Precast Concrete Bearing Wall Panel Design (Alternative Design Method) (Using ACI 318-11)
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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 spWall engineering software program from StructurePoint.

![Figure 1 – Reinforced Concrete Precast Wall Panel Geometry](image-url)
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Code

Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)

Reference


spWall Engineering Software Program Manual v5.01, STRUCTUREPOINT, 2016

Design Data

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

$f_y = 60,000$ psi

Wall length = 20 ft

Assumed wall thickness = 8 in.

Assumed vertical reinforcement: single layer of #4 bars at 9 in. ($A_{s,\text{vertical}} = 0.20 / 9 \text{ in.} \times 12 \text{ in.} = 0.27 \text{ in.}^2/\text{ft}$)
1. Minimum Vertical Reinforcement

\[
p_i = \frac{A_{v, \text{vertical}}}{h \times s_i} = \frac{0.27}{12 \times 8} = 0.0028
\]

\[
\rho_{i,\text{min}} = 0.0012
\]

\[
\rho_i = 0.0028 \geq \rho_{i,\text{min}} = 0.0012 \ (\text{o.k})
\]

\[
s_{i,\text{max}} = \text{smallest of } \frac{3 \times h}{18 \text{ in.}} = \text{smallest of } \frac{3 \times 8}{18 \text{ in.}} = \text{smallest of } \frac{24 \text{ in.}}{18 \text{ in.}} = 18 \text{ in.}
\]

\[
s_{i,\text{provided}} = 9 \text{ in.} \leq s_{i,\text{max}} = 18 \text{ in.} \ (\text{o.k.})
\]

2. Alternative Design Method Applicability

Precast concrete walls can be analyzed using the provisions of Chapter 14 of the ACI 318. Most walls, and especially slender walls, are widely evaluated using the “Alternative design of slender walls” in Section 14.8. The requirements of this procedure are summarized below:

- The cross section shall be constant over the height of the wall \textit{ACI 318-11 (14.8.2.2)}
- The wall can be designed as simply supported \textit{ACI 318-11 (14.8.2.1)}
- Maximum moments and deflections occurring at midspan \textit{ACI 318-11 (14.8.2.1)}
- The wall must be axially loaded \textit{ACI 318-11 (14.8.2.1)}
- The wall must be subjected to an out-of-plane uniform lateral load \textit{ACI 318-11 (14.8.2.1)}
- The wall shall be tension-controlled \textit{ACI 318-11 (14.8.2.3)}
- The reinforcement shall provide design strength greater than cracking strength \textit{ACI 318-11 (14.8.2.4)}

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:

\[
\text{Distribution Width of Concentrated Loads} = \min \left\{ \frac{W + \frac{l}{2}}{\text{spacing of the concentrated loads}} \right\}
\]

\[
\text{ACI 318-11 (14.8.2.5)}
\]

\[
\text{Distribution Width of Concentrated Loads} = \min \left\{ \frac{3.75 + \frac{20}{2}}{5.0 \text{ ft}} = 10.3 \text{ ft} \right\} = 5.0 \text{ ft}
\]
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 = \( \frac{8}{12} \times 20 \times 150 = 2,000 \text{ plf} \)

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

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

Eccentricity of the roof loads about the panel center line = \( \frac{2}{3} \times 4 = 2.7 \text{ in.} \)

3.2. Calculation of maximum wall forces

The calculation of maximum factored wall forces in accordance with 14.8.3 is summarized in Figure 2 including moment magnification due to second order (P-\( \Delta \)) effects.

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

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

\[
P_u = P_{u1} + \frac{P_{u2}}{2} = (1.4 \times 2.004) + \frac{(1.4 \times 2.000)}{2} = 4.2 \text{ kips}
\]
\[ M_{u} = \frac{M_{u}}{1 - \frac{5 \times P_t \times l_c^2}{0.75 \times 48 \times E_t \times I_{cr}}} \]  
\[ M_{u} = \frac{w_u \times l_c^2}{8} + \frac{P_t \times e}{2} = \frac{0.08 \times (20 \times 12)^2}{8} + \frac{2.8 \times 2.7}{2} = 3.8 \text{ in.-kips} \]

Where \( M_{u} \) is the maximum factored moment at midheight of wall due to lateral and eccentric vertical loads, not including \( P \Delta \) effects.  
\[ ACI\ 318-11\ (Eq.\ 14-6) \]

\[ E_t = 57,000 \times \sqrt{f'_c} = 57,000 \times \sqrt{4,000} = 3,605,000 \text{ psi} \]  
\[ ACI\ 318-11\ (8.5.1) \]

\[ I_{cr} = n \times A_{cr,n} \times (d - c)^2 + \frac{I_{w}}{3} \]  
\[ ACI\ 318-11\ (Eq.\ 14-7) \]

\[ n = \frac{E_t}{E_c} = \frac{29,000}{3,605} = 8.0 > 6.0 \ (\text{o.k.}) \]  
\[ ACI\ 318-11\ (14.8.3) \]

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

\[ A_{cr,n} = A_s + \frac{P_t \times h}{2 \times f_s \times d} = 0.27 + \frac{4.2 \times 8}{2 \times 60 \times 4} = 0.34 \text{ in.}^2/\text{ft} \]  
\[ ACI\ 318-11\ (R14.8.3) \]

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

\[ a = \frac{A_{cr,n} \times f_s}{0.85 \times f_c \times l_w} = \frac{0.34 \times 60}{0.85 \times 4 \times 12} = 0.50 \text{ in.} \]

\[ c = \frac{a}{\beta_l} = \frac{0.50}{0.85} = 0.59 \text{ in.} \]

\[ I_{cr} = 8.0 \times 0.34 \times (4 - 0.59)^2 + \frac{12 \times 0.59^3}{3} = 32.5 \text{ in.}^4 \]  
\[ ACI\ 318-11\ (Eq.\ 14-7) \]

\[ \varepsilon_s \left( \frac{0.003}{c} \right) \times d_t - 0.003 = \left( \frac{0.003}{0.59} \right) \times 4.0 - 0.003 = 0.0173 > 0.005 \]

Therefore, section is tension controlled  
\[ ACI\ 318-11\ (10.3.4) \]

\[ \phi = 0.9 \]  
\[ ACI\ 318-11\ (9.3.2) \]
\[ M_u = \frac{M_{su}}{1 - \frac{5 \times P_u \times I_c^2}{0.75 \times 48 \times E \times I_{cr}}} \]

*ACI 318-11 (Eq. 14-6)*

\[ M_u = \frac{3.9}{1 - \frac{5 \times 4.2 \times (20 \times 12)^2}{0.75 \times 48 \times 3,605 \times 32.5}} = 5.4 \text{ in.-kips} \]

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

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>( P_u ), kips</th>
<th>( M_{iu} ), in.-kips</th>
<th>( E_c ), ksi</th>
<th>( n )</th>
<th>( A_{sc, w} ), in.(^2)/ft</th>
<th>( a ), in.</th>
<th>( c ), in.</th>
<th>( I_{cr} ), in.(^4)</th>
<th>( \epsilon_t ), in./in.</th>
<th>( \phi )</th>
<th>( M_iu ), in.-kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4 D</td>
<td>4.2</td>
<td>3.8</td>
<td>3,605</td>
<td>8</td>
<td>0.34</td>
<td>0.50</td>
<td>0.59</td>
<td>32.5</td>
<td>0.0173</td>
<td>0.9</td>
<td>5.4</td>
</tr>
<tr>
<td>1.2 D + 1.6 L + 0.8 W</td>
<td>5.0</td>
<td>19.2</td>
<td>3,605</td>
<td>8</td>
<td>0.35</td>
<td>0.51</td>
<td>0.60</td>
<td>33.2</td>
<td>0.0170</td>
<td>0.9</td>
<td>28.8</td>
</tr>
<tr>
<td>1.2 D + 0.5 L +1.6 W</td>
<td>4.1</td>
<td>32.4</td>
<td>3,605</td>
<td>8</td>
<td>0.34</td>
<td>0.50</td>
<td>0.59</td>
<td>32.5</td>
<td>0.0173</td>
<td>0.9</td>
<td>45.0</td>
</tr>
<tr>
<td>0.9 D + 1.6 W</td>
<td>2.7</td>
<td>31.2</td>
<td>3,605</td>
<td>8</td>
<td>0.32</td>
<td>0.47</td>
<td>0.55</td>
<td>31.1</td>
<td>0.0188</td>
<td>0.9</td>
<td>38.7</td>
</tr>
</tbody>
</table>

3.3. Tension-controlled verification

*ACI 318-11 (14.8.2.3)*

For this check use the largest \( P_u \) (5.0 kips) from load combination 2 to envelop all the considered combinations.

\[ P_u = \frac{P}{\phi} = \frac{5.0}{0.9} = 5.56 \text{ kips} \]

\[ a = \frac{A_{sc, w} \times f_y}{0.85 \times f_y \times I_w} = \frac{P_u \times h}{2 \times d} + \frac{A_t \times f_y}{0.85 \times f_y \times I_w} = \frac{5.56 \times 8}{2 \times 4} + \frac{0.27 \times 60}{0.85 \times 4 \times 12} = 0.533 \text{ in.} \]

\[ c = \frac{a}{\beta_h} = \frac{0.533}{0.85} = 0.627 \text{ in.} \]

\[ \epsilon_t = \left( \frac{0.003}{c} \right) \times d_f = \left( \frac{0.003}{0.627} \right) \times 4.0 = 0.003 = 0.016 > 0.005 \]

Therefore, section is tension controlled  

*ACI 318-11 (10.3.4)*

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

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

\[ f_r = 7.5 \times \sqrt{f_y} = 7.5 \times 1.0 \times \sqrt{4,000} = 474.3 \text{ psi} \]

\[ I_g = \frac{l_h h^3}{12} = \frac{12 \times 8^3}{12} = 512 \text{ in.}^4 \]
\[ y_i = \frac{h}{2} = \frac{8}{2} = 4 \text{ in.} \]

\[ M_n = \frac{f_y I_s}{y_i} = \frac{474.3 \times 512}{4} \times \frac{1}{1000} = 60.7 \text{ in.-kip} \]

5. Wall Flexural Moment Capacity (\(\phi M_n\))

For load combination #1:

\[ M_n = A_{f,se} \times f_y \times \left( d - \frac{a}{2} \right) = 0.34 \times 60 \times \left( 4 - \frac{0.5}{2} \right) = 76.5 \text{ in.-kip} \]

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

\[ \phi M_n = \phi \times M_n = 0.9 \times 76.5 = 68.9 \text{ in.-kip} > M_n = 5.4 \text{ in.-kips (o.k.)} \]

\[ ACI 318-11 (14.8.3) \]

\[ \phi M_n = 68.9 \text{ in.-kip} > M_{cr} = 60.7 \text{ in.-kips (o.k.)} \]

\[ ACI 318-11 (14.8.2.4) \]

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>(M_n), in.-kips</th>
<th>(\phi)</th>
<th>(\phi M_n), in.-kips</th>
<th>(M_{cr}), in.-kips</th>
<th>14.8.3</th>
<th>14.8.2.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4 D</td>
<td>76.5</td>
<td>0.9</td>
<td>68.9</td>
<td>5.4 &lt; (\phi M_n)</td>
<td>o.k.</td>
<td>o.k.</td>
</tr>
<tr>
<td>1.2 D + 1.6 Lr + 0.8 W</td>
<td>78.7</td>
<td>0.9</td>
<td>70.8</td>
<td>28.8 &lt; (\phi M_n)</td>
<td>o.k.</td>
<td>o.k.</td>
</tr>
<tr>
<td>1.2 D + 0.5 Lr +1.6 W</td>
<td>76.5</td>
<td>0.9</td>
<td>68.9</td>
<td>45.0 &lt; (\phi M_n)</td>
<td>o.k.</td>
<td>o.k.</td>
</tr>
<tr>
<td>0.9 D + 1.6 W</td>
<td>72.3</td>
<td>0.9</td>
<td>65.1</td>
<td>38.7 &lt; (\phi M_n)</td>
<td>o.k.</td>
<td>o.k.</td>
</tr>
</tbody>
</table>
6. Wall Vertical Stress Check

Since load combination 2 provides the largest \( P_u \) (5.0 kips), load combination 2 controls.

\[
\frac{P_u}{A_g} = \frac{5,000}{8 \times 12} = 52.1 \text{ psi} < 0.06 \times f_c' = 0.06 \times 4,000 = 240 \text{ psi (o.k.)} \quad \text{ACI 318-11 (14.8.2.6)}
\]

7. 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 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 8 for detailed shear force diagram)

8. Wall Mid-Height Deflection \((\Delta_s)\)

The maximum out-of-plane deflection \((\Delta_s)\) due to service lateral and eccentric vertical loads, including P\(\Delta\) effects, shall not exceed \(l_c/150\). Where \(\Delta_s\) is calculated as follows: \quad \text{ACI 318-11 (14.8.4)}

\[
\Delta_s = \begin{cases} 
\frac{2}{3} \Delta_s + \frac{M_u - \frac{2}{3} M_{u\sigma}}{M_u - \frac{2}{3} M_{u\sigma}} \left( \frac{2}{3} \Delta_s \right) & \text{when } M_u > \frac{2}{3} M_{u\sigma} \\
\frac{M_u}{M_{u\sigma}} \Delta_s & \text{when } M_u \leq \frac{2}{3} M_{u\sigma}
\end{cases} \quad \text{ACI 318-11 (14.8.4)}
\]

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

\[
M_u = M_{u\sigma} + P_s\Delta_s
\]

\[
M_{u\sigma} = \frac{w_s \times l_s^2}{8} + \frac{P_{sl} \times e}{2} = \frac{0.030 \times (20)^2}{8} + \frac{(2.0 + 0.9) \times 2.7 / 12}{2} = 1.8 \text{ ft-kips} = 21.6 \text{ in.-kips}
\]

\[
P_s = P_{sl} + \frac{P_{e\sigma}}{2} = (2.004 + 0.9) + \frac{2.0}{2} = 3.9 \text{ kips}
\]

\[
M_{u\sigma} = \frac{f_c' I_s}{y_s} = 60.7 \text{ in.-kip (as calculated perviously)} \quad \text{ACI 318-11 (Eq. 9-9)}
\]

\[
\Delta_s = \frac{5}{48} \times \frac{M_{u\sigma} \times l_s^2}{E_s \times I_s} = \frac{5}{48} \times \frac{60.7 \times (20 \times 12)^2}{3,605 \times 512} = 0.20 \text{ in.} \quad \text{ACI 318-11 (Eq. 14-10)}
\]

\(\Delta_s\) will be calculated by trial and error method since \(\Delta_s\) is a function of \(M_s\) and \(M_s\) is a function of \(\Delta_s\).
Assume $M_{sa} < \frac{2}{3} M_{cr}$

Assume $\Delta_s = \left( \frac{M_{sa}}{M_{cr}} \right) \Delta_{cr} = \begin{pmatrix} 21.6 \\ 60.7 \end{pmatrix} \times 0.20 = 0.07$ in.

$M_s = M_{sa} + P \Delta_s = 21.6 + 3.9 \times 0.07 = 21.9$ in.-kips

$\Delta_s = \left( \frac{M_{sa}}{M_{cr}} \right) \Delta_{cr} = \begin{pmatrix} 21.9 \\ 60.7 \end{pmatrix} \times 0.20 = 0.07$ in.  

No further iterations are required

$M_s = 21.9$ in.-kips $< \frac{2}{3} M_{cr} = \frac{2}{3} \times 60.7 = 40.5$ in.-kips  (o.k.)

$\Delta_s = 0.07$ in. $< \frac{l}{150} = \frac{20 \times 12}{150} = 1.60$ in.  (o.k.)

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

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

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

- Wall geometry (including any number of openings and stiffeners)
- Material properties including cracking coefficients
- Wall loads (point, line, and area),
- 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-slimmer 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 forces.
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 318-11 is used in this example), and the user can specify one or two layers of wall reinforcement. In stiffeners and boundary elements, spWall calculates the required shear and torsion steel reinforcement. Wall concrete strength (in-plane and out-of-plane) is calculated for the applied loads and compared with the code permissible shear capacity.

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

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

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

Figure 3 – Defining Loads for Precast Wall Panel (spWall)
Figure 4 – Assigning Boundary Conditions for Precast Wall Panel (spWall)
Figure 5 – Factored Axial Forces Contour Normal to Precast Wall Panel Cross-Section (spWall)
Figure 6 – Precast Wall Panel Lateral Displacement Contour (Out-of-Plane) (spWall)
Figure 7 – Precast Wall Panel Axial Load Diagram (spWall)
Figure 8 – Out-of-plane Shear Diagram (spWall)
Figure 9 – Shear Wall Moment Diagram (spWall)
Figure 10 – Precast Wall Panel Vertical Reinforcement (spWall)

Figure 11 – Precast Wall Panel Cross-Sectional Forces (spWall)
Figure 12 – Precast Wall Panel Required Reinforcement (spWall)

<table>
<thead>
<tr>
<th>Elem</th>
<th>Curtains</th>
<th>Direction</th>
<th>Mu (k/N)</th>
<th>Nu (k/N)</th>
<th>Ld Combo</th>
<th>As (x/y)</th>
<th>%</th>
<th>Tie</th>
</tr>
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<tbody>
<tr>
<td>221</td>
<td>1</td>
<td>Horizontal</td>
<td>4.4202e+000</td>
<td>-9.7969e+000</td>
<td>1.2D+1.6</td>
<td>1.92e-001</td>
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<td>227</td>
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<td>Horizontal</td>
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<td>-9.7851e+000</td>
<td>1.2D+1.6</td>
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<td>Vertical</td>
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<tr>
<td>229</td>
<td>1</td>
<td>Horizontal</td>
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<td>1.2D+1.6</td>
<td>1.92e-001</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>230</td>
<td>1</td>
<td>Vertical</td>
<td>4.4202e+000</td>
<td>-9.7969e+000</td>
<td>1.2D+1.6</td>
<td>1.92e-001</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>231</td>
<td>1</td>
<td>Horizontal</td>
<td>4.4030e+000</td>
<td>-9.6825e+000</td>
<td>1.2D+1.6</td>
<td>1.92e-001</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vertical</td>
<td>4.2035e+000</td>
<td>-8.3333e+000</td>
<td>1.2D+1.6</td>
<td>1.92e-001</td>
<td>0.20</td>
<td></td>
</tr>
</tbody>
</table>

$A_{avg} = 0.27$ in.$^2$
**10. Design Results Comparison and Conclusions**

<table>
<thead>
<tr>
<th>Solution</th>
<th>$M_u$ (kip-ft)</th>
<th>$N_u$ (kips)</th>
<th>$A_{s,vertical}$ (in.$^2$)</th>
<th>$D_z$ (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hand</td>
<td>2.40</td>
<td>5.0</td>
<td>0.27</td>
<td>0.072</td>
</tr>
<tr>
<td>Reference</td>
<td>2.40</td>
<td>5.0</td>
<td>0.27</td>
<td>0.072</td>
</tr>
<tr>
<td>spWall</td>
<td>2.21</td>
<td>4.9</td>
<td>0.27</td>
<td>0.072</td>
</tr>
</tbody>
</table>

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

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

*ACI 318-11 (10.10.4.1)*

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 $I_{cr}/I_{gross}$ 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.

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 $I_g$ 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 32.5 in.$^4$ and 512 in.$^4$. Thus, the out-of-plane cracking coefficient for ultimate load combinations can be found as follows:

$$\alpha = \text{cracking coefficient} = \frac{I_{cr}}{I_g} = \frac{32.5}{512} = 0.06348$$
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 21.9 in.-kips which is less than $M_{cr} = 60.7$ in.-kips. That means the section is uncracked and the cracking coefficient can be taken equal to 1.

![Figure 13 – Defining Cracking Coefficient (spWall)](image)

In spWall, first-order or second-order analysis can be performed to obtain the design moment. In this model, the second order effects were included in order to compare the results with the reference and 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>
<thead>
<tr>
<th>Load Combination</th>
<th>$M_{ua}$, in.-kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hand &amp; Reference</td>
<td>spWall</td>
</tr>
<tr>
<td>1.4 D</td>
<td>3.8</td>
</tr>
<tr>
<td>1.2 D + 1.6 L + 0.8 W</td>
<td>19.2</td>
</tr>
<tr>
<td>1.2 D + 0.5 L + 1.6 W</td>
<td>32.4</td>
</tr>
<tr>
<td>0.9 D + 1.6 W</td>
<td>31.2</td>
</tr>
<tr>
<td></td>
<td>31.1</td>
</tr>
</tbody>
</table>
Figure 14 – Solver Module (spWall)