



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







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

- Weight of 10DT24 = 468 plf
- Roof dead load = 20 psf
- Roof live load = 30 psf
- Wind load = 30 psf

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





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Code

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

Reference

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

Design Data

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

 $f_y = 60,000 \text{ psi}$

Wall length = 20 ft

Assumed wall thickness = 8 in.

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

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





1. Minimum Vertical Reinforcement

$$\rho_{l} = \frac{A_{v,vertical}}{h \times s_{1}} = \frac{0.27}{12 \times 8} = 0.0028$$

$$ACI 318-14 (2.2)$$

$$\rho_{l,min} = 0.0012$$

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

$$s_{l,max} = \text{smallest of } \begin{cases} 3 \times h \\ 18 \text{ in.} \end{cases} = \text{smallest of } \begin{cases} 3 \times 8 \\ 18 \text{ in.} \end{cases} = \text{smallest of } \begin{cases} 3 \times 8 \\ 18 \text{ in.} \end{cases} = \text{smallest of } \begin{cases} 24 \text{ in.} \\ 18 \text{ in.} \end{cases} = 18 \text{ in.}$$

$$ACI 318-14 (11.7.2.1)$$

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

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

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

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

ACI 318 requires that concentrated gravity loads applied to the wall above the design flexural section shall be assumed to be distributed over a width:

 ACI 318 requires that concentrated gravity loads applied to the wall above the design flexural section shall be

- 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.

Distribution Width of Concentrated Loads = min $\begin{cases} W + \frac{l_c}{2} \\ \text{spacing of the concentrated loads} \end{cases}$

ACI 318-14 (11.8.2.2)

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

3. Wall Structural Analysis

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

3.1. Roof load per foot width of wall

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

 $D = \left[\frac{468}{2} + (20 \times 5)\right] \times \left(\frac{60}{2}\right) = 10,020 \text{ lbs / 5 ft} = 2,004 \text{ plf}$
 $L = (30 \times 5) \times \left(\frac{60}{2}\right) = 4,500 \text{ lbs / 5 ft} = 900 \text{ plf}$

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

3.2. Calculation of maximum wall forces

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







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

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

$$M_{u} = \frac{M_{ua}}{1 - \frac{5 \times P_{u} \times l_{c}^{2}}{0.75 \times 48 \times E_{c} \times I_{cr}}}$$

$$\underline{ACI 318-14 (11.8.3.1d)}$$

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

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

$$E_{c} = 57,000 \times \sqrt{f_{c}'} = 57,000 \times \sqrt{4,000} = 3,605,000 \text{ psi}$$

$$ACI 318-14 (19.2.2.1.b)$$

$$I_{cr} = n \times A_{se,w} \times (d-c)^{2} + \frac{l_{w} \times c^{3}}{3}$$

$$n = \frac{E_{s}}{E_{c}} = \frac{29,000}{3,605} = 8.0 > 6.0 \text{ (o.k.)}$$

$$ACI 318-14 (11.8.3.1)$$

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

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

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



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





ACI 318-14 (Table 21.2.2)

ACI 318-14 (Table 21.2.2)

ACI 318-14 (11.8.3.1d)

Therefore, section is tension controlled

$$\phi = 0.9$$

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

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

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} , inkips	E _c , ksi	n	A _{se,w} , in. ² /ft	a, in.	c, in.	I _{cr} , in. ⁴	ε _t , in./in.	ф	M _u , inkips	
1.4 D	4.21	3.74	3,605	8	0.34	0.50	0.58	32.4	0.0176	0.9	5.25	
$1.2 \text{ D} + 1.6 \text{ L}_r + 0.8 \text{ W}$	5.04	19.53	3,605	8	0.35	0.52	0.61	33.4	0.0168	0.9	29.38	
$1.2 \text{ D} + 0.5 \text{ L}_{r} + 1.6 \text{ W}$	4.05	32.61	3,605	8	0.33	0.49	0.58	32.3	0.0178	0.9	45.22	
0.9 D + 1.6 W	2.70	31.21	3,605	8	0.31	0.46	0.54	30.7	0.0193	0.9	38.80	

3.3. Tension-controlled verification

ACI 318-14 (11.8.1.1(b))

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

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

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

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

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

Therefore, section is tension controlled

ACI 318-14 (Table 21.2.2)

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4. Wall Cracking Moment Capacity (Mcr)

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

$$f_r = 7.5\lambda \sqrt{f_c'} = 7.5 \times 1.0 \times \sqrt{4,000} = 474.3 \text{ psi}$$

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

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

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{474.3 \times 512}{4} \times \frac{1}{1000} = 60.72 \text{ in.-kip}$$

ACI 318-14 (19.2.3.1)

<u>ACI 318-14 (24.2.3.5b)</u>

ACI 318-14 (11.5.1.1(b))

ACI 318-14 (11.8.1.1(c))

5. Wall Flexural Moment Capacity (ϕM_n)

For load combination #1:

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

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

$$\phi M_n = \phi \times M_n = 0.9 \times 75.82 = 68.24$$
 in.-kip > $M_n = 5.25$ in.-kips (o.k.)

 $\phi M_n = 68.24 \text{ in.-kip} > M_{cr} = 60.72 \text{ in.-kips} (0.k.)$

Table 2 - Design moment strength check M_n, Load Combination 11.5.1.1(b) ø φM_n, in.-kips M_u, in.-kips Mcr, in.-kips 11.8.1.1(c) in.-kips 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 W78.61 0.9 70.75 $29.38 < \phi M_n$ o.k. $60.72 < \phi M_n$ o.k. 75.29 1.2 D + 0.5 Lr + 1.6 W0.9 67.76 $45.22 < \phi M_n$ o.k. $60.72 < \phi M_n$ o.k. 70.53 0.9 0.9 D + 1.6 W $38.80 < \phi M_n$ $60.72 < \phi M_n$ 63.47 o.k. o.k.



6. Wall Vertical Stress Check

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

$$\frac{P_u}{A_g} = \frac{5,044.8}{8 \times 12} = 52.55 \text{ psi} < 0.06 \times f_c' = 0.06 \times 4,000 = 240 \text{ psi} \text{ (o.k.)}$$

$$\underline{ACI 318-14 (11.8.1.1(d))}$$

7. Wall Shear Stress Check

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

8. Wall Mid-Height Deflection (Δ_s)

11

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

$$\Delta_{s} = \begin{cases} \frac{2}{3}\Delta_{cr} + \frac{M_{a} - \frac{2}{3}M_{cr}}{M_{n} - \frac{2}{3}M_{cr}} \times \left(\Delta_{n} - \frac{2}{3}\Delta_{cr}\right) & \text{When} & M_{a} > \frac{2}{3}M_{cr} \\ \left(\frac{M_{a}}{M_{cr}}\right)\Delta_{cr} & \text{When} & M_{a} < \frac{2}{3}M_{cr} \end{cases} \end{cases}$$

$$\underbrace{ACI 318-14 (Table 11.8.4.1)}_{ACI 318-14 (Table 11.8.4.1)}$$

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

$$\begin{split} M_{a} &= M_{sa} + P_{s}\Delta_{s} \\ M_{sa} &= \frac{w_{s} \times l_{c}^{2}}{8} + \frac{P_{s1} \times e}{2} = \frac{0.030 \times (20)^{2}}{8} + \frac{(2.0 + 0.9) \times 2.7/12}{2} = 1.82 \text{ ft-kips} = 21.87 \text{ in-kips} \\ P_{s} &= P_{s1} + \frac{P_{s2}}{2} = (2.004 + 0.9) + \frac{2.0}{2} = 3.9 \text{ kips} \\ M_{cr} &= \frac{f_{r}I_{g}}{y_{t}} = 60.72 \text{ in-kip (as calculated perviously)} \\ \Delta_{cr} &= \frac{5}{48} \times \frac{M_{cr} \times l_{c}^{2}}{E_{c} \times I_{g}} = \frac{5}{48} \times \frac{60.72 \times (20 \times 12)^{2}}{3,605 \times 512} = 0.197 \text{ in.} \end{split}$$

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





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

Assume $\Delta_s = \left(\frac{M_{sa}}{M_{cr}}\right)\Delta_{cr} = \left(\frac{21.87}{60.72}\right) \times 0.197 = 0.071$ in.
 $M_a = M_{sa} + P_s\Delta_s = 21.87 + 3.9 \times 0.071 = 22.15$ in.-kips

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

No further iterations are required

$$M_{a} = 22.15 \text{ in.-kips} < \frac{2}{3}M_{cr} = \frac{2}{3} \times 60.72 = 40.48 \text{ in.-kips} \quad (\textbf{o.k.})$$
$$\Delta_{s} = 0.072 \text{ in.} < \frac{l_{c}}{150} = \frac{20 \times 12}{150} = 1.60 \text{ in.} \quad (\textbf{o.k.})$$

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

Structure Point

CONCRETE SOFTWARE SOLUTIONS



9. Precast Concrete Bearing Wall Panel Analysis and Design – <u>spWall</u> Software

<u>spWall</u> 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 <u>spWall</u>, 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, <u>spWall</u> 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 <u>spWall</u> 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.





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Figure 3 – spWall Interface







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







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





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







Figure 7 - Solve and Mesh Options (spWall)





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AC	31	8-14							4	.84, 19.53 (f	t) -		÷. •		Units:	English 🔻

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

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AC	1 318-1	4							1	.02, 18.27 (ft)	-		÷ -		Units:	English 🔻

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





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=↑				✓ Min/N	lax only							_			- er	
≡			Force	Size	200 %	1 1 1										
AC	318-	14								0.07, 20.97	'(ft) -		· ta · b	5	Units:	English 🔻

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





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





9 [5	<u>⊳ 8</u> א	Ċ,				:	spWall - Beari	ing Wall.wab	¢					- 0	×
File		Home														^
Pro	ो ट्र	Define	∰ Grid	↓ Select	Plates	Stiffeners	-i Nodes	 Restraints	↓ Loads	Solve	Results	Tables	Reporter	ジニ Display	Viewports	දිටු Settings
	RES	SULTS					Wall Cr	oss-Sectiona	l Forces - N	/lux - 1.2D+1	1.6Lr+0.8W	(kip-ft)				• ×
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	~	Wall Cross-Se	ctional Force	25										_		26,
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		✓ Mux 1.2D+	1.6Lr+0.8W											_		-q
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AC	1318	5-14							-0.	99, 21.02 (ft)					Units	English ₹

Figure 12 – Bearing Wall Moment Diagram (spWall)







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		100 C





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1. Project

1.1. General Information

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

1.2. Solver Options

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

2. Definitions

2.1. Grid Lines

2.1.1. Vertical

Spacing	Coordinate-X	Label	
ft	ft		
0.00	0.00	А	
5.00	5.00	В	

2.1.2. Horizontal

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

2.2. Objects

2.2.1. Plates

Label Thickness 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	Wc	Ec	v	Precast	Used
	ksi	pcf	ksi	-		
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 Cor	nbinations	Ultimate Co	mbinations	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

			Reinforcement Ratio				R				
Label	Curtains	Flags	Rmin (Hor)	Rmax (Hor)	Rmin (Ver)	Rmax (Ver)	Back H. (BH)	Back V. (BV)	Front H. (FH)	Front V. (FV)	Used
			%	%	%	%	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

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

2.5. Load Case/Combo.

2.5.1. Load Cases

NOTE: Self weight is included under Case A.

Case	Туре	Case Label	Load Defined?
A	Dead	DEAD	Yes
В	Live	LIVE	Yes
С	Wind	WIND	Yes

2.5.2. Load Combinations

Combo./Case	Α	В	С	D	E	F	G	н	I	Combo Type
Туре	Dead	Live	Wind							
Combo./Label	DEAD	LIVE	WIND							
1.2D+1.6L	1.000	1.000	1.000	-	-		-	-	-	Ser.
1.2D+1.6L	1.200	1.600	0.800	-	-	-	-	-	-	Ult.

3. Assignments

3.1. Nodes

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

3.2. Plates

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



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3.3. Stiffeners

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

3.4. Point Loads

		-	-	-				_
Nodes ID	Case	FX	Fy	FZ	Mx	Му	Mz	Ecc.
		kips	kips	kips	kip-ft	kip-ft	kip-ft	in
N1	А	0.00	-10.02	0.00	0.00	0.00	0.00	2.70
	в	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	С	0.00	0.00	-30.00

4. Results

4.1. Envelope

4.1.1. Plate Flexure Reinforcement

Coordinate System: Global

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





E	Element	Curtains	Direction	Mu (x/y)	Nu (x/y) Ld Comb.	As (x/y)	Rho
				kip-ft/ft	klf	in²/ft	%
			Vertical	2.44	-5.01 1.2D+1.6Lr+0	0.269	0.28
	205	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.44	-5.02 1.2D+1.6Lr+0	0.269	0.28
	206	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.44	-5.02 1.2D+1.6Lr+0	0.269	0.28
	207	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0	0.192	0.20
dheight			Vertical	2.44	-5.01 1.2D+1.6Lr+0	0.269	0.28
0.00 ft)	208	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.44	-5.01 1.2D+1.6Lr+0	0.269	0.28
	209	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.44	-5.01 1.2D+1.6Lr+0	0.269	0.28
	210	1	Horizontal	0.49	-1.00 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.44	-5.01 1.2D+1.6Lr+0	0.269	0.28
	211	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.95 1.2D+1.6Lr+0	0.269	0.28
	212	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.95 1.2D+1.6Lr+0	0.269	0.28
	213	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.95 1.2D+1.6Lr+0	0.269	0.28
	214	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.96 1.2D+1.6Lr+0	0.269	0.28
	215	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2 45	-4 96 1 2D+1 6Lr+0	0.269	0.28
	216	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.96 1.2D+1.6Lr+0	0.269	0.28
	217	1	Horizontal	0.49	-0.99 1.2D+1.6Lr+0	0.192	0.20
	2.17		Vertical	2 45	-4 96 1 2D+1 6Lr+0	0.269	0.28
	218	1	Horizontal	0.49	-0.99.1.2D+1.6Lr+0	0 192	0.20
	210		Vertical	2.45	-4 95 1 2D+1 6Lr+0	0.269	0.28
	219	1	Horizontal	0.49	-0.99 1 2D+1.6Lr+0	0.192	0.20
	210		Vertical	2 45	-4 95 1 2D+1 6Lr+0	0.269	0.28
critical	220	1	Horizontal	0.49	-0.99 1 2D+1.6Lr+0	0.192	0.20
section	220		Vertical	2 45	-4 95 1 2D+1 6Lr+0	0.269	0.28
1 00 ft)	221	1	Horizontal	0.49	-0.98 1 2D+1.6Lr+0	0.192	0.20
1.00 10)	221		Vertical	2.45	-4.89 1 2D+1.6Lr+0	0.269	0.28
	222	1	Horizontal	0.49	-4.03 1.2D+1.6Lr+0	0.102	0.20
	222		Vertical	2.45	-4.89 1 2D+1.6Lr+0	0.269	0.20
	222	1	Ventical	2.45	-4.09 1.2D+1.0Er+0	0.209	0.20
	223		Vertical	0.49	-0.98 1.2D+1.6L+0	0.192	0.20
	224		Ventical	2.45	-4.69 1.20+1.6L+0	0.209	0.20
	224	1	Honzontal	0.49	-0.98 1.2D+1.6Lr+0	0.192	0.20
	005		Vertical	2.45	-4.90 1.2D+1.6LF+0	0.269	0.28
	225	1	Horizontal	0.49	-0.98 1.2D+1.6LF+0	0.192	0.20
			Vertical	2.45	-4.90 1.2D+1.6Lr+0	0.269	0.28
	226	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.90 1.2D+1.6Lr+0	0.269	0.28
	227	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.90 1.2D+1.6Lr+0	0.269	0.28
	228	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.89 1.2D+1.6Lr+0	0.269	0.28
	229	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.89 1.2D+1.6Lr+0	0.269	0.28
	230	1	Horizontal	0.49	-0.98 1.2D+1.6Lr+0	0.192	0.20
			Vertical	2.45	-4.89 1.2D+1.6Lr+0	0.269	0.28

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

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





4.2. Service 4.2.1. Nodal Displacements

4.2.1.1. 1.2D+1.6Lr+0.8W

Coordinate System: Global

	Node	Dx	Dy	Dz	
		in	in	in	
	221	0.000	-0.001	-0.072	
	222	0.000	-0.001	-0.072	
	223	0.000	-0.001	-0.072	
	224	0.000	-0.001	-0.072	
	225	0.000	-0.001	-0.072	
at midheight	226	0.000	-0.001	-0.072	$D_{z,average} = 0.072$ in.
(v - 10.00 ft)	227	0.000	-0.001	-0.072	-,
() = 10.00 I()	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	
at critical	247	0.000	-0.002	-0.071	
acertion	248	0.000	-0.002	-0.071	$D_{z average} = 0.071$ in.
section	249	0.000	-0.002	-0.071	
(y = 11.00 ft)	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	

4.3. Ultimate

4.3.1. 4.3.1. Coordi	Plate Inter 1. 1.2D+1.6 nate System: 0	mal Forces SLr+0.8W Slobal	$f_{yy,avg} = 5.04 \text{ k}$	ips	My	_{y,avg} = 2.43 kij	p-ft/ft
	Element	Nxx	Nyy	Nxy	Mxx	Муу	Мху
		klf	klf	klf	kip-ft/ft	kip-ft/ft	kip-ft/f
	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
at midheig	ht 195	-1.01	-5.07	0.00	0.48	2.42	0.00
(y = 10.00 f	t) 196	-1.01	-5.07	0.00	0.48	2.42	0.00
-	197	-1.01	-5.07	0.00	0.48	2.42	0.00
	198	-1.01	-5.07	0.00	0.48	2.42	0.00

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El	ement	Nxx	Nvv	Nxv	Mxx	Mvv	Mxv
		klf	klf	klf	kip-ft/ft	kip-ft/ft	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
at midheight	205	-1.00	-5.02	0.00	0.49	2.44	0.00
(y = 10.00 ft)	206	-1.00	-5.02	0.00	0.49	2.44	0.00
	207	-1.00	-5.01	0.00	0.49	2.44	0.00
	208	-1.00	-5.01	0.00	0.49	2.44	0.00
	209	-1.00	-5.01	0.00	0.49	2.44	0.00
	210	-1.00	-5.01	0.00	0.49	2.44	0.00
	211	-0.99	-4.95	0.00	0.49	2.45	0.00
	212	-0.99	-4.95	0.00	0.49	2.45	0.00
	213	-0.99	-4.95	0.00	0.49	2.45	0.00
	214	-0.99	-4.96	0.00	0.49	2.45	0.00
	215	-0.99	-4.96	0.00	0.49	2.45	0.00
	216	-0.99	-4.96	0.00	0.49	2.45	0.00
	217	-0.99	-4.96	0.00	0.49	2.45	0.00
	218	-0.99	-4.95	0.00	0.49	2.45	0.00
	219	-0.99	-4.95	0.00	0.49	2.45	0.00
at critical	220	-0.99	-4.95	0.00	0.49	2.45	0.00
section	221	-0.98	-4.89	0.00	0.49	2.45	0.00
(11.00 ft)	222	-0.98	-4.89	0.00	0.49	2.45	0.00
(y = 11.00 ft)	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

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

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





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5. Screenshots 5.1. Extrude 3D view







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5.2. Plates & Stiffeners ID







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5.3. Nodes ID







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5.4. Restraints







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5.5. Loads - Case A - DEAD







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5.6. Loads - Case B - LIVE







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5.7. Loads - Case C - WIND





Table 3 – Comparison of Precast Wall Panel Analysis and Design Results (2 nd 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. 									

10. Design Results Comparison and Conclusions

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

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

ACI 318-14 (6.6.3.1.1)

In <u>spWall</u> 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-ofplane (bending and torsion) and in-plane (axial and shear) stiffness can be entered for plate elements. Because the values of the cracking coefficients can have a large effect on the analysis and design results, the user must take care in selecting values that best represent the state of cracking at the particular loading stage. Cracking coefficients are greater than 0 and less than 1.

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

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

Based on the previous discussion, the ratio between I_{cr} and I_g including a reduction factor of 0.75 is used as the cracking coefficient for the out-of-plane case for the ultimate load combinations. In this example, I_{cr} and I_g were found to be





equal to 33.38 in.⁴ and 512 in.⁴ (Load Combination #2). Thus, the out-of-plane cracking coefficient for ultimate load combinations can be found as follows:

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

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



Figure 13 – Defining Cracking Coefficient (spWall)





In <u>spWall</u>, 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} , inkips		
	Hand	<u>spWall</u>	
1.4 D	3.74	3.78	
$1.2 \text{ D} + 1.6 \text{ L}_r + 0.8 \text{ W}$	19.53	19.56	
$1.2 \text{ D} + 0.5 \text{ L}_r + 1.6 \text{ W}$	32.61	32.64	
0.9 D + 1.6 W	31.20	31.14	

SOLVE			
	Ē	Lun	
Run Options	Analysis & Design 🔹		
Include 2nd Order Effects	No *		Select "No" to perform
Max. allowed Out-of-Plane Deflection	H/150 *		first order analysis
User Defined allowed Deflection	279.400	in	
 Check Concrete Shear Street (ACI with Solid walls only) Use Simplified Equations Use Detailed Equations 	ngth of Wall Crossection Is (uniform walls only)	ns	

Figure 14 – Solver Module (spWall)