Single Storey Building Design

to NBC 2015

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Case 4

$$M_f = \frac{30.5 \times 6^2}{8} = 137\ \text{kN m} \quad C_f = 1.4 \times 66 = 92.4\ \text{kN}$$

Case 5

$$M_f = \frac{19.5 \times 6^2}{8} = 87.8\ \text{kN m} \quad C_f = 1.0 \times 145.4 = 145.4\ \text{kN}$$

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

$A = 4950\ \text{mm}^2$, $r_x = 159$ mm, $r_y = 28.4$ 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 $Fy = 345$ MPa, $\frac{Cr}{A} = 199$ MPa

$$C_r = \frac{199 \times 4950}{1000} = 985\ \text{kN}$$

From page 4-12 of the handbook, for $\frac{L}{r} = 70$, $\frac{Ce}{A} = 403$ 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 $U_1 = 1.01$

$$\frac{C_f}{C_r} + \frac{0.85 U_{1x} M_{fx}}{M_{rx}} = \frac{26.4}{985} + \frac{0.85 \times 1.01 \times 175}{216} = 0.72 < 1.0$$

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 mm^2, r_x = 117 mm, I_x = 37.0x10^6 mm^4

$$\frac{L_x}{r_x} = \frac{5250}{117} = 45$$

From page 4-7 of the handbook, for $\frac{L}{r} = 45$ and $Fy = 345$ MPa, $\frac{Cr}{A} = 263$ MPa

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

From page 4-12 of the handbook, for $\frac{L}{r} = 45$, $\frac{Ce}{A} = 975$ MPa

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

Case 3

From page 4-107

$$\frac{C_f}{C_e} = \frac{18.3}{2613} = 0.007 \text{ and } U_1 = 1.01$$

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} = \frac{18.3}{705} + \frac{0.85 \times 1.01 \times 65.5}{89.1} = 0.66$$

Case 4

$$\frac{C_f}{C_e} = \frac{64.1}{2613} = 0.02 \text{ and } U_1 = 1.02$$

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.

	Exterior Columns – All zones	
	Factored Inward Load	**Factored Outward Load**
ULS	0.91 kPa x 1.4 = 1.27 kPa	0.88 kPa x 1.4 = 1.23 kPa
SLS	0.91 kPa x 0.75 = 0.68 kPa	0.88 kPa x 0.75 = 0.66 kPa

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.

<u>Inward Acting Wind Pressure on Columns</u>

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_f = 1.27 x 5.25 = 6.67 kN/m

w_s = 0.68 x 5.25 = 3.57 kN/m

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

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

I required = 15.6 x 10^3 x 3.57 x 8.7^3 = 36.7 x 10^6 mm^4

M_f = $wL^2/8$ = 6.67x8.7^2/8 = 63.1 kNm

The eave member vertical load was previously calculated.

$w_f = 16.9\,kN/m$

$C_f = 16.9 \times 5.25 = 88.7\,kN$

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.

w_f = 1.23 x 5.25 = 6.46 kN/m

w_s = 0.66 x 5.25 = 3.47 kN/m

I required = 15.6 x 10^3 x 3.47 x 8.7^3 = 35.6 x 10^6 mm^4

M_f = $wL^2/8$ = 6.46x8.7^2/8 = 61.1 kNm

And C_f is the same at 310 kN.

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

w_f = 1.27 x 5.25 = 6.67 kN/m

w_s = 0.68 x 5.25 = 3.57 kN/m

I required = 15.6 x 10^3 x 3.57 x 9.0^3 = 40.6 x 10^6 mm^4

M_f = $wL^2/8$ = 6.67x9.0^2/8 = 67.5 kNm

$C_f = 88.7\ \text{kN}$

Outward Acting Wind Pressure on Columns

w_f = 1.23 x 5.25 = 6.46 kN/m

w_s = 0.66 x 5.25 = 3.47 kN/m

I required = 15.6 x 10^3 x 3.47 x 9.0^3 = 39.5 x 10^6 mm^4

M_f = $wL^2/8$ = 6.46x9.0^2/8 = 65.4 kNm

$C_f = 88.7\ \text{kN}$

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

w_f = 1.27 x 6.0 = 7.62 kN/m

w_s = 0.68 x 6.0 = 4.08 kN/m

I required = $15.6 \times 10^3 \times 4.08 \times 9.2^3 = 49.6 \times 10^6$ mm^4
$M_f = wL^2/8 = 7.62 \times 9.2^2/8 = 80.6$ kNm

The eave member vertical load for the end wall was previously calculated.
$$w_f = 30.5 \text{ kN/m}$$
$$C_f = 30.5 \times 6.0 = 183 \text{ kN}$$

Outward Acting Wind Pressure on Columns
$w_f = 1.23 \times 6.0 = 7.38$ kN/m
$w_s = 0.66 \times 6.0 = 3.96$ kN/m
I required = $15.6 \times 10^3 \times 3.96 \times 9.2^3 = 48.1 \times 10^6$ mm^4
$M_f = wL^2/8 = 7.38 \times 9.2^2/8 = 78.1$ kNm
$C_f = 183$ kN

Member Selection

Exterior columns on the side walls supporting the girders
Try W250x58

Check 1 - Inward Acting Wind Pressure on Columns
$C_f = 310$ kN; $M_f = 63.1$ kNm; I_x req'd = 36.7×10^6 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
$I_x = 87.3 \times 10^6$ mm^4 > 36.7×10^6 mm^4 and
$M_r = 239$ kNm for an unbraced length of 2500 mm > 63.1 kNm
W250x58 A = 7420 mm^2, $r_x = 108$ mm and $r_y = 50.4$ mm
$L_x = 8.7$ m and $L_y = 2.4$ 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):

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

where L is the column height and ω is a factor to account for bracing flexibility and is taken as 0.9.

$$F_e = 0.9\left[\frac{\pi^2(200000)(18.8x10^6)}{8700^2}\left(\frac{(252-13.5)^2}{4}+126^2\right) + (77000)(409x10^3)\right]\frac{1}{7420(108^2+50.4^2+126^2)}$$

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_e/A = 186 MPa

C_e = 186 MPa(7420 mm^2) = 1380.1 kN

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

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

<u>Check 2 - Outward Acting Wind Pressure on Columns</u>

C_f = 310 kN; M_f = 61.1 kNm; I_x req'd = 35.6x10^6 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 kN; M_f = 67.5 kNm; I_x req'd = 40.6 x10^6 mm^4

On page 5-24 of the handbook, W250x58 has

I_x = 87.3 x 10^6 mm^4 > 40.6 x 10^6 mm^4 and

M_r = 239 kNm for an unbraced length of 2500 mm > 67.5 kNm

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

L_x = 9.0 m and L_y = 2.4 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.

$$F_e = 0.9\left[\frac{\pi^2(200000)(18.8x10^6)}{9000^2}\left(\frac{(252-13.5)^2}{4} + 126^2\right) + (77000)(409x10^3)\right]\frac{1}{7420(108^2 + 50.4^2 + 126^2)}$$

F_e = 182.6 MPa

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

L/r = 104

From page 4-9 of the handbook, for L/r = 104, and F_y = 345 MPa

C_r/A = 126 MPa

C_r = 126 MPa(7420 mm^2) = 934.9 kN

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

C_e/A = 183 MPa

C_e = 183 MPa(7420 mm^2) = 1357.9 kN

From page 4-107 for C_f/C_e = 88.7/1357.9 = 0.07, U = 1.08

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}=\frac{88.7}{934.9}+\frac{0.85(1.08)(67.5)}{239}=0.35<1.0$$

<u>Check 2 - Outward Acting Wind Pressure on Columns</u>

C_f = 88.7 kN; M_f = 65.4 kNm; I_x req'd = 39.5x10^6 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 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

<u>Check 1 - Inward Acting Wind Pressure on Columns</u>

C_f = 183 kNm; M_f = 80.6 kNm; I_x req'd = 49.6 x 10^6 mm^4

On page 5-24 of the handbook, W250x58 has

I_x = 87.3 x 10^6 mm^4 > 49.6 x 10^6 mm^4 and

M_r = 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.

$$F_e = 0.9\left[\frac{\pi^2(200000)(18.8x10^6)}{9200^2}\left(\frac{(252-13.5)^2}{4}+126^2\right)+(77000)(409x10^3)\right]\frac{1}{7420(108^2+50.4^2+126^2)}$$

F_e = 180.2 MPa

Set this equal to $F_e=\frac{\pi^2 E}{(kL/r)^2}=180.2\ MPa$ 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_r/A = 124 MPa

C_r = 124 MPa(7420 mm^2) = 920.1 kN

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

C_e/A = 179 MPa

C_e = 179 MPa(7420 mm^2) = 1328.2 kN

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

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

<u>Check 2 - Outward Acting Wind Pressure on Columns</u>

C_f = 183 kN; M_f = 78.1 kNm; I_x req'd = 48.1x10^6 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 9000mm = 119 kNm

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

Interpolating for an unbraced length of 9200mm

M'_r = 116.2 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.

$R_f = w_fL/2$ = [(6.67 kN/m)(9.3m)]/2 = 31.0 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 mm^2

Using the information on page 4-153 through 4-156 of the handbook

b=203 for W250x58

0.8b = 0.8(203) = 162.4

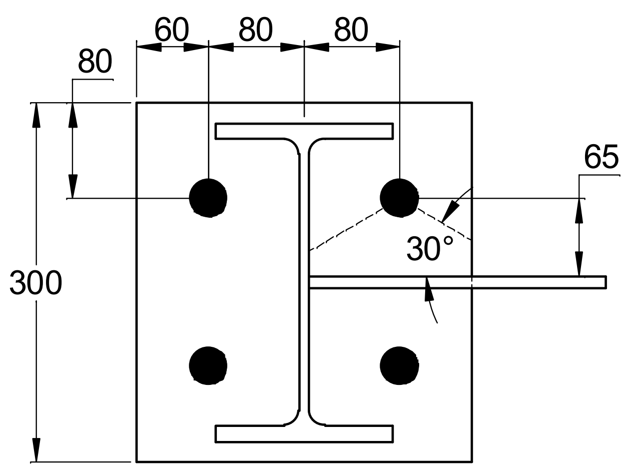
n = (280-162.4)/2 = 58.8 mm

$$t_p = \sqrt{\frac{2C_f n^2}{BC\phi F_y}} = \sqrt{\frac{2(365x10^3)58.8^2}{(280)(300)(0.9)(300)}} = 10.5\ mm$$

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



$$M_f = \frac{T_f}{4} \times 65 = \frac{131 \times 10^3}{4} \times 65 = 2.13 \times 10^6\ \text{N mm}$$

Effective width in bending

$B' = 80 + 60 = 140$ mm

$$t_p = \sqrt{\frac{4M_f}{B'\phi F_y}} = \sqrt{\frac{4 \times 2.13 \times 10^6}{140 \times 0.9 \times 300}} = 15.0\ \text{mm}$$

Use 19 mm plate

Anchor rods

For each bolt $V_f = \dfrac{319.8}{4} = 80.0$ kN and $Tf = \dfrac{131}{4} = 32.8$ kN

QUANTITY TAKEOFF

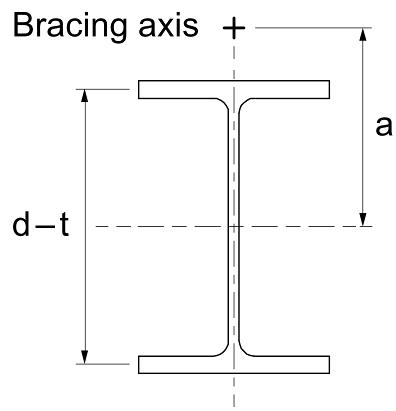
Member Size	Number of Pieces	Member Length (m)	Unit Mass (kg/m)	Gross Mass (kg)
Joists				
OWSJ 700 deep	232	10.5	22.9	55,784
OWSJ 800 deep	39	10.5	32.6	13,350
Girders				
W460x60	14	7.3	60	6,132
W610x101	7	16.4	101	11,595
W610x101	14	14.5	101	20,503
Office beams				
W460x52	8	6	52	2,496
W410x46	4	12	46	2,208
W310x21	2	10.5	21	441
Eave Members				
W410x39	10	12	39	4,680
W310x21	16	10.5	21	3,528
Interior Columns				
HSS203x203x6.4	14	8.6	38.4	4,623
HSS203x203x8.0	14	8.6	47.5	5,719
L38x38x3.2 BCE	84	0.7	1.82	107
Perimeter Columns				
HSS127x127x6.4	11	4.6	23.2	1174
W250x58	52	9.3	58	28049
L76x76x6.4	16	4	7.28	466
HSS127x127x6.4	11	4.3	23.2	1,097
Girts				
C180x15	30	12	15	5,400
C150x12	48	10.5	12	6,048
Sag rods	136	7.3	0.89	884
Bracing				
2L89x64x9.5	8	18	21.4	3082
L89x89x6.4	2	7.5	8.55	128
Roof edge				
L76x51x6.4		405	6.01	2,434
HSS102x102x6.4	60	1.2	18.2	1,310
HSS102x102x6.4	64	0.8	18.2	932
Total				182,161

Steel content based on 5544 m^2 = 32.9 kg/m^2

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:

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

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

References

AISC. 2016. Specification for Structural Steel Buildings. ANSI/AISC 360-16. American Institute of Steel Construction, Chicago, Illinois.

Liu, D., Davis, B., Arber, L. and Sabelli, Rafael. 2013. Torsional and Constrained-Axis Flexural-Torsional Buckling Tables for Steel W-Shapes in Compression. AISC Engineering Journal, 4th Quarter.

Ziemian, R.D. (Editor). 2010. Guide to Stability Design Criteria for Metal Structures, 6th Edition. John Wiley & Sons.