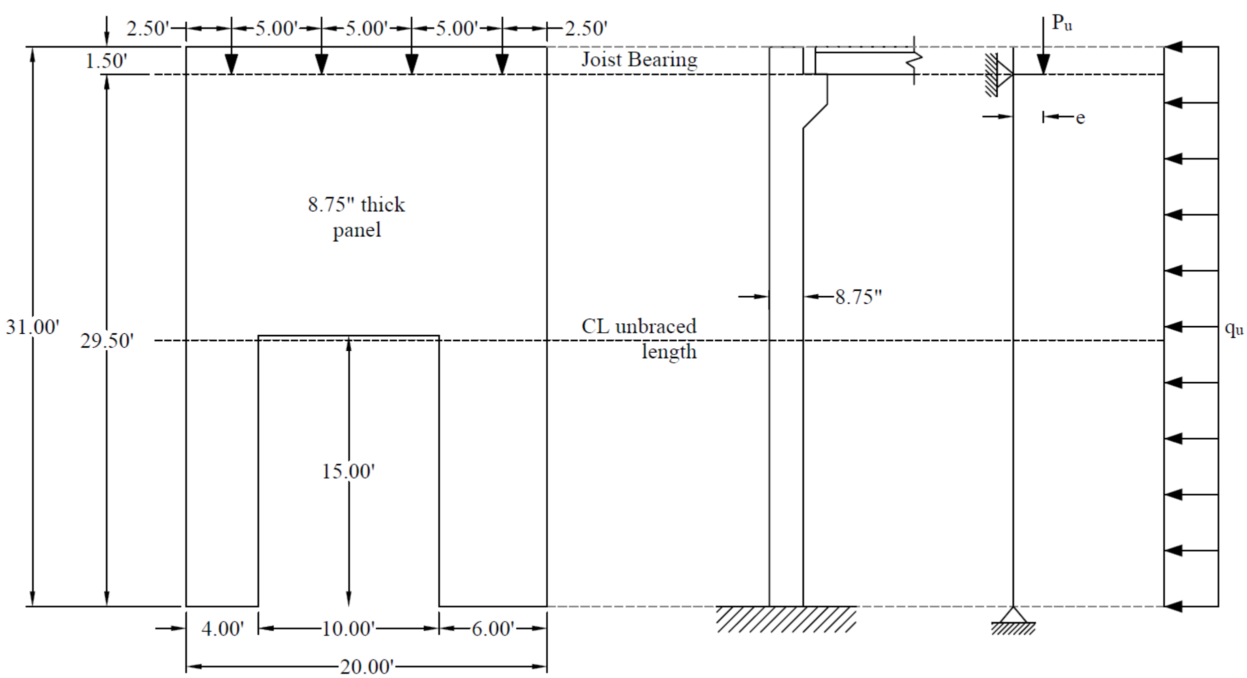


Figure 1 – Reinforced Concrete Tilt-Up Wall Panel Geometry (with 10 x 15 ft Door Opening)

Contents

Code

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

Reference

- Design Guide for Tilt-Up Concrete Panels, ACI 551.2R-15, 2015, Example B.2
- 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 = l_c = 31 ft – 1.5 ft = 29.5 ft

Assumed wall thickness = 8.75 in. (Note: reference example started with a thickness of 6.25 in. that was deemed not sufficient to meet tension control condition to use Alternative Method for Out-of-Plane Slender Wall Analysis).

Assumed eccentricity = e_{cc} = 3 in.

Assumed vertical reinforcement: 7 #6 (single layer) for the left leg (design strip)
7 #6 (single layer) for the right leg (design strip)

Solution

The effect of openings on out-of-plane bending in tilt-up panels can be approximated in hand calculations by a simple, one-dimensional strip analysis that provides accuracy and economy for most designs. Where openings occur, the entire lateral and axial load, including self-weight above the critical section, is distributed to supporting legs or design strips at each side of the opening (sometimes referred to as wall piers). ***ACI 551.2R-15 (7.2)***

The effective width of the strip should be limited to approximately 12 times the panel thickness to avoid localized stress concentrations along the edge of the opening. This limit is not mandated by ACI 318, but is included as a practical guideline where the opening width is less than one-half the clear vertical span. In most cases the tributary width for loads can be taken as the width of the strip plus one-half the width of adjacent openings. Tilt-up design strips should have constant properties for the full height and the reinforcement should not be cut off just above or below the opening. Thickened vertical or horizontal sections can be introduced within the panel where openings are large or where there are deep recesses on the exterior face. Some conditions may require ties around all vertical reinforcement bars in a vertical pilaster for the full height of the tilt-up panel. ***ACI 551.2R-15 (7.2)***

Left Leg Analysis and Design

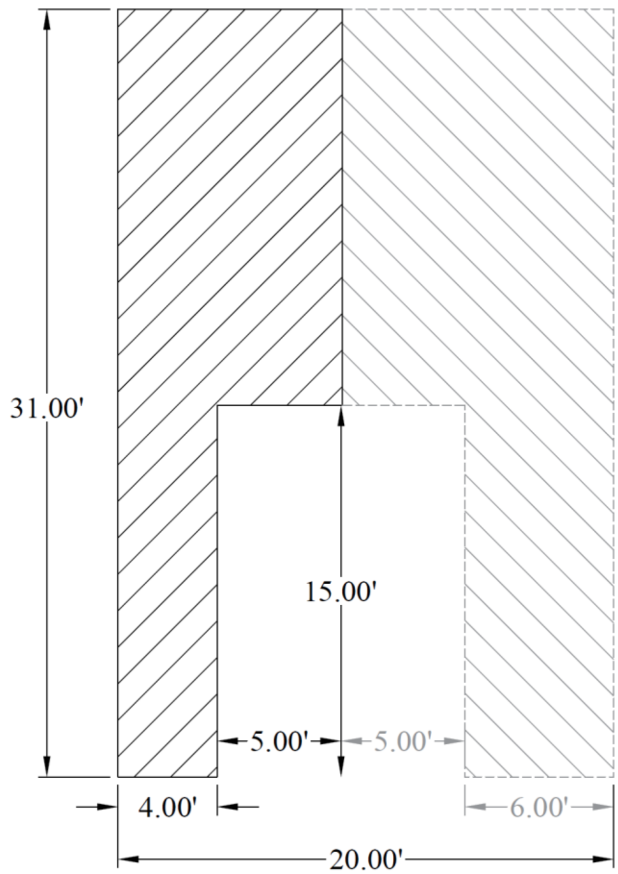


Figure 2 – Tilt-Up Design Strips Tributary Widths for Loads

1. Minimum Vertical Reinforcement

$$\rho_l = \frac{A_{v,vertical}}{b \times h} = \frac{3.08}{(4 \times 12) \times 8.75} = 0.0073$$ ***ACI 318-14 (2.2)***

$$\rho_{l,\min} = 0.0015$$ ***ACI 318-14 (Table 11.6.1)***

$$\rho_l = 0.0073 \geq \rho_{l,\min} = 0.0015 \ (\mathbf{o.k.})$$

$$s_{l,\max} = \text{smallest of} \begin{Bmatrix} 3 \times h \\ 18 \text{ in.} \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 3 \times 8.75 \\ 18 \text{ in.} \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 26.25 \text{ in.} \\ 18 \text{ in.} \end{Bmatrix} = 18 \text{ in.}$$ ***ACI 318-14 (11.7.2.1)***

$$s_{l,provided} = \frac{4 \times 12}{7} = 6.86 \text{ in.} \leq s_{l,\max} = 18 \text{ in.} \ (\mathbf{o.k.})$$

2. Alternative Method for Out-of-Plane Slender Wall Analysis ACI 318 Provisions

The design guide for tilt-up concrete panels ACI 551 states that tilt-up concrete walls can be analyzed using the provisions of Chapter 14 of the ACI 318-11, the same provisions are presented in 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 of the ACI 318-14. The method is applicable when the conditions summarized below are met:

- 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 $0.06f_c{'}A_g$ **_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))_**

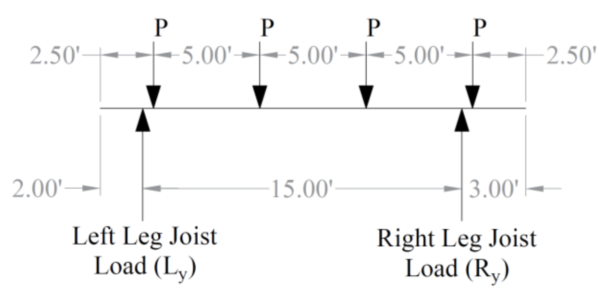
3. Tilt-Up Wall Structural Analysis

3.1. Applied loads

The tributary width for loads can be taken as the width of the strip plus one-half the width of adjacent openings.

$$\text{Wall self-weight} = \frac{8.75}{12}\times 150\times\left[4\times\left(\frac{29.5}{2}+1.5\right)+5\times(31-15)\right]\times\frac{1\text{ kip}}{1000\text{ lb}} = 15.86\text{ kip}$$

Joist loads are divided between the individual legs assuming an equivalent simply supported beam across the top of the panel with the supports at the centerline of each leg.



$$P_{DL} = 4.48\text{ kip (for the left leg)}$$

$$P_{LL} = 4.67\text{ kip (for the left leg)}$$

$$w = 27.2\text{ lb/ft}^2$$

3.2. Maximum wall forces

The calculation of maximum factored wall forces in accordance with 11.8.3.1 including moment magnification due to second order (P-Δ) effects is shown below (load combination U = 1.2 D + 1.6 L_r + 0.5 W is considered in this example):

$$P_{ua} = 1.2\times 4.48 + 1.6\times 4.67 = 12.84 \text{ kip}$$

$$P_{um} = 12.84 + 1.2\times 15.86 = 31.87 \text{ kip}$$

$$w_u = 0.5\times 27.2\times(4+5)\times\frac{1\text{ kip}}{1000\text{ lb}} = 0.122 \text{ kip/ft}$$

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

$$M_{ua} = \frac{w_u\times l_c^2}{8}+\frac{P_{ua}\times e}{2} = \frac{0.122\times(29.5)^2}{8}+\frac{12.84\times 3}{2\times 12} = 14.92 \text{ ft-kip}$$

Where M_{ua} is the maximum factored moment at midheight of wall due to lateral and eccentric vertical loads, not including $P\Delta$ effects. **<u>*ACI 318-14 (11.8.3.1)*</u>**

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

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

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

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

$$A_{se} = A_s+\frac{P_{um}\times h}{2\times f_y\times d} = 3.08+\frac{31.87\times 8.75}{2\times 60\times(8.75/2)} = 3.61 \text{ in.}^2$$ **<u>*ACI 318-14 (R11.8.3.1)*</u>**

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

$$a = \frac{A_{se}\times f_y}{0.85\times f_c^{'}\times b} = \frac{3.61\times 60}{0.85\times 4\times(4\times 12)} = 1.328 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{1.328}{0.85} = 1.562 \text{ in.}$$

$$\frac{c}{d} = \frac{1.562}{4.375} = 0.357 < 0.375 \therefore \text{ tension-controlled}$$ **<u>*ACI 318-14 (R21.2.2)*</u>**

$$\phi = 0.9$$ **<u>*ACI 318-14 (Table 21.2.2)*</u>**

$$I_{cr} = 8.0\times 3.61\times(4.375-1.562)^2+\frac{(4\times 12)\times 1.562^3}{3} = 290.85 \text{ in.}^4$$ **<u>*ACI 318-14 (11.8.3.1(c))*</u>**

$$M_u = \frac{M_{ua}}{1-\dfrac{P_{um}}{0.75 \times K_b}}$$ **_ACI 318-14 (Eq. 11.8.3.1(d))_**

$$K_b = \frac{48 \times E_c \times I_{cr}}{5 \times l_c^2} = \frac{48 \times 3605 \times 290.85}{5 \times (29.5 \times 12)^2} = 80.32 \text{ kip}$$

$$M_u = \frac{14.92}{1-\dfrac{31.87}{0.75 \times 80.32}} = 31.68 \text{ ft-kip}$$

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

$$P_n = \frac{P_{um}}{\phi} = \frac{31.87}{0.9} = 35.42 \text{ kips}$$

$$a = \frac{A_{se,w} \times f_y}{0.85 \times f_c^{'} \times l_w} = \frac{\dfrac{P_n \times h}{2 \times d} + A_s \times f_y}{0.85 \times f_c^{'} \times l_w} = \frac{\dfrac{35.42 \times 8.75}{2 \times 4.375} + 3.08 \times 60}{0.85 \times 4 \times 4 \times 12} = 1.349 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{1.349}{0.85} = 1.587 \text{ in.}$$

$$\varepsilon_t = \left(\frac{0.003}{c}\right) \times d_t - 0.003 = \left(\frac{0.003}{1.587}\right) \times 4.375 - 0.003 = 0.0053 > 0.0050$$

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

4. Tilt-Up 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}

$$f_r = 7.5\lambda\sqrt{f_c^{'}} = 7.5 \times 1.0 \times \sqrt{4{,}000} = 474.34 \text{ psi}$$ **_ACI 318-14 (19.2.3.1)_**

$$I_g = \frac{l_w h^3}{12} = \frac{(4 \times 12) \times 8.75^3}{12} = 2679.69 \text{ in.}^4$$

$$y_t = \frac{h}{2} = \frac{8.75}{2} = 4.375 \text{ in.}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{474.34 \times 2679.69}{4.375} \times \frac{1}{1000} \times \frac{1}{12} = 24.21 \text{ ft-kip}$$ **_ACI 318-14 (24.2.3.5(b))_**

5. Tilt-Up Wall Flexural Moment Capacity (ϕM_n)

$$M_n = A_{se} \times f_y \times \left(d - \frac{a}{2}\right) = 3.61 \times 60 \times \left(4.375 - \frac{1.349}{2}\right) = 801.76 \text{ in.-kip} = 66.81 \text{ ft-kip}$$

It was shown previously that the section is tension controlled → $\phi = 0.9$

$\phi M_n = \phi \times M_n = 0.9 \times 66.81 = 60.13 \text{ ft-kip} > M_u = 31.68 \text{ ft-kip } (\textbf{o.k.})$ **ACI 318-14 (11.5.1.1(b))**

$\phi M_n = 60.13 \text{ ft-kip} > M_{cr} = 24.21 \text{ ft-kip } (\textbf{o.k.})$ **ACI 318-14 (11.8.1.1(c))**

$$\Delta_u = \frac{M_u}{0.75 \times K_b} = \frac{31.68 \times 12}{0.75 \times 80.32} = 6.311 \text{ in.}$$ **ACI 318-14 (11.8.3.1(b))**

6. Tilt-Up Wall Vertical Stress Check

$$\frac{P_{um}}{A_g} = \frac{31.87 \times 1000}{8.75 \times (4 \times 12)} = 75.89 \text{ psi} < 0.06 \times f_c' = 0.06 \times 4{,}000 = 240 \text{ psi } (\textbf{o.k.})$$ **ACI 318-14 (11.8.1.1(d))**

7. Tilt-Up Wall Shear Stress Check

In-plane shear is not evaluated 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, it can be determined that the maximum shear force is under 5 kip. The wall left leg (the weakest section) has a shear capacity approximately 50 kip and no detailed calculations are required by engineering judgement. (See figure 10 for detailed shear force diagram)

8. Tilt-Up 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 318-11 (11.8.1.1(e))**

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

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

$$M_{sa} = \frac{w_s \times l_c^2}{8} + \frac{P_a \times e}{2} = \frac{\left[0.7 \times \frac{27.2}{1.6} \times (4+5)\right] \times (29.5)^2}{8 \times 1000} + \frac{(4.48) \times 3/12}{2} = 12.21 \text{ ft-kip}$$

$$P_s = P_{DL} + \text{wall self-weight} = 4.48 + 15.86 = 20.34 \text{ kip}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = 24.21 \text{ ft-kip (as calculated perviously)}$$ **ACI 318-14 (24.2.3.5(b))**

$$\Delta_{cr} = \frac{5}{48} \times \frac{M_{cr} \times l_c^2}{E_c \times I_g} = \frac{5}{48} \times \frac{24.21 \times 12 \times (29.5 \times 12)^2}{3{,}605 \times 2679.69} = 0.393 \text{ in.}$$ ***<u>ACI 318-14 (11.8.4.3(a))</u>***

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 $\Delta_s = \left(\frac{M_{sa}}{M_{cr}}\right)\Delta_{cr} = \left(\frac{12.21}{24.21}\right) \times 0.393 = 0.198 \text{ in.}$

$$M_a = M_{sa} + P_s \Delta_s = 12.21 \times 12 + 20.34 \times 0.198 = 150.55 \text{ in.-kip} = 12.55 \text{ ft-kip}$$

$$\Delta_s = \left(\frac{M_a}{M_{cr}}\right)\Delta_{cr} = \frac{12.55}{24.21} \times 0.393 = 0.203 \text{ in.}$$ ***<u>ACI 318-14 (Table 11.8.4.1)</u>***

No further iterations are required

$$M_a = 12.55 \text{ ft-kip} < \frac{2}{3} M_{cr} = \frac{2}{3} \times 24.21 = 16.14 \text{ ft-kip} \quad (\textbf{o.k.})$$

$$\Delta_s = 0.203 \text{ in.} < \frac{l_c}{150} = \frac{29.5 \times 12}{150} = 2.36 \text{ in.} \quad (\textbf{o.k.})$$

The wall left leg is adequate with 7 #6 vertical reinforcement and 8.75 in. thickness.

Right Leg Analysis and Design

Repeating the same process for the right leg (right design strip) leads to the following results:

$P_{DL} = 5.12$ kip (for the right leg)

$P_{LL} = 5.33$ kip (for the right leg)

$w = 27.2\text{ lb/ft}^2$

$P_{ua} = 14.68$ kip

$P_{um} = 37.97$ kip

$w_u = 0.150$ kip/ft

$M_{ua} = 18.11$ ft-kip

$A_{se} = 3.71\text{ in.}^2$

$a = 0.910$ in.

$c = 1.071$ in.

$\frac{c}{d} = 0.245 < 0.375 \therefore$ tension-controlled

$I_{cr} = 355.58\text{ in.}^4$

$K_b = 98.20$ kip

$M_u = 37.38$ ft-kip

$I_g = 4019.53\text{ in.}^4$

$M_{cr} = 36.32$ ft-kip

$\phi M_n = 65.35\text{ ft-kip} > M_u = 37.38\text{ ft-kip}\ (\textbf{o.k.})$

$\phi M_n = 65.35\text{ ft-kip} > M_{cr} = 36.32\text{ ft-kip}\ (\textbf{o.k.})$

$\Delta_u = 6.091$ in.

$\frac{P_{um}}{A_g} = 60.28\text{ psi} < 0.06 \times f_c^{'} = 240\text{ psi}\ (\textbf{o.k.})$

$M_{sa} = 14.88$ ft-kip

$\Delta_{cr} = 0.393$ in.

$M_a = 15.21\text{ ft-kip} < \frac{2}{3} M_{cr} = 24.21\text{ ft-kip}\quad (\textbf{o.k.})$

$\Delta_s = 0.164\text{ in.} < \frac{l_c}{150} = 2.36\text{ in.}\quad (\textbf{o.k.})$

The wall right leg is adequate with 7 #6 vertical reinforcement and 8.75 in. thickness.

9. Analysis and Design of the Section between the Design Strips

For the vertical reinforcement for the section between the design strips, minimum area of steel should be provided as follows:

$$\rho_{l,\min} = 0.0015$$ ***ACI 318-14 (Table 11.6.1)***

Try single layer panel reinforcement of 9 #4.

$$\rho_l = \frac{A_{v,vertical}}{b \times h} = \frac{9 \times 0.20}{(10 \times 12) \times 8.75} = 0.0017$$ ***ACI 318-14 (2.2)***

$$\rho_l = 0.0017 \geq \rho_{l,\min} = 0.0015 \ (\textbf{o.k.})$$

$$s_{l,\max} = \text{smallest of} \begin{Bmatrix} 3 \times h \\ 18 \text{ in.} \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 3 \times 8.75 \\ 18 \text{ in.} \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 26.25 \text{ in.} \\ 18 \text{ in.} \end{Bmatrix} = 18 \text{ in.}$$ ***ACI 318-14 (11.7.2.1)***

$$s_{l,provided} = \frac{10 \times 12}{9} = 13.3 \text{ in.} \leq s_{l,\max} = 18 \text{ in.} \ (\textbf{o.k.})$$

10. Horizontal Reinforcement

$$\rho_{h,\min} = 0.00200$$ ***ACI 318-14 (Table 11.6.1)***

Try single layer panel reinforcement of 33 #4.

$$\rho_h = \frac{A_{h,vertical}}{b \times h} = \frac{33 \times 0.20}{(31 \times 12) \times 8.75} = 0.00203$$ ***ACI 318-11 (2.2)***

$$\rho_h = 0.00203 \geq \rho_{h,\min} = 0.00200 \ (\textbf{o.k.})$$

Additional reinforcement requirements are outlined in ***ACI 318-14 (11.7.5.1)*** for header and jambs of openings.

11. Tilt-Up 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),
- 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.

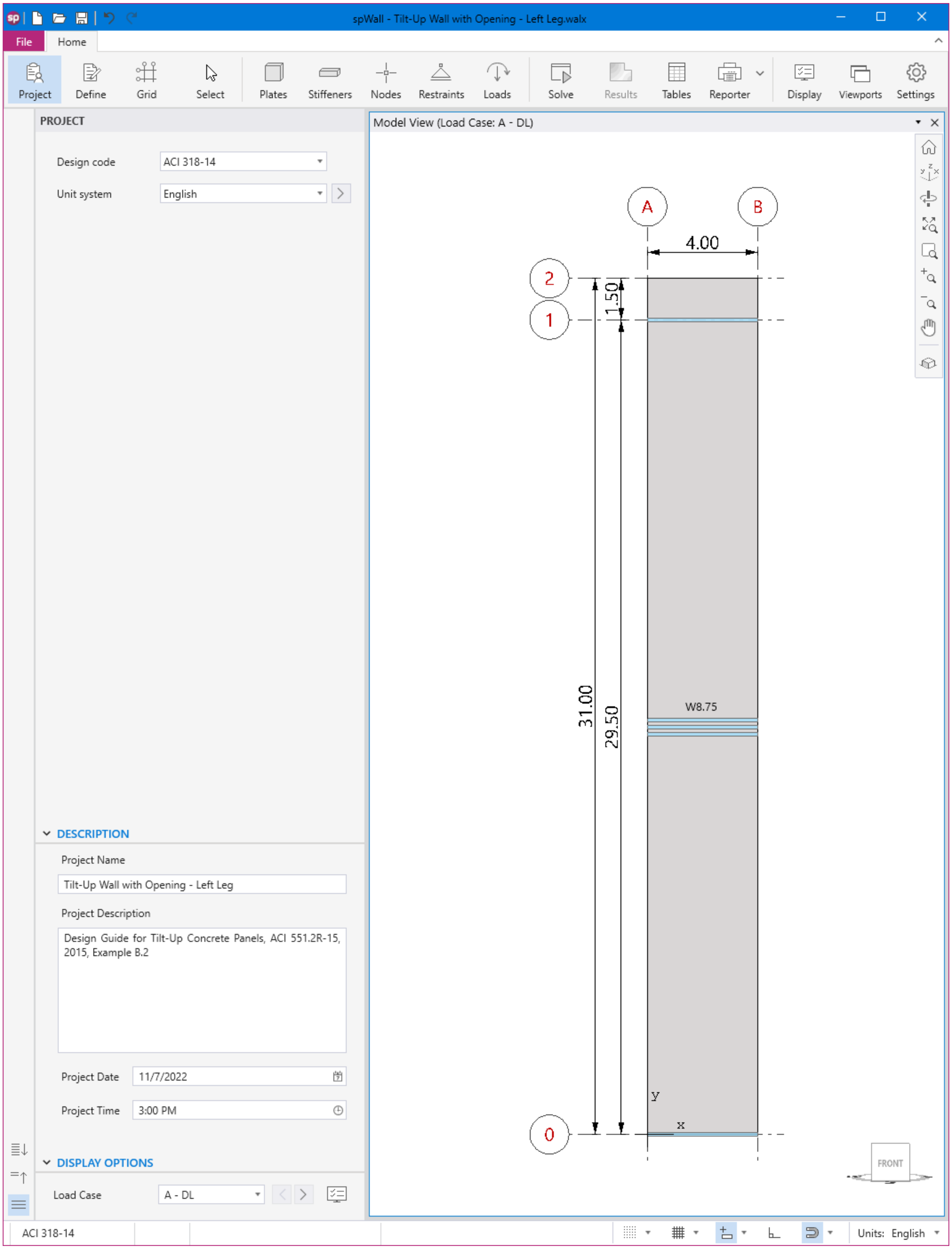


Figure 3 – spWall Interface

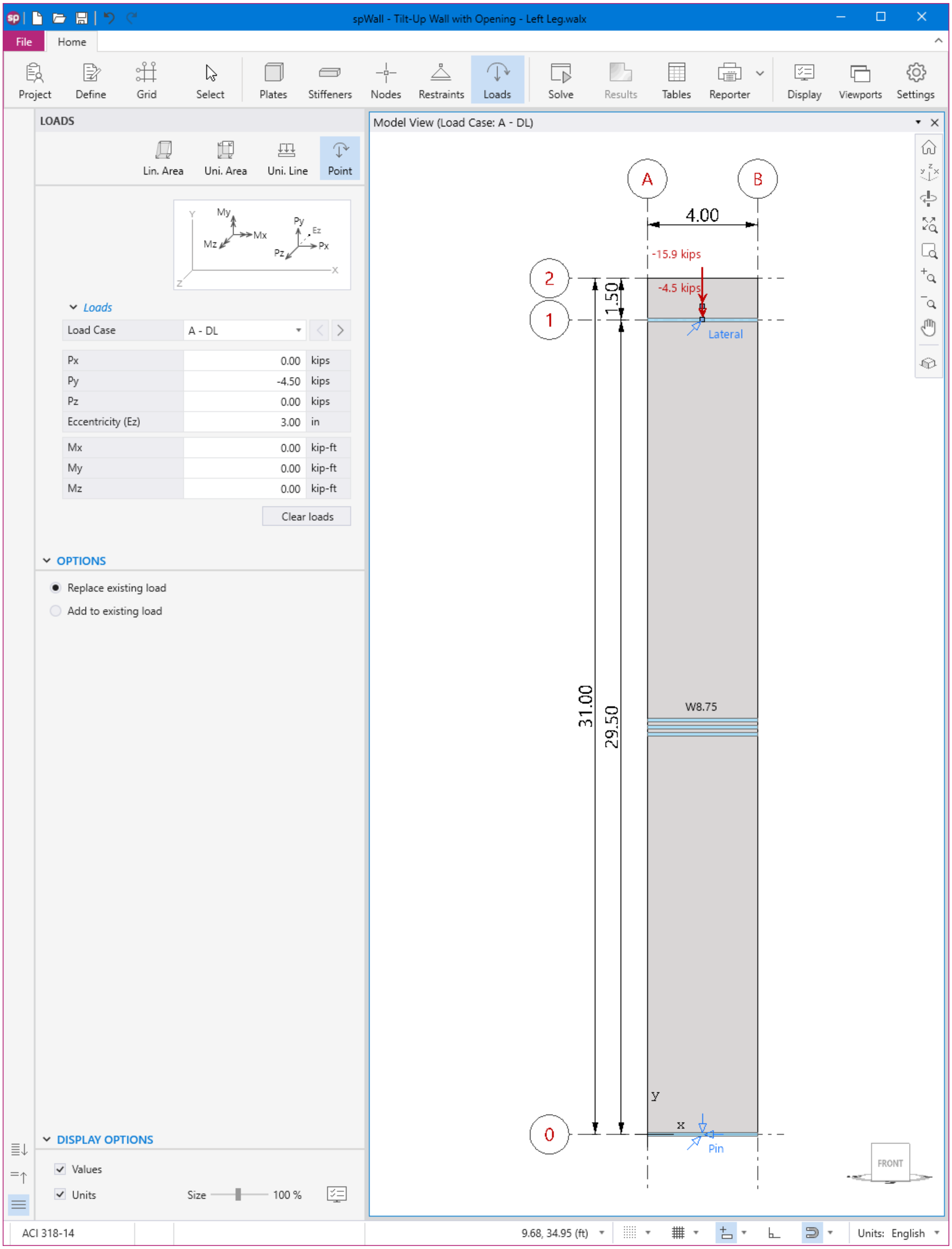


Figure 4 – Assigning Roof Dead Loads for Tilt-Up Wall (spWall)

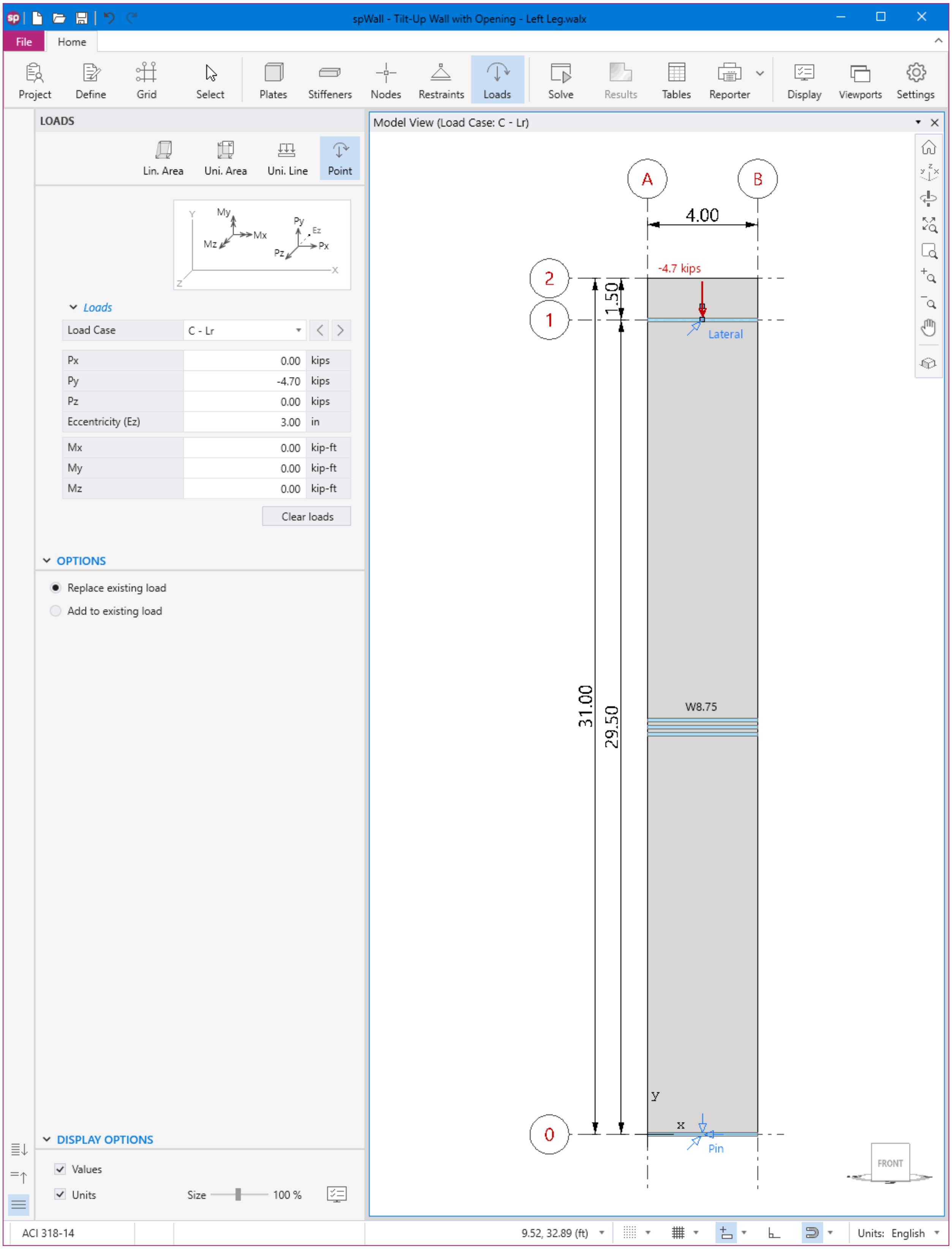


Figure 5 – Assigning Roof Live Loads for Tilt-Up Wall (spWall)

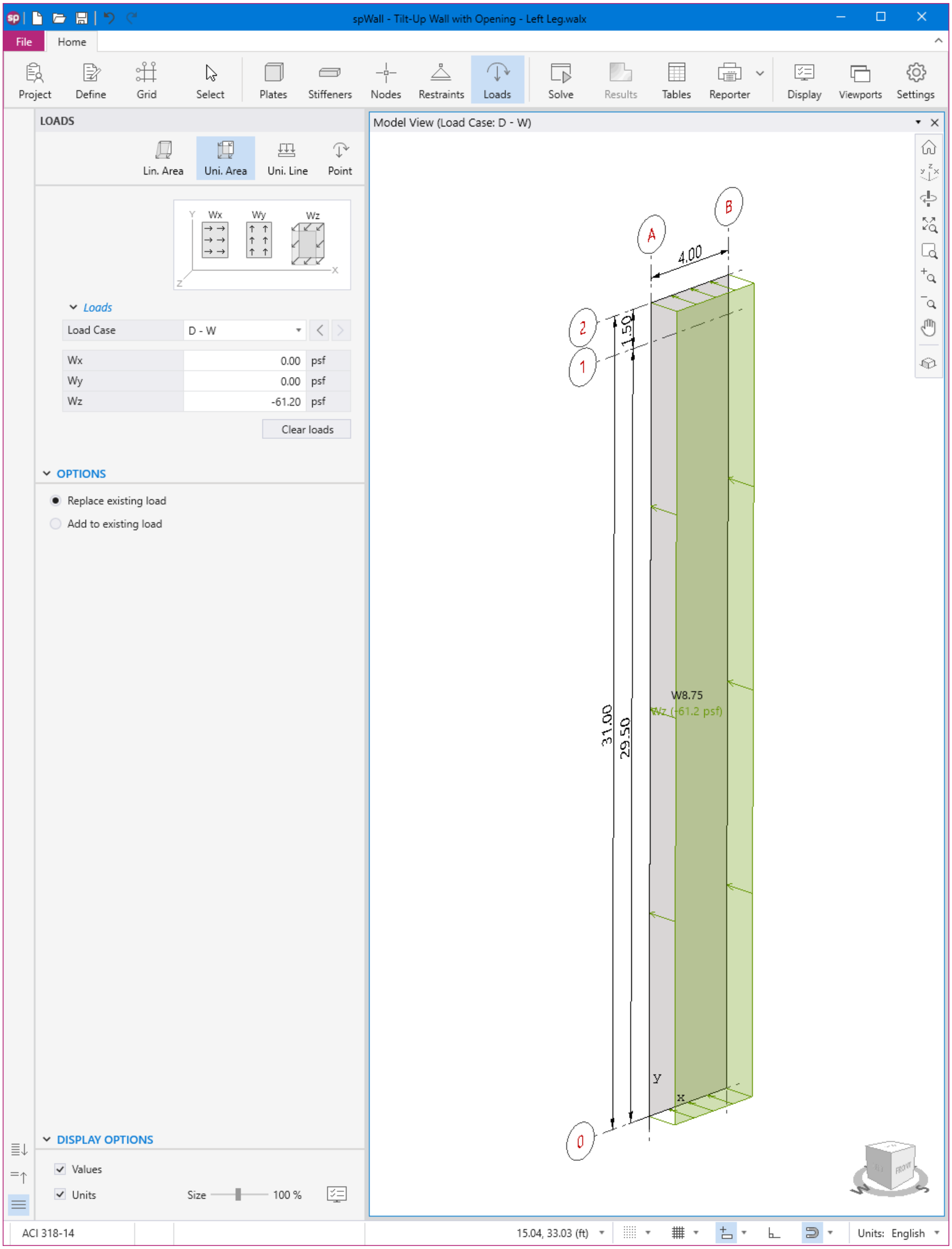


Figure 6 – Assigning Wind Loads for Tilt-Up Wall (spWall)

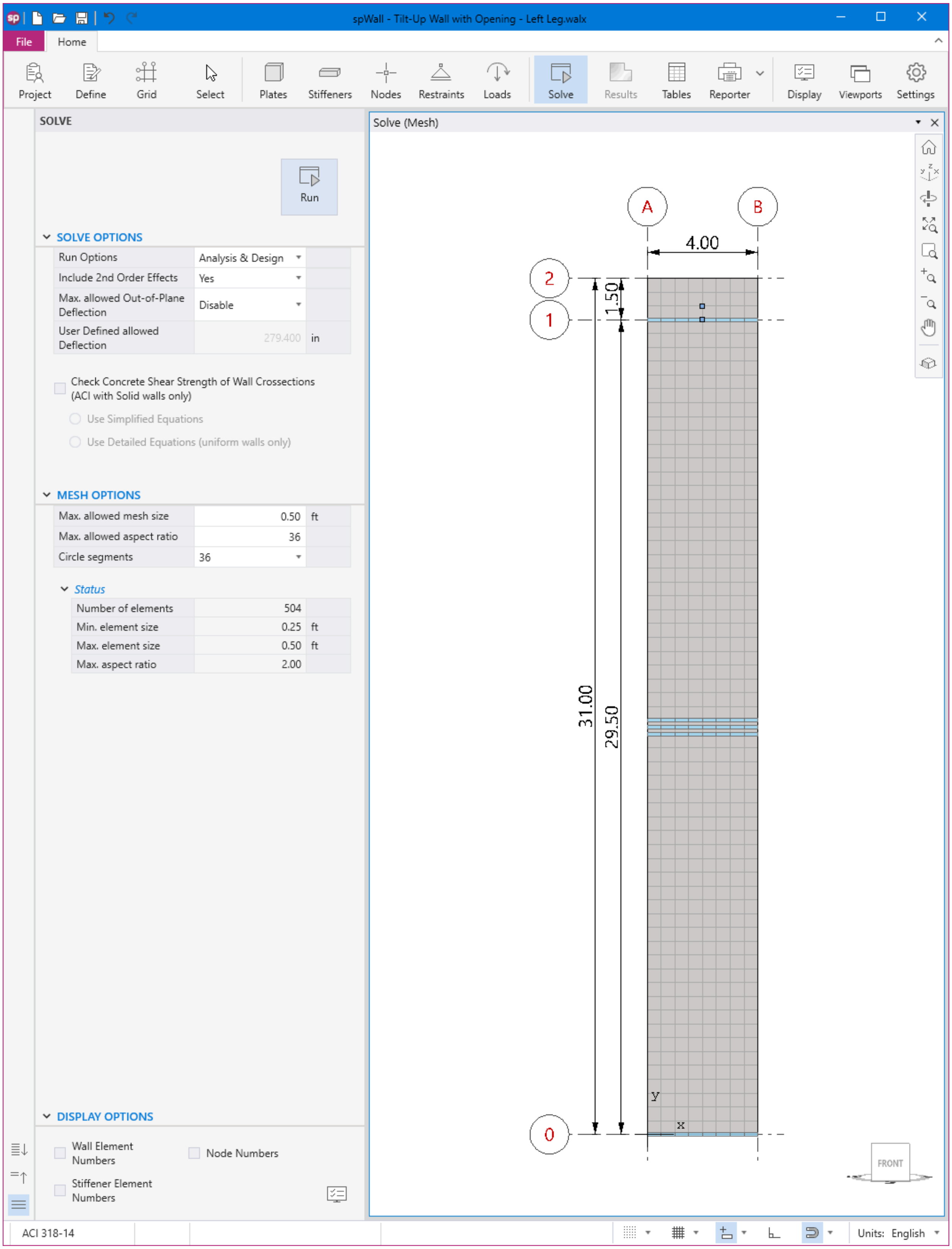


Figure 7 – Solve and Mesh Options (spWall)

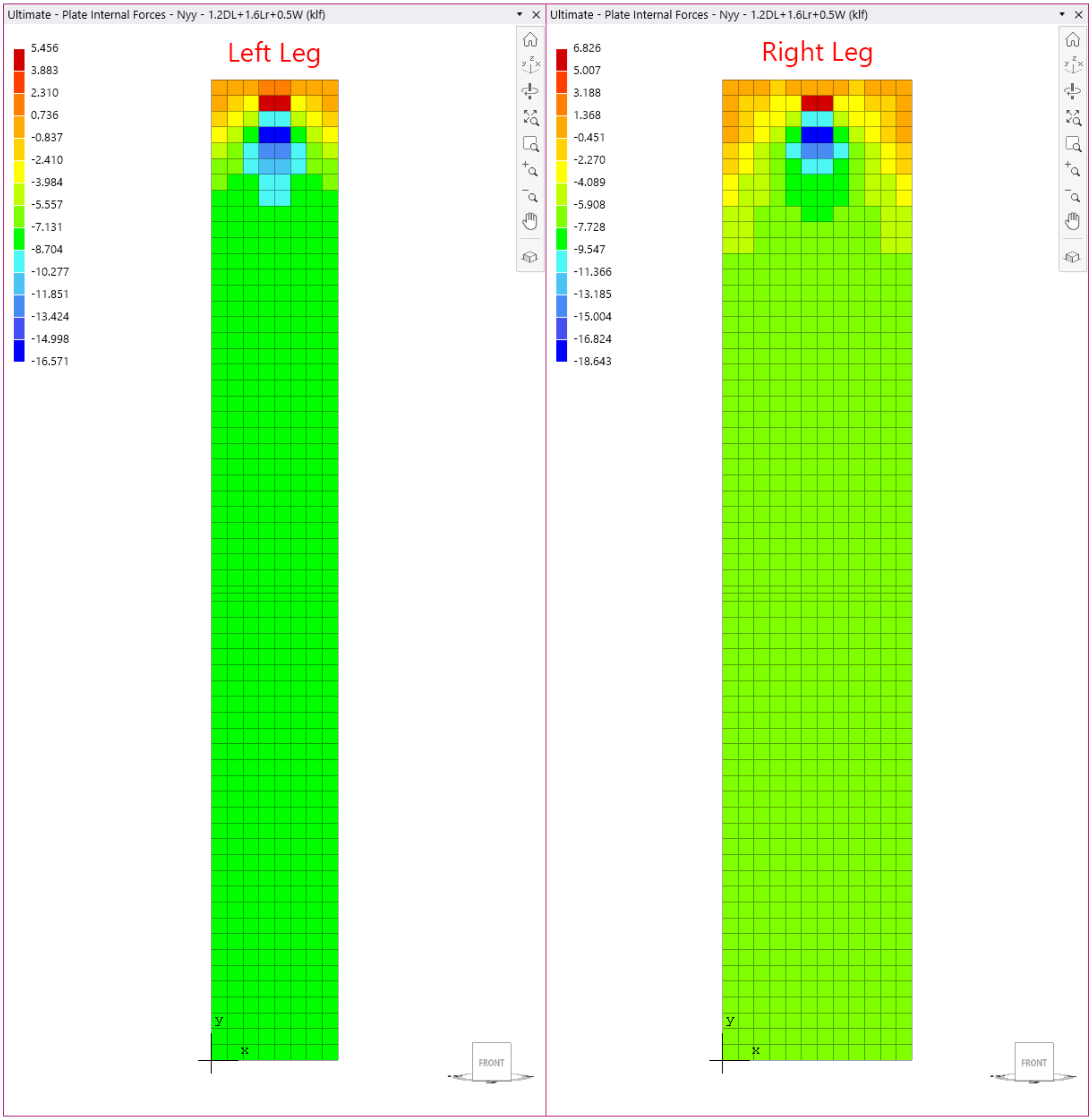


Figure 8 – Factored Axial Forces Contour Normal to Tilt-Up Wall Panel Design Strips Cross-Sections (spWall)

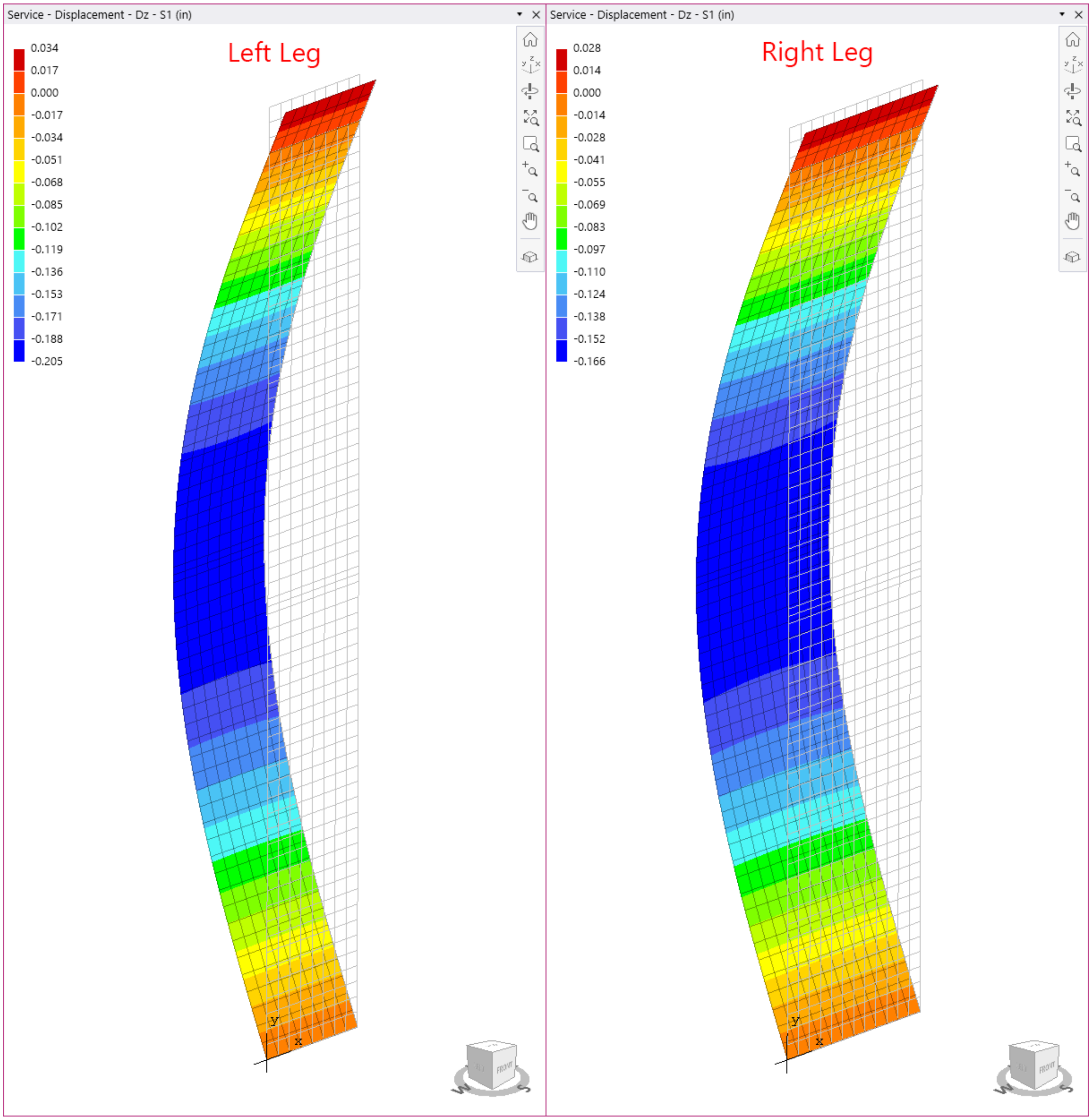


Figure 9 – Tilt-Up Wall Panel Service Lateral Displacement Contour (Out-of-Plane) (spWall)

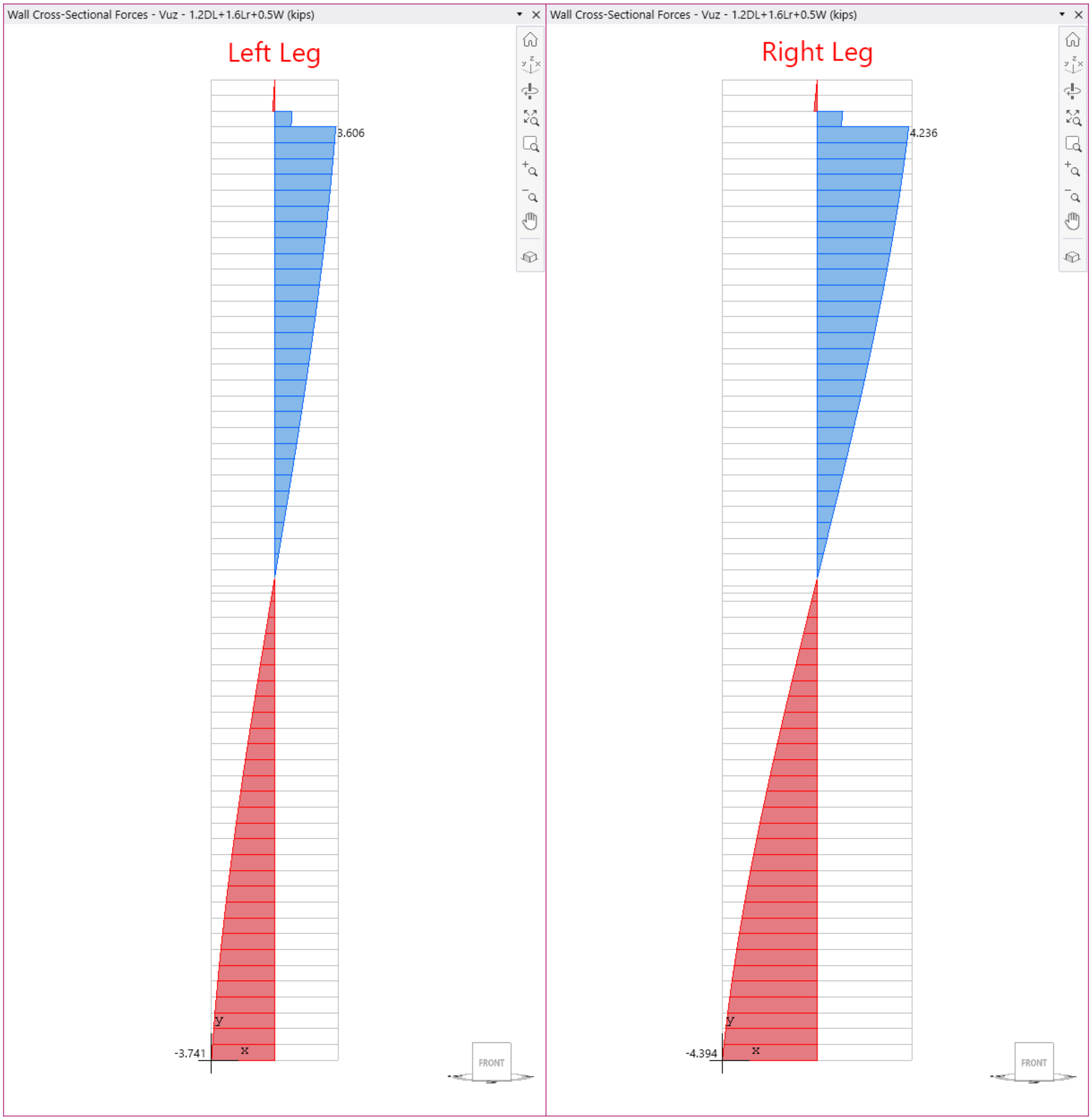


Figure 10 – Out-of-plane Shear Diagram (spWall)

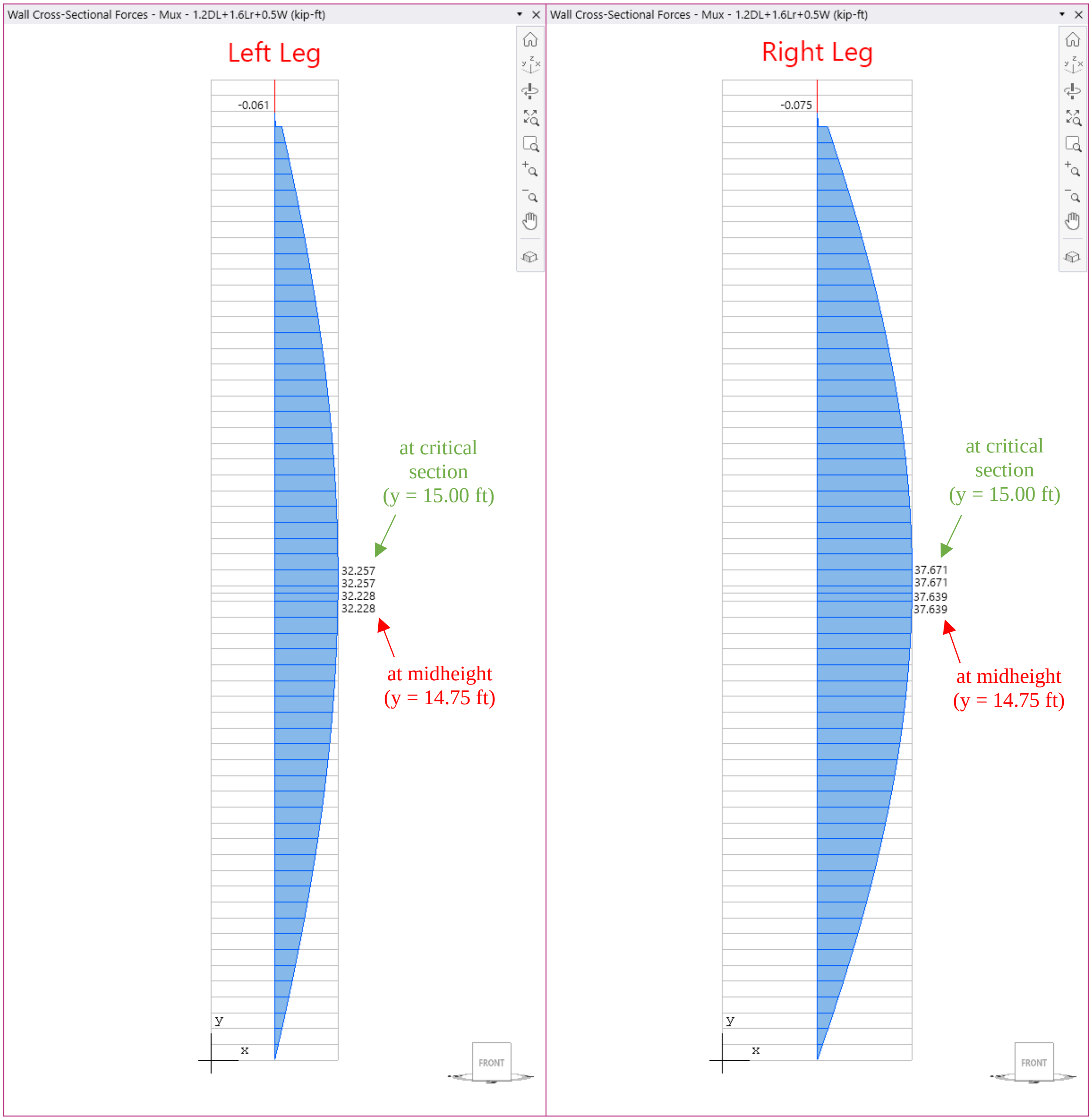


Figure 11 – Tilt-Up Wall Panel with Opening Moment Diagram (spWall)

1.1. Service

1.1.1. Nodal Displacements

1.1.1.1. S1

Coordinate System: Global

Left Leg

Node	Dx in	Dy in	Dz in
271	0.000	-0.002	-0.205
272	0.000	-0.002	-0.204
273	0.000	-0.002	-0.204
274	0.000	-0.002	-0.204
275	0.000	-0.002	-0.204
276	0.000	-0.002	-0.204
277	0.000	-0.002	-0.204
278	0.000	-0.002	-0.204
279	0.000	-0.002	-0.205
280	0.000	-0.002	-0.205
281	0.000	-0.002	-0.204
282	0.000	-0.002	-0.204
283	0.000	-0.002	-0.204
284	0.000	-0.002	-0.204
285	0.000	-0.002	-0.204
286	0.000	-0.002	-0.204
287	0.000	-0.002	-0.204
288	0.000	-0.002	-0.205

at midheight (y = 14.75 ft)

at critical section (y = 15.00 ft)

1.1. Service

1.1.1. Nodal Displacements

1.1.1.1. S1

Coordinate System: Global

Right Leg

Node	Dx in	Dy in	Dz in
391	0.000	-0.002	-0.166
392	0.000	-0.002	-0.165
393	0.000	-0.002	-0.165
394	0.000	-0.002	-0.164
395	0.000	-0.002	-0.164
396	0.000	-0.002	-0.164
397	0.000	-0.002	-0.164
398	0.000	-0.002	-0.164
399	0.000	-0.002	-0.164
400	0.000	-0.002	-0.164
401	0.000	-0.002	-0.165
402	0.000	-0.002	-0.165
403	0.000	-0.002	-0.166
404	0.000	-0.002	-0.166
405	0.000	-0.002	-0.165
406	0.000	-0.002	-0.165
407	0.000	-0.002	-0.164
408	0.000	-0.002	-0.164
409	0.000	-0.002	-0.164
410	0.000	-0.002	-0.164
411	0.000	-0.002	-0.164
412	0.000	-0.002	-0.164
413	0.000	-0.002	-0.164
414	0.000	-0.002	-0.165
415	0.000	-0.002	-0.165
416	0.000	-0.002	-0.166

Figure 12 – Tilt-Up Wall Panel with Opening Displacement at Critical Sections (Service Combinations) (spWall)

1.2. Ultimate

1.2.1. Nodal Displacements

1.2.1.1. 1.2DL+1.6Lr+0.5W

Coordinate System: Global

Left Leg

Node	Dx in	Dy in	Dz in
271	0.000	-0.004	-6.423
272	0.000	-0.004	-6.411
273	0.000	-0.004	-6.402
274	0.000	-0.004	-6.397
275	0.000	-0.004	-6.395
276	0.000	-0.004	-6.397
277	0.000	-0.004	-6.402
278	0.000	-0.004	-6.411
279	0.000	-0.004	-6.423
280	0.000	-0.004	-6.424
281	0.000	-0.004	-6.411
282	0.000	-0.004	-6.403
283	0.000	-0.004	-6.398
284	0.000	-0.004	-6.396
285	0.000	-0.004	-6.398
286	0.000	-0.004	-6.403
287	0.000	-0.004	-6.411
288	0.000	-0.004	-6.424

at midheight (y = 14.75 ft)

at critical section (y = 15.00 ft)

1.2. Ultimate

1.2.1. Nodal Displacements

1.2.1.1. 1.2DL+1.6Lr+0.5W

Coordinate System: Global

Right Leg

Node	Dx in	Dy in	Dz in
391	0.000	-0.003	-6.149
392	0.000	-0.003	-6.131
393	0.000	-0.003	-6.117
394	0.000	-0.003	-6.106
395	0.000	-0.003	-6.098
396	0.000	-0.003	-6.093
397	0.000	-0.003	-6.091
398	0.000	-0.003	-6.093
399	0.000	-0.003	-6.098
400	0.000	-0.003	-6.106
401	0.000	-0.003	-6.117
402	0.000	-0.003	-6.131
403	0.000	-0.003	-6.149
404	0.000	-0.003	-6.149
405	0.000	-0.003	-6.132
406	0.000	-0.003	-6.117
407	0.000	-0.003	-6.106
408	0.000	-0.003	-6.098
409	0.000	-0.003	-6.094
410	0.000	-0.003	-6.092
411	0.000	-0.003	-6.094
412	0.000	-0.003	-6.098
413	0.000	-0.003	-6.106
414	0.000	-0.003	-6.117
415	0.000	-0.003	-6.132
416	0.000	-0.003	-6.149

Figure 13 – Tilt-Up Wall Panel with Opening Displacement at Critical Sections (Ultimate Combinations) (spWall)

1.2.2. Wall Cross-Sectional Forces

1.2.2.1. 1.2DL+1.6Lr+0.5W

Coordinate System: Global
(+) Horizontal cross-section above Y-coordinate
(-) Horizontal cross-section below Y-coordinate

Left Leg

at midheight (y = 14.75 ft)

at critical section (y = 15.00 ft)

	Wall Crossection		In-Plane Forces			Out-Of-Plane Forces		
No.	Y coordinate	X-Centroid	Vux	Nuy	Muz	Vuz	Mux	Muy
	ft	ft	kips	kips	kip-ft	kips	kip-ft	kip-ft
31-	14.75	2.00	0.00	-32.00	0.00	-0.16	32.23	0.00
31+	14.75	2.00	0.00	-32.00	0.00	-0.16	32.23	0.00
32-	15.00	2.00	0.00	-32.00	0.00	-0.08	32.26	0.00
32+	15.00	2.00	0.00	-32.00	0.00	-0.08	32.26	0.00

Right Leg

	Wall Crossection		In-Plane Forces			Out-Of-Plane Forces		
No.	Y coordinate	X-Centroid	Vux	Nuy	Muz	Vuz	Mux	Muy
	ft	ft	kips	kips	kip-ft	kips	kip-ft	kip-ft
31-	14.75	3.00	0.00	-37.88	0.00	-0.17	37.64	0.00
31+	14.75	3.00	0.00	-37.88	0.00	-0.17	37.64	0.00
32-	15.00	3.00	0.00	-37.88	0.00	-0.08	37.67	0.00
32+	15.00	3.00	0.00	-37.88	0.00	-0.08	37.67	0.00

Figure 14 – Tilt-Up Wall Panel with Opening Cross-Sectional Forces (spWall)

12. Design Results Comparison and Conclusions

The model shown above was created in spWall taking into account the ACI 318-14 provisions (Alternative Method for Out-of-Plane Slender Wall Analysis) and ACI 551 recommendations regarding the analysis and design of tilt-up wall panels with openings. In this model the left and right design strips are modeled such that the entire lateral and axial load, including self-weight above the critical section, are distributed to the two strips at each side of the opening. The tributary width for loads was taken as the width of the strip plus one-half the width of the opening. The following table shows the comparison between the hand calculation results and spWall model results.

Table 1 – Comparison of Tilt-Up Wall Panel with Opening Analysis and Design Results

Solution	M_u (kip-ft)		N_u (kip)		$D_{z,service}$ (in.)		$D_{z,ultimate}$ (in.)	
Design Strip	Left	Right	Left	Right	Left	Right	Left	Right
Hand (at midheight)	31.68	37.38	31.87	37.97	0.203	0.164	6.311	6.091
spWall (at midheight)*	32.23	37.64	32.00	37.88	0.204	0.164	6.395	6.091
spWall (at critical section)**	32.26	37.67	32.00	37.88	0.204	0.164	6.396	6.092

* Values are taken at midheight (y = 14.75 ft) for comparison purposes with hand calculations.
** Values are taken at critical section (y = 15.00 ft) with maximum moment value.

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

12.1. Comparison of Wall Modeling Methods

ACI 318 provides the Alternative Method for Out-of-Plane Slender Wall Analysis as a simple and accurate option for analysis and design of simple walls meeting the method conditions. Other methods such as finite element analysis can be used to address panels not meeting the numerous limitations of the Alternative Method for Out-of-Plane Slender Wall Analysis (cantilevered walls, variable thickness and width, walls with openings, non-standard boundary conditions, walls with high compressive loads, in-plane lateral loads, non-standard concentrated load position from attachments of piping, racking etc., concentrated out of plane loads).

The exact wall geometry and applied loads were modeled using spWall engineering software to investigate the differences between the simplified approximate method and the finite element method. For illustration and comparison purposes, the following figures provide a sample of the results obtained from an spWall model created for the reinforced concrete wall in this example using exact wall geometry and applied loads.

It is very important to consider the wind load applied to the door opening and how it must be considered and applied in the model based on the door boundary condition. In this example, the door support reactions are assumed along the left and right side of the door opening. Load is modeled as an equivalent uniform line load applied along the right edge of the left leg and the left side of the right leg. The magnitude of this load is calculated as follows:

$$W_{door} = 27.2 \times \frac{10}{2} \times \frac{1}{1000} = 0.136 \text{ kip/ft}$$

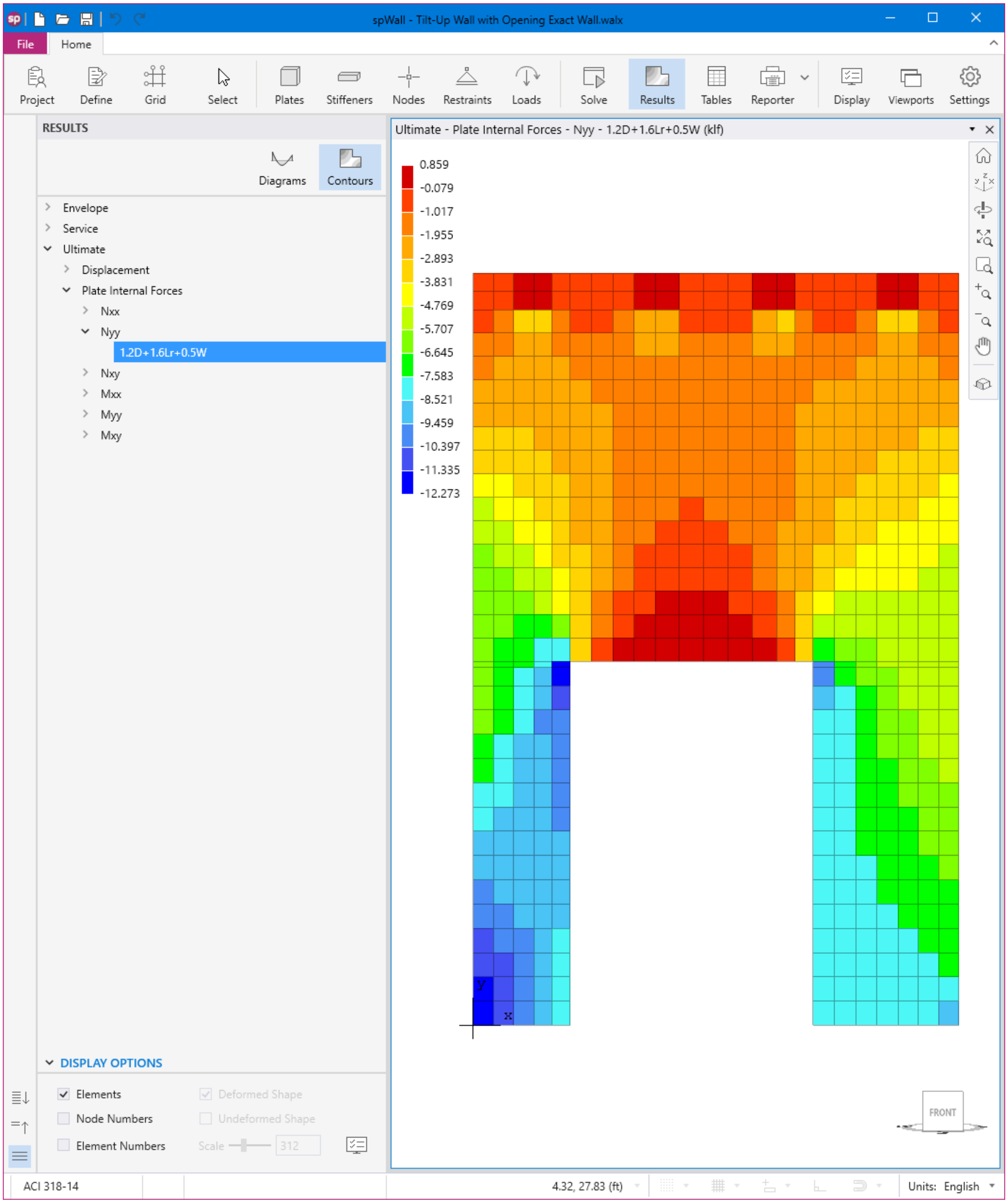


Figure 15 – Factored Axial Forces Contour - Exact Geometry and Loads (spWall)

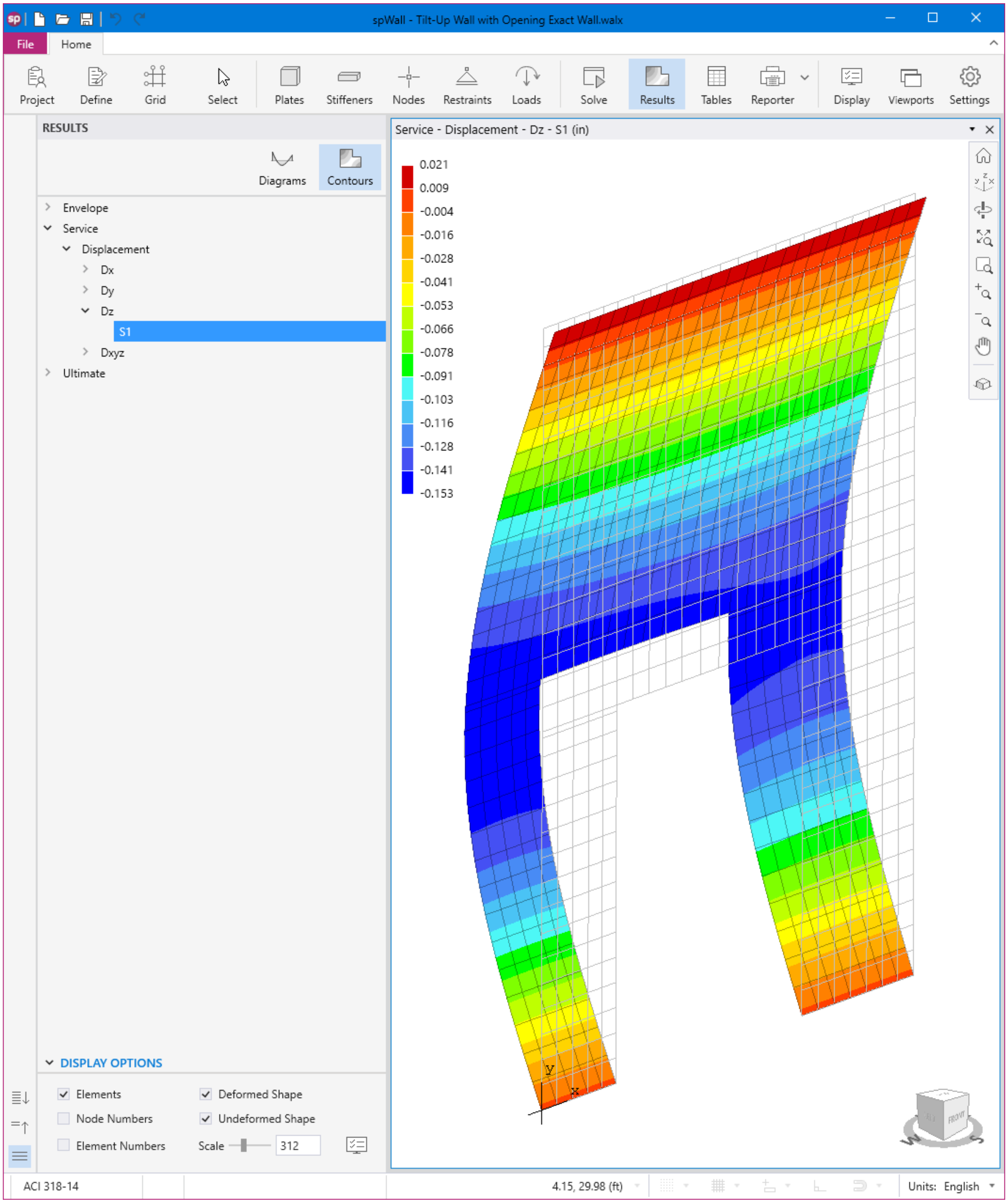


Figure 16 – Tilt-Up Wall Panel Service Lateral Displacement Contour (Out-of-Plane) - Exact Geometry and Loads (spWall)

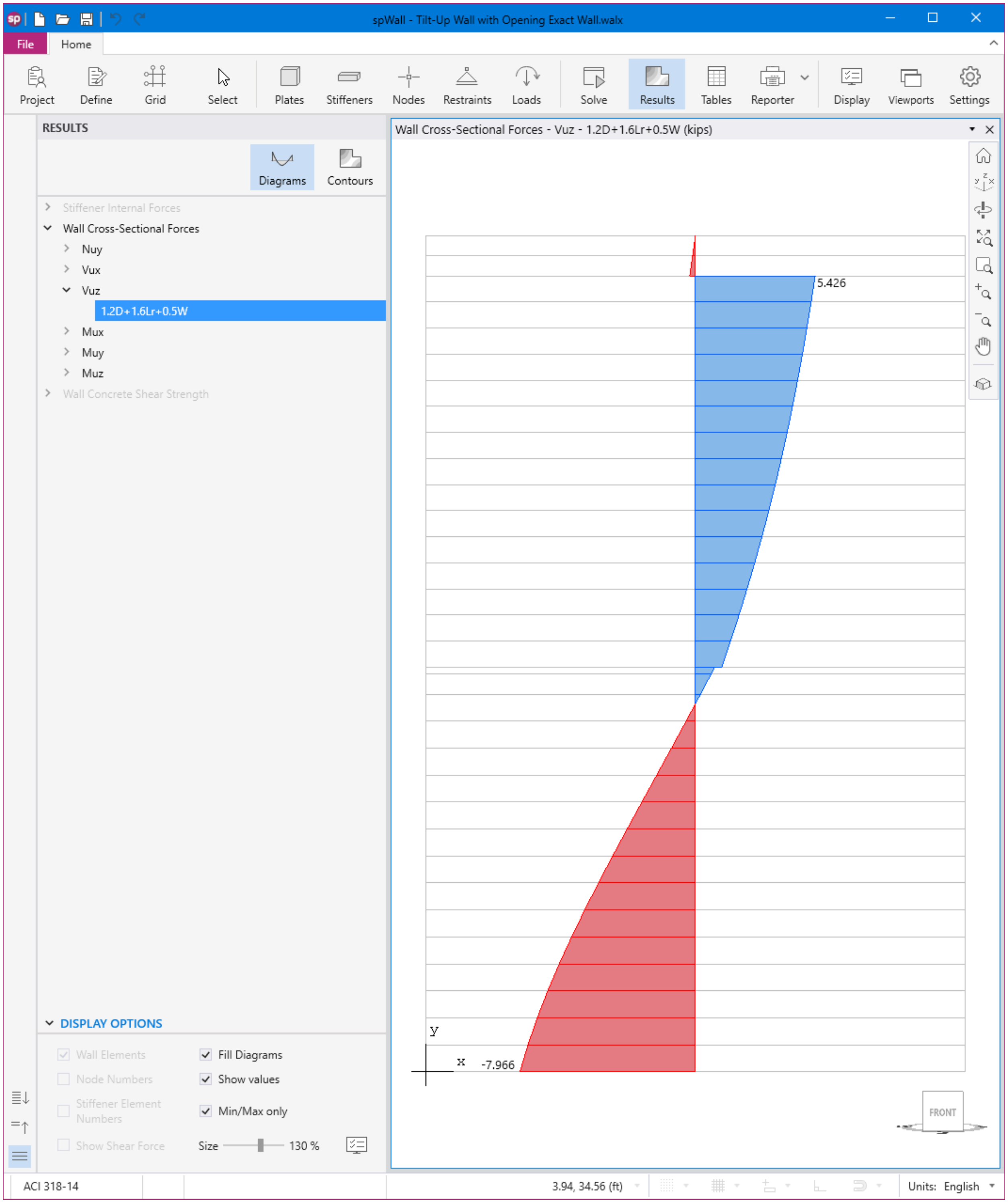


Figure 17 – Out-of-plane Shear Diagram - Exact Geometry and Loads (spWall)

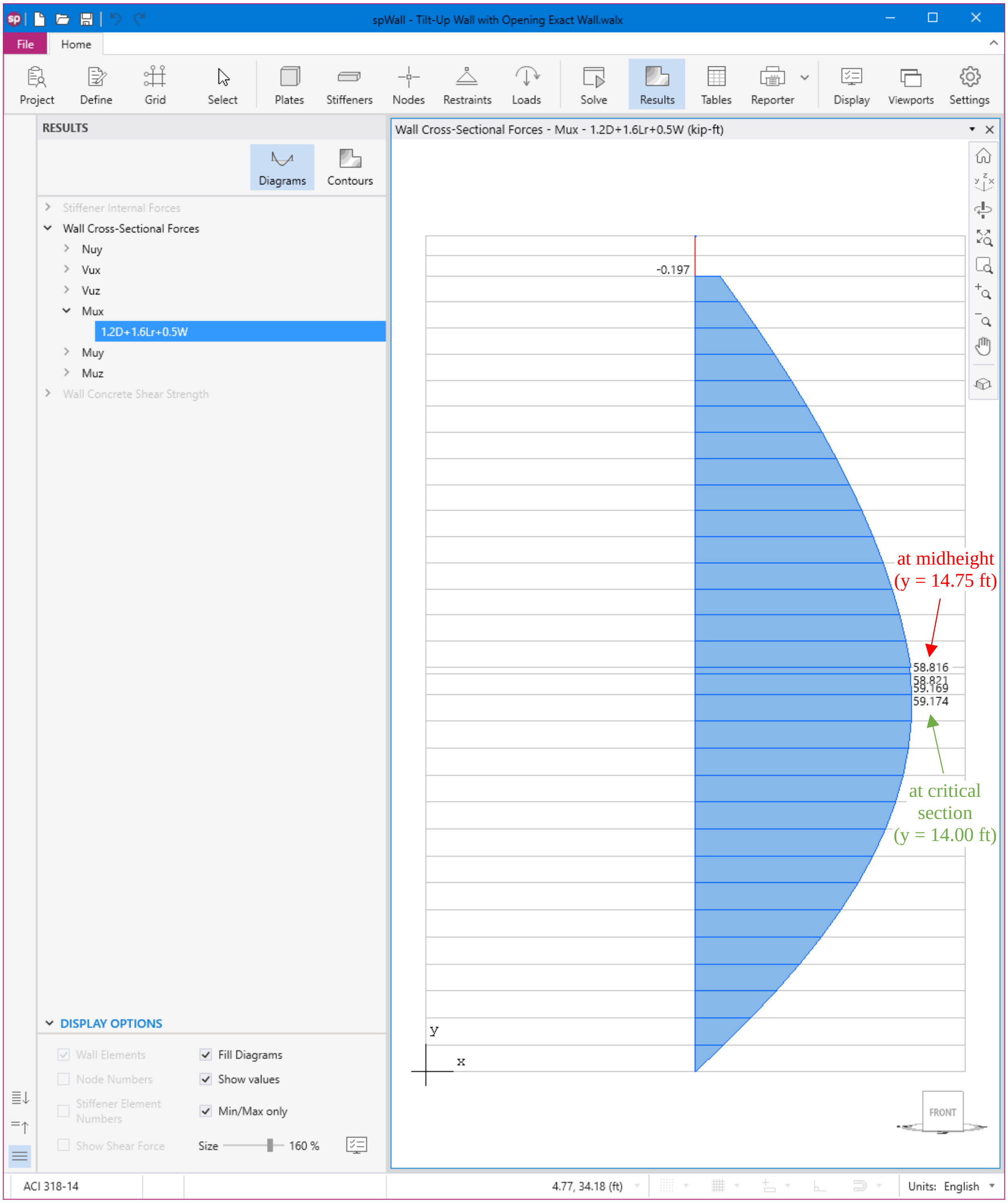


Figure 18 – Tilt-Up Wall Panel with Opening Moment Diagram – Exact Geometry and Loads (spWall)

1.1. Service

1.1.1. Nodal Displacements

1.1.1.1. S1

Coordinate System: Global

	Node	Dx	Dy	Dz	
		in	in	in	
	197	-0.001	-0.003	-0.153	
	198	-0.001	-0.003	-0.153	
at critical section (y = 14.00 ft)	199	-0.001	-0.003	-0.152	Left Leg
	200	-0.001	-0.003	-0.152	
	201	-0.001	-0.003	-0.153	
	202	-0.001	-0.003	-0.153	
	203	0.000	-0.002	-0.147	
	204	0.000	-0.002	-0.146	
	205	0.000	-0.002	-0.145	
at critical section (y = 14.00 ft)	206	0.000	-0.002	-0.144	Right Leg
	207	0.000	-0.002	-0.144	
	208	0.000	-0.002	-0.144	
	209	0.000	-0.002	-0.144	
	210	0.000	-0.002	-0.144	
	211	-0.001	-0.003	-0.152	
	212	-0.001	-0.003	-0.152	
at midheight (y = 14.75 ft)	213	-0.001	-0.003	-0.151	Left Leg
	214	-0.001	-0.003	-0.151	
	215	-0.001	-0.003	-0.151	
	216	-0.001	-0.003	-0.152	
	217	0.000	-0.003	-0.146	
	218	0.000	-0.002	-0.145	
	219	0.000	-0.002	-0.144	
at midheight (y = 14.75 ft)	220	0.000	-0.002	-0.143	Right Leg
	221	0.000	-0.002	-0.143	
	222	0.000	-0.002	-0.143	
	223	0.000	-0.002	-0.143	
	224	0.000	-0.002	-0.144	

Figure 19 – Displacement at Critical Sections – Exact Geometry and Loads (Service Combinations) (spWall)

1.2. Ultimate

1.2.1. Nodal Displacements

1.2.1.1. 1.2D+1.6Lr+0.5W

Coordinate System: Global

	Node	Dx in	Dy in	Dz in	
	197	-0.002	-0.004	-5.045	
	198	-0.002	-0.004	-5.018	
at critical section (y = 14.00 ft)	199	-0.002	-0.004	-5.001	Left Leg
	200	-0.002	-0.004	-4.994	
	201	-0.002	-0.004	-4.997	
	202	-0.002	-0.004	-5.009	
	203	0.000	-0.004	-4.755	
	204	0.000	-0.004	-4.712	
	205	0.000	-0.003	-4.680	
at critical section (y = 14.00 ft)	206	0.000	-0.003	-4.659	Right Leg
	207	0.000	-0.003	-4.650	
	208	0.000	-0.003	-4.649	
	209	0.000	-0.003	-4.657	
	210	0.000	-0.003	-4.673	
	211	-0.002	-0.004	-4.997	
	212	-0.002	-0.004	-4.969	
at midheight (y = 14.75 ft)	213	-0.002	-0.004	-4.949	Left Leg
	214	-0.002	-0.004	-4.939	
	215	-0.002	-0.004	-4.937	
	216	-0.002	-0.004	-4.946	
	217	0.000	-0.004	-4.713	
	218	0.000	-0.004	-4.671	
	219	0.000	-0.004	-4.643	
at midheight (y = 14.75 ft)	220	0.000	-0.003	-4.627	Right Leg
	221	0.000	-0.003	-4.620	
	222	0.000	-0.003	-4.621	
	223	0.000	-0.003	-4.631	
	224	0.000	-0.003	-4.648	

Figure 20 – Displacement at Critical Sections – Exact Geometry and Loads (Ultimate Combinations) (spWall)

1.2.2. Wall Cross-Sectional Forces

1.2.2.1. 1.2D+1.6Lr+0.5W

at midheight (y = 14.75 ft)

at critical section (y = 14.00 ft)

Coordinate System: Global
(+) Horizontal cross-section above Y-coordinate
(-) Horizontal cross-section below Y-coordinate

Wall Crossection			In-Plane Forces			Out-Of-Plane Forces		
No.	Y coordinate ft	X-Centroid ft	Vux kips	Nuy kips	Muz kip-ft	Vuz kips	Mux kip-ft	Muy kip-ft
15-	14.00	11.00	0.00	-70.83	69.52	0.23	59.17	1.08
15+	14.00	11.00	0.00	-70.83	69.53	0.23	59.17	1.08
16-	14.75	11.00	0.00	-69.85	69.52	0.71	58.82	1.23
16+	14.75	11.00	0.00	-69.85	69.53	0.71	58.82	1.23

Figure 21 – Tilt-Up Wall Panel with Opening Cross-Sectional Forces – Exact Geometry and Loads (spWall)

Table 2 – Comparison of Analysis Methods (at midheight)										
Solution	M_u (kip-ft)			N_u (kips)			$D_{z,service}$ (in.)		$D_{z,ultimate}$ (in.)	
	Left	Right	Total	Left	Right	Total	Left	Right	Left	Right
Simplified Model Approximate Design Strips (at y = 14.75 ft)	32.23	37.64	69.87	32.00	37.88	69.88	0.204	0.164	6.395	6.091
Complete Model Exact Geometry and Loads (at y = 14.75 ft)	---	---	58.82	---	---	69.85	0.151	0.143	4.949	4.627

Table 3 – Comparison of Analysis Methods (at critical section)										
Solution	M_u (kip-ft)			N_u (kips)			$D_{z,service}$ (in.)		$D_{z,ultimate}$ (in.)	
	Left	Right	Total	Left	Right	Total	Left	Right	Left	Right
Simplified Model Approximate Design Strips (at y = 15.00 ft)	32.26	37.67	69.93	32.00	37.88	69.88	0.204	0.164	6.396	6.092
Complete Model Exact Geometry and Loads (at y = 14.00 ft)	---	---	59.17	---	---	70.83	0.152	0.144	5.001	4.659

Using the complete model with the exact wall geometry and applied loads compared with the simplified model of two equivalent design strips results in:

1. Reduction in the required moment capacity by approximately 16%.
2. Reduction in the out-of-plane displacements, at service and ultimate levels by approximately 19% to 23% respectively.

The complete model, as shown in the following figure, displays a complete view of the torsional moment distribution indicating areas of torsional stress concentration at opening edges. This corresponds to the additional reinforcement requirements outlined in ***ACI 318-14 (11.7.5.1)*** for header and jambs of openings for improved serviceability.

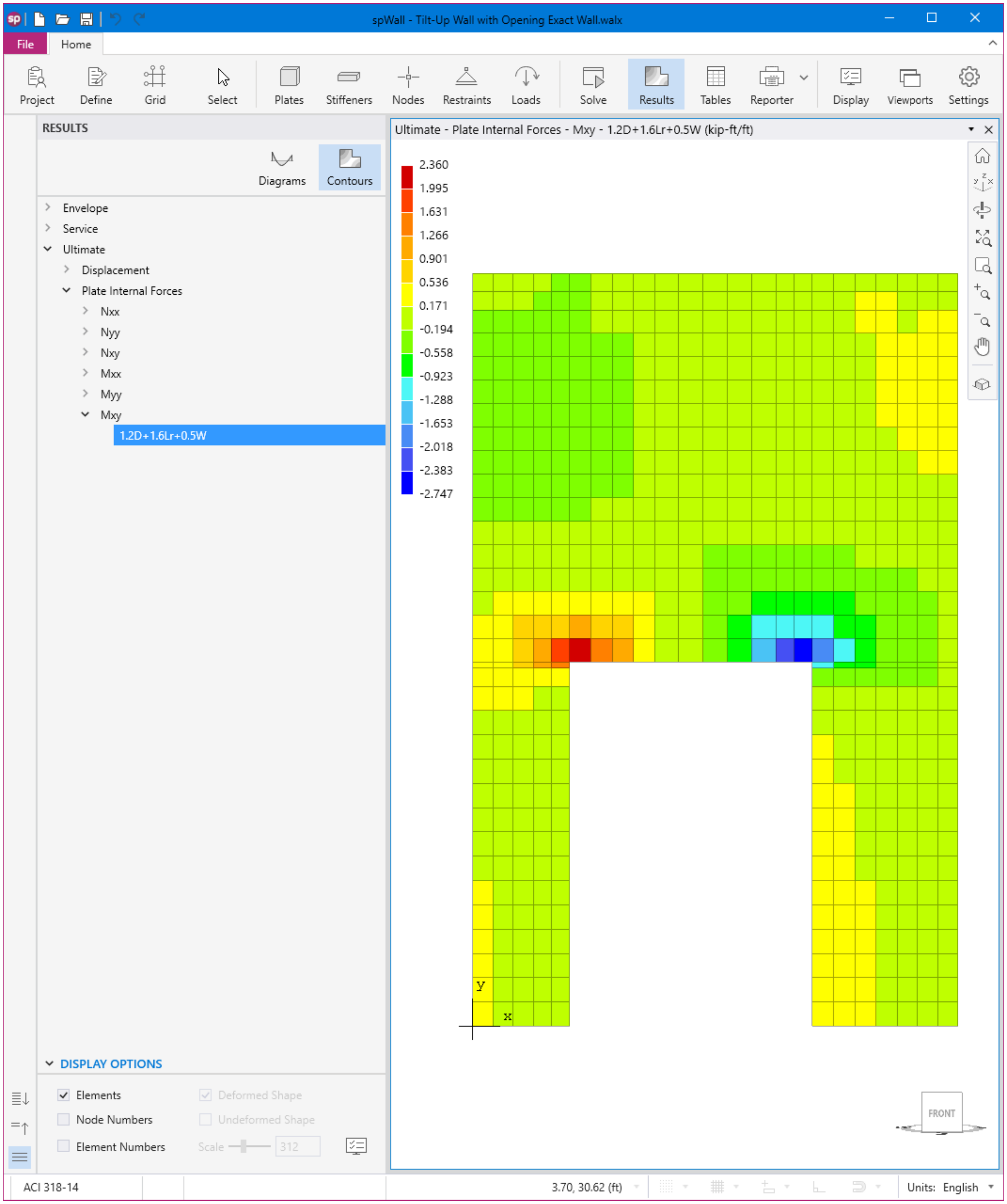


Figure 22 – Tilt-Up Wall Panel with Opening Torsional Moment Contour (spWall)

12.2. Tilt-up Wall Stiffness Reduction

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 reduced 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. A factor 0.75 can be also used to reduce the calculated bending stiffness of the concrete section in accordance with ACI 318-14 Chapter 11. It is intended to account for variations in material properties and workmanship. This reduction factor in bending stiffness should be incorporated by all other alternate design methods to comply with the requirements of ACI 318 as ACI 551 committee stated.

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 the reduction factor (0.75) can be 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 290.85 in.^{4} and 2,679.69 in.^{4} for the left leg (design strip). Thus, the out-of-plane cracking coefficient for ultimate load combinations for the left leg can be found as follows:

$$\alpha = \text{ cracking coefficient} = \frac{0.75\times I_{cr}}{I_g} = \frac{0.75\times 290.85}{2,679.69} = 0.08140$$

For the service load combinations, it was found that M_a for the left leg equals to 12.55 ft-kip which is less than M_{cr} = 24.21 ft-kip. That means the left leg section is uncracked and the cracking coefficient can be taken equal to 1.

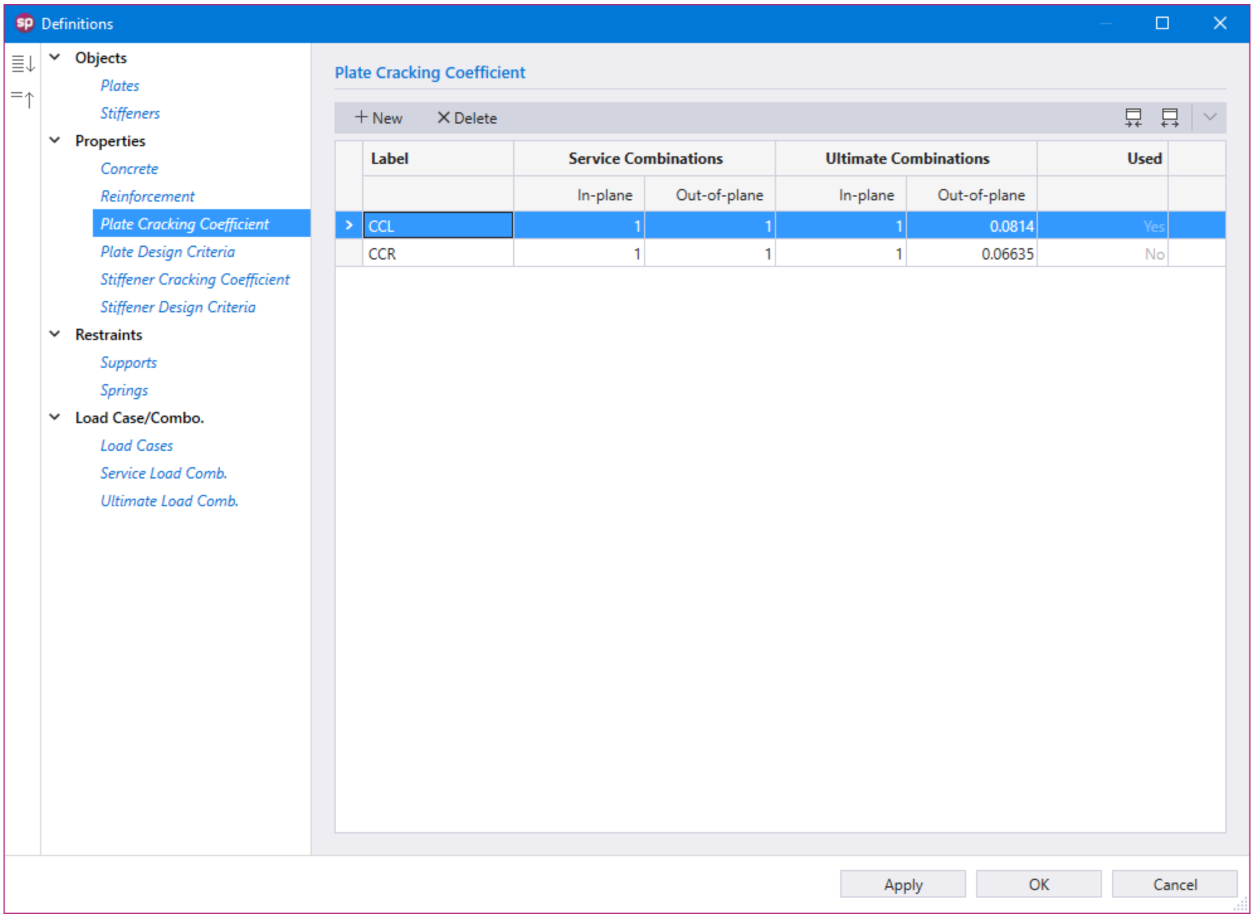


Figure 23 – Defining Cracking Coefficient (spWall)

12.3. Comparison of Load Type Effects

During the process of analyzing the tilt-up wall panels, the effect of load type on the wall behavior at the critical section (mid-height of the unbrace wall length) was investigated in terms of out-of-plane deflection at service and ultimate level, required axial capacity, and required out-of-plane moment capacity.

Table 4 – Effect of Load Type on the Wall Behavior (at midheight)

Solution	M_u (kip-ft)			N_u (kips)			$D_{z,service}$ (in.)		$D_{z,ultimate}$ (in.)	
	Left	Right	Total	Left	Right	Total	Left	Right	Left	Right
Actual Joists Point Loads	---	---	58.82	---	---	69.85	0.151	0.143	4.949	4.627
Equivalent Uniform Line Load	---	---	58.82	---	---	69.85	0.151	0.143	4.953	4.620

Using equivalent uniform line load along the section width to represent the actual joists point loads has only a slight effect on the results obtained at the critical section (mid-height of the unbrace wall length). However, modeling point loads to reflect actual behavior and stress distribution is beneficial in cases where there are openings, variable thicknesses, changes in geometry, intermediate supports, and other variations from a simply supported wall with constant width and thickness.

12.4. Cracking Coefficient and Effective Flexural Stiffness of Concrete Walls

The cracking coefficient for tilt-up wall panels can be calculated using different ACI 318 provisions. The following shows the commonly used provisions to calculate the cracked (or effective) moment of inertia used in the cracking coefficient calculations required for spWall models:

1. 0.35 I_g for cracked walls and 0.70 I_g for uncracked walls **ACI 318-14 (Table 6.6.3.1.1(a))**
2. When treating the wall as compression member:

$$0.35\times I_g \le \left(0.80+25\times\frac{A_{st}}{A_g}\right)\times\left(1-\frac{M_u}{P_u\times h}-0.5\times\frac{P_u}{P_o}\right)\times I_g \le 0.875\times I_g$$ **ACI 318-14 (Table 6.6.3.1.1(b1))**

3. When treating the wall as flexural member:

$$0.25\times I_g \le \left(0.10+25\times\rho\right)\times\left(1.2-0.2\times\frac{b_w}{d}\right)\times I_g \le 0.5\times I_g$$ **ACI 318-14 (Table 6.6.3.1.1(b2))**

4. Using the moment magnification procedure for nonsway frames:

$$\frac{0.2\times E_c\times I_g+E_s\times I_{se}}{\left(1+\beta_{dns}\right)\times E_c}$$ **ACI 318-14 (6.6.4.4.4(b))**

5. Using the moment magnification procedure for nonsway frames:

$$\frac{0.4\times E_c\times I_g}{\left(1+\beta_{dns}\right)\times E_c}$$ **ACI 318-14 (6.6.4.4.4(a))**

6. Using the Alternative Method for Out-of-Plane Slender Wall Analysis:

$$n\times A_{se}\times\left(d-c\right)^2+\frac{l_w\times c^3}{3}$$ **ACI 318-14 (11.8.3.1(c))**

11.8.3.1(c) is used in this example to calculate the cracking coefficient for the wall section modeled in spWall. This is intended to best match the reference approach using the Alternative Method for Out-of-Plane Slender Wall Analysis to analyze and design the tilt-up wall panels.

The variation in the magnitude of the cracking coefficient has a significant effect on the analysis results and specifically the wall moments and displacements. The following table illustrates the effect of using the above equations on the program results.

Table 5 – Comparison of (I_{cr} or I_{eff}) Effect on Results											
Method	I_{cr} or I_{eff}, in.^{4}		Cracking coefficient (α) for spWall		M_u, kip-ft			$D_{z,service}$, in.		$D_{z,ultimate}$, in.	
	Left	Right	Left	Right	Left	Right	Total	Left	Right	Left	Right
Table 6.6.3.1.1(a)	938	1407	0.350	0.350	17.03	20.03	37.06	0.204	0.164	0.80	0.63
Table 6.6.3.1.1(b1)	2345	3517	0.875	0.875	15.66	18.77	34.43	0.204	0.164	0.30	0.24
Table 6.6.3.1.1(b2)	670	1005	0.250	0.250	18.08	20.96	39.04	0.204	0.164	1.19	0.91
6.6.4.4.4b	268	402	0.100	0.100	26.56	27.65	54.21	0.204	0.164	4.31	2.99
6.6.4.4.4a	536	804	0.200	0.200	19.11	21.85	40.96	0.204	0.164	1.56	1.19
11.8.3.1c	291	356	0.109	0.088	25.04	29.69	59.73	0.204	0.164	3.75	3.62
11.8.3.1c with reduction factor (from 11.8.3.1d)	218	267	0.081	0.066	32.23	37.64	69.87	0.204	0.164	6.40	6.09

From the table above the following can be observed:

1. The values above reveal the necessity to carefully select I_{cr} or I_{eff} values (and the corresponding α value) to ensure the wall moment capacity and estimated deflections are calculated with sufficient conservatism ensuring adequate strength and stability.
2. The $D_{z,service}$ values are unaffected by the method used to calculate I_{cr} or I_{eff} since the section is uncracked and the cracking coefficient α is taken as 1.
3. The $D_{z,ultimate}$, values are calculated however are not used in any calculations and the deflection limits are given for $D_{z,service}$ only.
4. The range of the cracking coefficient and the cracked (or effective) moment of inertia values vary widely based on the equation used.
5. In this example the spWall model utilized the value of the cracked moment of inertia using the alternative analysis method equation 11.8.3.1(c) with reduction factor from 11.8.3.1(d).

13. Tilt-Up Wall Reinforcement and Cracking Coefficient Optimization

In the previous models, the cracking coefficients were selected based on the area of steel used by the reference and equation 11.8.3.1c with the reduction factor to best match the reference. The reinforcement selected in the reference is conservative and results in a higher cracking moment of inertia leading to lower values of reinforcement to be obtained by spWall.

To explore this topic in further details, the left leg (design strip) model results will be used. I_{cr} for this model based on 7 #6 bars (A_s = 3.08 in.^{2}) vertical reinforcement was found to be equal to 290.85 in.^{4} which leads to a 0.08140 cracking coefficient (the model outputs are highly dependent on and sensitive to the cracking coefficient and up to 5 significant figures is recommended). Using this value, the required area of steel of 1.180 in.^{2} is less than the provided area of steel used to calculate the cracking coefficient by 61.7%. This is expected since the provided area of steel in reference example is much higher than the required (ϕM_n = 60.13 ft-kip >> M_u = 31.68 ft-kip).

The use of the required area of steel from this model in this case is insufficient because it is based on a high assumed value of the cracking coefficient. To confirm this, a model was reanalyzed using the new required area of steel (1.180 in.^{2}) to calculate the cracking coefficient (0.05272). spWall in this case shows that the model is failing and the following warning will be provided:

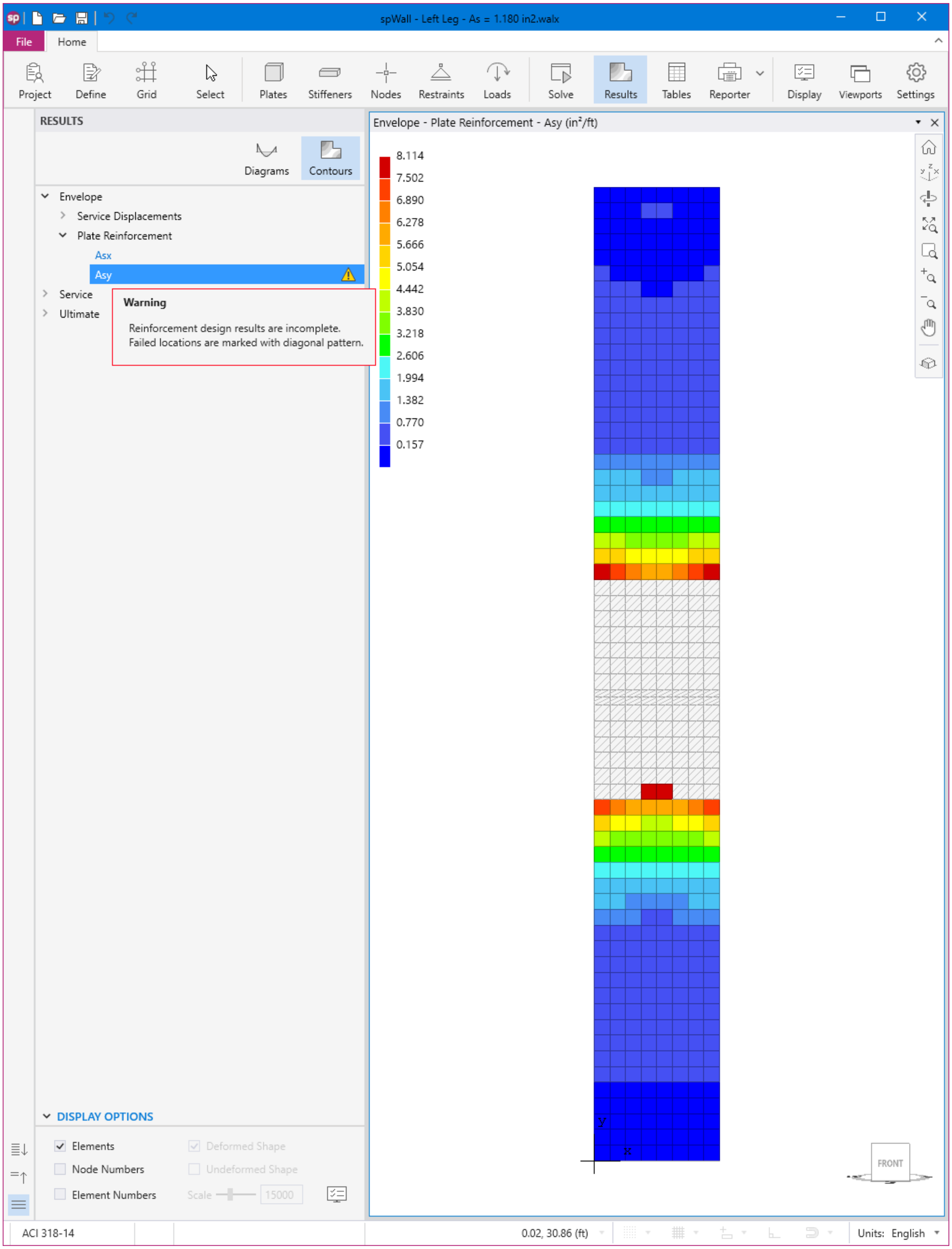


Figure 24 – Failing Reinforcement Error (spWall)

In order to find the optimum required area of steel and the associated cracking coefficient for ultimate combinations using spWall, the following procedure should be followed: ***spWall Manual v10 Chapter 2***

1. Estimate the value of A_s.
2. Calculate A_{se} using the following equation:

$$A_{se} = A_s + \frac{P_{um} \times h}{2 \times f_y \times d}$$ ***ACI 318-14 (R11.8.3.1)***

   Where A_{se} is the effective area of longitudinal reinforcement in a slender wall.

3. Calculate I_{cr} using the following equation:

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

4. Calculate the cracking coefficient using the following equation:

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

   Where the 0.75 is bending stiffness reduction factor of the concrete section in accordance with ACI 318-14 Chapter 11. It is intended to account for variations in material properties and workmanship.

5. Run the first model in spWall using the initial cracking coefficient. After analysis and design, if the computed value of A_s ($A_{s,n+1}$) is greatly differs from the estimated value of A_s ($A_{s,n}$), the analysis should be performed again with new values of A_s and cracking coefficient until $A_{s,n} \approx A_{s,n+1}$.

The following table shows the iteration stages to obtain the optimum area of steel for the left leg (design strip) wall of this example using the procedure described above:

Table 6 - Area of Steel Optimization (Using the Proposed Procedure)				
Iteration #	$A_{s,n}$, in.^{2}	Cracking Coefficient	$A_{s,n+1}$, in.^{2}	Difference, %
1	3.080	0.08140	1.180*	61.7
2	1.220**	0.05355	27.885	-2185.7
3	2.150	0.06939	1.664	22.6
4	1.664	0.06183	2.320	-39.4
5	1.907	0.06578	1.916	-0.5
6	1.912	0.06585	1.912	0.0
* Model wall reinforcement design failed ** The lowest wall reinforcement value that will produce a viable model				

Using this procedure above for the left leg, we started with 3.080 in.^{2}, the value used by the reference. After a few iterations with averaging of two consecutive reinforcement areas, it was found that the solution converged at 1.912 in.^{2} as the optimum reinforcement area. For illustration and comparison purposes, the following figures provide a sample of the results obtained from the spWall model created for the reinforced concrete wall with the optimum area of steel (1.912 in.^{2}).

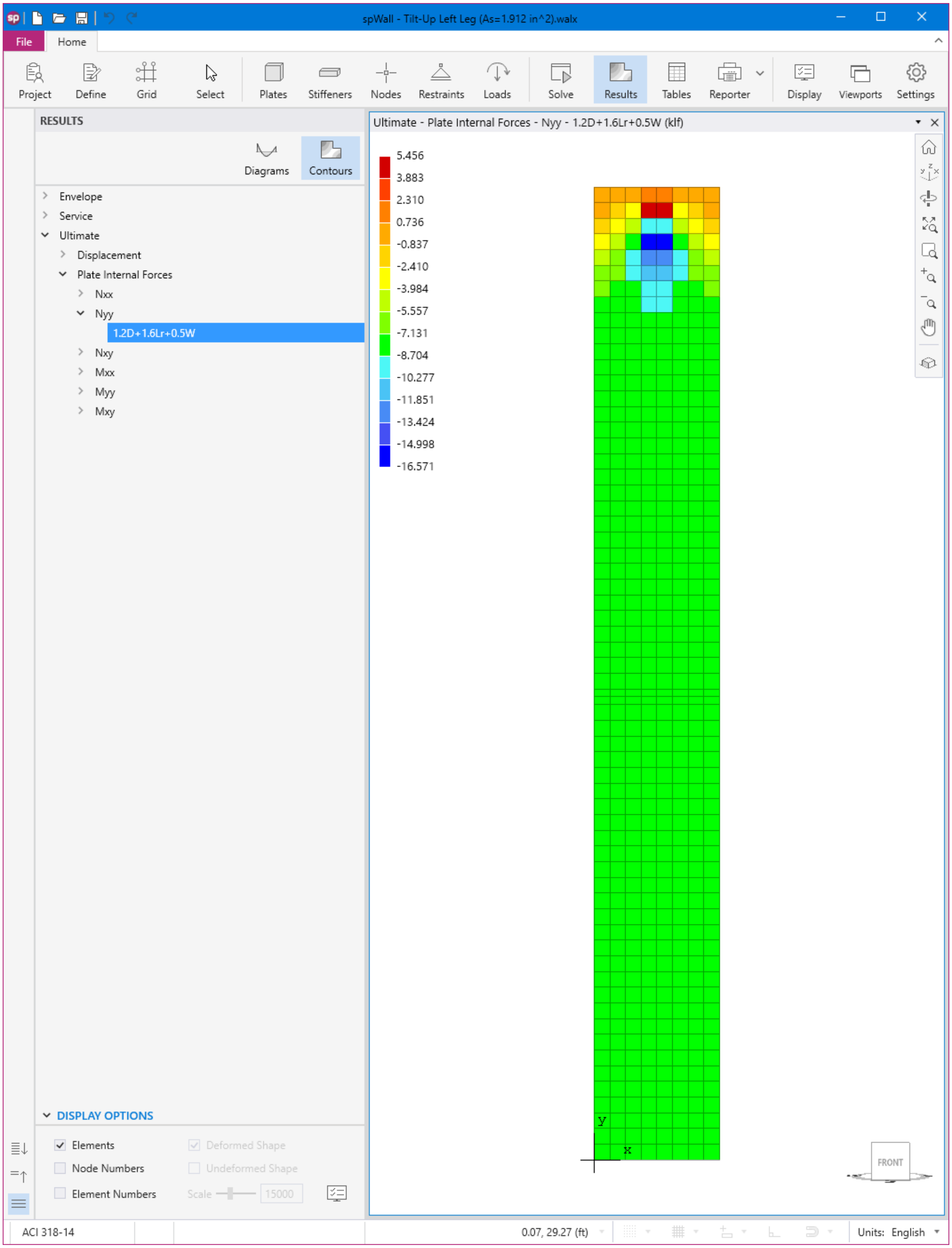


Figure 25 – Factored Axial Forces Contour Normal to the Left Design Strip Cross-Section (spWall)

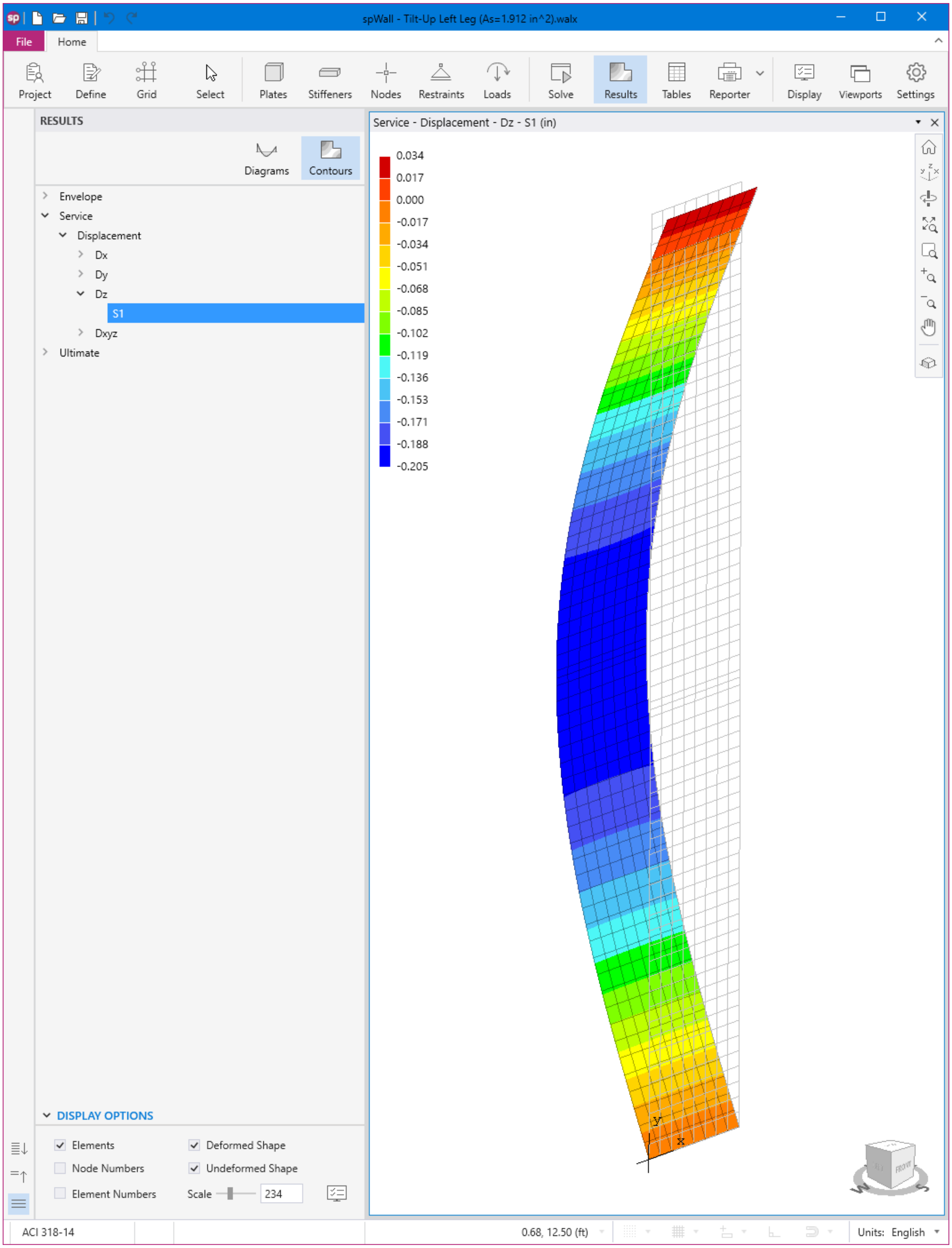


Figure 26 – Service Lateral Displacement Contour (Out-of-Plane) (spWall)

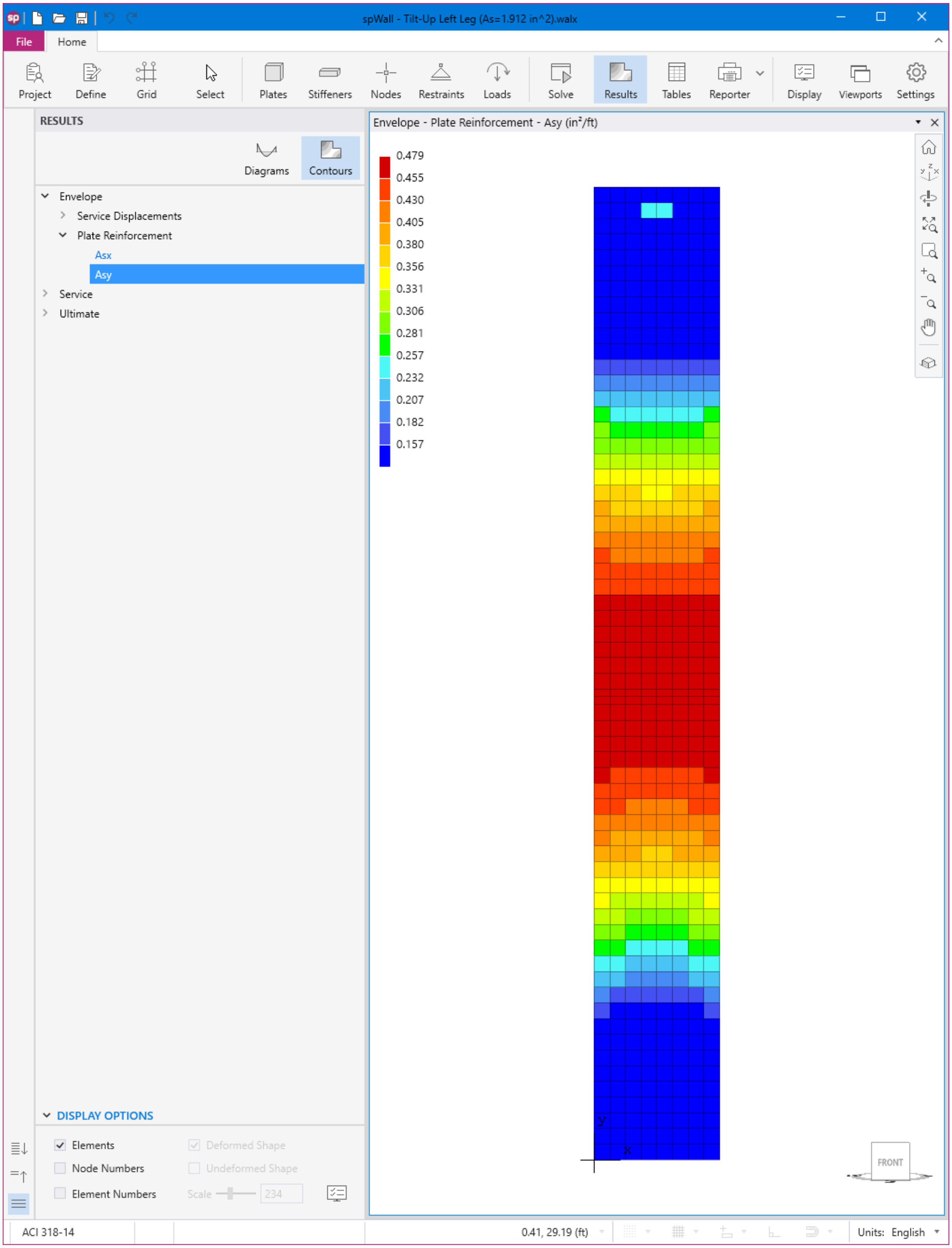


Figure 27 – Vertical Reinforcement Contour (in.^{2}/ft) (spWall)

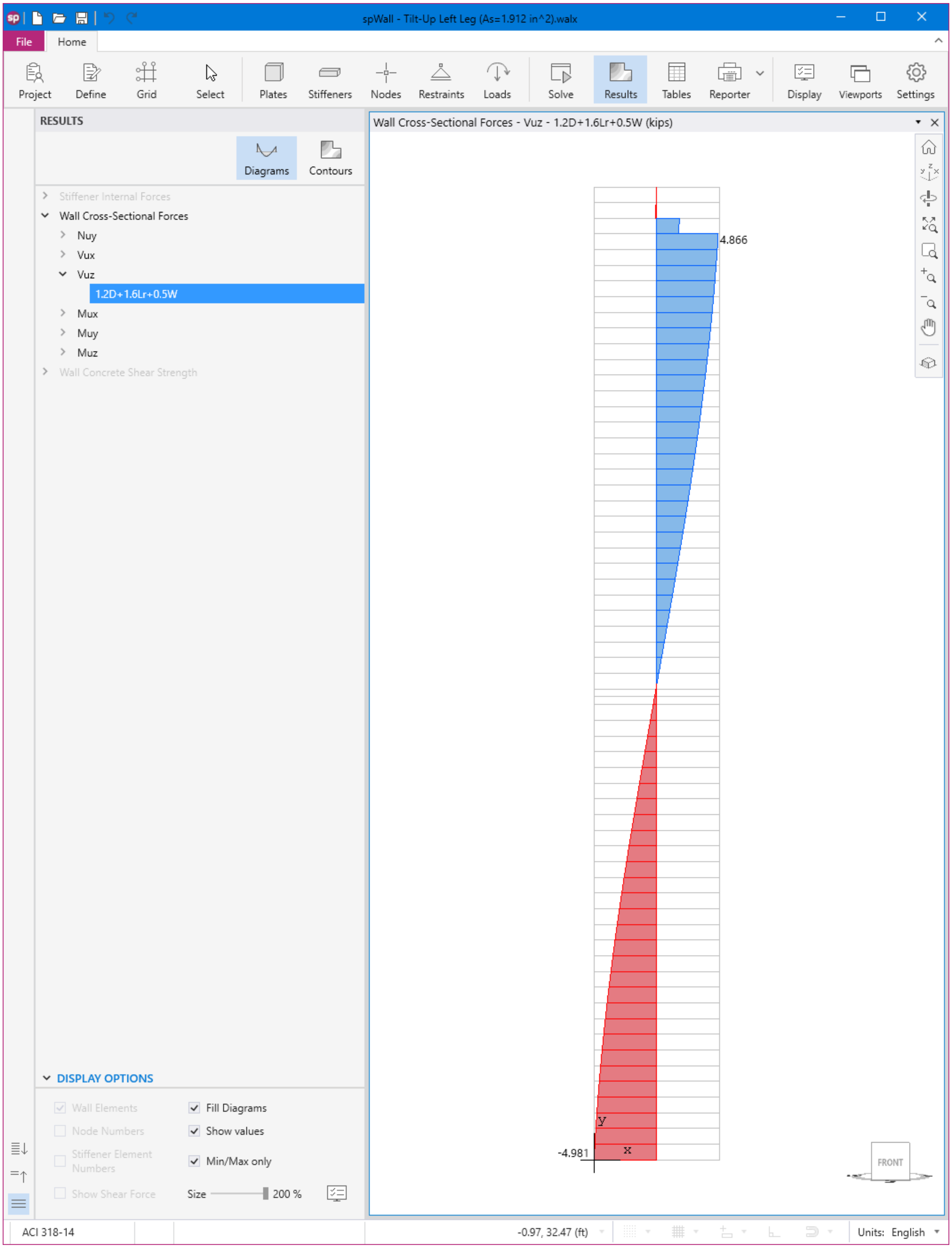


Figure 28 – Out-of-plane Shear Diagram (spWall)

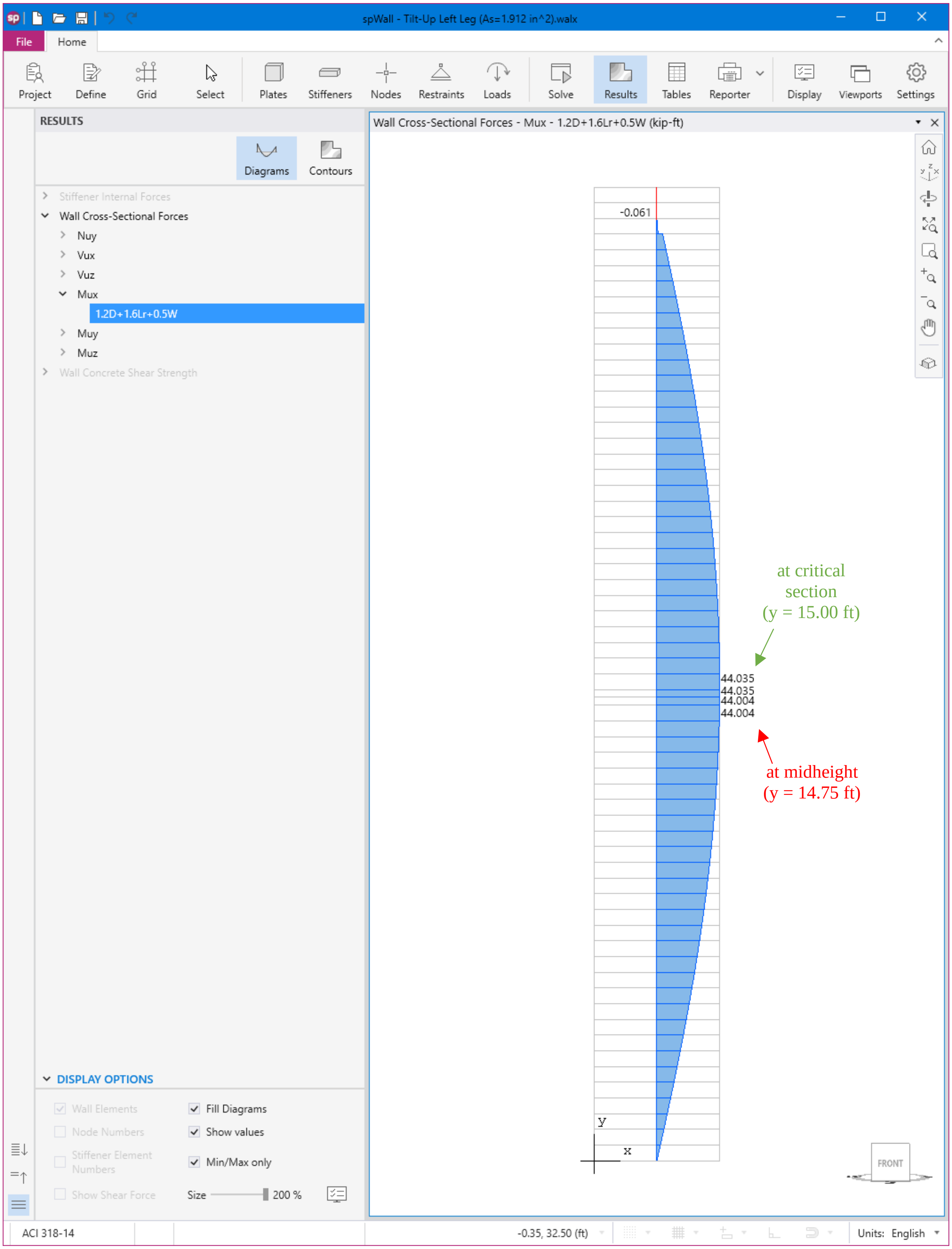


Figure 29 – Tilt-Up Wall Panel with Opening Moment Diagram (spWall)

1. Results

1.1. Envelope

1.1.1. 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
	233	1	Horizontal	0.03	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.02	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	234	1	Horizontal	0.06	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.01	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	235	1	Horizontal	0.07	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.00	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
at midheight	236	1	Horizontal	0.07	0.00	1.2D+1.6Lr+0...	0.210	0.20	
(y = 14.75 ft)			Vertical	10.99	-8.00	1.2D+1.6Lr+0...	0.475	0.45	
	237	1	Horizontal	0.07	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	10.99	-8.00	1.2D+1.6Lr+0...	0.475	0.45	
	238	1	Horizontal	0.07	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.00	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	239	1	Horizontal	0.06	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.01	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	240	1	Horizontal	0.03	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.02	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	241	1	Horizontal	0.03	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.03	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	242	1	Horizontal	0.05	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.01	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	243	1	Horizontal	0.07	0.00	1.2D+1.6Lr+0...	0.210	0.20	
at critical			Vertical	11.00	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
section	244	1	Horizontal	0.07	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.00	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
(y = 15.00 ft)	245	1	Horizontal	0.07	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.00	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	246	1	Horizontal	0.07	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.00	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	247	1	Horizontal	0.05	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.01	-8.00	1.2D+1.6Lr+0...	0.479	0.46	
	248	1	Horizontal	0.03	0.00	1.2D+1.6Lr+0...	0.210	0.20	
			Vertical	11.03	-8.00	1.2D+1.6Lr+0...	0.479	0.46	

$\sum A_{s,i}$ = 3.824 in.^{2}/ft

Element width = 0.5 ft

$\sum A_{s,i}$ = 3.824 in.^{2}/ft x 0.5 ft = 1.912 in.^{2}

$\sum A_{s,i}$ = 3.832 in.^{2}/ft

Element width = 0.5 ft

$\sum A_{s,i}$ = 3.832 in.^{2}/ft x 0.5 ft = 1.916 in.^{2}

Figure 30 – Cross-Sectional Vertical Reinforcement (spWall)

1.2. Service

1.2.1. Nodal Displacements

1.2.1.1. S1

Coordinate System: Global

	Node	Dx in	Dy in	Dz in
	271	0.000	-0.002	-0.205
	272	0.000	-0.002	-0.204
	273	0.000	-0.002	-0.204
	274	0.000	-0.002	-0.204
at midheight (y = 14.75 ft)	275	0.000	-0.002	-0.204
	276	0.000	-0.002	-0.204
	277	0.000	-0.002	-0.204
	278	0.000	-0.002	-0.204
	279	0.000	-0.002	-0.205
	280	0.000	-0.002	-0.205
	281	0.000	-0.002	-0.204
	282	0.000	-0.002	-0.204
	283	0.000	-0.002	-0.204
at critical section (y = 15.00 ft)	284	0.000	-0.002	-0.204
	285	0.000	-0.002	-0.204
	286	0.000	-0.002	-0.204
	287	0.000	-0.002	-0.204
	288	0.000	-0.002	-0.205

Figure 31 – Lateral Displacement at Critical Sections (Service Combinations) (spWall)

1.3. Ultimate

1.3.1. Nodal Displacements

1.3.1.1. 1.2D+1.6Lr+0.5W

Coordinate System: Global

	Node	Dx in	Dy in	Dz in
	271	0.000	-0.004	-10.780
	272	0.000	-0.004	-10.760
	273	0.000	-0.004	-10.745
	274	0.000	-0.004	-10.736
at midheight (y = 14.75 ft)	275	0.000	-0.004	-10.734
	276	0.000	-0.004	-10.736
	277	0.000	-0.004	-10.745
	278	0.000	-0.004	-10.760
	279	0.000	-0.004	-10.780
	280	0.000	-0.004	-10.780
	281	0.000	-0.004	-10.760
	282	0.000	-0.004	-10.745
	283	0.000	-0.004	-10.737
at critical section (y = 15.00 ft)	284	0.000	-0.004	-10.734
	285	0.000	-0.004	-10.737
	286	0.000	-0.004	-10.745
	287	0.000	-0.004	-10.760
	288	0.000	-0.004	-10.780

Figure 32 – Lateral Displacement at Critical Sections (Ultimate Combinations) (spWall)

1.3.2. Wall Cross-Sectional Forces

1.3.2.1. 1.2D+1.6Lr+0.5W

Coordinate System: Global
(+) Horizontal cross-section above Y-coordinate
(-) Horizontal cross-section below Y-coordinate

at midheight (y = 14.75 ft)

at critical section (y = 15.00 ft)

Wall Crossection			In-Plane Forces			Out-Of-Plane Forces		
No.	Y coordinate	X-Centroid	Vux	Nuy	Muz	Vuz	Mux	Muy
	ft	ft	kips	kips	kip-ft	kips	kip-ft	kip-ft
31-	14.75	2.00	0.00	-32.00	0.00	-0.18	44.00	0.00
31+	14.75	2.00	0.00	-32.00	0.00	-0.18	44.00	0.00
32-	15.00	2.00	0.00	-32.00	0.00	-0.07	44.04	0.00
32+	15.00	2.00	0.00	-32.00	0.00	-0.07	44.04	0.00

Figure 33 – Cross-Sectional Forces (spWall)

The hand calculation procedure shown earlier is repeated for the left leg based on the optimum area of steel (A_s = 1.912 in.^{2}) as follows:

$$P_{DL} = 4.48 \text{ kip (for the left leg)}$$

$$P_{LL} = 4.67 \text{ kip (for the left leg)}$$

$$w = 27.2 \text{ lb/ft}^2$$

$$P_{ua} = 12.84 \text{ kip}$$

$$P_{um} = 31.87 \text{ kip}$$

$$w_u = 0.122 \text{ kip/ft}$$

$$M_{ua} = 14.92 \text{ ft-kip}$$

$$A_{se} = 2.44 \text{ in.}^2$$

$$a = 0.898 \text{ in.}$$

$$c = 1.057 \text{ in.}$$

$$\frac{c}{d} = 0.242 < 0.375 \therefore \text{ tension-controlled}$$

$$I_{cr} = 235.29 \text{ in.}^4$$

$$K_b = 64.98 \text{ kip}$$

$$M_u = 43.13 \text{ ft-kip}$$

$$M_{cr} = 24.21 \text{ ft-kip}$$

$$\phi M_n = 43.16 \text{ ft-kip} > M_u = 43.13 \text{ ft-kip} \quad (\textbf{o.k.})$$

$$\phi M_n = 43.16 \text{ ft-kip} > M_{cr} = 24.21 \text{ ft-kip} \quad (\textbf{o.k.})$$

$$\Delta_u = 10.619 \text{ in.}$$

$$\frac{P_{um}}{A_g} = 75.89 \text{ psi} < 0.06 \times f_c' = 240 \text{ psi} \quad (\textbf{o.k.})$$

$$M_{sa} = 12.21 \text{ ft-kip}$$

$$\Delta_{cr} = 0.393 \text{ in.}$$

$$M_a = 12.55 \text{ ft-kip} < \frac{2}{3} M_{cr} = 16.14 \text{ ft-kip} \quad (\textbf{o.k.})$$

$$\Delta_s = 0.203 \text{ in.} < \frac{l_c}{150} = 2.36 \text{ in.} \quad (\textbf{o.k.})$$

The above calculations reveal a reduction in the cracked moment of inertia resulting in an increase in the M_u applied. Note that the moment capacity is now very close to the required moment.

The following table shows the comparison between hand results with spWall model results for the optimum area of steel.

Table 7 – Comparison of Analysis and Design Results for the Tilt-Up Wall with Optimum Area of Steel					
Solution	M_u (kip-ft)	N_u (kip)	$D_{z,service}$ (in.)	$D_{z,ultimate}$ (in.)	$A_{s,required}$ (in.^{2})
Hand (at midheight)	43.13	31.87	0.203	10.619	1.912
spWall (at midheight)*	44.00	32.00	0.204	10.734	1.912
spWall (at critical section)**	44.04	32.00	0.204	10.734	1.916

* Values are taken at midheight (y = 14.75 ft) for comparison purposes with hand calculations.
** Values are taken at critical section (y = 15.00 ft) with maximum moment value.

After following the reinforcement optimization procedure, the results of all the hand calculations used above are in agreement with the automated exact results obtained from the spWall program including the required area of steel.