CSA-S16-09 Example-001

HANDBOOK OF STEEL CONSTRUCTION, TENTH EDITION - COMPOSITE BEAM

EXAMPLE DESCRIPTION
Design a simply supported composite beam to span 12 m and carry a uniformly distributed specified load of 18 kN/m live load and 12 kN/m dead load. Beams are spaced at 3 m on center and support a 75 mm steel deck (ribs perpendicular to the beam) with a 65 mm cover slab of 25 MPa normal density concrete. Calculations are based on $F_y = 345$ MPa. Live load deflections are limited to L/300.

GEOMETRY, PROPERTIES AND LOADING

Member Properties
- W460x74
- $E = 205,000$ MPa
- $F_y = 345$ MPa

Loading
- $w = 8.0$ kN/m (Dead Load)
- $w = 2.5$ kN/m (Construction)
- $w = 4.0$ kN/m (Superimposed Load)
- $w = 18.00$ kN/m (Live Load)

Geometry
- Span, L = 12 m

TECHNICAL FEATURES OF ETABS TESTED

Composite beam design, including:
- Selection of steel section, camber and shear stud distribution
- Member bending capacities, at construction and in service
- Member deflections, at construction and in service
RESULTS COMPARISON

Independent results are referenced from the design example on page 5-25 of the *Handbook of Steel Construction, Tenth Edition*.

<table>
<thead>
<tr>
<th>Output Parameter</th>
<th>ETABS</th>
<th>Independent</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction Design Moment (kN-m)</td>
<td>247.4</td>
<td>247.5</td>
<td>0.04%</td>
</tr>
<tr>
<td>Construction M_s (kN-m)</td>
<td>512.3</td>
<td>512.3</td>
<td>0.00%</td>
</tr>
<tr>
<td>Construction Deflection (mm)</td>
<td>32.4</td>
<td>32.4</td>
<td>0.00%</td>
</tr>
<tr>
<td>Design Moment (kN-m)</td>
<td>755.8</td>
<td>756</td>
<td>0.02%</td>
</tr>
<tr>
<td>Full Composite M_rc (kN-m)</td>
<td>946.7</td>
<td>946.7</td>
<td>0.00%</td>
</tr>
<tr>
<td>Partial Composite M_rc (kN-m)</td>
<td>783.6</td>
<td>783.6</td>
<td>0.00%</td>
</tr>
<tr>
<td>Shear Stud Capacity Q_s (kN)</td>
<td>68.7</td>
<td>68.7</td>
<td>0.00%</td>
</tr>
<tr>
<td>Shear Stud Distribution</td>
<td>30</td>
<td>30</td>
<td>0.00%</td>
</tr>
<tr>
<td>Live Load Deflection (mm)</td>
<td>32.9</td>
<td>32.9</td>
<td>0.00%</td>
</tr>
<tr>
<td>Bottom Flange Tension (MPa)</td>
<td>267.2</td>
<td>267.1</td>
<td>0.04%</td>
</tr>
<tr>
<td>Design Shear Force V_f (kN)</td>
<td>251.9</td>
<td>251.9</td>
<td>0.00%</td>
</tr>
<tr>
<td>Shear Resistance V_r (kN)</td>
<td>842.9</td>
<td>842.9</td>
<td>0.00%</td>
</tr>
</tbody>
</table>

COMPUTER FILE: CSA-S16-09 EXAMPLE 001.edb

CONCLUSION

The ETABS results show an acceptable comparison with the independent results.
HAND CALCULATION

Properties:

Materials:
ASTM A992 Grade 50 Steel
\[ E = 200,000 \text{ MPa}, \quad F_y = 345 \text{ MPa}, \quad \gamma_s = 7850 \text{ kg/m}^3 \]
Normal weight concrete
\[ E = 23,400 \text{ MPa}, \quad f_{cu} = 20 \text{ MPa}, \quad \gamma_c = 2300 \text{ kg/m}^3 \]

Section:
W460x74
\[ d = 457 \text{ mm}, \quad b_f = 190 \text{ mm}, \quad t_f = 14.5 \text{ mm}, \quad t_w = 9 \text{ mm}, \quad T = 395 \text{ mm}, \quad r_{fille} = 16.5 \text{ mm} \]
\[ A_s = 9,450 \text{ mm}^2, \quad Z_s = 1,650 \times 10^3 \text{ mm}^3, \quad I_s = 333 \times 10^6 \text{ mm}^4 \]

Deck:
\[ t_c = 65 \text{ mm}, \quad h_r = 75 \text{ mm}, \quad s_r = 300 \text{ mm}, \quad w_r = 150 \text{ mm} \]

Shear Connectors:
\[ d = 19 \text{ mm}, \quad h = 115 \text{ mm}, \quad F_u = 450 \text{ MPa} \]

Loadings:
Self weight slab = 2.42 kN/m²
Self weight beam = 0.73 kN/m
Construction load = 0.83 kN/m²
Superimposed dead load = 1.33 kN/m²
Live load = 6.0 kN/m²

Design for Pre-Composite Condition:
Construction Required Flexural Strength:
\[ w_{fconstruction} = 1.25 \times 0.73 + (1.25 \times 2.42 + 1.5 \times 0.83) \times 3.0 = 13.75 \text{ kN/m} \]
\[ M_{fconstruction} = \frac{w_{fconstruction} \times L^2}{8} = \frac{13.75 \times 12^2}{8} = 247.5 \text{ kN-m} \]
Moment Capacity:

\[ M_x = Z_x \cdot 0.9 \cdot F_y = 1.650 \cdot 10^3 \cdot 0.9 \cdot 345 \cdot 10^{-6} = 512.3 \text{ kN-m} \]

Pre-Composite Deflection:

\[ w_{\text{construction}} = 2.42 \cdot 3.0 + 0.73 = 8.0 \text{ kN/m} \]
\[ \delta = \frac{5 \cdot w_{\text{construction}} \cdot L^4}{384 \cdot E \cdot I} = \frac{5 \cdot 8.0 \cdot 12,000^4}{384 \cdot 200,000 \cdot 33,300 \cdot 10^4} = 32.4 \text{ mm} \]

Camber = 0.8 \cdot \delta = 25.9 \text{ mm, which is rounded down to 25 mm}

Design for Composite Flexural Strength:

Required Flexural Strength:

\[ w_f = 1.25 \cdot 0.73 + (1.25 \cdot 2.42 + 1.25 \cdot 1.33 + 1.5 \cdot 6) \cdot 3.0 = 42 \text{ kN/m} \]
\[ M_f = \frac{w_f \cdot L^2}{8} = \frac{42 \cdot 12^2}{8} = 756.0 \text{ kN-m} \]

Full Composite Action Available Flexural Strength:

Effective width of slab:

\[ b_l = \frac{L}{4} = \frac{12,000}{4} = 3,000 \text{ mm} \leq 3,000 \text{ mm} \]

Resistance of slab in compression:

\[ \alpha_1 = 0.85 - 0.0015 \cdot f'_{c} = 0.8125 \]
\[ C'_{r} = \alpha_1 \cdot \Phi_c \cdot t \cdot b_f \cdot f'_{c} = 0.8125 \cdot 0.65 \cdot 65 \cdot 3,000 \cdot 25 \cdot 10^{-3} = 2,574 \text{ kN controls} \]

Resistance of steel in tension:

\[ \Phi \cdot A_s \cdot F_y = 0.9 \cdot 9,450 \cdot 345 \cdot 10^{-3} = 2,934 \text{ kN} \]

Depth of compression block within steel section top flange:

\[ x = \frac{(\Phi \cdot A_s \cdot F_y - C'_{r})/2}{\Phi \cdot F_y \cdot b_f} = \frac{(2,934 - 2,547) \cdot 10^3/2}{0.9 \cdot 345 \cdot 190} = 3.05 \text{ mm} \]
Moment resistance of composite beam for full composite action:

\[ M_{rc} = C' \left( h + \frac{t_c}{2} + \frac{x}{2} \right) + \Phi \cdot A_s \cdot F_y \left( \frac{d - x}{2} \right) \]

\[ = 2,574 \left( 75 + \frac{65}{2} + \frac{3}{2} \right) \cdot 10^{-3} + 2,934 \left( \frac{457}{2} - \frac{3}{2} \right) \cdot 10^{-3} = 946.7 \text{ kN-m} \]

Partial Composite Action Available Flexural Strength:

Assume 40.0% composite action:

\[ Q_r = 0.4 \cdot R_c = 0.4 \cdot 2,574 = 1,031 \text{ kN} \]

Depth of compression block within concrete slab:

\[ a = \frac{Q_r}{\alpha_t \cdot \Phi_c \cdot b_{eff} \cdot f'_c} = \frac{1,031 \cdot 10^3}{0.8125 \cdot 0.65 \cdot 3,000 \cdot 25} = 26 \text{ mm} \]

Compression force within steel section:

\[ C_r = (P_y - Q_r)/2 = (2,934 - 1,031)/2 = 951.6 \text{ kN} \]

Tensile resistance of one flange:

\[ F_{flange} = \Phi \cdot f_s \cdot t_f \cdot F_y = 0.9 \cdot 190 \cdot 14.5 \cdot 345 \cdot 10^{-3} = 855.4 \text{ kN} \]

Tensile resistance of web:

\[ F_{web} = \Phi \cdot T \cdot t_w \cdot F_y = 0.9 \cdot 395 \cdot 9 \cdot 345 \cdot 10^{-3} = 1,103.8 \text{ kN} \]

Tensile resistance of one fillet area:

\[ F_{fillet} = (P_y - 2 \cdot F_{flange} - F_{web})/2 = (2,934 - 2 \cdot 855.4 - 1,103.8)/2 = 59.8 \text{ kN} \]

Compression force in web:

\[ C_{web} = C_r - F_{flange} - F_{fillet} = 951.6 - 855.4 - 59.7 = 36.4 \text{ kN} \]

Depth of compression block in web:

\[ x = \frac{C_{web} \cdot T}{F_{web}} = \frac{36.4}{1,103.8} \cdot 395 = 13 \text{ mm} \]
Location of centroid of compressive force within steel section measured from top of steel section:

\[
d_2 = \frac{0.5 \cdot t_f \cdot F_{\text{flange}} + (t_f + 0.5 \cdot r_{\text{fillet}}) \cdot F_{\text{fillet}} + (t_f + r_{\text{fillet}} + 0.5 \cdot x) \cdot C_{\text{web}}}{C_r}
\]

\[
= \frac{0.5 \cdot 14 \cdot 855 + (14 + 0.5 \cdot 16.5) \cdot 60 + (14 + 16.5 + 0.5 \cdot 44) \cdot 36.4}{951.6} = 9.4 \text{ mm}
\]

Moment resistance of composite beam for partial composite action:

\[
M_{rc} = Q_r \cdot \left( h_r + t_c - \frac{a}{2} + d_2 \right) + P_y \cdot \left( \frac{d}{2} - d_2 \right)
\]

\[
= 1,031 \cdot \left( 75 + 65 - \frac{26}{2} + 9.4 \right) \cdot 10^{-3} + 2,934 \cdot \left( \frac{457}{2} - 9.4 \right) \cdot 10^{-3} = 783.6 \text{ kN-m}
\]

Shear Stud Strength:

From CISC Handbook of Steel Construction Tenth Edition for 19-mm-diameter studs,

\[
h_d = 75 \text{ mm}, \quad w_d/h_d = 2.0, \quad 25 \text{ MPa}, \quad 2,3000 \text{ kg/m}^3 \text{ concrete:}
\]

\[
q_{rr} = 68.7 \text{ kN}
\]

Total number of studs required \( \frac{2 \cdot Q_r}{q_{rr}} = \frac{2 \cdot 1,031}{68.7} = 30 \)

Live Load Deflection:

Modulus of elasticity ratio:

\[
n = \frac{E_c}{E_c} = 200,000/23,400 = 8.55
\]

Transformed elastic moment of inertia assuming full composite action:

<table>
<thead>
<tr>
<th>Element</th>
<th>Transformed Area</th>
<th>Moment Arm from Centroid</th>
<th>Ay</th>
<th>Ay²</th>
<th>I₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab</td>
<td>22,815</td>
<td>336</td>
<td>7,666</td>
<td>2,576</td>
<td>8</td>
</tr>
<tr>
<td>W460x74</td>
<td>9,450</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>333</td>
</tr>
<tr>
<td></td>
<td>32,265</td>
<td>7,666</td>
<td>2,576</td>
<td>341</td>
<td></td>
</tr>
</tbody>
</table>

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\[ I_x = I_0 + Ay^2 = 341 \times 10^6 + 2,576 \times 10^6 = 2,917 \times 10^6 \text{ mm}^4 \]

\[ y = \frac{7,666 \times 10^6}{32,265} = 238 \text{ mm} \]

\[ I_{xx} = I_x - A \cdot y^2 = 2,917 \times 10^6 - 32,265 \times 238^2 = 1,095 \times 10^6 \text{ mm}^4 \]

Effective moment of inertia assuming partial composite action:

\[ I_{eff} = I_x + 0.85 \rho^{0.25} (I_{xx} - I_x) \]

\[ = 333 + 0.85 \times 0.40^{0.25} \times (1,095 - 333) \]

\[ = 848 \times 10^6 \text{ mm}^4 \]

\[ \Delta_{LL} = 1.15 \frac{5w_L L^4}{384EI_{eff}} = 1.15 \frac{5 \times 18 \times (12,000)^4}{384 \times 200,000 \times 848 \times 10^6} = 32.9 \text{ mm} \]

**Bottom Flange Tension:**

Stress in tension flange due to specified load acting on steel beam alone:

\[ f_1 = \frac{M_1}{S_x} = \frac{8 \times 12,000^2}{8 \times 1460 \times 10^3} = 98.6 \text{ MPa} \]

Bottom section modulus based on transformed elastic moment of inertia assuming, per the original example, full composite action:

\[ S_x = \frac{I_{xx}}{\left( d \times 2 + y \right)} = \frac{1,095 \times 10^6}{(228.5 + 237.6)} = 1350 \text{ mm} \]

Stress in tension flange due to specified live and superimposed dead loads acting on composite section:

\[ f_2 = \frac{M_2}{S_x} = \frac{(18 + 4) \times 12,000^2}{8 \times 2350 \times 10^3} = 168.5 \text{ MPa} \]

\[ f_1 + f_2 = 98.6 + 168.5 = 267.1 \text{ MPa} \]
Design for Shear Strength:

Required Shear Strength:

\[ V_f = \frac{w_{\text{factored}} \cdot L}{2} = \frac{42 \cdot 12}{2} = 252 \text{ kN} \]

Shear Resistance of Steel Section:

\[ V_c = \Phi \cdot A_w \cdot F_s = 0.9 \cdot d \cdot t_w \cdot 0.66 \cdot F_y = 0.9 \cdot 457 \cdot 9 \cdot 0.66 \cdot 345 = 842.9 \text{ kN} \]