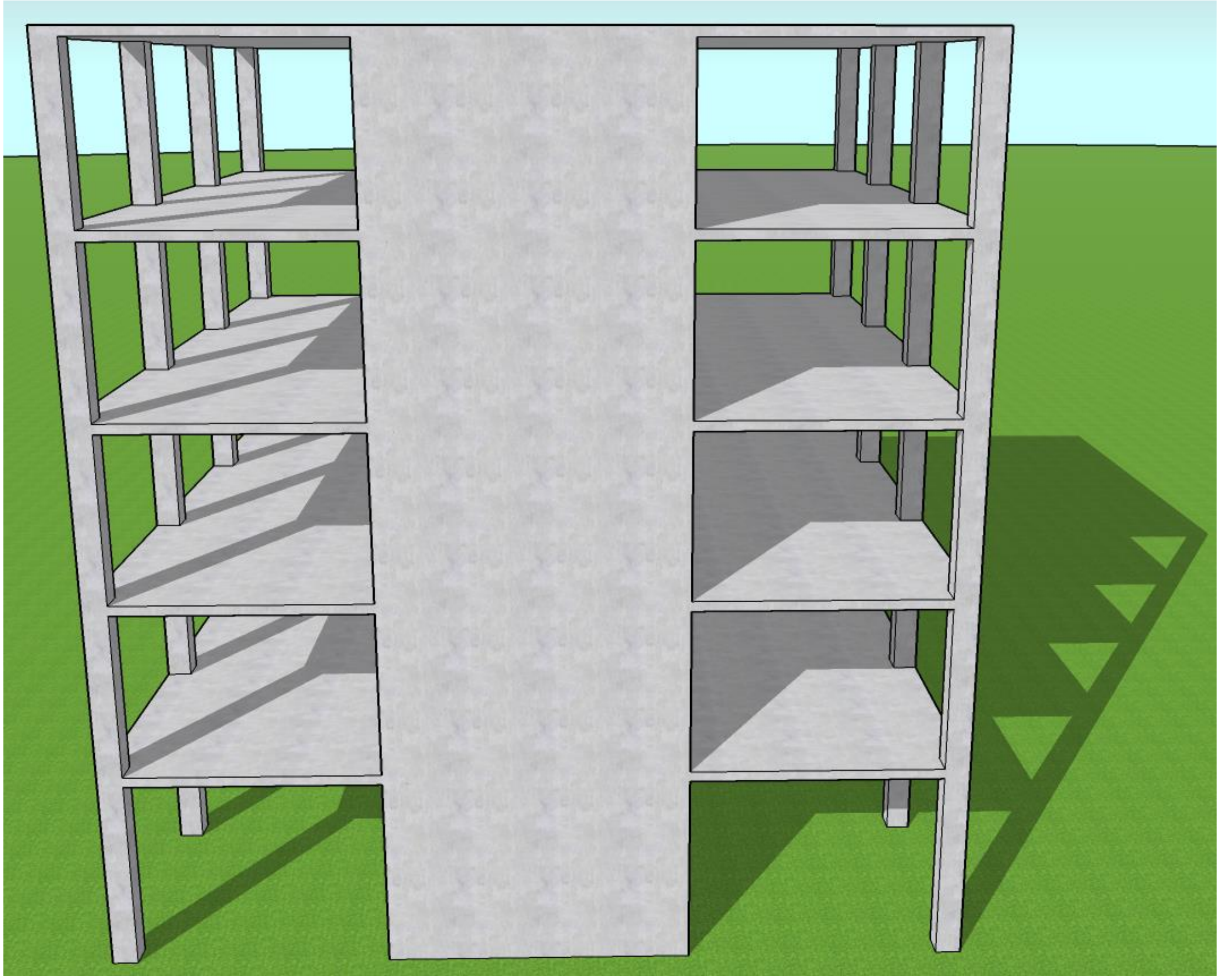
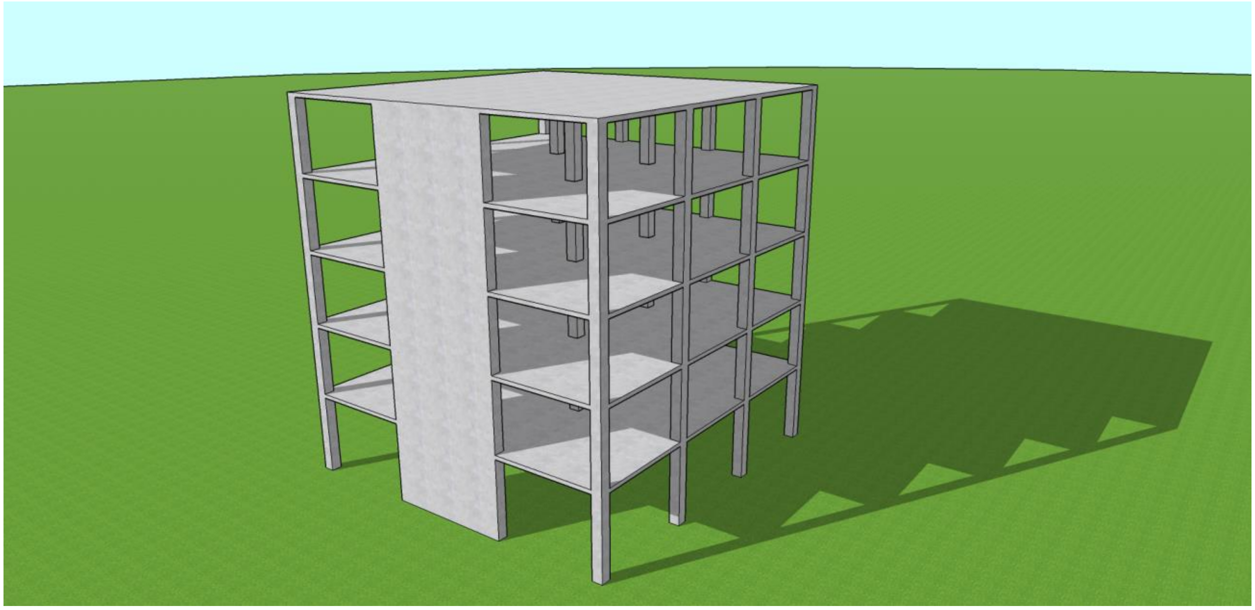
Reinforced Concrete Shear Wall Analysis and Design





Reinforced Concrete Shear Wall Analysis and Design

A structural reinforced concrete shear wall in a 5-story building provides lateral and gravity load resistance for the applied load as shown in the figure below. Shear wall section and assumed reinforcement is investigated after analysis to verify suitability for the applied loads.

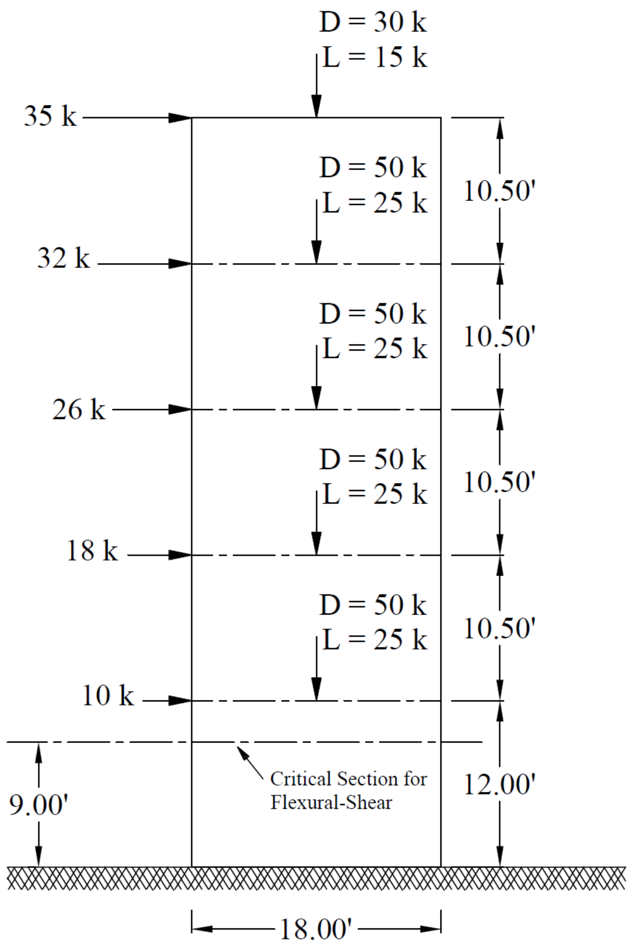


Figure 1 – Reinforced Concrete Shear Wall Geometry and Loading

Contents

Code

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

Reference

Reinforced Concrete Mechanics and Design, 7th Edition, 2016, James Wight, Pearson, Example 18-2

Design Data

f_c' = 4,000 psi normal weight concrete

f_y = 60,000 psi

Slab thickness = 7 in.

Wall thickness = 10 in.

Wall length = 18 ft

Vertical reinforcement: #5 bars at 18 in. on centers in each face ($A_{s,\ vertical}$ = #5 @ 18 in.)

Horizontal reinforcement: #4 bars at 16 in. on centers in each face ($A_{s,\ horizontal}$ = #4 @ 16 in.)

1. Minimum Reinforcement Requirements (Reinforcement Percentage and Spacing)

1.1. Horizontal Reinforcement Check

$$\rho_t = \frac{A_{v,horizontal}}{h \times s_2} = \frac{2\times 0.2}{10\times 16} = 0.0025$$ ***ACI 318-14 (2.2)***

$$\rho_t = 0.0025 \geq \rho_{t,\min} = 0.0025 \text{ (o.k)}$$ ***ACI 318-14 (11.6.2(b))***

$$s_{t,\max} = \text{smallest of} \begin{Bmatrix} 3\times h \\ 18\text{ in.} \\ l_w/5 \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 3\times 10 \\ 18\text{ in.} \\ 18/5 \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 30\text{ in.} \\ 18\text{ in.} \\ 43.2\text{ in.} \end{Bmatrix} = 18\text{ in.}$$ ***ACI 318-14 (11.7.3.1)***

$$s_{t,provided} = 16\text{ in.} < s_{t,\max} = 18\text{ in. (o.k)}$$

1.2. Vertical Reinforcement Check

$$\rho_l = \frac{A_{v,vertical}}{h \times s_1} = \frac{2\times 0.31}{10\times 18} = 0.00344$$ ***ACI 318-14 (2.2)***

$$\rho_{l,\min} = \text{greater of} \begin{Bmatrix} 0.0025 + 0.5\left(2.5 - \dfrac{h_w}{l_w}\right)(\rho_t - 0.0025) \\ 0.0025 \end{Bmatrix}$$ ***ACI 318-14 (11.6.2(a))***

$$\rho_{l,\min} = \text{greater of} \begin{Bmatrix} 0.0025 + 0.5\left(2.5 - \dfrac{h_w}{l_w}\right)(0.0025 - 0.0025) \\ 0.0025 \end{Bmatrix} = \text{greater of} \begin{Bmatrix} 0.0025 \\ 0.0025 \end{Bmatrix} = 0.0025$$

$$\rho_l = 0.00344 \geq \rho_{l,\min} = 0.0025 \text{ (o.k)}$$ ***ACI 318-14 (11.6.2(a))***

$$s_{l,\max} = \text{smallest of} \begin{Bmatrix} 3\times h \\ 18\text{ in.} \\ l_w/3 \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 3\times 10 \\ 18\text{ in.} \\ 18/3 \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 30\text{ in.} \\ 18\text{ in.} \\ 72\text{ in.} \end{Bmatrix} = 18\text{ in.}$$ ***ACI 318-14 (11.7.2.1)***

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

2. Neutral Axis Depth Determination

$$M_{base} = 35\times54+32\times43.5+26\times33+18\times22.5+10\times12 = 4,670 \text{ kip-ft}$$

The load factor for strength-level wind force = 1.0

$$M_{u,base} = 1.0\times4,670 = 4,670 \text{ kip-ft}$$

$$N_u = 0.9\times N_D = 0.9\times(30+50+50+50+50) = 207 \text{ kips}$$ ***ACI 318-14 (Eq.5.3.1f)***

$$\beta_1 = 0.85 - \frac{0.05\times(f_c^{'}-4000)}{1000} = 0.85 - \frac{0.05\times(4000-4000)}{1000} = 0.85$$ ***ACI 318-14 (Table 22.2.2.4.3)***

$$\omega = \rho_l\frac{f_y}{f_c^{'}} = 0.00344\times\frac{60}{4} = 0.0516$$

$$\alpha = \frac{N_u}{h\times l_w\times f_c^{'}} = \frac{207}{10\times216\times4} = 0.0240$$

$$c = \left(\frac{\alpha+\omega}{0.85\beta_1+2\omega}\right)l_w = \left(\frac{0.0240+0.0516}{0.85\times0.85+2\times0.0516}\right)\times216 = 19.8 \text{ in.}$$

Assume the effective flexural depth (d) is approximately equal to $0.8l_w$ = 173 in. ***ACI 318-14 (11.5.4.2)***

$$c = 19.8 \text{ in.} << d = 173 \text{ in.} \rightarrow \text{Tension controlled section}$$

$$\therefore \phi = 0.90$$ ***ACI 318-14 (Table 21.2.2)***

3. Moment Capacity Check

$$A_{st} = A_{v,vertical} \frac{l_w}{s_{l,provided}} = 2 \times 0.31 \times \frac{216}{18} = 7.44 \text{ in.}^4$$

$$T = A_{st} \times f_y \left(\frac{l_w - c}{l_w} \right) = 7.44 \times 60 \times \left(\frac{216 - 19.8}{216} \right) = 405 \text{ kips}$$

Taking into account the applied axial force and summing force moments about the compression force (C), the moment capacity can be computed as follows:

$$M_n = T\left(\frac{l_w}{2}\right) + N_u\left(\frac{l_w - c}{2}\right) = 405\left(\frac{216}{2}\right) + 207\left(\frac{216 - 19.8}{2}\right) = 64,000 \text{ kips-in.} = 5,340 \text{ kips-ft}$$

$$\phi M_n = 0.9 \times 5,340 = 4,800 \text{ kips-ft} > M_u = 4,670 \text{ kips-ft}$$

Since ϕM_n is greater than M_u, the wall has adequate flexural strength.

To further confirm the moment capacity is adequate with detailed consideration for the axial compression, an interaction diagram using spColumn can be created easily as shown below for the wall section. The location of the neutral axis, maximum tensile strain, and the phi factor can all be also verified from the spColumn model results output parameters. As can be seen from the interaction diagram a comprehensive view of the wall behavior for any combination of axial force and applied moment.

For a factored axial and moment of 207 kips and 4670 kip-ft the interaction diagram shows a capacity factor of 1.139 (ϕM_n = 5,320 kip-ft for $\phi P_n = P_u$), see Figures 11 and 12.

4. Shear Capacity Check

$$V_u = 35 + 32 + 26 + 18 + 10 = 121 \text{ kips}$$

$$V_c = \text{lesser of} \left\{ \begin{array}{ll} 3.3 \times \lambda \times \sqrt{f_c^{'}} \times h \times d + \dfrac{N_u \times d}{4 \times l_w} & \text{(d)} \\ \left[0.6 \times \lambda \times \sqrt{f_c^{'}} + \dfrac{l_w \left(1.25 \times \lambda \times \sqrt{f_c^{'}} + 0.2 \dfrac{N_u}{l_w \times h} \right)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}} \right] \times h \times d & \text{(e)} \end{array} \right\}$$

ACI 318-14 (Table 11.5.4.6)

$$V_c = \text{lesser of} \left\{ \begin{array}{l} 3.3 \times 1.0 \times \sqrt{4,000} \times 10 \times 173 + \dfrac{207,000 \times 173}{4 \times 216} \\ \left[0.6 \times 1.0 \times \sqrt{4,000} + \dfrac{216 \times \left(1.25 \times 1.0 \times \sqrt{4,000} + 0.2 \dfrac{207,000}{216 \times 10} \right)}{\dfrac{3,580}{121} - \dfrac{216}{2}} \right] \times 10 \times 173 \end{array} \right\}$$

$$V_c = \text{lesser of} \left\{ \begin{array}{l} 402 \text{ kips} \\ 214 \text{ kips} \end{array} \right\} = 214 \text{ kips}$$

Where M_u/V_u ratio used in equation (e) was calculated at the critical section above the base of the wall (see Figure 1).

$$\text{distance to the critical section} = \text{smaller of} \left\{ \begin{array}{l} \dfrac{l_w}{2} \\ \dfrac{h_w}{2} \\ \text{one story height} \end{array} \right\}$$

ACI 318-14 (11.5.4.7)

$$\text{distance to the critical section} = \text{smaller of} \left\{ \begin{array}{l} \dfrac{18}{2} = 9 \text{ ft} \\ \dfrac{54}{2} = 27 \text{ ft} \\ 12 \text{ ft} \end{array} \right\} = 9 \text{ ft}$$

The factored moment at the ultimate section is equals to:

$$M_u = M_{u,base} - V_{u,base} \times \frac{l_w}{2} = 4,670 - 121 \times 9 = 3,580 \text{ kip-ft}$$

$$\phi V_c = \phi \times V_c = 0.75 \times 214 = 161 \text{ kips}$$

Where $\phi = 0.75$ for shear **_ACI 318-14 (Table 21.2.1)_**

$$\phi V_c = 161 \text{ kips} > V_u = 121 \text{ kips}$$

Thus, it is not required to calculate the additional shear strength provided by the horizontal reinforcement (V_s)

$$0.5 \times \phi V_c = 80.5 \text{ kips} < V_u = 121 \text{ kips}$$

Since $0.5\phi V_c$ is less than V_u, ρ_l shall be at least the greater of Equation 11.6.2 in the Code and 0.0025 but need not to exceed ρ_t required by Equation 11.5.4.8. and ρ_t shall be at least 0.0025. **_ACI 318-14 (11.6.2)_**

(Those requirements were checked in step 1).

5. Shear Wall Analysis and Design – spWall Software

spWall is a program for the analysis and design of reinforced concrete shear walls, tilt-up walls, precast wall 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.

After the Finite Element Analysis (FEA) is completed 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 shear wall reinforcement. In stiffeners and boundary elements, spWall calculates the required shear and torsion steel reinforcement. Shear 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 the FEA results obtained from an spWall model created for the reinforced concrete shear wall in this example.

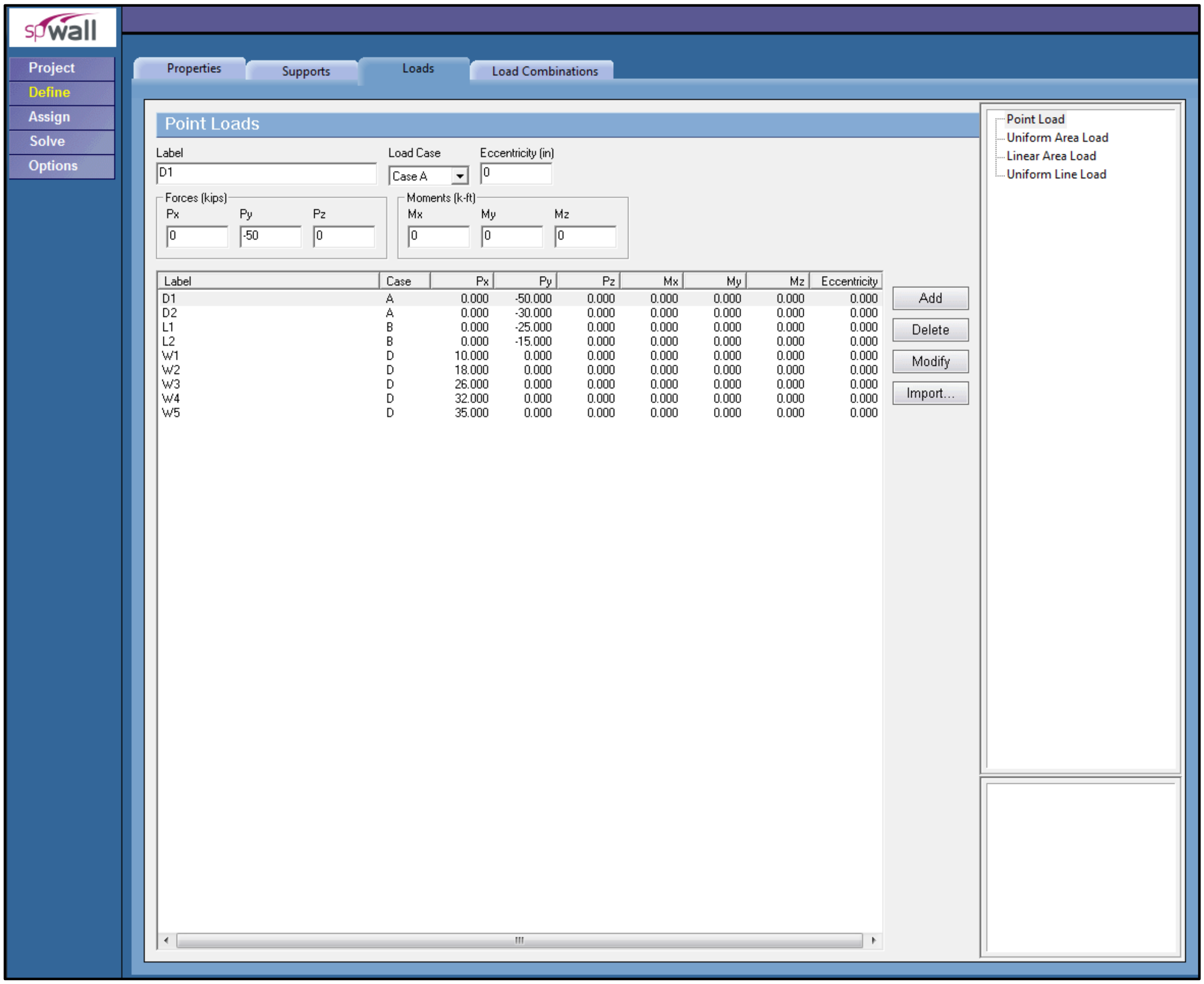


Figure 2 –Defining Loads for Shear Wall (spWall)

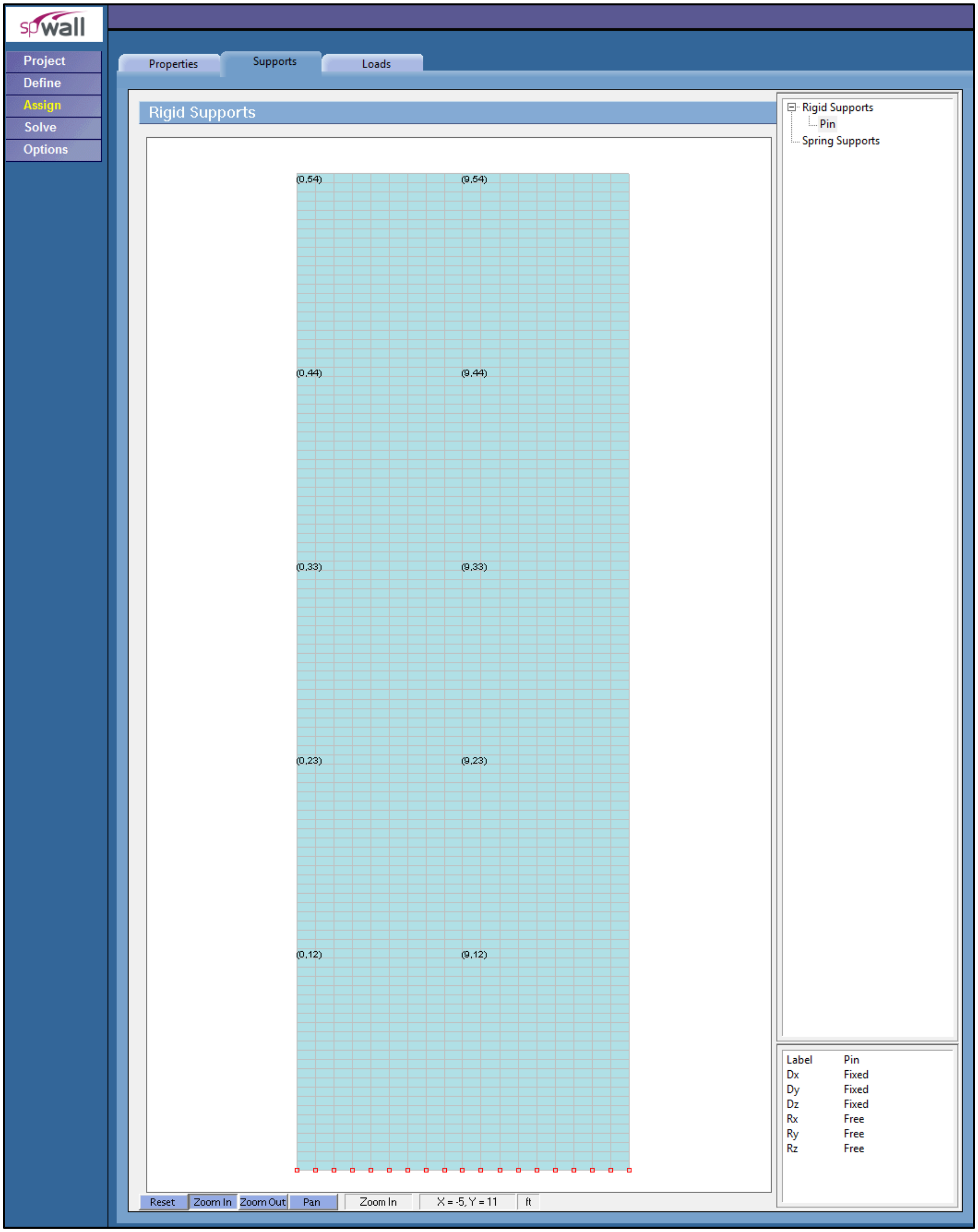


Figure 3 – Assigning Boundary Conditions for Shear Wall (spWall)

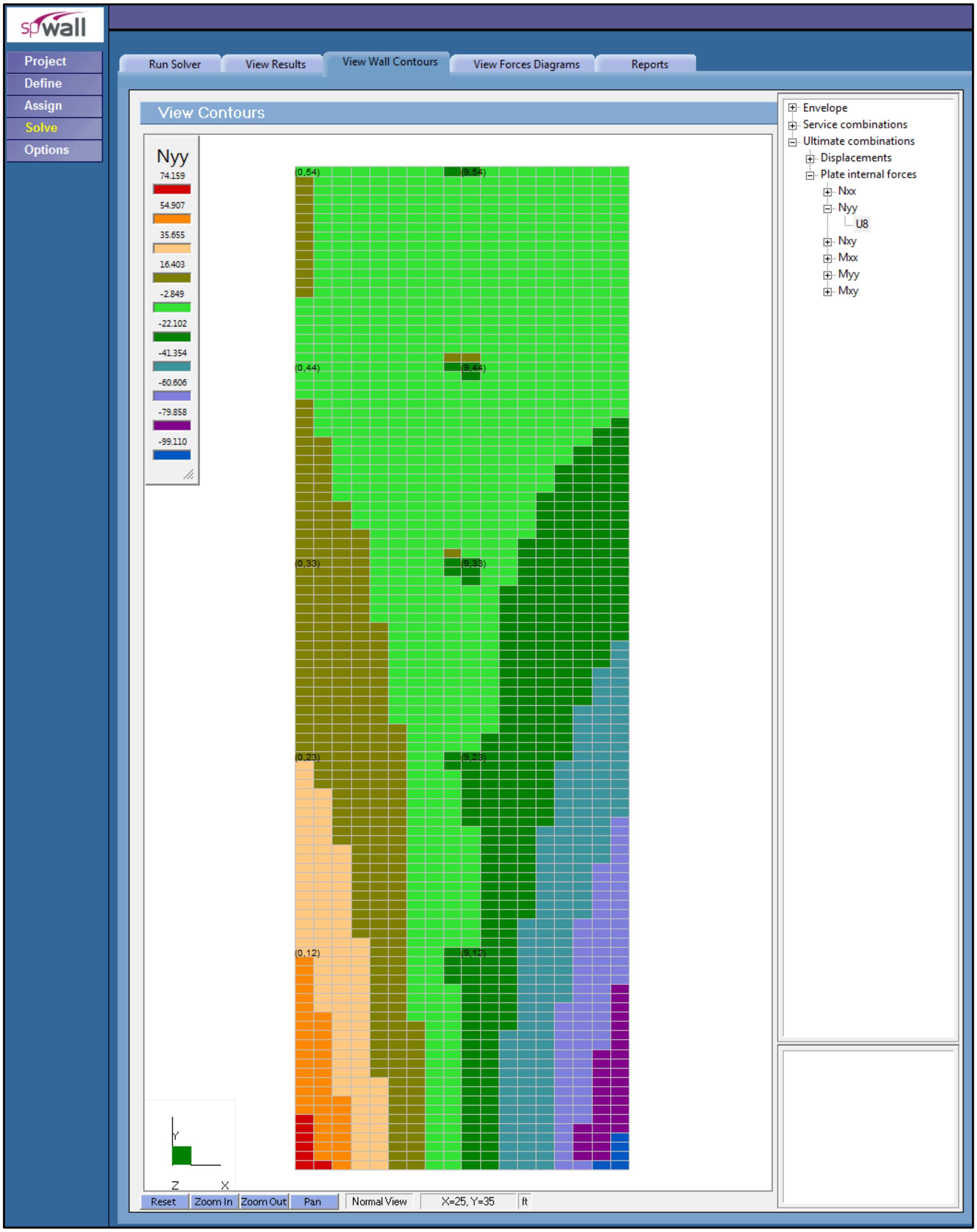


Figure 4 –Factored Axial Forces Contour Normal to Shear Wall Cross-Section (spWall)

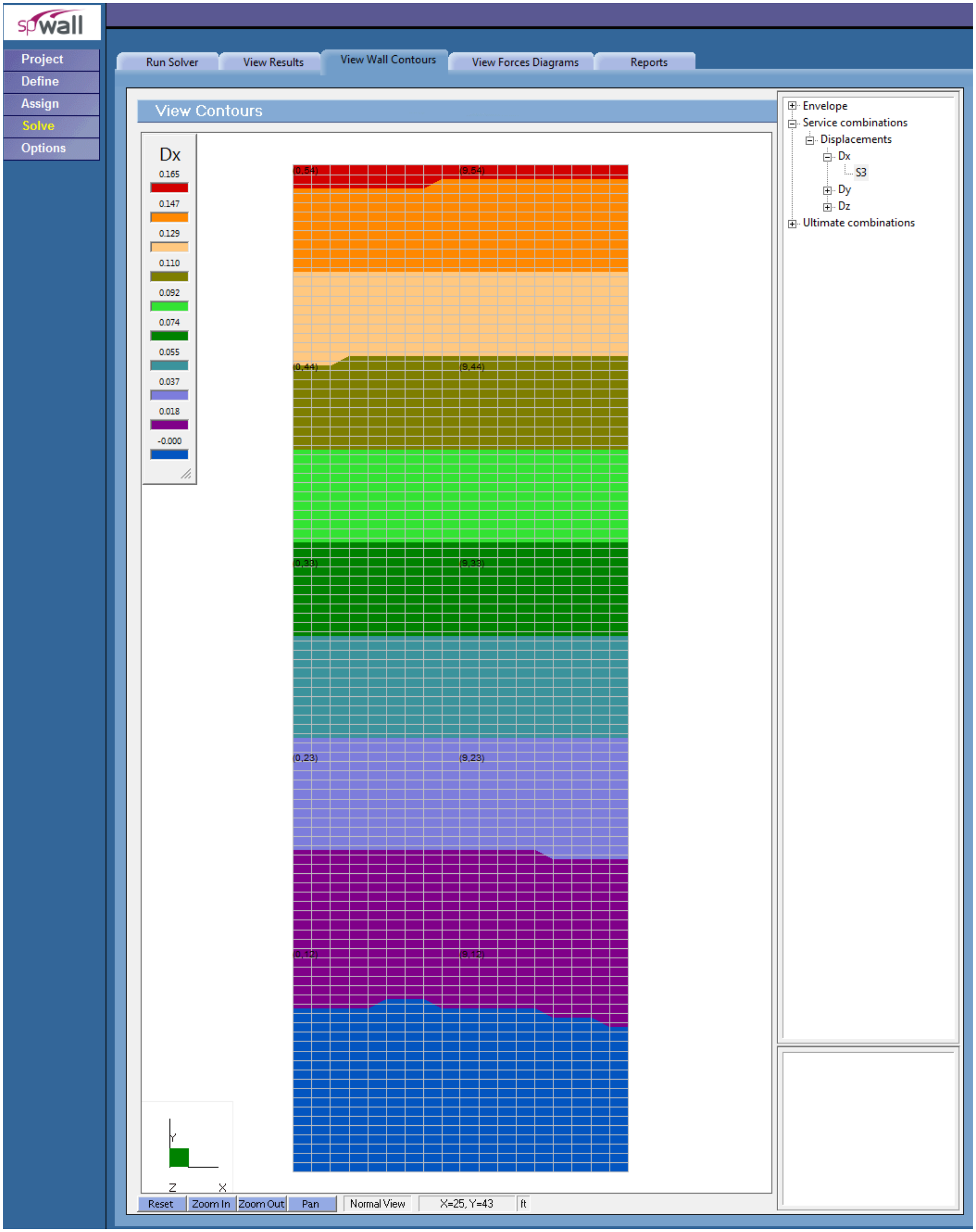


Figure 5 – Shear Wall Lateral Displacement Contour (spWall)

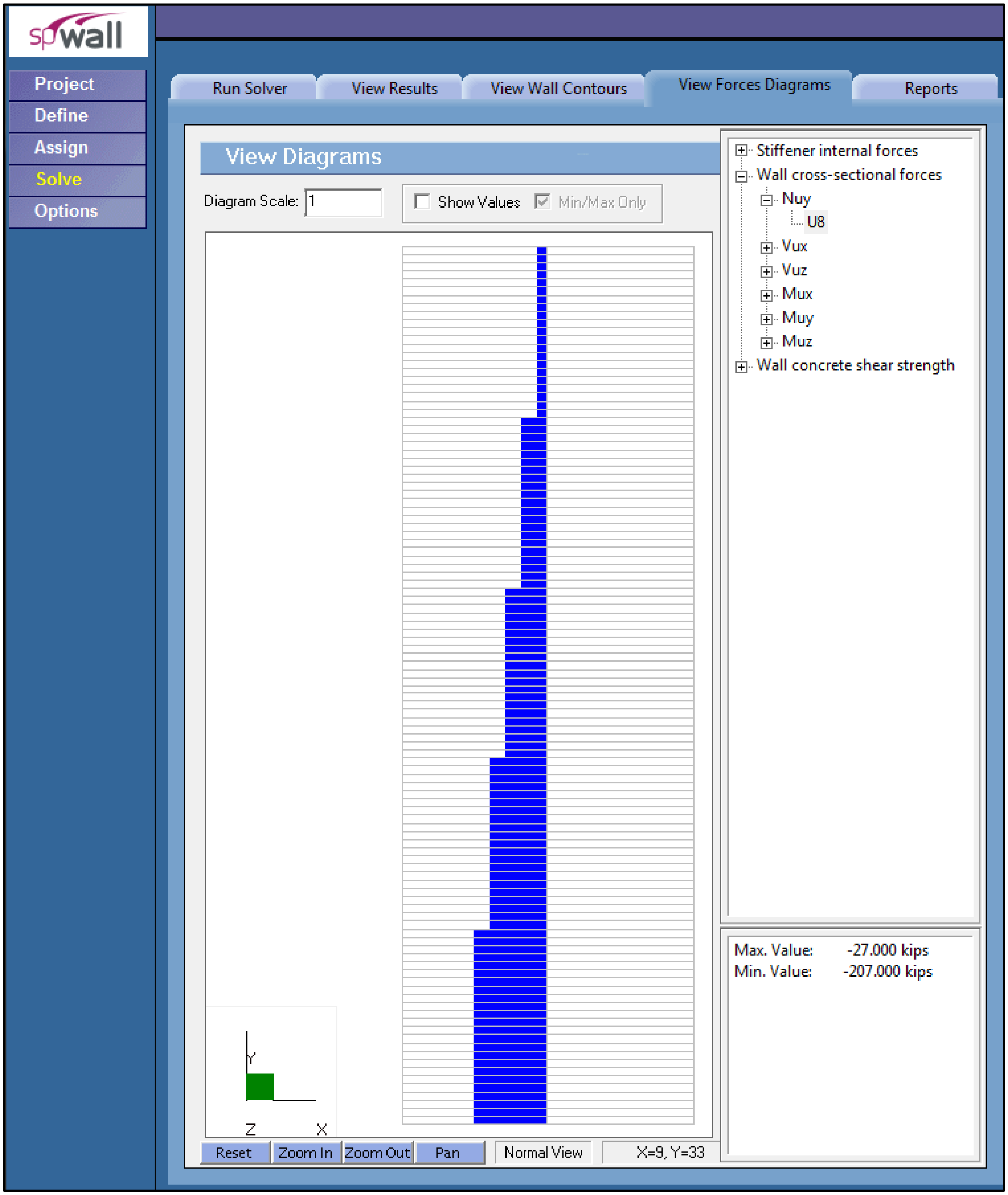


Figure 6 – Shear Wall Axial Load Diagram (spWall)

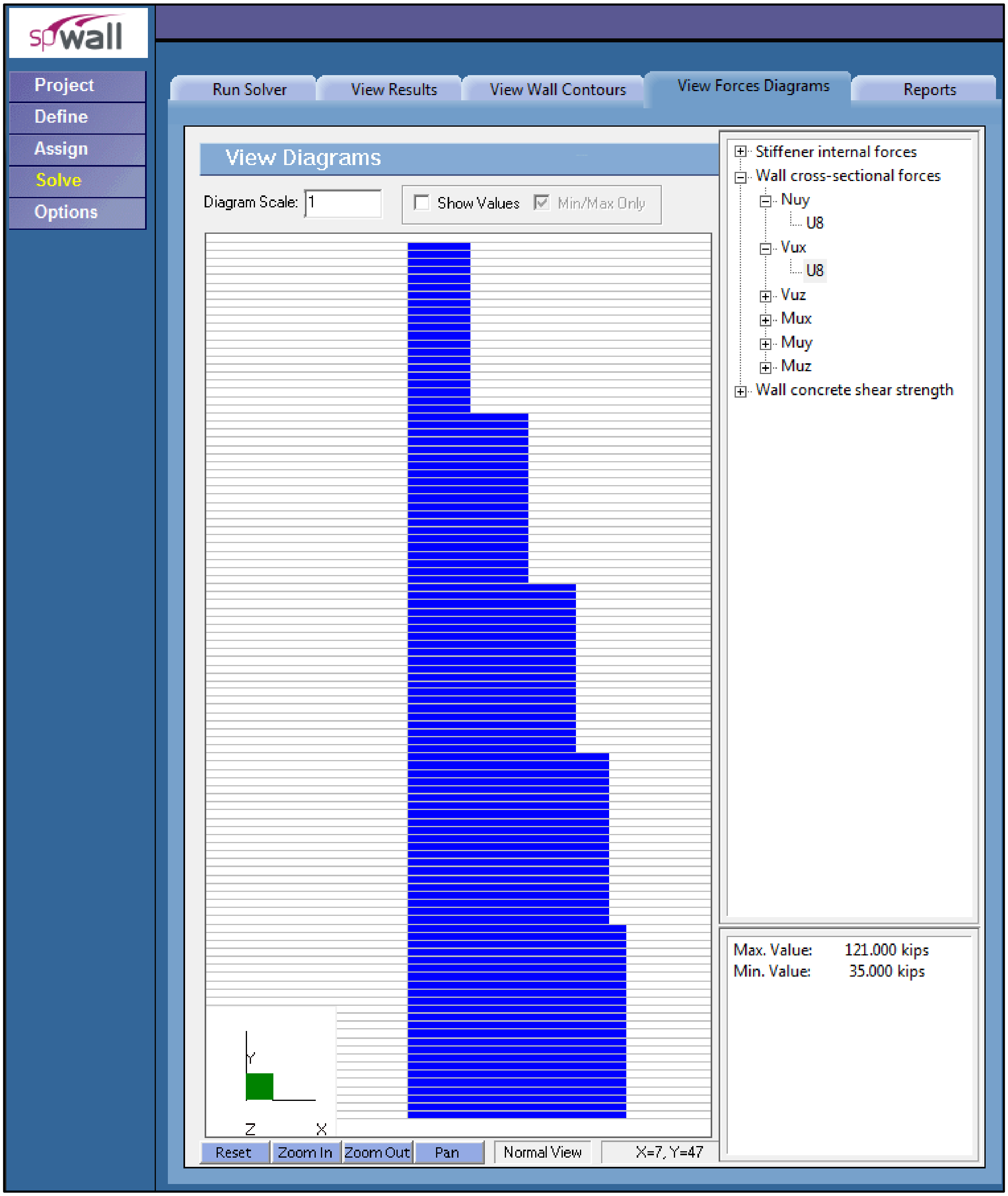


Figure 7 – In-plane Shear Diagram (spWall)

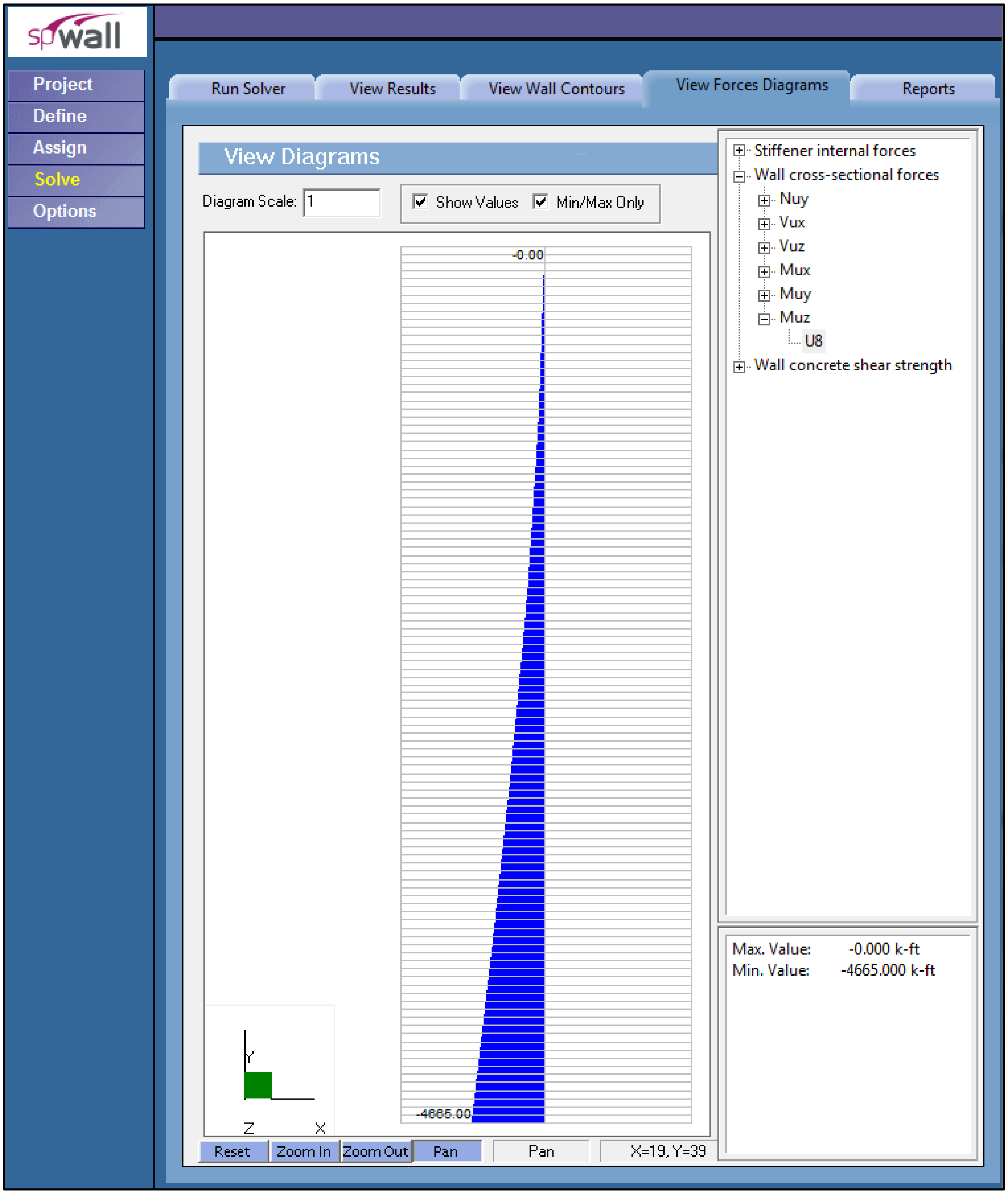


Figure 8 – Shear Wall Moment Diagram (spWall)

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Envelope | Plate reinforcement

Coordinate System: Global

Units:

Elements along the wall base

Total required area of steel (As): in^2/ft
Bending moment (Mu): k-ft/ft, axial force (Nu): klf

Elem	Curtains	Direction	Mu (x/y)	Nu (x/y)	Ld_combo	As (x/y)	ro(%)	Tie
1	2	Horizontal	0.0000e+000	2.9550e+001	U8	5.53e-001	0.46	
		Vertical	0.0000e+000	7.7378e+001	U8	1.45e+000	1.21	
2	2	Horizontal	0.0000e+000	1.8772e+001	U8	3.51e-001	0.29	
		Vertical	0.0000e+000	7.3781e+001	U8	1.38e+000	1.15	
3	2	Horizontal	0.0000e+000	1.5581e+001	U8	2.91e-001	0.24	
		Vertical	0.0000e+000	5.4554e+001	U8	1.02e+000	0.85	
4	2	Horizontal	0.0000e+000	1.2902e+001	U8	2.41e-001	0.20	
		Vertical	0.0000e+000	4.2835e+001	U8	8.01e-001	0.67	
5	2	Horizontal	0.0000e+000	1.0669e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	3.2675e+001	U8	6.11e-001	0.51	
6	2	Horizontal	0.0000e+000	8.7408e+000	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	2.3359e+001	U8	4.37e-001	0.36	
7	2	Horizontal	0.0000e+000	7.0193e+000	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	1.4611e+001	U8	2.73e-001	0.23	
8	2	Horizontal	0.0000e+000	5.4510e+000	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	6.2406e+000	U8	1.44e-001	0.12	
9	2	Horizontal	0.0000e+000	2.6129e+000	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-1.2546e+001	U8	1.44e-001	0.12	
10	2	Horizontal	0.0000e+000	-8.4802e+000	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-2.0955e+001	U8	1.44e-001	0.12	
11	2	Horizontal	0.0000e+000	-1.0370e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-2.9458e+001	U8	1.44e-001	0.12	
12	2	Horizontal	0.0000e+000	-1.2361e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-3.8160e+001	U8	1.44e-001	0.12	
13	2	Horizontal	0.0000e+000	-1.4490e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-4.7212e+001	U8	1.44e-001	0.12	
14	2	Horizontal	0.0000e+000	-1.6819e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-5.6863e+001	U8	1.44e-001	0.12	
15	2	Horizontal	0.0000e+000	-1.9473e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-6.7480e+001	U8	1.44e-001	0.12	
16	2	Horizontal	0.0000e+000	-2.2634e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-8.0022e+001	U8	1.44e-001	0.12	
17	2	Horizontal	0.0000e+000	-2.6373e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-1.0247e+002	U8	1.44e-001	0.12	
18	2	Horizontal	0.0000e+000	-4.0107e+001	U8	2.40e-001	0.20	
		Vertical	0.0000e+000	-1.0437e+002	U8	1.44e-001	0.12	

$\sum A_{s,vertical} = 7.56 \text{ in.}^2$

Figure 9 – Shear Wall Vertical Reinforcement (spWall)

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Envelope | Wall concrete shear strength | In-plane shear

Coordinate System: Global

Units:
Y-coordinate: ft
(+) Horizontal cross-section above Y-coordinate
(-) Horizontal cross-section below Y-coordinate
Force (Nuy, Vux): kips
Moment (Muz): k-ft
Wall concrete shear strength (Vcx): kips

Cross-section			Cross-sectional Forces			Strength	
No.	Y-coordinate	Ld_combo	Nuy	Muz	Vux	Phi*Vcx	
1+	0.000	U8	-2.0700e+002	-4.6650e+003	1.2100e+002	1.6393e+002	#
19+	9.000	U8	-2.0700e+002	-3.5760e+003	1.2100e+002	1.6393e+002	#

Notes:
\# - Shear force Vux exceeds 0.5*Phi*Vcx

Figure 10 – Concrete Shear Strength and Shear Wall Cross-Sectional Forces (spWall)

6. Design Results Comparison and Conclusions

Table 1 – Comparison of Shear Wall Analysis and Design Results						
Solution	Wall Cross-Sectional Forces			ϕV_c (kips)	$A_{s,vertical}$ (in.^{2})	ϕM_n, kip-ft
	M_u (kip-ft)	N_u (kips)	V_u (kips)			
Hand	4,670	207	121	161	7.44	4,800
Reference	4,670	207	121	161	7.44	4,800
spWall	4,665	207	121	164	7.56	4,669*
* minimum required capacity						

The results of all the hand calculations and the reference used illustrated above are in precise agreement with the automated results obtained from the spWall FEA. It is worth noting that the minimum area of steel is governed by the minimum reinforcement ratio stipulated by the code. The same can be seen in spWall output for elements 9 through 18.

To calculate the wall moment capacity, the design forces in each finite element can be employed to sum force moments about the center of the wall section as follows:

$$\phi M_n = \phi \times \sum_{i=1}^{18} \left(N_{u,i} \times \mathrm{d}_i \right)$$

$$= 0.9 \times \begin{bmatrix} (77.4\times8.5)+(73.8\times7.5)+(54.6\times6.5)+(42.8\times5.5)+(32.7\times4.5)+(23.4\times3.5)+(14.5\times2.5) \\ +(6.2\times1.5)+(-12.5\times0.5)+(-21\times-0.5)+(-29.5\times-1.5)+(-38.2\times-2.5)+(-47.2\times-3.5) \\ +(-56.9\times-2.5)+(-67.5\times-5.5) \end{bmatrix} = 4,669\,\text{kip-ft}$$

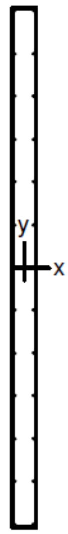
7. Appendix – Commentary on Reinforcement Arrangement Impact on Wall Capacity

In the hand calculations and the reference, a simplified procedure to calculate the nominal flexural strength was used (A. E. Cardenas et al.). In this procedure, several broad assumptions are made to avoid tedious detailed calculations:

- All steel in the tension zone yields in tension.
- All steel in the compression zone yields in compression.
- The tension force acts at mid-depth of the tension zone.
- The total compression force (sum of steel and concrete contributions) acts at mid-depth of the compression zone.

To investigate the shear wall cross section capacity using the interaction diagram method, a model generated by spColumn is made. This approach considers the entire wall section and employs the provisions of the Strength Design Method and Unified Design Provisions with all conditions of strength satisfying the applicable conditions of equilibrium and strain compatibility.

For illustration and comparison purposes, following figures provide a sample of the input and output of the results obtained from an spColumn model created for the shear wall in this example. spColumn calculates the values of strain at each layer of steel (in tension and compression zones) with location of the total tension and compression forces leading to the value for nominal and design strengths (axial and flexural strengths).



10 x 216 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

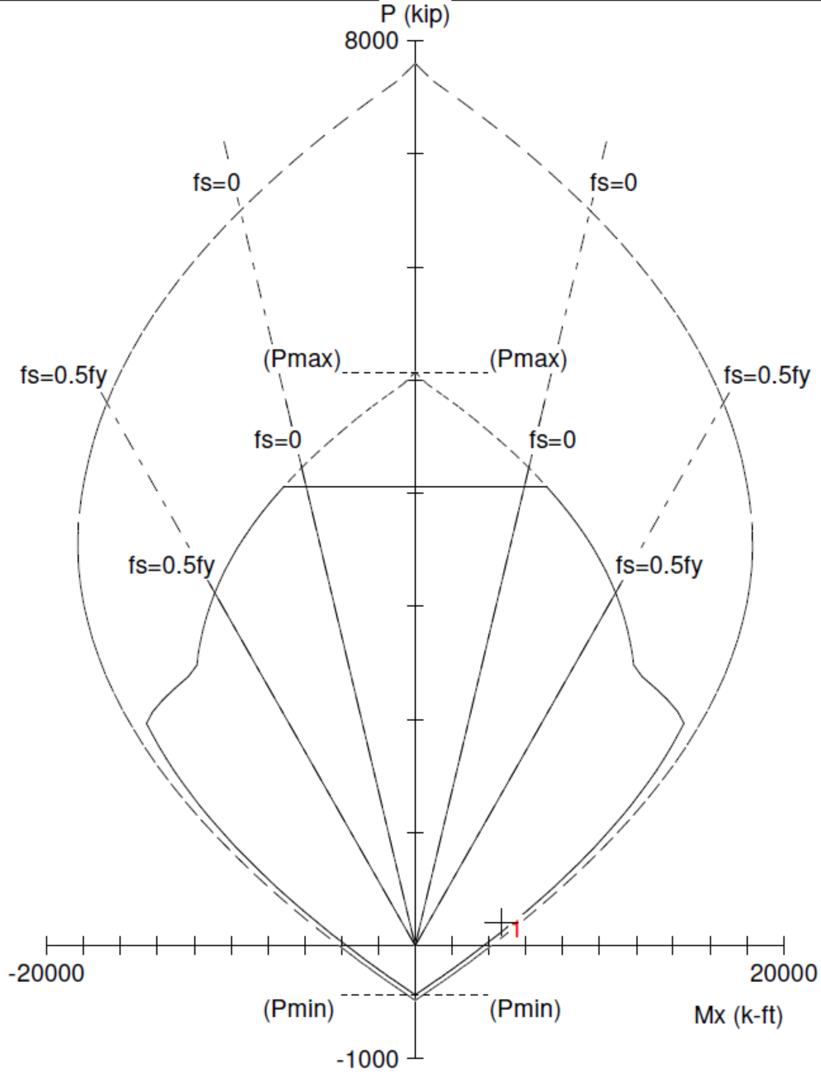
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 08/07/17

Time: 13:45:35



STRUCTUREPOINT - spColumn v5.50 (TM). Licensed to: StructurePoint. License ID: 00000-0000000-4-25EF2-2C6B6

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Project: Structural Wall

Column: Shear Wall		Engineer: SP	
f'c = 4 ksi	fy = 60 ksi	Ag = 2160 in^2	26 #5 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 8.06 in^2	rho = 0.37%
fc = 3.4 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 8.39808e+006 in^4
e_u = 0.003 in/in		Yo = -0.00 in	Iy = 18000 in^4
Beta1 = 0.85		Min clear spacing = 7.37 in	Clear cover = N/A
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

Figure 11 – Shear Wall Interaction Diagram (X-Axis, In-Plane) (spColumn)

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General Information:
====================
  File Name: C:\TSDA\Structural Wall Under Lateral Load\spColumn\Structural Wall.col
  Project:  Structural Wall
  Column:   Shear Wall                        Engineer: SP
  Code:     ACI 318-14                        Units: English

  Run Option: Investigation                   Slenderness: Not considered
  Run Axis:   X-axis                          Column Type: Structural

Material Properties:
====================
  Concrete: Standard                          Steel: Standard
  f'c    = 4 ksi                              fy     = 60 ksi
  Ec     = 3605 ksi                           Es     = 29000 ksi
  fc     = 3.4 ksi                            Eps_yt = 0.00206897 in/in
  Eps_u = 0.003 in/in
  Beta1 = 0.85

Section:
========
  Exterior Points
   No.     X (in)     Y (in)    No.     X (in)     Y (in)    No.     X (in)     Y (in)
  ----- ---------- ---------- ----- ---------- ---------- ----- ---------- ----------
     1       -5.0     -108.0     2         5.0     -108.0     3         5.0      108.0
     4       -5.0      108.0

  Gross section area, Ag =  2160 in^2
  Ix =  8.39808e+006 in^4                     Iy =  18000 in^4
  rx =  62.3538 in                            ry =  2.88675 in
  Xo =  0 in                                  Yo =  -0 in

Reinforcement:
==============
  Bar Set: ASTM A615
  Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
  ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
  #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
  #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
  #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
  # 14      1.69        2.25   # 18      2.26        4.00

  Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
  phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

  Pattern: Irregular
  Total steel area: As = 8.06 in^2 at rho = 0.37% (Note: rho < 0.50%)
  Minimum clear spacing = 7.37 in

  Area in^2  X (in)   Y (in)  Area in^2  X (in)   Y (in)  Area in^2  X (in)   Y (in)
  --------- -------- -------- --------- -------- -------- --------- -------- --------
       0.31     -4.0    107.0      0.31      4.0    107.0      0.31      4.0     89.2
       0.31      4.0     71.3      0.31      4.0     53.5      0.31      4.0     35.7
       0.31      4.0     17.8      0.31      4.0     -0.0      0.31      4.0    -17.8
       0.31      4.0    -35.7      0.31      4.0    -53.5      0.31      4.0    -71.3
       0.31      4.0    -89.2      0.31      4.0   -107.0      0.31     -4.0   -107.0
       0.31     -4.0    -89.2      0.31     -4.0    -71.3      0.31     -4.0    -53.5
       0.31     -4.0    -35.7      0.31     -4.0    -17.8      0.31     -4.0     -0.0
       0.31     -4.0     17.8      0.31     -4.0     35.7      0.31     -4.0     53.5
       0.31     -4.0     71.3      0.31     -4.0     89.2

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Factored Loads and Moments with Corresponding Capacities:
=========================================================
                Pu          Mux       PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.          kip         k-ft         k-ft               in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
   1        207.00      4670.00      5319.18    1.139    20.73   215.00  0.02811  0.900

  *** End of output ***
```

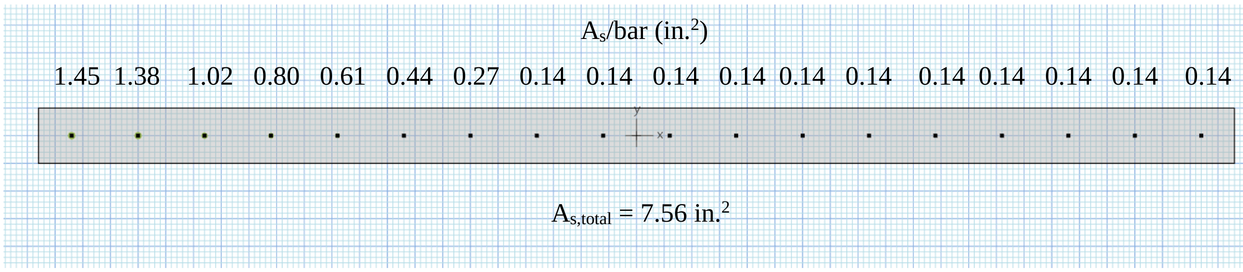
Figure 12 – Load & Moment Capacities Output from spColumn

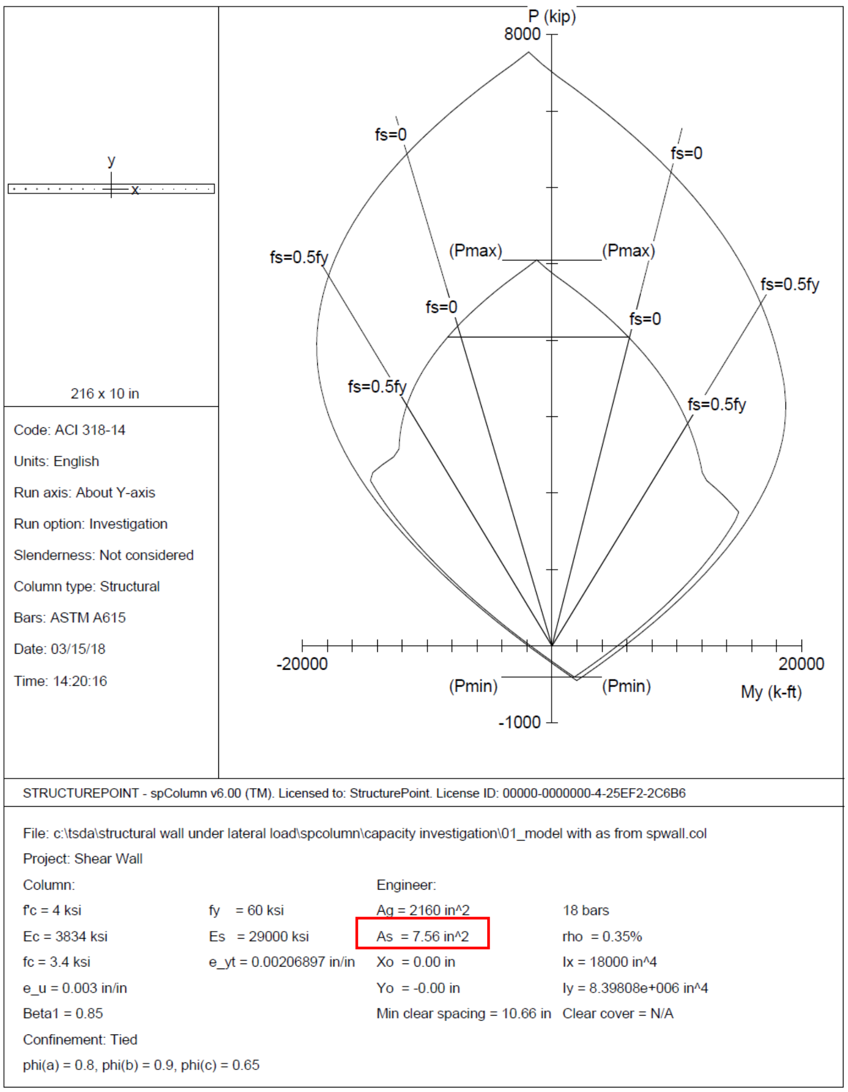
Using spColumn, calculate the expected wall capacity based on various reinforcement distributions obtained from the FEA results from spWall. Three reinforcement distributions are evaluated below.

Table 2 - Comparative capacity calculations (using spColumn) based on FEA suggested reinforcement distribution		
Reinforcement Distribution	c, in.	ϕM_n, kip-ft
Reference	20.73	5,319 > 4,800* (110.8%)
Non-Uniform	22.54	6,824 > 4,800* (142.2%)
Uniform	20.63	5,064 > 4,800* (105.5%)
Suggested	22.52	6,744 > 4,800* (140.5%)
* Wall flexural capacity calculated using simplified reference method		

Wall Capacity – Non-Uniform Reinforcement from FEA

Using the method of solution in spColumn where one section is used the finite element analysis model can be investigated as one section and not as individual finite elements as calculated by spWall.



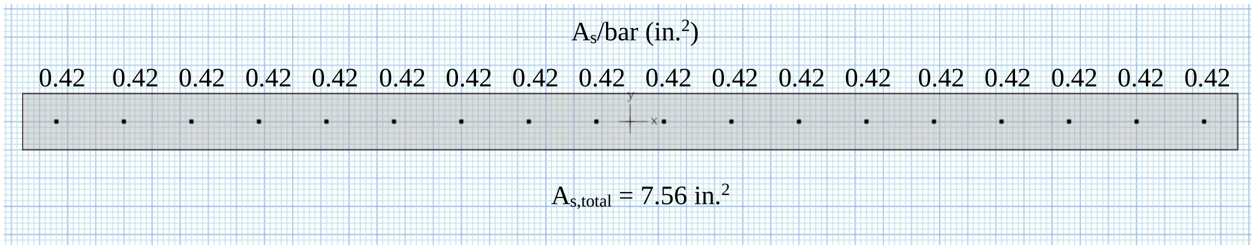


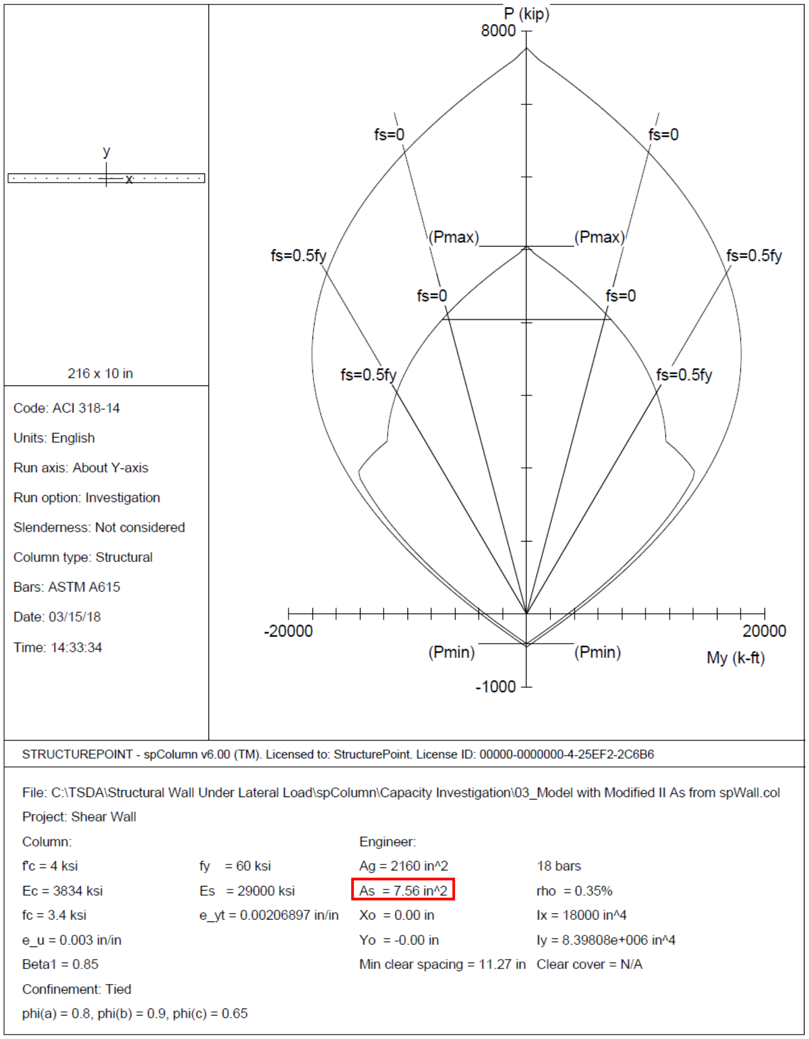
Axial Loads and Moments with Corresponding Capacities

No	ϕPn	ϕMny	NA Depth	dt Depth	εt	ϕ
	kip	k-ft	in	in		
1	207.0	6824.06	22.540	210.000	0.02495	0.900
2	207.0	-3354.73	15.809	210.000	0.03685	0.900

Wall Capacity – Uniform Reinforcement from FEA

Taking the total area of non-uniform reinforcement obtained from FEA and redistributing it in a uniform bar pattern to represent a reinforcement arrangement very comparable to the reference example distribution, the wall capacity can be calculated and is expected to be very similar to the results obtained from the reinforcement configuration used by the reference.



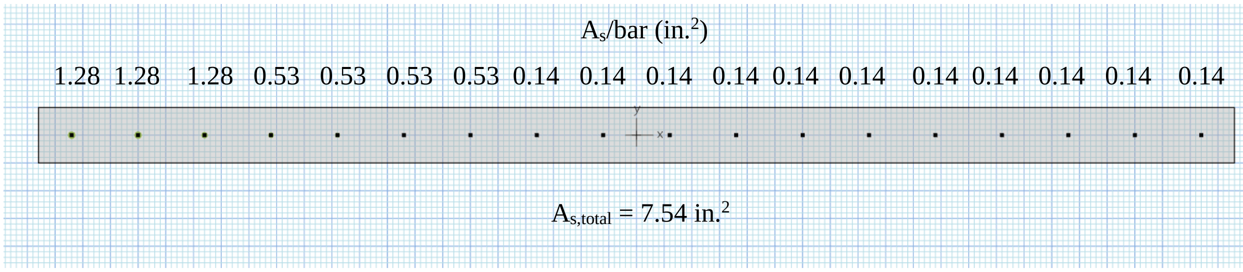


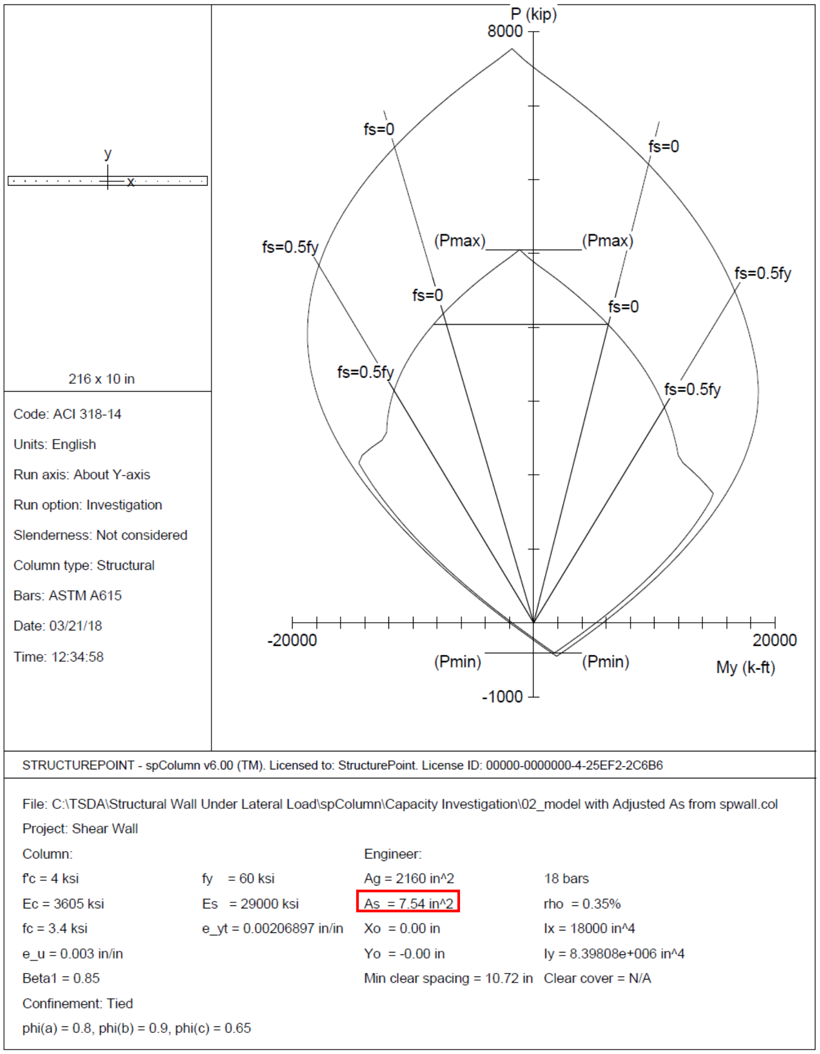
Axial Loads and Moments with Corresponding Capacities

No	ϕPn	ϕMny	NA Depth	dt Depth	εt	ϕ
	kip	k-ft	in	in		
1	207.0	5063.52	20.629	210.000	0.02754	0.900
2	207.0	-5063.52	20.629	210.000	0.02754	0.900

Wall Capacity – Suggested Reinforcement

Taking the total area of non-uniform reinforcement obtained from FEA and redistributing it in a banded approach where the suggested reinforcement is averaged over the first 3 elements and the following 4 elements resulting in the suggested bar pattern below to represent a practical reinforcement arrangement, a new wall capacity can be calculated.





Axial Loads and Moments with Corresponding Capacities

No	ϕPn	ϕMny	NA Depth	dt Depth	εt	ϕ
	kip	k-ft	in	in		
1	207.0	6744.11	22.516	210.000	0.02498	0.900
2	207.0	-3415.01	16.392	210.000	0.03543	0.900

Conclusions & Observations:

As can be seen from the three options above the engineers can evaluate several options when arriving at the reinforcing bar arrangement from an FEA model. The following conclusions and observations can be used to better understand designing and investigating shear walls using spWall:

1. In finite element analysis, selecting mesh size has a crucial impact on the results accuracy (as an example the amount and distribution of reinforcement). The mesh size should be optimized in a way that changing the element size has slight effect on the results obtained. However, the optimum element size is dependent on multiple parameters in the model which makes it difficult to find a generalized procedure to select the optimum size. Multiple studies conducted by StructurePoint showed that the element length should not be greater than 10% of the total wall length and a coarser mesh should be used with caution and engineering judgement.

2. spWall calculates the required area of steel for each element along the section. This area of steel is selected in a way that it should be enough to satisfy the strength requirements under a specific sets of extreme design forces. This approach will lead to placing most of the reinforcement at wall section ends as was shown in this example leading to the highest possible flexural capacity that can be achieved for the section with the same amount of steel. In practice, having a uniform distribution of reinforcement along the wall section is more common and the flexural capacity of the concrete wall is usually calculated based on it.

3. Concrete Shear walls can be analyzed and designed using simplified structural analysis approaches as the one used in this example. However, as the level of complexity of the wall increases, analyzing and designing shear walls using hand solution become more challenging and less effective. Computer software utilizing FEA (e.g. spWall) is an efficient solution to analyze and design concrete shear walls regardless of the level of complexity. spWall selects the minimum required area of steel with the optimum reinforcement distribution for the wall section in which the highest bending capacity of the wall section is achieved. spColumn software can be also utilized to obtain the wall interaction diagram to help better understand the behavior of the section selected.