Final Design Project

CIV312 – Steel and Timber Design

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Group 16

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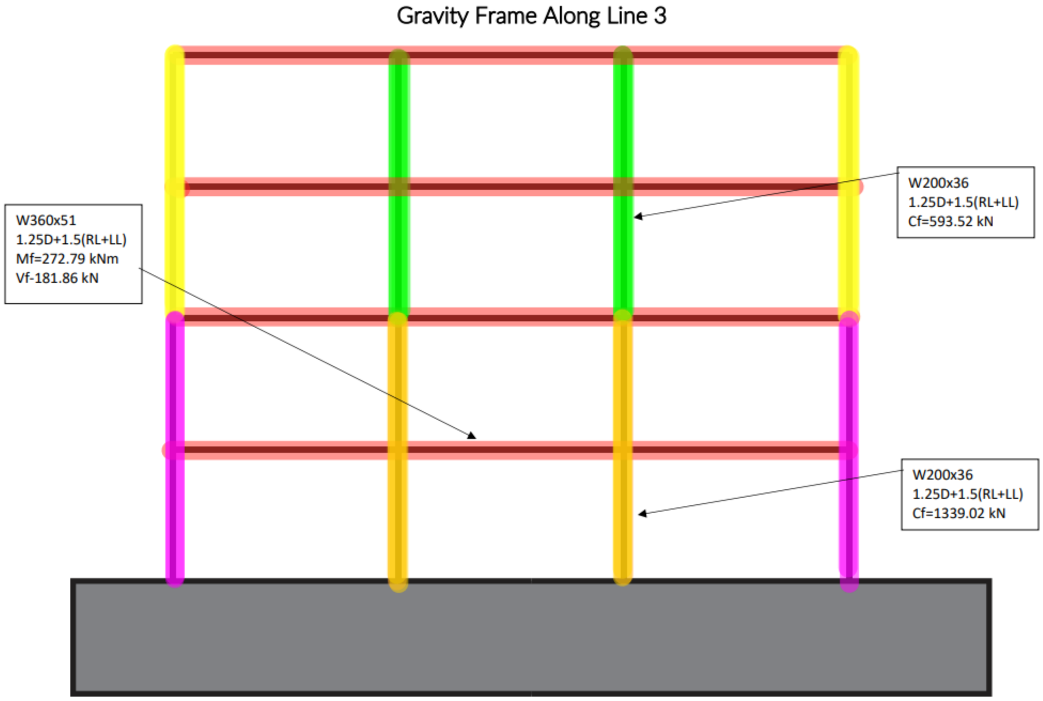
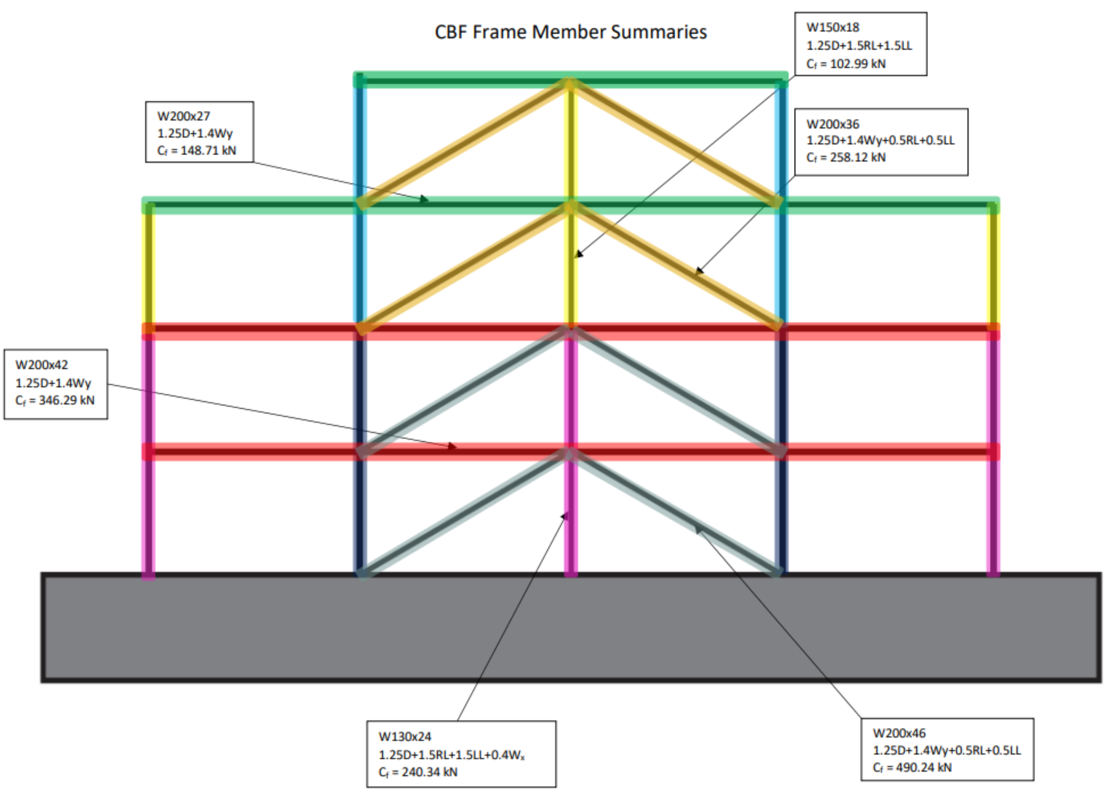
1.0 Introduction

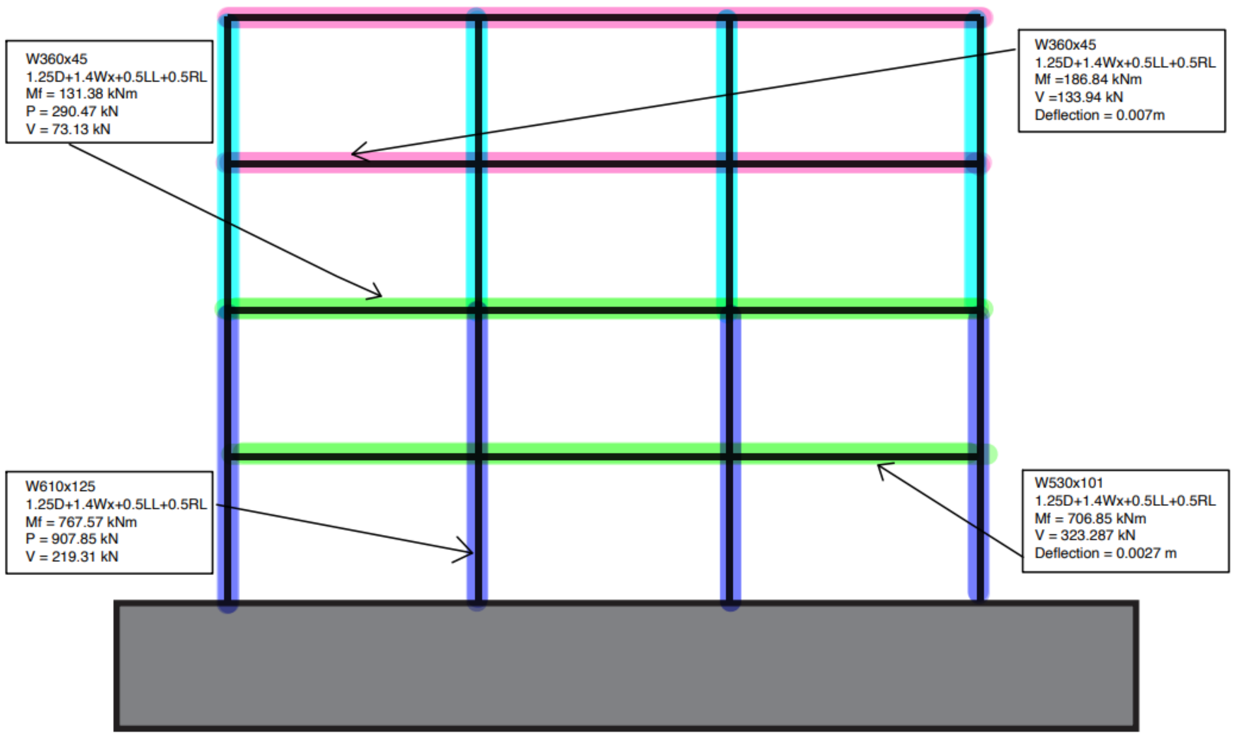
This report summarizes the design procedure for a four-storey steel building for use as an office space in North York. Sample calculations are shown for governing design forces, structural member design, and connection details. In addition, scaled drawings of each designed connection is provided.

2.0 Final Member Selection





MRF Along Line 2



2.1 Sample Calculations

2.1.1 MRF Beam on 1st Floor (W530x101)

Governing Load Combination: 1.25D + 1.4Wx + 0.5RL + 0.5LL

Loading Action: Moment 3-3, Shear 2-2

Section Class (Table 2):

$$\frac{b_{el}}{t} = \frac{b/2}{t} = \frac{210/2}{17.4} = 6.03 \qquad \frac{170}{\sqrt{F_y}} = \frac{170}{\sqrt{350}} = 9.1 \qquad \because \frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$$

$$\therefore flange\ is\ better\ than\ class\ 2$$

$$\frac{h}{w} = \frac{d-2t}{w} = \frac{352 - 2\times 9.8}{6.9} = 48.17 \qquad \frac{1700}{\sqrt{Fy}} = \frac{1700}{\sqrt{350}} = 90.9 \qquad \because \frac{h}{w} \leq \frac{1700}{\sqrt{Fy}}$$

$$\therefore web\ is\ better\ than\ class\ 2$$

Moment Capacity:

[§ 13.5.a] $M_P = Z_x F_y = 2.62 \times 10^6 \times 350 = 917\ kNm$

[§ 13.6.a.ii] $\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2+4M_a^2+7M_b^2+4M_c^2}} = \frac{4\times 706.85}{\sqrt{706.85^2+4\times 273.68^2+7\times 52.23^2+4\times 274.62^2}} = 2.67 > 2.5$

$\because \omega_2 > 2.5 \qquad \therefore \omega_2 = 2.5$

[§ 13.6.a.ii] $M_u = \frac{\omega_2 \pi}{L}\sqrt{EIyGJ + (\frac{\pi E}{L})^2 IyCw}$

$$= \frac{2.5\times\pi}{6000}\sqrt{(200{,}000)(2.69\times10^7)(7.7\times10^4)(1.02\times10^6) + \left(\frac{200{,}000\pi}{6000}\right)^2(2.69\times10^7)(1.82\times10^{12})} = 1282.17\ kNm$$

Check Mu > 0.67Mp: $1282.17\ KNm > 0.67\times 917\ KNm = 614.39\ KNm$

[§ 13.6.a.i] Mr = 1.15ϕMp (1 - $\frac{0.28Mp}{Mu}$) $= 1.15(0.9)(917)\left[1 - \frac{0.28(917)}{1282.17}\right] = 759.04\ kNm$

Check Mr < ϕMp : $759.04\ KNm < 0.9 \times 917\ KNm = 825.3\ KNm$

$\therefore Mr = 759.04\ kNm$

$\because Mr > Mf = 706.85\ KNm$ $\therefore Moment\ check\ passes$

Shear Resistance:

[§ 13.4.1.1] $A_w = dw = 537(10.9) = 5853.3\ mm^2$

$\frac{1014}{\sqrt{F_y}} = \frac{1014}{\sqrt{350}} = 54.20$ $\therefore \frac{h}{w} = 48.17 \leq \frac{1014}{\sqrt{F_y}}$

[§ 13.4.1.1.a.i] $\therefore F_s = 0.66F_y = 0.66(350) = 231\ MPa$

[§ 13.4.1] $V_r = \Phi A_w F_s = 0.9(5853.3)(231) = 1216.90\ kN$

$\because Vr > Vf = 323.29\ KN$ $\therefore Shear\ check\ passes$

Deflection Check:

[Table D.1] $Deflection\ Limit = \frac{L}{300} = \frac{6}{300} = 0.02\ m$

$Deflection\ (reading\ from\ SAP) = 0.0027\ m < 0.02\ m\ \therefore pass$

2.1.2 CBF Beam on First Floor (W200x42)

Governing Load Combination: 1.25D + 1.4Wy

Loading Action: Axial (Treat the beam as an axially loaded member due to insignificant moment acting on the beam)

Variables: $L = 6000\ mm,\ n = 1.34$

$r_x = 87.7\ mm,\ r_y = 41.2\ mm,\ d = 205\ mm,\ b = 166\ mm,\ t = 11.8\ mm,\ w = 7.2\ mm$

$C_w = 84x10^9\ mm^6, J = 222x10^3\ mm^4, A = 5320\ mm^2,\ \bar{r}_0^2 = 87.7^2 + 41.2^2 = 9389\ mm^2$

Slenderness Limit

$\frac{k_x L_x}{r_x} = \frac{1.0*6000\ mm}{87.7\ mm} = 68.4 < 200\ \therefore ok$ $\frac{k_y L_y}{r_y} = \frac{1.0*6000\ mm}{41.2\ mm} = 145.6 < 200\ \therefore ok$

Local Buckling Checks

Flange: $\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}, \frac{b_{el}}{t} = \frac{\frac{b}{2}}{t} = \frac{\frac{166\ mm}{2}}{11.8\ mm} = 7.03 < 10.69\ \therefore ok$

Web: $\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}, \frac{b_{el}}{t} = \frac{d-2*t}{w} = \frac{205\ mm-(2*11.8\ mm)}{7.2\ mm} = 25.19 < 35.8\ \therefore ok$

Compressive Capacity

$F_{ex} = \frac{\pi^2 * E}{\left[\frac{k_x L_x}{r_x}\right]^2} = \frac{\pi^2 * 200{,}000}{[68.4]^2} = 421.91\ MPa$ $F_{ey} = \frac{\pi^2 * E}{\left[\frac{k_y L_y}{r_y}\right]^2} = \frac{\pi^2 * 200{,}000}{[145.6]^2} = 93.11\ MPa$

$$F_{ez} = \left[\frac{\pi^2 * E * C_w}{[k_z L_z]^2} + JG\right]\frac{1}{A * \bar{r}_0^2} = 434.45\ MPa$$

$$F_e = 93.11\ MPa,\ \lambda = \sqrt{\frac{350}{93.11}} = 1.94$$

$$C_r = \frac{\phi A F_y}{[1 + \lambda^{2n}]^{\frac{1}{n}}} = 396\ KN$$

2.1.3 MRF Column on First Floor (W610x125)

Governing Load Combination: 1.25D + 1.4Wx + 0.5RL + 0.5LL

Loading Action: Moment 3-3, Axial, Shear 2-2

Section Class (Table 2):

$$\frac{b_{el}}{t} = \frac{b/2}{t} = \frac{229/2}{19.6} = 5.84 \qquad \frac{170}{\sqrt{F_y}} = \frac{170}{\sqrt{350}} = 9.1 \qquad \because \frac{bel}{t} \leq \frac{170}{\sqrt{F_y}}$$

$\therefore$ *flange is better than class 2*

$$\frac{h}{w} = \frac{d-2t}{w} = \frac{612 - 2 \times 19.6}{11.9} = 48.13 \qquad \frac{1700}{\sqrt{Fy}} = \frac{1700}{\sqrt{350}} = 90.9 \qquad \because \frac{h}{\omega} \leq \frac{1700}{\sqrt{Fy}}$$

$\therefore$ *web is better than class 2*

Cross Sectional Strength (Compression):

[§ 13.3.1] $\lambda = 0,\ Cr = \frac{\Phi AFy}{(1+\lambda^{2n})^{\frac{1}{n}}} = 0.9(15900)(350) = 5{,}008.5\ kN$

[§ 13.5.a] $M_r = \Phi Z_x F_y = 0.9(3.67 \times 10^6)(350) = 1{,}156.5\ kNm$

[§ 13.8.5.a] $\kappa = \frac{M_{small}}{M_{large}} = 0$

[§ 13.8.5.a] $\omega_1 = 0.6 - 0.4\kappa = 0.6 - 0 = 0.6 > 0.4$

$\because \omega_1 > 0.4 \qquad \therefore \omega_1 = 0.6$

[§ 13.8.4] $C_{ex} = \frac{\pi^2 EI_x}{l^2} = \frac{\pi^2(200{,}000)(9.85\times10^8)}{3.5^2} = 1.59 \times 10^5\ kN$

[§ 13.8.4] $U_{1x} = \frac{\omega_1}{1-\frac{C_f}{C_e}} = \frac{0.6}{1-\frac{907.85}{1.59\times10^5}} = 0.6 < 1$

$\because U_{1x} < 1 \qquad \therefore U_{1x} = 1$

[§ 13.8.2] $\frac{Cf}{Cr} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} = \frac{907.85}{5008.5} + \frac{0.85(1)(767.57)}{1156.5} + 0 = 0.75 \leq 1.0\ \therefore pass$

Overall Member Strength (Strong Axis):

[§ 13.3.1] $F_{ex} = \frac{\pi^2 E}{\left(\frac{kL}{rx}\right)^2} = \frac{\pi^2(200{,}000)}{\left(\frac{1(3500)}{249}\right)^2} = 9960.62\ MPa \quad ; \lambda = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{350}{9960.82}} = 0.19$

[§ 13.3.1] $Cr = \frac{\varphi AFy}{(1+\lambda^{2n})^{\frac{1}{n}}} = \frac{0.9(15900)(350)}{(1+0.19^{2\times1.34})^{\frac{1}{1.34}}} = 4967.01\ kN$

[§ 13.8.4] $U_{1x} = \frac{\omega_1}{1-\frac{C_f}{C_e}} = \frac{0.6}{1-\frac{907.85}{1.59\times10^5}} = 0.6$

[§ 13.5.a] $M_r = \Phi Z_x F_y = 0.9(3.67 \times 10^6)(350) = 1156.5\ KNm$

[§ 13.8.2] $\frac{Cf}{Cr} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} = \frac{907.85}{4967.01} + \frac{0.85(0.6)(767.57)}{1156.5} + 0 = 0.52 < 1.0\ \therefore pass$

Lateral Torsional Buckling Strength (Weak axis):

[§ 13.3.1] $F_{ey} = \frac{\pi^2 E}{\left(\frac{kL}{ry}\right)^2} = \frac{\pi^2(200{,}000)}{\left(\frac{1(3500)}{49.7}\right)^2} = 398.02\ MPa \quad ; \lambda = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{350}{398.02}} = 0.94$

[§ 13.3.1] $Cr = \frac{\varphi AFy}{(1+\lambda^{2n})^{\frac{1}{n}}} = \frac{0.9(15900)(350)}{(1+0.94^{2\times1.34})^{\frac{1}{1.34}}} = 3175.24\ kN$

[§ 13.5.a] $M_P = Z_x F_y = (3.67 \times 10^6)(350) = 1284.50\ kNm$

[§ 13.8.5.a] $\kappa = \frac{M_{small}}{M_{large}} = 0$ [§ 13.6.a.ii] $\omega_2 = 1.75 + 1.05\kappa + 0.3\kappa^2 = 1.75 < 2.5$

[§ 13.6.a.ii] $M_u = \frac{\omega_2 \pi}{L}\sqrt{EIyGJ + (\frac{\pi E}{L})^2 IyCw}$

$$= \frac{1.75 \times \pi}{3500}\sqrt{(200{,}000)(3.93 \times 10^7)(7.7 \times 10^4)(1.54 \times 10^6) + \left(\frac{200{,}000\pi}{3500}\right)^2 (3.93 \times 10^7)(3.45 \times 10^{12})}$$

$= 3616.78\ kNm$

Check if Mu > 0.67*Mp : $3616.78\ KNm > 0.67 \times 1284.50\ KNm = 860.62\ KNm$

[§ 13.6.a.i] Mr = 1.15ϕMp (1 - $\frac{0.28Mp}{Mu}$) $= 1.15(0.9)(1284.50)\left(1 - \frac{0.28(1284.50)}{3616.78}\right) = 1197.25\ kNm$

Check if Mr < ϕMp : $1197.25 > 0.9 \times 1284.50 = 1156.05\ kNm$

$\because M_r > \phi M_p$ $\therefore M_r = \phi M_p = 1156.05\ kNm$

[§ 13.8.4] $C_{ey} = \frac{\pi^2 EI_y}{L^2} = \frac{\pi^2(200{,}000)(3.93\times10^7)}{3.5^2} = 6332.66\ kN$

[§ 13.8.5.a] $\omega_1 = 0.6 - 0.4\kappa = 0.6 - 0 = 0.6 > 0.4$

$\because \omega_1 > 0.4$ $\therefore \omega_1 = 0.6$

[§ 13.8.4] $U_{1x} = \frac{\omega_1}{1-\frac{Cf}{Ce}} = \frac{0.6}{1-\frac{907.85}{6332.66}} = 0.70 < 1$

$\because U_{1x} < 1$ $\therefore U_{1x} = 1$

[$ 13.8.2] $\frac{Cf}{Cr} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} = \frac{907.85}{3175.24} + \frac{0.85(1)(767.57)}{1156.05} + 0 = 0.85 < 1.0\ \therefore pass$

<u>Check for Low Axial Loads with High Moments:</u>

$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \le 1$ $\frac{767.57}{1156.05} + 0 < 1\ \therefore pass$

<u>Shear Resistance:</u>

[§ 13.4.1.1] $A_w = dw = 612(11.9) = 7282.8\ mm^2$

$\frac{1014}{\sqrt{F_y}} = \frac{1014}{\sqrt{350}} = 54.20$ $\therefore \frac{h}{w} = 48.13 \le \frac{1014}{\sqrt{F_y}}$

[§ 13.4.1.1.a.i] $\therefore F_s = 0.66F_y = 0.66(350) = 231\ MPa$

[§ 13.4.1] $V_r = \Phi A_w F_s = 0.9(7282.8)(231) = 1514.09\ kN$

$\because Vr > Vf = 219.31\ KN$ $\therefore$ *Shear check passes*

Note: *W610x125 is the most economical section when we checked the demand to capacity ratio in SAP. Smaller section, such as W610x113, failed in SAP (d/c ratio =1.04). Therefore, we chose W610x125 as the optimal section. However, when we checked later in Excel, it showed that W610x113 works, with a d/c ratio of 0.95, while the d/c ratio of W610x125 is only 0.85. SAP and Excel showed different results because SAP takes more variables into consideration. While W610x113 is the most economical section according to Excel, we decided to stick with W620x125.*

2.1.4 CBF Column on First Floor (W130x24)

Governing Load Combination:1.25D + 1.5RL + 1.5LL

Loading Action: Axial

Section: W130x24, n = 1.34

Slenderness Limit

$r_x = 54.1\,mm, r_y = 32.2\,mm$

$\frac{K_x L_x}{r_x} = \frac{1.0*3500\,mm}{54.1\,mm} = 64.6 < 200 \therefore ok$ $\frac{K_y L_y}{r_y} = \frac{1.0*3500\,mm}{32.2\,mm} = 108.96 < 200 \therefore ok$

Local Buckling Checks

$d = 127\ mm, b = 127\ mm, t = 9.1\ mm, w = 6.1\,mm$

Flange: $\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}, \frac{b_{el}}{t} = \frac{\frac{b}{2}}{t} = \frac{\frac{127\ mm}{2}}{9.1\ mm} = 6.98 < 10.69 \therefore ok$

Web: $\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}, \frac{b_{el}}{t} = \frac{d-2*t}{w} = \frac{127\ mm-(2*9.1\ mm)}{6.1\ mm} = 17.84 < 35.8 \therefore ok$

Compressive Capacity

$C_w = 10.8x10^9\ mm^6, J = 76.2x10^3\ mm^4, A = 3040\ mm^2,\ \bar{r}_0^2 = 54.1^2 + 32.2^2 = 3964\ mm^2$

$F_{ex} = \frac{\pi^2*E}{\left[\frac{K_xL_x}{r_x}\right]^2} = \frac{\pi^2*200{,}000}{[64.6]^2} = 473\ MPa$ $F_{ey} = \frac{\pi^2*E}{\left[\frac{K_yL_y}{r_y}\right]^2} = \frac{\pi^2*200{,}000}{[108.96]^2} = 166.3\ MPa$

$F_{ez} = \left[\frac{\pi^2 * E * C_w}{[K_zL_z]^2} + JG\right]\frac{1}{A*\bar{r}_0^2} = 631.4\ MPa$

$F_e = 166.3\,MPa,\ \lambda = \sqrt{\frac{350}{166.3}} = 1.45$ $C_r = \frac{\phi A F_y}{[1+\lambda^{2n}]^{\frac{1}{n}}} = 361\ KN$

2.1.5 CBF Brace on First Floor (W200x46)

Governing Load Combination:1.25D + 1.4Wy + 0.5RL + 0.5LL

Loading Action: Axial

Section: W200x46, n = 1.34

Slenderness Limit

$r_x = 88.1\,mm, r_y = 51.2\,mm$

$\frac{K_x L_x}{r_x} = \frac{1.0*6946\ mm}{88.1\,mm} = 78.8 < 200 \therefore ok$ $\frac{K_y L_y}{r_y} = \frac{1.0*6946\,mm}{51.2\,mm} = 135.7 < 200 \therefore ok$

Local Buckling Checks

$d = 203\ mm, b = 203\ mm, t = 11.0\,mm, w = 7.2\ mm$

Flange: $\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}, \frac{b_{el}}{t} = \frac{\frac{b}{2}}{t} = \frac{\frac{203\ mm}{2}}{11.0\,mm} = 9.23 < 10.69 \therefore ok$

Web: $\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}, \frac{b_{el}}{t} = \frac{d-2*t}{w} = \frac{203\ mm-(2*11.0\ mm)}{7.2\ mm} = 25.14 < 35.8 \therefore ok$

Compressive Capacity

$C_w = 141x10^9\ mm^6, J = 220x10^3\ mm^4, A = 5890\ mm^2,\ \bar{r}_0^2 = 88.1^2 + 51.2^2 = 10{,}383.05\ mm^2$

$F_{ex} = \frac{\pi^2*E}{\left[\frac{K_xL_x}{r_x}\right]^2} = \frac{\pi^2*200{,}000}{[78.8]^2} = 317.89\ MPa$ $F_{ey} = \frac{\pi^2*E}{\left[\frac{K_yL_y}{r_y}\right]^2} = \frac{\pi^2*200{,}000}{[135.7]^2} = 107.19\ MPa$

$F_{ez} = \left[\frac{\pi^2*E*C_w}{[K_zL_z]^2} + JG\right]\frac{1}{A*\bar{r}_0^2} = 371.3\ MPa$ $F_e = 107.19\ MPa,\ \lambda = \sqrt{\frac{350}{107.19}} = 1.81$

$C_r = \frac{\phi A F_y}{[1+\lambda^{2n}]^{\frac{1}{n}}} = 495\ KN$

2.1.6 Gravity Beam on First Floor (W360x51)

Governing Load Combination: 1.25D + 1.5RL + 1.5LL

Loading Action: Moment 3-3, Shear 2-2

Section Class (Table 2):

$\frac{b_{el}}{t} = \frac{b/2}{t} = \frac{171/2}{11.6} = 7.37$ $\frac{170}{\sqrt{F_y}} = \frac{170}{\sqrt{350}} = 9.1$ $\because \frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$

$\therefore flange\ is\ better\ than\ class\ 2$

$\frac{h}{w} = \frac{d-2t}{w} = \frac{355 - 2 \times 11.6}{7.2} = 46.1$ $\frac{1700}{\sqrt{Fy}} = \frac{1700}{\sqrt{350}} = 90.9$ $\because \frac{h}{w} \leq \frac{1700}{\sqrt{Fy}}$

$\therefore web\ is\ better\ than\ class\ 2$

Moment Capacity:

[§ 13.5.a] $M_P = Z_x F_y = 8.93 \times 10^5 \times 350 = 312.55\ kNm$

[§ 13.6.a.ii] $\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2}} = \frac{4\times 273.94}{\sqrt{273.94^2 + 4\times 235.42^2 + 7\times 256.82^2 + 4\times 269.66^2}} = 1.07 > 2.5$

$\therefore \omega_2 = 1.07$

[§ 13.6.a.ii] $M_u = \frac{\omega_2 \pi}{L}\sqrt{EIyGJ + (\frac{\pi E}{L})^2 IyCw}$

$$= \frac{1.07 \times \pi}{1500}\sqrt{(200{,}000)(9.68\times 10^6)(7.7\times 10^4)(2.37\times 10^5) + \left(\frac{200{,}000\pi}{1500}\right)^2 (9.68\times 10^6)(2.85\times 10^{11})} = 1615.05\ kNm$$

Check Mu > 0.67Mp: $1616.05 > 0.67 \times 312.55 = 209.4\ KNm$

[§ 13.6.a.i] Mr = 1.15ϕMp (1 - $\frac{0.28Mp}{Mu}$) $= 1.15(0.9)(312.55)\left[1 - \frac{0.28(312.55)}{1615.05}\right] = 305.96\ kNm$

Check Mr < ϕMp : $305.96 < 0.9 \times 312.55\ KNm = 281.3\ kNm$

$\therefore Mr = 281.3\ kNm$

$\because Mr > Mf = 273.94\ kNm$ $\therefore Moment\ check\ passes$

Shear Resistance:

[§ 13.4.1.1] $A_w = dw = 355(7.2) = 2556\ mm^2$

$\frac{1014}{\sqrt{F_y}} = \frac{1014}{\sqrt{350}} = 54.20$ $\therefore \frac{h}{w} = 46.1 \leq \frac{1014}{\sqrt{F_y}}$

[§ 13.4.1.1.a.i] $\therefore F_s = 0.66F_y = 0.66(350) = 231\ MPa$

[§ 13.4.1] $V_r = \Phi A_w F_s = 0.9(2556)(231) = 531.4\ kN$

$\because Vr > Vf = 181.862\ kN$ $\therefore Shear\ check\ passes$

Deflection Check:

[Table D.1] $Deflection\ Limit = \frac{L}{300} = \frac{6\ m}{300} = 0.02\ m$

$w_{max}\ of\ (snow, wind, live) = 2.4kPa * 6m = 14.4\frac{KN}{m}$

$Deflection = \frac{5wL^4}{384EI} = \frac{5(14.4)6000^4}{384(200000)(141\times 10^6)} = 0.0086\ m < 0.02\ m\ \therefore pass$

2.1.7 Gravity Column on First Floor (W200x36)

Governing Load Combination: 1.25D + 1.5RL + 1.5LL

Loading Action: Axial

<u>Slenderness Limit</u>

$r_x = 89.0\,mm, r_y = 51.8\,mm$

$\frac{K_xL_x}{r_x} = \frac{1.0*3500\,mm}{89.0\,mm} = 39.33 < 200 \therefore ok$ $\frac{K_yL_y}{r_y} = \frac{1.0*3500\,mm}{51.8\,mm} = 67.57 < 200 \therefore ok$

<u>Local Buckling Checks</u>

$d = 206\,mm, b = 204\,mm, t = 12.6\,mm, w = 7.9\,mm$

Flange: $\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}, \frac{b_{el}}{t} = \frac{\frac{b}{2}}{t} = \frac{\frac{204\,mm}{2}}{12.6\,mm} = 8.095 < 10.69 \therefore ok$

Web: $\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}, \frac{b_{el}}{t} = \frac{d-2*t}{w} = \frac{204\,mm-(2*12.6\,mm)}{7.9\,mm} = 22.89 < 35.8 \therefore ok$

<u>Compressive Capacity</u>

$$C_w = 167 \times 10^9\,mm^6, J = 323 \times 10^3\,mm^4, A = 6650\,mm^2$$
$$\bar{r}_0^2 = 89.0^2 + 51.8^2 = 10604.2\,mm^2$$

$F_{ex} = \frac{\pi^2*E}{\left[\frac{K_xL_x}{r_x}\right]^2} = \frac{\pi^2*200{,}000}{[39.33]^2} = 1283.3\,MPa$ $F_{ey} = \frac{\pi^2*E}{\left[\frac{K_yL_y}{r_y}\right]^2} = \frac{\pi^2*200{,}000}{[67.57]^2} = 432.34\,MPa$

$F_{ez} = \left[\frac{\pi^2*E*C_w}{[K_zL_z]^2} + JG\right]\frac{1}{A*\bar{r}_0^2} = 734.3$ MPa $F_e = 432.34\,MPa,\ \lambda = \sqrt{\frac{350}{432.34}} = 0.90, n = 1.34$

$C_r = \frac{\phi AF_y}{[1+\lambda^{2n}]^{\frac{1}{n}}} = 1377.3\,KN$ $\because Cr > Cf = 1339\,kN$ $\therefore$ *passes*

3.0 Connection Details

3.1 Shear Tab

Shear Force: $V_f = 181.862\,kN$ 2 Shear Tabs: $\frac{V_f}{2} = 90.931\,kN$

Beam: W360x51 Column: W200x52

<u>Bolt Requirement:</u>

[§13.12.1.2] $V_r = 0.60\phi_b nm A_b F_u 0.7$
$181862 = 0.60(0.80)(2)(2)A_b(825)(0.7)$
$A_b = 80.4\,mm^2$

[§22.3.1] Min. Pitch Distance:
$2.7d_h = 2.7(0.5)(25.4) = 12.7\,mm$

[Table 3-1] $F_u = 825\,MPa\ (A325\ d > 1")$
Assume that threads are intercepted
Assume plate thickness: t=20
Use A325 ½" Bolts
75>12.7

Using the Tables in Part 3 HSC (p. 3-31 to 3-37)

$$[\S 13.12.1.2]\ Vr = 0.60\Phi_b nm A_b F_u = 100.58\,kN$$

$$C = \frac{P_f}{V_r} = \frac{90.931}{100.58} = 0.904$$

Using b = 75 mm, L=50 mm
$Pr = (1.2)(100.6) = 120.7\,kN$

C=1.20
120.7 > 90.9

Using ICR Method

$$V_r = 100.58\ kN$$

Assume ICR, x = 80 mm

$$V_1 = V_w = 100.58\ kN$$

$$\theta_1 = \theta_2 = \tan^{-1}(\frac{b}{x})$$

$$\sum F_y = 0$$

$$F_1 \cos\theta + F_2 \cos\theta - P = 0$$

$$\Delta_{max} = 8.64\ mm$$

$$\Delta_i = \frac{r_i}{r_{max}} \Delta_{max}$$

$$r_1 = r_2 = \sqrt{x^2 + b^2}$$

$$\sum M_{ICR} = 0, P = \frac{F_1 r_1 + F_2 r_2}{(x + e)}$$

Iterations:

x (mm)	r1 = r2 (mm)	θ (°)	P (bolts) (kN)	P (M) (kN)
80	110	43.2	147.1	169.8
90	117	39.8	154.6	168.4
110	133	34.3	166.2	167.4

Error < 3% $P_{rt} = 166.2(2) = 332.2\ kN$

Weld Requirement:

[§13.13.2.2] One Weld (M_w=1.0): $A_w = 0.707DL$

$$V_r = 0.60\Phi_b A_w X_u (1 + 0.5 \sin^{1.5}\theta)\ M_w$$

$$90931 = 0.67(0.67)(0.707D)(131)(490)(1)(1)$$

$$4.464 = 0.707D \qquad D = 6.31 \qquad Spec. \rightarrow D = 6\ mm$$

Use L103x90x20

Net Section Fracture

[§13.2.a.iii] $T_r = \emptyset_U A_{ne} F_u$

$$= (0.75)(1561.6)(450) = 527\ kN$$

[§12.3.1.b] $A_n = (w_n)t$

$$= (28 + 75 + 28 - 2((0.5)(25.4) + 2 + 2))(20)$$

$$= 1952\ mm^2$$

Assume bolt holes are punched
[Table 6-3] F_u = 450 MPa
[Table 6-3] F_y = 350 MPa
[§13.1] $\emptyset_U = 0.75$
[§12.3.3.2.b.ii] $A_{ne} = 0.8A_n$

Block Shear

[§13.11] $T_r = \emptyset_U [U_t A_n F_u + 0.6 A_{gv} \frac{(F_y + F_u)}{2}]$

$$= (0.75)\left[(0.6)(893)(450) + (0.6)(2060)\frac{(350 + 450)}{2}\right]$$

$$= 551.6\ kN$$

$$A_n = [(25 + 28) - 0.5(0.5)((25.4) + 2 + 2)](20)$$

$$= 893\ mm^2$$

$$A_{gv} = (75 + 28)(20) = 2060\ mm^2$$

[§13.11] $U_t = 0.6$

Bolt Bearing

[§13.12.1.2]

$$B_r = 3\emptyset_{br} ntd F_u = (3)(0.8)(2)(20)(12.7)(450) = 548.6\ kN$$

$$d_b = 0.5(25.4) = 12.7\ mm$$

n = 2
[Table 6-3] F_u = 450 MPa
[§13.1] $\emptyset_{br} = 0.8$

3.2 Gusset Plate

Max Tensile Force = 348 kN

Slenderness Limit Check

[§10.4.2.2] $\frac{L_x}{r_x} = \frac{6946.2}{88.1} = 78.8 < 300;\ \frac{L_y}{r_y} = \frac{6946.2}{51.2} = 135.67 < 300;$

Gross Area Yielding

[§13.2] $T_r = \phi A_g F_y = 0.9(5890)(350) = 1855\ kN$

Selection of Number of Bolts

[§13.12.1.2] $V_r = 0.6\phi_b nm A_b F_u$

Iteration 1: let n = 4, select M16 bolts

$V_r = 0.6(0.8)(4)(2)(201)(825)(0.7) = 446\ kN > T_f = 348\ kN\ \therefore 4\ bolts\ are\ sufficient$

Net Section Fracture Check

Assume holes are punched: $A_n = 5890 - 2(16+2+2)(7.2) = 5602\ mm^2$

[§12.3.3.2] $A_{ne} = 0.75A_n = 0.75(5602) = 4201.5\ mm^2$

[§13.2] $T_r = \phi_u A_{ne} F_u = 0.75(4201.5)(450) = 1418\ kN > 348\ kN\ \therefore pass$

Block Shear

Determine proper bolt spacing r

[§22.3.1] $Min\ Pitch = 2.7d_b = 2.7(16) = 43.2\ mm\ \therefore set\ pitch\ dist = 60\ mm$

[§22.3.2, Table 6] $Min\ Edge = 28\ mm$

[§22.3.3] $Max\ Edge < \min(12t, 150) < \min(12 \times 7.2), 150) < 86.4\ \therefore \max\ edge\ dist = 86.4\ mm$

[§22.3.4] $Min\ End = 1.5d_b = 1.5(16) = 24\ mm$

$A_n = [60 - (16+4)](7.2) = 288\ mm^2$

$A_{gv} = 2(40+60)(7.2) = 1440\ mm^2$

[§13.11] $T_r = \phi_u[U_t A_n F_u + 0.6A_{gv}\frac{(F_y+F_u)}{2}] = 0.75[1(288)(450) + 0.6(1440)\frac{(350+450)}{2} = 356(kN) > 348\ kN\ \therefore pass$

Bolt Bearing

[§13.12.1.2] $B_r = 3\phi_{br} ntd F_u = 3(0.8)(4)(7.2)(16)(450) = 497\ kN$

Bolt Tearout

[§13.11] $T_r = \phi_u 0.6A_{gv}\frac{(F_y+F_u)}{2} = 0.75(0.6)[4(40+60)(7.2)]\left[\frac{(350+450)}{2}\right] = 518\ kN > 348\ kN\ \therefore pass$

3.2.1 Splice Plate (Assume thickness of each splice plate = 6 mm)

Gross Area Yielding

[§13.2] $T_r = \phi A_g F_y = 0.9(140)(12)(350) = 529\ kN > 348\ kN\ \therefore pass$

Net Section Fracture Check

Assume holes are punched: $A_n = 140(12) - 2(16+2+2)(12) = 1200\ mm^2$

[§13.2] $T_r = \phi_u A_{ne} F_u = 0.75(1200)(450) = 405\ kN > 348\ kN\ \therefore pass$

3.3 Base Plate

Governing Load Combination: C_f = -1339 kN

Variables:

d = 206 mm, b = 204 mm, t = 12.6 mm, w = 7.9 mm b_{el} = b/2 = 204/2 = 102 mm, h = d – 2t = 180.8 mm

Dimensions:

[§ 25.3.1] $A = \frac{C_f}{B_r}; B_r = 0.85\Phi f'c$ A = $\frac{1339 \times 10^3}{0.85 \times 0.65 \times 20} = 121{,}176 mm^2$

Choose C = 380 mm, B = 330 mm, Area = 125,400 mm^2 > 121,176 mm^2

Plate thickness:

m = (C – 0.95d)/2 = 92.15 mm n = (B – 0.80b)/2 = 83.4 mm n<m, use n for design

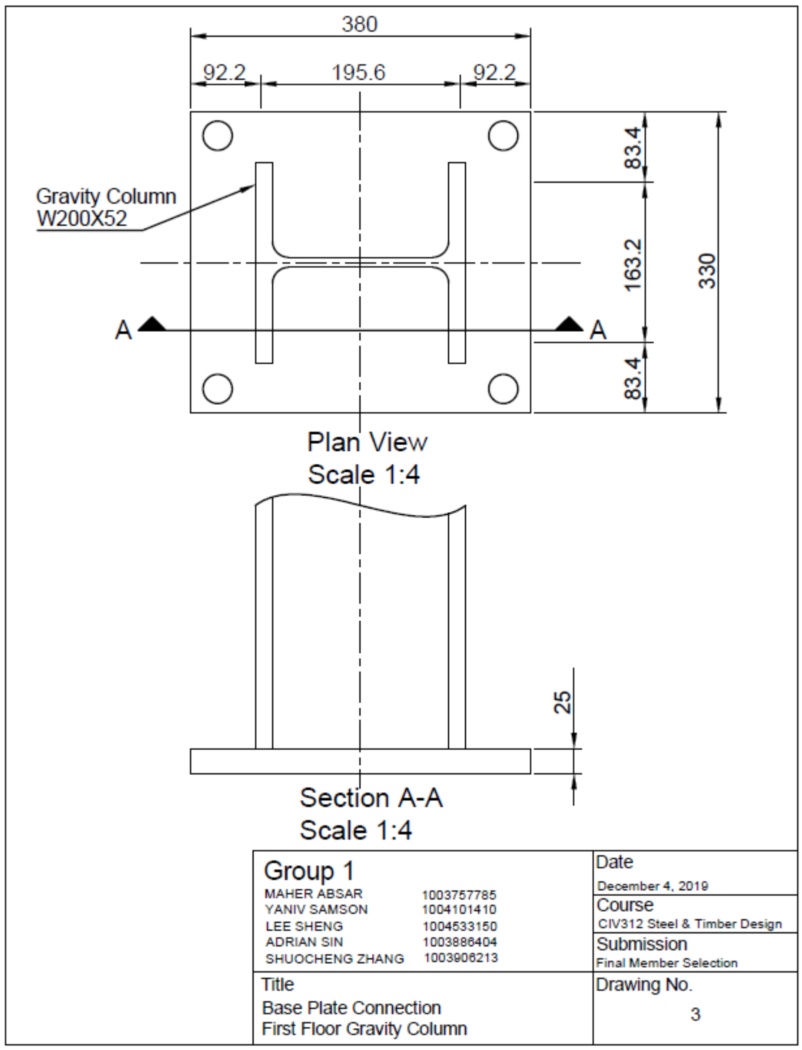
Plate thickness required = $t_p = \sqrt{\frac{2\,C_f n^2}{BC\varphi F_y}} = \sqrt{\frac{2\times1339\times10^3\times83.4^2}{330\times380\times0.9\times300}} = 23.4\,mm$

n/5 = 16.7 mm < 23.4 mm OK Use 25 mm (25 mm < 65 mm, Fy = 300 MPa)

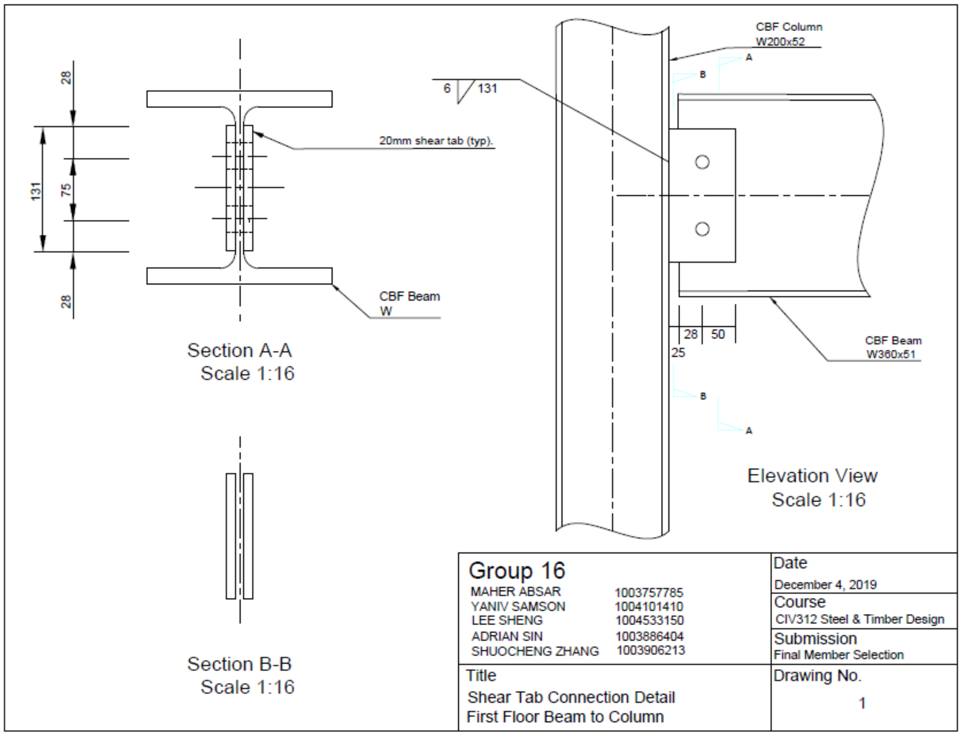
Use PL 25x380x330

4.0 Connection Drawings

4.1 Base Plate



4.2 Shear Tab



4.3 Gusset Plate