Single Storey Building Design

to NBC 2015

by R.M Lasby

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Case 4
\[ M_t = \frac{30.5 \times 6^2}{8} = 137 \text{ kN m} \quad C_t = 1.4 \times 66 = 92.4 \text{ kN} \]

Case 5
\[ M_t = \frac{19.5 \times 6^2}{8} = 87.8 \text{ kN m} \quad C_t = 1.0 \times 145.4 = 145.4 \text{ kN} \]

Member selection
The compression flange and the weak axis for compression load are braced at 2000 mm on center by the joists. At each joist, a bottom chord extension has been added to the bottom flange of the eave member to prevent constrained axis torsional buckling. These are noted as “BCE” on drawing S2.02 - Roof Plan.

If an eave member, subject to bending and axial compression, is not braced at both the top and bottom flange, constrained-axis torsional buckling may govern the design. Refer to Appendix A for additional flexural-torsional buckling checks required for eave members braced on one flange only.

There is no net uplift due to upward wind (calculations not shown).

The unsupported length for strong axis compression load is the full length, 6.0 m.

From page 5-24 of the handbook, \( M'r \) for W410 x 39 = 216 kN m
\[ A = 4950 \text{ mm}^2, r_x = 159 \text{ mm}, r_y = 28.4 \text{ mm}, I_x = 126 \times 10^6 \text{ mm}^4 \]
\[ \frac{L_x}{r_x} = \frac{6000}{159} = 37.7 \quad \frac{L_y}{r_y} = \frac{2000}{28.4} = 70.4 \]

From page 4-18 of the handbook, for \( \frac{L}{r} = 70 \) and \( F_y = 345 \text{ MPa}, \frac{C_r}{A} = 199 \text{ MPa} \]
\[ C_r = \frac{199 \times 4950}{1000} = 985 \text{ kN} \]

From page 4-12 of the handbook, for \( \frac{L}{r} = 70 \), \( C_e = 403 \text{ MPa} \)
\[ C_e = \frac{403 \times 4950}{1000} = 1995 \text{ kN} \]

Case 3

From table 4-9 on page 4-107 \( \frac{C_f}{C_e} = \frac{26.6}{1995} = 0.013 \) and \( U_1 = 1.01 \)
\[ \frac{C_f}{C_r} + \frac{0.85 U_1 M_{t X}}{M_{t X}} = \frac{26.4}{985} + \frac{0.85 \times 1.01 \times 175}{216} = 0.72 < 1.0 \]
**Member selection**

The compression flange is braced continuously by the roof deck. A brace has been added from the bottom chord of the eave member to the top of the first interior joists to prevent rotation of the bottom flange. These are noted as “BR2” on drawing S2.02 - Roof Plan.

If an eave member, subject to bending and axial compression, is not braced at both the top and bottom flange, constrained axis torsional buckling may govern the design. Refer to Appendix A for additional flexural-torsional buckling checks required for eave members braced on one flange only.

There is no net uplift due to upward wind (calculations not shown).

The unsupported length for strong axis compression load is the full length, 5.25 m.

From page 5-24 of the handbook, \( M_r \) for W310x21 = 89.1 kN m

\[
A = 2680 \text{ mm}^2, \quad r_x = 117 \text{ mm}, \quad I_x = 37.0 \times 10^6 \text{ mm}^4
\]

\[
\frac{L_x}{r_x} = \frac{5250}{117} = 45
\]

From page 4-7 of the handbook, for \( \frac{L}{r} = 45 \) and \( F_y = 345 \text{ MPa}, \frac{C_r}{A} = 263 \text{ MPa} \)

\[
C_r = \frac{263 \times 2680}{1000} = 705 \text{ kN}
\]

From page 4-12 of the handbook, for \( \frac{L}{r} = 45 \), \( \frac{C_e}{A} = 975 \text{ MPa} \)

\[
C_e = \frac{975 \times 2680}{1000} = 2613 \text{ kN}
\]

**Case 3**

From page 4-107

\[
\frac{C_f}{C_e} = \frac{18.3}{2613} = 0.007 \text{ and } U_i = 1.01
\]

\[
\frac{C_f}{C_r} + \frac{0.85U_iM_fx}{M_rx} = \frac{18.3}{705} + \frac{0.85 \times 1.01 \times 65.5}{89.1} = 0.66
\]

**Case 4**

\[
\frac{C_f}{C_e} = \frac{64.1}{2613} = 0.02 \text{ and } U_i = 1.02
\]
EXTERIOR COLUMNS
The exterior columns are designed as vertical beam-columns carrying the wind loads from the girts up to the roof diaphragm and down to the foundation, as well as vertical loads from the edge of the roof and the cladding.

Loads
Calculate the inward and outward wind loads acting on the exterior columns using $C_pC_g = -1.5, 1.25$. This is taken from NBCC 2015 Figure 4.1.7.6-B (see page 29) for a tributary wind load area of 48.4 $m^2$ for the side walls, and 55.8 $m^2$ for the end walls.

<table>
<thead>
<tr>
<th></th>
<th>Exterior Columns – All zones</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Factored Inward Load</td>
</tr>
<tr>
<td>ULS</td>
<td>0.91 kPa x 1.4 = 1.27 kPa</td>
</tr>
<tr>
<td>SLS</td>
<td>0.91 kPa x 0.75 = 0.68 kPa</td>
</tr>
</tbody>
</table>

Exterior columns on the side walls supporting the girders
Two checks will have to be done on the columns:
1. Inward acting wind pressure, where the compression flange is the exterior flange which is braced by the wind girts.
2. Outward acting wind pressure, where the compression flange is the interior flange which is unbraced.

Inward Acting Wind Pressure on Columns
The highest inward pressure on the girts is 1.27 kPa. The tributary width is 5.25 m and the height to the underside of the eave members is 8.7 m.

$w_r = 1.27 \times 5.25 = 6.67 \text{ kN/m}$
$w_s = 0.68 \times 5.25 = 3.57 \text{ kN/m}$

Plugging $\delta = \frac{L}{240}$ into $\delta = \frac{5wL^4}{384EI}$

And solving for $I = \frac{5wL^3 \times 240}{384E} = 15.6 \times 10^3 wL^3$ where $w$ is in kN/m and $L$ is in m

$I$ required = $15.6 \times 10^3 \times 3.57 \times 8.7^3 = 36.7 \times 10^6 \text{ mm}^4$

$M_r = wL^2/8 = 6.67 \times 8.7^2/8 = 63.1 \text{ kNm}$

The eave member vertical load was previously calculated.

$w_r = 16.9 \text{ kN/m}$

$C_f = 16.9 \times 5.25 = 88.7 \text{ kN}$
From the girder analysis, the end reaction is 220.9 to which the eave member vertical load of 88.7 is added.

\[ C_f = 220.9 + 88.7 = 310 \text{ kN} \]

**Outward Acting Wind Pressure on Columns**

The highest outward pressure (suction) on the column is 1.23 kPa. The tributary width is 5.25 m and the height to the underside of the eave members is 8.7 m.

\[ \begin{align*}
    w_f &= 1.23 \times 5.25 = 6.46 \text{ kN/m} \\
    w_s &= 0.66 \times 5.25 = 3.47 \text{ kN/m} \\
    I \text{ required} &= 15.6 \times 10^3 \times 3.47 \times 8.7^3 = 35.6 \times 10^6 \text{ mm}^4 \\
    M_f &= wL^2/8 = 6.46 \times 8.7^2/8 = 61.1 \text{ kNm} \\
    \text{And } C_f \text{ is the same at } 310 \text{ kN.}
\end{align*} \]

**Exterior columns on the side walls between the girders**

The tributary width is 5.25 m and the height to the underside of the eave members is 9.0 m.

**Inward Acting Wind Pressure on Columns**

\[ \begin{align*}
    w_f &= 1.27 \times 5.25 = 6.67 \text{ kN/m} \\
    w_s &= 0.68 \times 5.25 = 3.57 \text{ kN/m} \\
    I \text{ required} &= 15.6 \times 10^3 \times 3.57 \times 9.0^3 = 40.6 \times 10^6 \text{ mm}^4 \\
    M_f &= wL^2/8 = 6.67 \times 9.0^2/8 = 67.5 \text{ kNm} \\
    C_f &= 88.7 \text{ kN}
\end{align*} \]

**Outward Acting Wind Pressure on Columns**

\[ \begin{align*}
    w_f &= 1.23 \times 5.25 = 6.46 \text{ kN/m} \\
    w_s &= 0.66 \times 5.25 = 3.47 \text{ kN/m} \\
    I \text{ required} &= 15.6 \times 10^3 \times 3.47 \times 9.0^3 = 39.5 \times 10^6 \text{ mm}^4 \\
    M_f &= wL^2/8 = 6.46 \times 9.0^2/8 = 65.4 \text{ kNm} \\
    C_f &= 88.7 \text{ kN}
\end{align*} \]

**Exterior columns on the end walls**

The tributary width is 6.0 m and the height to the joist seat is 9.2 m.

**Inward Acting Wind Pressure on Columns**

\[ \begin{align*}
    w_f &= 1.27 \times 6.0 = 7.62 \text{ kN/m} \\
    w_s &= 0.68 \times 6.0 = 4.08 \text{ kN/m}
\end{align*} \]
I required = 15.6 × 10³ × 4.08 × 9.2³ = 49.6 × 10⁶ mm⁴

\[ M_t = wL^2/8 = 7.62 \times 9.2^2/8 = 80.6 \text{ kNm} \]

The eave member vertical load for the end wall was previously calculated.

\[ w_f = 30.5 \text{ kN/m} \]

\[ C_t = 30.5 \times 6.0 = 183 \text{ kN} \]

**Outward Acting Wind Pressure on Columns**

\[ w_f = 1.23 \times 6.0 = 7.38 \text{ kN/m} \]

\[ w_s = 0.66 \times 6.0 = 3.96 \text{ kN/m} \]

I required = 15.6 × 10³ × 3.96 × 9.2³ = 48.1 × 10⁶ mm⁴

\[ M_f = wL^2/8 = 7.38 \times 9.2^2/8 = 78.1 \text{ kNm} \]

\[ C_f = 183 \text{ kN} \]

**Member Selection**

**Exterior columns on the side walls supporting the girders**

Try W250x58

**Check 1 - Inward Acting Wind Pressure on Columns**

\[ C_f = 310 \text{ kN}; \quad M_f = 63.1 \text{ kNm}; \quad I_x \text{ req'd} = 36.7 \times 10^6 \text{ mm}^4 \]

Note: for bending, compression flange of column is braced at 2400 by wind girts.

On page 5-24 of the handbook, W250x58 has

\[ I_x = 87.3 \times 10^6 \text{ mm}^4 > 36.7 \times 10^6 \text{ mm}^4 \]

\[ M_f = 239 \text{ kNm for an unbraced length of 2500 mm > 63.1 \text{ kNm}} \]

W250x58 \( A = 7420 \text{ mm}^2, \quad r_x = 108 \text{ mm and } r_y = 50.4 \text{ mm} \)

\[ L_x = 8.7 \text{ m and } L_y = 2.4 \text{ m} \]

\[ L_x/r_x = 8700/108 = 80.6 \]

\[ L_y/r_y = 2400/50.4 = 47.6 \]

Due to the fact that only the outside flange of the column is supported, we must also consider constrained-axis flexural-torsional buckling. Calculate the equivalent \( L/r \) ratio using the expression for Elastic Buckling Stress \( F_e = \frac{\pi^2 E}{(kL/r)^2} \)
taken from Clause 13.3.1 of S16-14 and the following expression (See Appendix A):

\[
F_e = \omega \left[ \frac{\pi^2 EI_y}{L^2} \left( \frac{(d - t)^2}{4} + a^2 \right) + GJ \right] \frac{1}{A(r_x^2 + r_y^2 + a^2)}
\]
where $L$ is the column height and $\omega$ is a factor to account for bracing flexibility and is taken as 0.9.

$$F_e = 0.9 \left[ \frac{\pi^2 (200000)(18.8 \times 10^6)}{8700^2} \left( \frac{(252 - 13.5)^2}{4} + 126^2 \right) \right. + \left(77000)(409 \times 10^3) \right] \frac{1}{7420(108^2 + 50.4^2 + 126^2)}$$

$F_e = 186.5$ MPa

Set this equal to $F_e = \frac{\pi^2 E}{(kL/r)^2} = 186.5$ MPa and solve for $L/r$

$L/r = 102.9$ (say 103).

This value is greater than $L_x/r_x$ and $L_y/r_y$, therefore constrained axis flexural-torsional buckling will govern the design.

From page 4-9 of the handbook, for $L/r = 103$, and $F_y = 345$ MPa

$C_r/A = 128$ MPa

$C_r = 128$ MPa$(7420$ mm$^2) = 949.8$ kN

From page 4-12 of the handbook, for $L/r = 103$

$C_r/A = 186$ MPa

$C_r = 186$ MPa$(7420$ mm$^2) = 1380.1$ kN

From page 4-107 for $C_f/C_r = 310/1380.1 = 0.22$, $U = 1.28$

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} = \frac{310}{949.8} + \frac{0.85(1.28)(63.1)}{239} = 0.61 < 1.0$$

**Check 2 - Outward Acting Wind Pressure on Columns**

$C_r = 310$ kN; $M_r = 61.1$ kNm; $I_x$ req’d = $35.6 \times 10^6$ mm$^4$

Note: for bending, compression flange of column is not braced and $M_r$ must be based on the full unsupported height of the column.

From page 5-25

$M'_r$ for an unbraced length of 8000mm = 137 kNm

$M'_r$ for an unbraced length of 9000mm = 119 kNm

Interpolating for an unbraced length of 8700mm

$M'_r = 124.4$ kNm
\[
\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} = \frac{310}{949.8} + \frac{0.85(1.28)(61.1)}{124.4} = 0.86 < 1.0
\]

W250x58 is acceptable.

**Exterior columns on the side walls between the girders**

Try W250x58

**Check 1 - Inward Acting Wind Pressure on Columns**

\(C_f = 88.7 \text{ kN}; \ M_f = 67.5 \text{ kNm}; \ I_x \text{ req'd} = 40.6 \times 10^6 \text{ mm}^4\)

On page 5-24 of the handbook, W250x58 has

\(I_x = 87.3 \times 10^6 \text{ mm}^4 > 40.6 \times 10^6 \text{ mm}^4\) and

\(M_r = 239 \text{ kNm} \text{ for an unbraced length of } 2500 \text{ mm} > 67.5 \text{ kNm}\)

W250x58 \(A = 7420 \text{ mm}^2, \ r_x = 108 \text{ mm} \text{ and } r_y = 50.4 \text{ mm}\)

\(L_x = 9.0 \text{ m} \text{ and } L_y = 2.4 \text{ m}\)

\(L_x/r_x = 9000/108 = 83.3\)

\(L_y/r_y = 2400/50.4 = 47.4\)

Calculate \(L/r\) for constrained-axis flexural-torsional buckling.

\[
F_e = 0.9 \left[ \frac{\pi^2(200000)(18.8 \times 10^6)}{9000^2} \left( \frac{(252 - 13.5)^2}{4} + 126^2 \right) \right.
\]

\[
\left. + (77000)(409 \times 10^3) \right] \frac{1}{7420(108^2 + 50.4^2 + 126^2)}
\]

\(F_e = 182.6 \text{ MPa}\)

Set this equal to \(F_e = \frac{\pi^2E}{(kL/r)^2} = 182.6 \text{ MPa}\) and solve for \(L/r\)

\(L/r = 104\)

From page 4-9 of the handbook, for \(L/r = 104\), and \(F_y = 345 \text{ MPa}\)

\(C_f/\text{A} = 126 \text{ MPa}\)

\(C_r = 126 \text{ MPa}(7420 \text{ mm}^2) = 934.9 \text{ kN}\)

From page 4-12 of the handbook, for \(L/r = 104\)

\(C_f/\text{A} = 183 \text{ MPa}\)

\(C_e = 183 \text{ MPa}(7420 \text{ mm}^2) = 1357.9 \text{ kN}\)

From page 4-107 for \(C_f/C_e = 88.7/1357.9 = 0.07, \ U = 1.08\)
Check 2 - Outward Acting Wind Pressure on Columns
\[ \frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} = \frac{88.7}{934.9} + \frac{0.85(1.08)(67.5)}{239} = 0.35 < 1.0 \]

Note: for bending, compression flange of column is not braced and \( M_r \) must be based on the full unsupported height of the column.

From page 5-25
\( M'_r \) for an unbraced length of 9000mm = 119 kNm

\[ \frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} = \frac{88.7}{934.9} + \frac{0.85(1.08)(65.4)}{119} = 0.60 < 1.0 \]

W250x58 is acceptable.

**Exterior columns on the end walls**

Try W250x58

Check 1 - Inward Acting Wind Pressure on Columns
\( C_r = 183 \) kNm; \( M_t = 80.6 \) kNm; \( I_x \) req’d = \( 39.5 \times 10^6 \) mm\(^4\)

On page 5-24 of the handbook, W250x58 has
\( I_x = 87.3 \times 10^6 \) mm\(^4\) > \( 49.6 \times 10^6 \) mm\(^4\) and
\( M_t = 239 \) kNm for an unbraced length of 2500 mm > 80.6 kNm

W250x58
\( A = 7420 \) mm\(^2\), \( r_x = 108 \) mm and \( r_y = 50.4 \) mm

\( L_x = 9.2 \) m and \( L_y = 2.4 \) m

\( L_x/r_x = 9200/108 = 85.2 \)

\( L_y/r_y = 2400/50.4 = 47.6 \)

Calculate \( L/r \) for constrained axis flexural-torsional buckling.

\[ F_e = 0.9 \left[ \frac{\pi^2(200000)(18.8\times10^6)}{9200^2} \left( \frac{(252 - 13.5)^2}{4} + 126^2 \right) + \frac{(77000)(409\times10^3)}{7420(108^2 + 50.4^2 + 126^2)} \right] \]

\( F_e = 180.2 \) MPa

Set this equal to \( F_e = \frac{\pi^2E}{(kL/r)^2} = 180.2 \) MPa and solve for \( L/r \)

\( L/r = 104.7 \) (say 105)
From page 4-9 of the handbook, for $L/r = 105$, and $F_y = 345$ MPa

$C_i/A = 124$ MPa

$C_r = 124$ MPa$(7420 \text{ mm}^2) = 920.1 \text{ kN}$

From page 4-12 of the handbook, for $L/r = 105$

$C_e/A = 179$ MPa

$C_e = 179$ MPa$(7420 \text{ mm}^2) = 1328.2 \text{ kN}$

From page 4-107 for $C_i/C_e = 183/1328.2 = 0.14$, $U = 1.16$

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} = \frac{183}{920.1} + \frac{0.85(1.16)(80.6)}{239} = 0.53 < 1.0$$

Check 2 - Outward Acting Wind Pressure on Columns

$C_r = 183 \text{ kN}; M_r = 78.1 \text{ kNm}; I_x \text{ req'd} = 48.1 \times 10^6 \text{ mm}^4$

Note: for bending, compression flange of column is not braced and $M_r$ must be based on the full unsupported height of the column.

From page 5-25

$M'_r$ for an unbraced length of 9000mm = 119 kNm

$M'_r$ for an unbraced length of 10000mm = 105 kNm

Interpolating for an unbraced length of 9200mm

$M'_r = 116.2 \text{ kNm}$

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} = \frac{183}{920.1} + \frac{0.85(1.16)(78.1)}{116.2} = 0.86 < 1.0$$

W250x58 is acceptable.

Load Transfer at the Top

The exterior columns on the end walls transfer the lateral load at the top of the column into the joist top chord. The exterior columns on the side walls supporting girders transfer the lateral load at the top of the column into the girder.

There is no structural member in the roof to transfer the lateral load from the exterior column on the side walls between the girders. A brace from the top of the column to the top chord of the next open web steel joist and sufficient welds to the roof deck are required to transfer the lateral load.

$$R_f = wL/2 = [(6.67 \text{ kN/m})(9.3\text{ m})]/2 = 31.0 \text{ kN}$$
From the Canam Diaphragm publication, one 19 mm diameter puddle weld has a factored shear strength of 4.75 kN. Seven welds are required to transfer the 31.0 kN reaction at the top of the column to the roof deck. Provide a L76x76x6.4 between the top of the column and the first joist. Weld the L76 between the column and first joist to the roof deck with at least 7 – 19 mm diameter puddle welds.
Base plate used is
300 x 280 = 84000 mm²
Using the information on page 4-153 through 4-156 of the handbook
b=203 for W250x58
0.8b = 0.8(203) = 162.4
n = (280-162.4)/2 = 58.8 mm

\[
t_p = \frac{2C_f n^2}{BC\phi F_y} = \frac{2(365\times10^3)58.8^2}{(280)(300)(0.9)(300)} = 10.5 \text{ mm}
\]

Thickness required for deflection = n/5 = 58.8/5 = 11.8 mm

**Uplift**
The factored bending moment at the face of the gusset plate due to uplift

![Diagram of a structural component showing uplift forces and deflection](image)

\[
M_f = \frac{T_f}{4} \times 65 = \frac{131\times10^3}{4} \times 65 = 2.13 \times 10^6 \text{ Nmm}
\]

Effective width in bending
B' = 80 + 60 = 140 mm

\[
t_p = \frac{4M_f}{B'\phi F_y} = \frac{4\times2.13\times10^6}{140\times0.9\times300} = 15.0 \text{ mm}
\]

Use 19 mm plate

**Anchor rods**
For each bolt \( V_f = \frac{319.8}{4} = 80.0 \text{ kN} \) and \( T_f = \frac{131}{4} = 32.8 \text{ kN} \)
## QUANTITY TAKEOFF

<table>
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<tr>
<th>Member Size</th>
<th>Number of Pieces</th>
<th>Member Length (m)</th>
<th>Unit Mass (kg/m)</th>
<th>Gross Mass (kg)</th>
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</thead>
<tbody>
<tr>
<td><strong>Joists</strong></td>
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<tr>
<td>OWSJ 700 deep</td>
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Steel content based on 5544 m² = 32.9 kg/m²
APPENDIX A – MEMBERS BRACED ON ONE FLANGE

Columns

Exterior columns carry wind loads from the girts which provide bracing to the exterior flange only. The bracing axis is located at a distance \( a \) from the shear centre, as shown on the accompanying figure, taken equal to half the member depth \( a = d / 2 \). Since the girts do not prevent twist, columns in axial compression are subject to constrained-axis flexural-torsional buckling (Liu et al., 2013).

Beams

Eave members are braced on the top flange by the roof deck or by the top chords of open-web steel joists, depending on their location around the building perimeter. Flexural-torsional buckling in compression is again a possible failure mode, with the section twisting about an axis located at a distance, \( a = d / 2 \), from the shear centre.

For doubly-symmetric sections, the elastic buckling stress is given by:

\[
F_e = \omega \left[ \frac{\pi^2 E I_y}{L^2} \left( \frac{(d - t)^2}{4} + a^2 \right) + G J \right] \frac{1}{A \left( r_x^2 + r_y^2 + a^2 \right)}
\]

where \( L \) is the column height or beam length, and \( \omega \) is a factor to account for bracing flexibility, taken as 0.9 in AISC (2016).

References

