Design an I-shaped structural wall consisting of a web with two columns at its end to carry gravity and seismic forces.

**Section:** Web with Columns
- 234" x 16" web
- 30" x 30" columns

Wall height: \( H_w = 150.0 \) ft

Design displacement: \( \delta u = 14.5 \) in

Concrete: \( f'_c = 4.0 \) ksi

Mild steel: \( f_y = 60 \) ksi, \( E_s = 29,000 \) ksi

Clear cover: 1.5" for the column, 0.75" for the web

Code: ACI 318-02

Loads:
- Dead: \( P = 2,870 \) kips
- Live: \( P = 400 \) kips
- EQ: \( P = 0.0, M_x = 49,200 \) ft-k, \( V_y = 815 \) kips

Load combinations:
- 1.2D
- 1.2D + 16L
- 12 D + 0.5 L ± 10 E
- 12 D ± 10 E

The problem is solved using the program in the Design run mode. The reinforcement suggested by the program is as follows:

**Web:** No. 5 @ 12” Vert + No. 5 @ 12” Horiz
Within boundary, 2-No. 5 ties @ 3”

**Columns:** 24 No. 9 Vert + No. 4 ties and cross-ties @ 5”
Load-Moment Capacity:
The following table lists, for each load combination, the factored loads, moments and shears. The corresponding effective depth, \( d \), and the neutral axis depth corresponding to the nominal moment strength, \( C_n \), are also listed.

<table>
<thead>
<tr>
<th>Combo</th>
<th>( Pu ) (kip)</th>
<th>( Mu ) (ft-K)</th>
<th>( Vu ) (kip)</th>
<th>( d ) (in)</th>
<th>( C_n ) (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>U1 = 12D</td>
<td>4,018</td>
<td>0.0</td>
<td>0.0</td>
<td>254.8</td>
<td>69.39</td>
</tr>
<tr>
<td>U2 = 12 D + 6 L</td>
<td>4,084</td>
<td>0.0</td>
<td>0.0</td>
<td>255.3</td>
<td>70.55</td>
</tr>
<tr>
<td>U3 = 12 D + 0.5 L</td>
<td>3,644</td>
<td>0.0</td>
<td>0.0</td>
<td>252.5</td>
<td>62.78</td>
</tr>
<tr>
<td>U4 = 12 D + 0.5 L + 10 E</td>
<td>3,644</td>
<td>49,200</td>
<td>815</td>
<td>252.5</td>
<td>62.78</td>
</tr>
<tr>
<td>U5 = 12 D + 0.5 L - 10 E</td>
<td>3,644</td>
<td>-49,200</td>
<td>-815</td>
<td>252.5</td>
<td>62.78</td>
</tr>
<tr>
<td>U6 = 12 D + 11E</td>
<td>3,644</td>
<td>49,200</td>
<td>815</td>
<td>25125</td>
<td>59.35</td>
</tr>
<tr>
<td>U7 = 12 D - 11E</td>
<td>3,644</td>
<td>-49,200</td>
<td>-815</td>
<td>25125</td>
<td>59.35</td>
</tr>
</tbody>
</table>

Looking at the P-M interaction diagram of the section, all of the load points fall well within the capacity curve. It is clear that the suggested reinforcement is sufficient to carry the factored axial loads and bending moments.
Shear Design:

$Lw = 234 + 30 + 30 = 294$ in.

$Hw = 150 \times 12 = 1800$ in

Spacing of web vertical bars, $Sv = 12$ in, $< S_{v \text{max}} = 18$ in, Ok

Spacing of web horizontal bars, $Sh = 12$ in, $< S_{h \text{max}} = 18$ in, Ok

\[ \rho_h = \frac{(2 \times 0.31)}{(16 \times 12)} = 0.0032 > \rho_{h \text{min}} = 0.0025, \text{Ok} \]

\[ \rho_v = \frac{(2 \times 0.31)}{(16 \times 12)} = 0.0032 > \rho_{v \text{min}} = 0.0025, \text{Ok} \]

\[ 2 Lw \sqrt{f/c} = 2 \times 294 \times 16 \times \sqrt{4000 / 1000} = 559 \text{ kips} \]

$V_{\text{max}} = \text{Max factored shear force} = 815$ kips $> 559$ kips.

Two curtains of reinforcement must be provided within the web.

Nominal shear strength:

\[ V_n = (t Lw)(\alpha_c \sqrt{f/c} + \rho_h f_y) \]

\[ Hw / Lw = 1800 / 294 = 6.1 > 2, \alpha_c = 2.0 \]

\[ V_n = (16 \times 294)(2.0 \times \sqrt{4000 / 1000} + 0.0032 \times 60) = 1506 \text{ kips} \]

Max allowable shear = $8 Lw \sqrt{f/c} = 8 \times 294 \times 16 \times \sqrt{4000 / 1000} = 2380$ kips

$V_n < \text{Max allowable, Ok.}$

$Hw / Lw = 1800 / 294 = 6.12 > 2.0$, use $\phi = 0.75$ for shear.

$\phi V_n = 0.75 V_n = 1129.8$ kips $> V_{\text{max}} = 815$ kips, Ok
Boundary Element Check:
For the seismic load combinations (U4 through U7), the neutral axis depth corresponding
to the nominal moment strength, Cn, is listed in the table above.
Cmax = 62.78 in (largest value of Cn)
Climit = Lw / (600 δu / Hw)
Climit = 294 / (600 x 14.5 / 1800) = 60.83 in
Cmax > Climit. Therefore, special boundary elements are required.

The vertical extent of the boundary element is the larger of:
Lw = 294 in (governs)
Mu / 4Vu = 49,200 x 12 / (4 x 815) = 181.1 in

The horizontal extent of the boundary element is the largest of:
Cmax / 2 = 62.78 / 2 = 31.4 in
Cmax - 0.1 Lw = 62.78 - 0.1 x 294 = 33.4 in
12 in into the web = 30 + 12 = 42 in (governs)
Confine at least 42 in at each end of the wall section.
Boundary Element Confinement:
Columns: \( \rho = \frac{(24 \times 10)}{(30 \times 30)} = 0.027 \)

Center-to-center spacing of bars on each face = \( \frac{(30 - 2x15 - 1.128 - 2x0.5)}{6} = 4.15 \) in.  
Clear bar spacing in each direction = \( 4.15 - 1.128 = 3.02 \) in. < 6 in.

Provide a tie or cross-tie at every corner and alternate longitudinal bar. With 24 bars, we have 5 bars on each face (excluding the corner bars). A total of 4-No. 4 ties are provided in each direction. \( N_x = N_y = 4. \)

Center-to-center spacing of two consecutive ties = \( 2 \times 4.15 + 1128 + 0.5 = 9.92 \) in < 14 in., Ok

\[
A_{sh} = 0.09 \left( Sh \cdot Hc \cdot f'c / fy \right)
\]
\[Sh = 5.0\text{"}, Hc = 30 - 2 \times 15 - 0.5 = 26.5\text{"} \]
\[
A_{sh} = 0.09 \times 5.0 \times 26.5 \times 4 / 60 = 0.795 \text{ in}^2 
\]
Provided \( A_v = 4 \times 0.31 = 1.24 \text{ in}^2 > A_{sh}, \text{Ok} \)

Max allowable spacing, \( S_{max} \), is the lesser of:
\[
\begin{align*}
0.25 \times 30 & = 7.5 \text{ in} \\
6 \times \text{Vert bar diameter} & = 6 \times 1128 = 6.8 \text{ in} \\
S_x & = 4 + (14 - h_x) / 3 = 4 + (14 - 9.92) / 3 = 5.35 \text{ in} \quad \text{(governs)} \\
6.0 & \text{ in.}
\end{align*}
\]
The provided spacing is less than \( S_{max} \), Ok.

Web zone within boundary element:
Extent of boundary element into web = 42 - 30 = 12 in. By inspection, there are 2-No. 5 bars within this distance. Since the bar spacing is greater than 6 in, one No. 5 tie is provided at each bar.

Center-to-center spacing of two consecutive ties in the y-direction = \( 12 + 0.625 + 0.625 = 13.25 \) in < 14 in., Ok
Center-to-center spacing of ties (horizontal bars) in the x-direction = \( 16 - 2 \times 0.75 - 0.625 = 13.875 \) in < 14 in., Ok

Confinement in the y-direction:
\[
A_{sh} = 0.09 \left( Sh \cdot Hc \cdot f'c / fy \right)
\]
\[Sh = 3.0 \text{ in}, Hc = 16 - 2 \times 0.75 - 0.625 = 13.875 \text{ in} \\
A_{sh} = 0.09 \times 3.0 \times 13.875 \times 4 / 60 = 0.25 \text{ in}^2 \\
Provided \( A_v = 2 \times 0.31 = 0.62 \text{ in}^2 > A_{sh}, \text{Ok} \)
Confinement in the x-direction:

\[ Ash = 0.09 \left( Sh Hc f'c / fy \right) \]
\[ Sh = 3.0 \text{ in}, \quad Hc = 12 + 0.625 + 0.625 = 13.25 \text{ in} \]
\[ Ash = 0.09 \times 3.0 \times 13.25 \times 4 / 60 = 0.238 \text{ in}^2 \]

Provided \( Av = 2 \times 0.31 = 0.62 \text{ in}^2 > Ash, \text{Ok} \)

Max allowable spacing, \( S_{max} \), is the least of:

\[ 0.25 \times 16 = 4.0 \text{ in} \]
\[ 0.25 \times 234 = 58.5 \text{ in} \]
\[ 6 \times \text{Vert bar diameter} = 6 \times 0.625 = 3.75 \text{ in (governs)} \]
\[ S_x = 4 + (14 - hx) / 3 = 4 + (14 - 13.25) / 3 = 4.25 \text{ in} \]

6.0 in.

The provided spacing is less than \( S_{max} \), Ok.

No. 5 @ 12"
No. 5 @ 3"
24-No. 9

No. 4 hoops and ties @ 5"
Development Length and Splice Length:
For No. 9 bars in the columns, the lap splice length is 13 times the development length of the bar (assuming Class B splice).
\[
L_d = \alpha \beta \lambda \delta \left( \frac{f_y}{\sqrt{f'_c}} \right) \left( \frac{3}{40} \right) \frac{d_b}{C_d}
\]
\[
\alpha = \text{location factor} = 10
\]
\[
\beta = \text{coating factor} = 10
\]
\[
\lambda = \text{size factor} = 10 \text{ for No. 9 bars}
\]
\[
\delta = \text{lightweight concrete factor} = 10
\]
\[
c = \text{smaller of cover (15 + 0.625 + 1128 / 2 = 2.69 in) or half the spacing (4.1 / 2 = 2.05 in)}
\]
\[
C_d = \frac{c}{d_b} = \frac{2.05}{1.128} = 1.82
\]
\[
L_d = \frac{60}{\sqrt{4,000/1,000}} \left( \frac{3}{40} \right) \frac{1.128}{1.82} = 44.1 \text{ in} > 12 \text{ in}
\]
\[
\text{Splice length} = 1.3 \times 44.1 = 57.4 \text{ in}
\]

For No. 5 vertical bars in the web, the lap splice length is 13 times the development length of the bar (assuming Class B splice).
\[
L_d = \alpha \beta \lambda \delta \left( \frac{f_y}{\sqrt{f'_c}} \right) \left( \frac{3}{40} \right) \frac{d_b}{C_d}
\]
\[
\alpha = \beta = \delta = 1.0
\]
\[
\lambda = \text{size factor} = 0.8 \text{ for No. 5 bars}
\]
\[
c = \text{smaller of cover (0.75 + 0.625 + 0.625 / 2 = 1.6875 in) or half the spacing (12 / 2 = 6.0 in)}
\]
\[
C_d = \frac{c}{d_b} = \frac{1.69}{0.625} = 2.7 > 2.5, \text{ use 2.5}
\]
\[
L_d = 0.8 \times 60 \left( \sqrt{4,000/1,000} \right) \left( \frac{3}{40} \right) \frac{0.625}{2.5} = 14.2 \text{ in} > 12 \text{ in}
\]
\[
\text{Splice length} = 13 \times 14.2 = 185.4 \text{ in}
\]

For No. 5 horizontal bars in the web, the development length assuming a hooked bar is:
\[
L_{dh} = \frac{d_b f_y}{65 \sqrt{f'_c}} = \frac{0.625 \times 60}{65 \times \sqrt{4,000/1,000}} = 9.12 \text{ in}
\]
\[
L_{dh} > 8 \text{ db} = 8 \times 0.625 = 5 \text{ in, Ok}
\]
\[
L_{dh} > 6 \text{ in, Ok}
\]

For No. 5 horizontal bars in the web, the development length assuming a straight bar is 3.5
\[
L_{dh} \text{ computed in step above} = 3.5 \times 9.12 = 31.9 \text{ in}
\]
Settings and Options:
File: C:\StrucTools\Data\StrucWall\Verification_01.dat
Title: Program verification
Example 1
Code: ACI 318-02 Units: English Seismic design provisions

Material Properties:
\( f'c(ksi) = 4 \) \( E_c(ksi) = 3605 \) Beta1 = 0.850
\( fy(ksi) = 60 \) \( E_s(ksi) = 29000 \) ASTM A615

Section:
Web \( W_1(\text{in}) = 16.00 \) \( H_1(\text{in}) = 234.00 \)
Columns \( W_2(\text{in}) = 30.00 \) \( H_2(\text{in}) = 30.00 \)
\( H_w(\text{ft}) = 150.00 \) \( L_w(\text{in}) = 294.00 \)

Seismic Design displacement = 14.5 in
Web
No.5 @ 12.00 in Horiz. Clear cover = 0.75 in
No.5 @ 12.00 in Vert. 2 layers
Min Ratio = 0.20%. Max Ratio = 2.00%
Clear spacing is larger of 1.00 Db or 1.00 in
Ratio = 0.33%. Least clear spacing: \( C_x(\text{in}) = 12.00, C_y(\text{in}) = 11.38 \)
Columns
No.4 @ 5.00 in Horiz. Clear cover = 1.50 in
24 No.9 Vert.
Min Ratio = 0.50%. Max Ratio = 6.00%
Clear spacing is larger of 1.50 Db or 1.50 in
Ratio = 2.67%. Least clear spacing: \( C_x(\text{in}) = 3.02, C_y(\text{in}) = 3.02 \)

Ag(in^2) = 5544 \( I_x(\text{in}^4) = 4.85821e+007 \) \( I_y(\text{in}^4) = 214872 \)

Load Cases and Combinations:

Service Loads:
Case P(kip) Mx(k-ft) Vy(kip)
Dead 2870 0 0
Live 400 0 0
EQ 0 49200 815

Factored Loads:

Web shear:
Max Vu = 815,00 kip < Phi*Vn = 1129.81 kip
Max Pu = 3644.00 kip [U4], Mn = 76479.14 k-ft, PhiV = 0.75

Seismic provisions:
Largest neutral axis depth = 62.95 in > 60.83 in
Special confinement zone is required.
Horizontal extent = 42.00 in from extreme compression fiber.
Vertical extent of special confinement zone = 24.500 ft.
End zone confinement reinforcement:

Required area of transverse reinf: $A_{shX} = 0.79 \text{ in}^2$ (4 ties provided).

$A_{shY} = 0.79 \text{ in}^2$ (4 ties provided).

Maximum allowable spacing of transverse reinf, $S_{max} = 5.36 \text{ in}$

Special zone within web:

2 No.5 at 3.00 in (horizontal)
2 No.5 at 3.00 in (crossties)

Lap splice for No.9 vertical bars in end zone = 56.78 in

Lap splice for No.5 vertical bars in web = 18.50 in

Development length for No.5 straight horizontal bars in web = 31.93 in

Development length for No.5 hooked horizontal bars in web = 9.12 in

Available length for development of web horizontal bars = 24.87 in
16 x 234 + 30 x 30
Web: No.5 @ 12.00 V, No.5 @ 12.00 H
Columns: 24 No.9 V, No.4 @ 5.00 H