

Final Design Project

CIV312 – Steel and Timber Design

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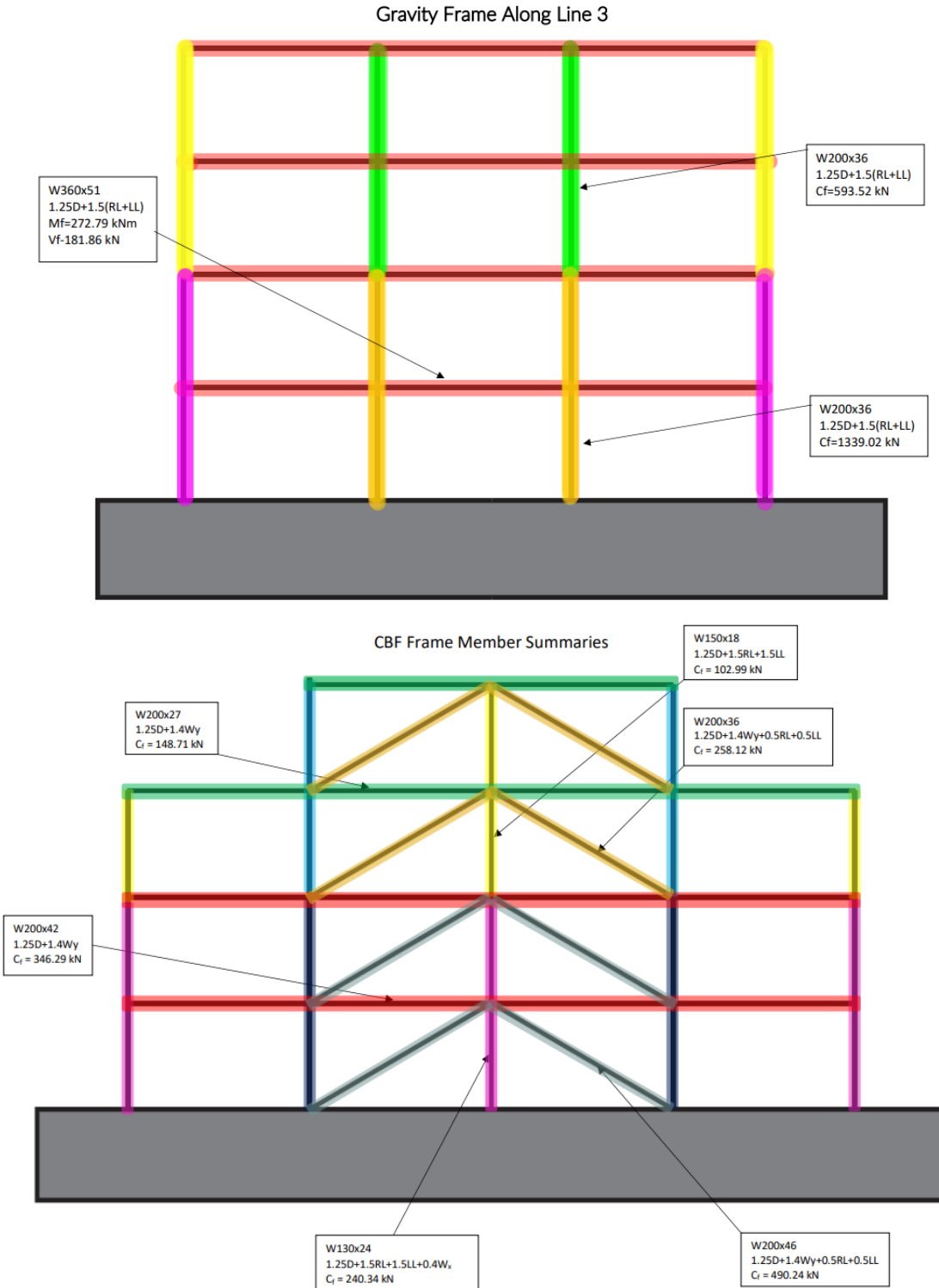
Group 16

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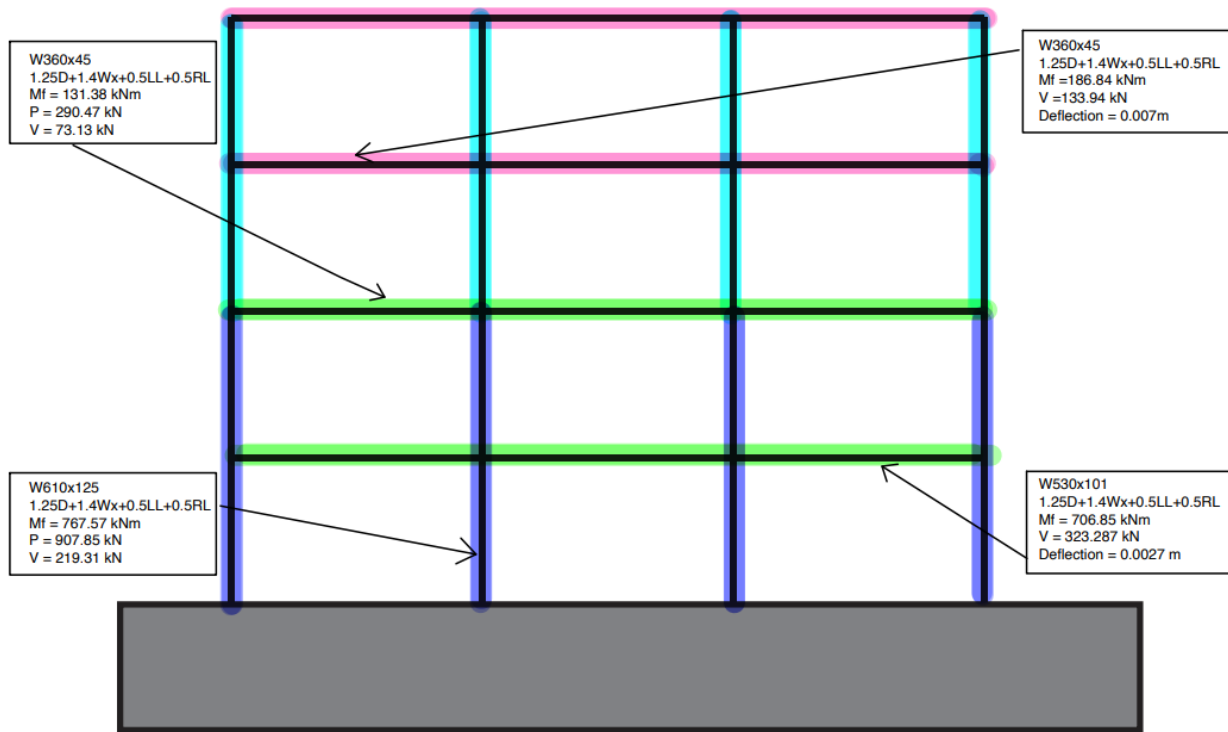
## 1.0 Introduction

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 1<sup>st</sup> 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):

$$\frac{b_{el}}{t} = \frac{b/2}{t} = \frac{210/2}{17.4} = 6.03$$

$$\frac{h}{w} = \frac{d - 2t}{w} = \frac{352 - 2 \times 9.8}{6.9} = 48.17$$

$$\frac{170}{\sqrt{F_y}} = \frac{170}{\sqrt{350}} = 9.1$$

$$\frac{1700}{\sqrt{F_y}} = \frac{1700}{\sqrt{350}} = 90.9$$

$$\therefore \frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$$

$\therefore$  flange is better than class 2

$$\therefore \frac{h}{w} \leq \frac{1700}{\sqrt{F_y}}$$

$\therefore$  web is better than class 2

Moment Capacity:

$$[\S 13.5.a] M_p = Z_x F_y = 2.62 \times 10^6 \times 350 = 917 \text{ kNm}$$

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

$$\therefore \omega_2 > 2.5$$

$$\therefore \omega_2 = 2.5$$

$$[\S 13.6.a.ii] M_u = \frac{\omega_2 \pi}{L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

$$= \frac{2.5 \times \pi}{6000} \sqrt{(200,000)(2.69 \times 10^7)(7.7 \times 10^4)(1.02 \times 10^6) + \left(\frac{200,000\pi}{6000}\right)^2 (2.69 \times 10^7)(1.82 \times 10^{12})} = 1282.17 \text{ kNm}$$

$$\text{Check } \mu > 0.67\mu_p: 1282.17 \text{ kNm} > 0.67 \times 917 \text{ kNm} = 614.39 \text{ kNm}$$

$$[\S 13.6.a.i] M_r = 1.15\phi M_p \left(1 - \frac{0.28M_p}{M_u}\right) = 1.15(0.9)(917) \left[1 - \frac{0.28(917)}{1282.17}\right] = 759.04 \text{ kNm}$$

$$\text{Check } M_r < \phi M_p : 759.04 \text{ kNm} < 0.9 \times 917 \text{ kNm} = 825.3 \text{ kNm}$$

$$\therefore M_r = 759.04 \text{ kNm}$$

$$\therefore M_r > M_f = 706.85 \text{ kNm} \quad \therefore \text{Moment check passes}$$

### Shear Resistance:

$$[\S 13.4.1.1] A_w = d w = 537(10.9) = 5853.3 \text{ mm}^2$$

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

$$[\S 13.4.1.1.a.i] \therefore F_s = 0.66F_y = 0.66(350) = 231 \text{ MPa}$$

$$[\S 13.4.1] V_r = \Phi A_w F_s = 0.9(5853.3)(231) = 1216.90 \text{ kN}$$

$$\therefore V_r > V_f = 323.29 \text{ kN} \quad \therefore \text{Shear check passes}$$

### Deflection Check:

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

$$\text{Deflection (reading from SAP)} = 0.0027 \text{ m} < 0.02 \text{ m} \quad \therefore \text{pass}$$

### **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:**  $L = 6000 \text{ mm}$ ,  $n = 1.34$

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

$$C_w = 84 \times 10^9 \text{ mm}^6, J = 222 \times 10^3 \text{ mm}^4, A = 5320 \text{ mm}^2, \bar{r}_0^2 = 87.7^2 + 41.2^2 = 9389 \text{ mm}^2$$

### Slenderness Limit

$$\frac{k_x L_x}{r_x} = \frac{1.0 \times 6000 \text{ mm}}{87.7 \text{ mm}} = 68.4 < 200 \quad \therefore \text{ok}$$

$$\frac{k_y L_y}{r_y} = \frac{1.0 \times 6000 \text{ mm}}{41.2 \text{ mm}} = 145.6 < 200 \quad \therefore \text{ok}$$

### Local Buckling Checks

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

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

### Compressive Capacity

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

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

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

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

**2.1.3 MRF Column on First Floor (W610x125)****Governing Load Combination:**  $1.25D + 1.4W_x + 0.5RL + 0.5LL$ **Loading Action:** *Moment 3-3, Axial, Shear 2-2***Section Class (Table 2):**

$$\frac{b_{el}}{t} = \frac{b/2}{t} = \frac{229/2}{19.6} = 5.84$$

$$\frac{170}{\sqrt{F_y}} = \frac{170}{\sqrt{350}} = 9.1$$

$$\therefore \frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}} \\ \therefore \text{flange is better than class 2}$$

$$\frac{h}{w} = \frac{d - 2t}{w} = \frac{612 - 2 \times 19.6}{11.9} = 48.13$$

$$\frac{1700}{\sqrt{F_y}} = \frac{1700}{\sqrt{350}} = 90.9$$

$$\therefore \frac{h}{w} \leq \frac{1700}{\sqrt{F_y}} \\ \therefore \text{web is better than class 2}$$

**Cross Sectional Strength (Compression):**

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

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

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

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

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

$$[\S 13.8.4] C_{ex} = \frac{\pi^2 E I_x}{L^2} = \frac{\pi^2 (200,000)(9.85 \times 10^8)}{3.5^2} = 1.59 \times 10^5 \text{ kN}$$

$$[\S 13.8.4] U_{1x} = \frac{\omega_1}{1 - \frac{C_f}{C_e}} = \frac{0.6}{1 - \frac{907.85}{1.59 \times 10^5}} = 0.6 < 1$$

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

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

**Overall Member Strength (Strong Axis):**

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

$$[\S 13.3.1] Cr = \frac{\Phi A F_y}{(1 + \lambda^2 n)^{\frac{1}{n}}} = \frac{0.9(15900)(350)}{(1 + 0.19^2 \times 1.34)^{\frac{1}{1.34}}} = 4967.01 \text{ kN}$$

$$[\S 13.8.4] U_{1x} = \frac{\omega_1}{1 - \frac{C_f}{C_e}} = \frac{0.6}{1 - \frac{907.85}{1.59 \times 10^5}} = 0.6$$

$$[\S 13.5.a] M_r = \Phi Z_x F_y = 0.9(3.67 \times 10^6)(350) = 1156.5 \text{ kNm}$$

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

**Lateral Torsional Buckling Strength (Weak axis):**

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

$$[\S 13.3.1] Cr = \frac{\Phi A F_y}{(1 + \lambda^2 n)^{\frac{1}{n}}} = \frac{0.9(15900)(350)}{(1 + 0.94^2 \times 1.34)^{\frac{1}{1.34}}} = 3175.24 \text{ kN}$$

$$[\S 13.5.a] M_p = Z_x F_y = (3.67 \times 10^6)(350) = 1284.50 \text{ kNm}$$

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

$$[\S 13.6.a.ii] M_u = \frac{\omega_2 \pi}{L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

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

$$= 3616.78 \text{ kNm}$$

$$\text{Check if } M_u > 0.67 M_p : 3616.78 \text{ kNm} > 0.67 \times 1284.50 \text{ kNm} = 860.62 \text{ kNm}$$

$$[\S 13.6.a.i] M_r = 1.15 \phi M_p \left(1 - \frac{0.28 M_p}{M_u}\right) = 1.15(0.9)(1284.50) \left(1 - \frac{0.28(1284.50)}{3616.78}\right) = 1197.25 \text{ kNm}$$

$$\text{Check if } M_r < \phi M_p : 1197.25 > 0.9 \times 1284.50 = 1156.05 \text{ kNm}$$

$$\therefore M_r > \phi M_p \quad \therefore M_r = \phi M_p = 1156.05 \text{ kNm}$$

$$[\S 13.8.4] C_{ey} = \frac{\pi^2 EI_y}{L^2} = \frac{\pi^2 (200,000)(3.93 \times 10^7)}{3.5^2} = 6332.66 \text{ kN}$$

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

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

$$[\S 13.8.4] U_{1x} = \frac{\omega_1}{C_r} = \frac{0.6}{1 - \frac{f}{C_e}} = \frac{0.6}{1 - \frac{907.85}{6332.66}} = 0.70 < 1$$

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

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

#### **Check for Low Axial Loads with High Moments:**

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

#### **Shear Resistance:**

$$[\S 13.4.1.1] A_w = dw = 612(11.9) = 7282.8 \text{ mm}^2$$

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

$$[\S 13.4.1.1.a.i] \therefore F_s = 0.66 F_y = 0.66(350) = 231 \text{ MPa}$$

$$[\S 13.4.1] V_r = \phi A_w F_s = 0.9(7282.8)(231) = 1514.09 \text{ kN}$$

$$\therefore V_r > V_f = 219.31 \text{ kN} \quad \therefore \text{Shear check passes}$$

**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**

$$r_x = 54.1 \text{ mm}, r_y = 32.2 \text{ mm}$$

$$\frac{K_x L_x}{r_x} = \frac{1.0 \cdot 3500 \text{ mm}}{54.1 \text{ mm}} = 64.6 < 200 \therefore \text{ok}$$

$$\frac{K_y L_y}{r_y} = \frac{1.0 \cdot 3500 \text{ mm}}{32.2 \text{ mm}} = 108.96 < 200 \therefore \text{ok}$$

**Local Buckling Checks**

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

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

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

**Compressive Capacity**

$$C_w = 10.8 \times 10^9 \text{ mm}^6, J = 76.2 \times 10^3 \text{ mm}^4, A = 3040 \text{ mm}^2, \bar{r}_0^2 = 54.1^2 + 32.2^2 = 3964 \text{ mm}^2$$

$$F_{ex} = \frac{\pi^2 E}{\left[\frac{K_x L_x}{r_x}\right]^2} = \frac{\pi^2 \cdot 200,000}{[64.6]^2} = 473 \text{ MPa}$$

$$F_{ey} = \frac{\pi^2 E}{\left[\frac{K_y L_y}{r_y}\right]^2} = \frac{\pi^2 \cdot 200,000}{[108.96]^2} = 166.3 \text{ MPa}$$

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

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

**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**

$$r_x = 88.1 \text{ mm}, r_y = 51.2 \text{ mm}$$

$$\frac{K_x L_x}{r_x} = \frac{1.0 \cdot 6946 \text{ mm}}{88.1 \text{ mm}} = 78.8 < 200 \therefore \text{ok}$$

$$\frac{K_y L_y}{r_y} = \frac{1.0 \cdot 6946 \text{ mm}}{51.2 \text{ mm}} = 135.7 < 200 \therefore \text{ok}$$

**Local Buckling Checks**

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

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

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

**Compressive Capacity**

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

$$F_{ex} = \frac{\pi^2 E}{\left[\frac{K_x L_x}{r_x}\right]^2} = \frac{\pi^2 \cdot 200,000}{[78.8]^2} = 317.89 \text{ MPa}$$

$$F_{ey} = \frac{\pi^2 E}{\left[\frac{K_y L_y}{r_y}\right]^2} = \frac{\pi^2 \cdot 200,000}{[135.7]^2} = 107.19 \text{ MPa}$$

$$F_{ez} = \left[ \frac{\pi^2 E C_w}{[K_z L_z]^2} + JG \right] \frac{1}{A \cdot \bar{r}_0^2} = 371.3 \text{ MPa}$$

$$F_e = 107.19 \text{ MPa}, \lambda = \sqrt{\frac{350}{107.19}} = 1.81$$

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

**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):**

$$\frac{b_{el}}{t} = \frac{b/2}{t} = \frac{171/2}{11.6} = 7.37$$

$$\frac{170}{\sqrt{F_y}} = \frac{170}{\sqrt{350}} = 9.1$$

$$\therefore \frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}} \\ \therefore \text{flange is better than class 2}$$

$$\frac{h}{w} = \frac{d - 2t}{w} = \frac{355 - 2 \times 11.6}{7.2} = 46.1$$

$$\frac{1700}{\sqrt{F_y}} = \frac{1700}{\sqrt{350}} = 90.9$$

$$\therefore \frac{h}{w} \leq \frac{1700}{\sqrt{F_y}} \\ \therefore \text{web is better than class 2}$$

**Moment Capacity:**

$$[\S 13.5.a] M_p = Z_x F_y = 8.93 \times 10^5 \times 350 = 312.55 \text{ kNm}$$

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

$$\therefore \omega_2 = 1.07$$

$$[\S 13.6.a.ii] M_u = \frac{\omega_2 \pi}{L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

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

$$\text{Check } M_u > 0.67M_p: 1615.05 > 0.67 \times 312.55 = 209.4 \text{ kNm}$$

$$[\S 13.6.a.i] M_r = 1.15\phi M_p \left(1 - \frac{0.28M_p}{M_u}\right) = 1.15(0.9)(312.55) \left[1 - \frac{0.28(312.55)}{1615.05}\right] = 305.96 \text{ kNm}$$

$$\text{Check } M_r < \phi M_p : 305.96 < 0.9 \times 312.55 \text{ kNm} = 281.3 \text{ kNm}$$

$$\therefore M_r = 281.3 \text{ kNm}$$

$$\therefore M_r > M_f = 273.94 \text{ kNm} \quad \therefore \text{Moment check passes}$$

**Shear Resistance:**

$$[\S 13.4.1.1] A_w = dw = 355(7.2) = 2556 \text{ mm}^2$$

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

$$[\S 13.4.1.1.a.i] \therefore F_s = 0.66F_y = 0.66(350) = 231 \text{ MPa}$$

$$[\S 13.4.1] V_r = \Phi A_w F_s = 0.9(2556)(231) = 531.4 \text{ kN}$$

$$\therefore V_r > V_f = 181.862 \text{ kN} \quad \therefore \text{Shear check passes}$$

**Deflection Check:**

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

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

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



**2.1.7 Gravity Column on First Floor (W200x36)****Governing Load Combination:**  $1.25D + 1.5RL + 1.5LL$ **Loading Action:** Axial**Slenderness Limit**

$$r_x = 89.0 \text{ mm}, r_y = 51.8 \text{ mm}$$

$$\frac{K_x L_x}{r_x} = \frac{1.0 \times 3500 \text{ mm}}{89.0 \text{ mm}} = 39.33 < 200 \therefore \text{ok}$$

$$\frac{K_y L_y}{r_y} = \frac{1.0 \times 3500 \text{ mm}}{51.8 \text{ mm}} = 67.57 < 200 \therefore \text{ok}$$

**Local Buckling Checks**

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

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

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

**Compressive Capacity**

$$C_w = 167 \times 10^9 \text{ mm}^6, J = 323 \times 10^3 \text{ mm}^4, A = 6650 \text{ mm}^2$$

$$\bar{r}_0^2 = 89.0^2 + 51.8^2 = 10604.2 \text{ mm}^2$$

$$F_{ex} = \frac{\pi^2 E}{\left[\frac{K_x L_x}{r_x}\right]^2} = \frac{\pi^2 \times 200,000}{[39.33]^2} = 1283.3 \text{ MPa}$$

$$F_{ey} = \frac{\pi^2 E}{\left[\frac{K_y L_y}{r_y}\right]^2} = \frac{\pi^2 \times 200,000}{[67.57]^2} = 432.34 \text{ MPa}$$

$$F_{ez} = \left[ \frac{\pi^2 E C_w}{[K_z L_z]^2} + JG \right] \frac{1}{A \bar{r}_0^2} = 734.3 \text{ MPa}$$

$$F_e = 432.34 \text{ MPa}, \lambda = \sqrt{\frac{350}{432.34}} = 0.90, n = 1.34$$

$$C_r = \frac{\phi A F_y}{[1 + \lambda^{2n}]^{\frac{1}{n}}} = 1377.3 \text{ kN}$$

$$\therefore C_r > C_f = 1339 \text{ kN}$$

 $\therefore \text{passes}$ **3.0 Connection Details****3.1 Shear Tab**

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

Beam: W360x51

Column: W200x52

**Bolt Requirement:**

$$[\S 13.12.1.2] V_r = 0.60 \phi_b n m A_b F_u 0.7$$

$$181862 = 0.60(0.80)(2)(2) A_b (825)(0.7)$$

$$A_b = 80.4 \text{ mm}^2$$

$$[\text{Table 3-1}] F_u = 825 \text{ MPa (A325 } d > 1")$$

Assume that threads are intercepted

Assume plate thickness:  $t=20$ Use A325  $\frac{1}{2}"$  Bolts

$$75 > 12.7$$

$$[\S 22.3.1] \text{ Min. Pitch Distance:}$$

$$2.7 d_b = 2.7(0.5)(25.4) = 12.7 \text{ mm}$$

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

$$[\S 13.12.1.2] V_r = 0.60 \phi_b n m A_b F_u = 100.58 \text{ kN}$$

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

$$\text{Using } b = 75 \text{ mm, } L = 50 \text{ mm}$$

$$P_r = (1.2)(100.6) = 120.7 \text{ kN}$$

$$C = 1.20$$

$$120.7 > 90.9$$

Using ICR Method

$$\begin{aligned}
 V_r &= 100.58 \text{ kN} \\
 \text{Assume ICR, } x &= 80 \text{ mm} \\
 V_1 &= V_w = 100.58 \text{ kN} \\
 \theta_1 &= \theta_2 = \tan^{-1}\left(\frac{b}{x}\right) \\
 \sum F_y &= 0 \\
 F_1 \cos \theta + F_2 \cos \theta - P &= 0
 \end{aligned}$$

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

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

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

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

Iterations:

x (mm)	r <sub>1</sub> = r <sub>2</sub> (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%  $P_{rt} = 166.2(2) = 332.2 \text{ kN}$ **Weld Requirement:**

$$\begin{aligned}
 [\S 13.13.2.2] \text{ One Weld } (M_w=1.0): & & A_w &= 0.707DL \\
 V_r &= 0.60 \Phi_b A_w X_u (1 + 0.5 \sin^{1.5} \theta) M_w \\
 90931 &= 0.67(0.67)(0.707D)(131)(490)(1)(1) \\
 4.464 &= 0.707D & D &= 6.31 & \text{Spec.} \rightarrow D = 6 \text{ mm}
 \end{aligned}$$

Use L103x90x20

**Net Section Fracture**

$$\begin{aligned}
 [\S 13.2.a.iii] T_r &= \Phi_U A_{ne} F_u \\
 &= (0.75)(1561.6)(450) = 527 \text{ kN} \\
 [\S 12.3.1.b] A_n &= (w_n)t \\
 &= (28 + 75 + 28 - 2((0.5)(25.4) + 2 + 2))(20) \\
 &= 1952 \text{ mm}^2
 \end{aligned}$$

Assume bolt holes are punched

[Table 6-3]  $F_u = 450 \text{ MPa}$ [Table 6-3]  $F_y = 350 \text{ MPa}$ [§13.1]  $\Phi_U = 0.75$ [§12.3.3.2.b.ii]  $A_{ne} = 0.8A_n$ **Block Shear**

$$\begin{aligned}
 [\S 13.11] T_r &= \Phi_U [U_t A_n F_u + 0.6 A_{gv} \frac{(F_y + F_u)}{2}] \\
 &= (0.75) \left[ (0.6)(893)(450) + (0.6)(2060) \frac{(350 + 450)}{2} \right] \\
 &= 551.6 \text{ kN}
 \end{aligned}$$

$$\begin{aligned}
 A_n &= [(25 + 28) - 0.5(0.5)((25.4) + 2 + 2)](20) \\
 &= 893 \text{ mm}^2
 \end{aligned}$$

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

[§13.11]  $U_t = 0.6$ **Bolt Bearing**

$$\begin{aligned}
 [\S 13.12.1.2] \\
 B_r &= 3 \Phi_{br} n t d F_u = (3)(0.8)(2)(20)(12.7)(450) = 548.6 \text{ kN}
 \end{aligned}$$

$$d_b = 0.5(25.4) = 12.7 \text{ mm}$$

n = 2

[Table 6-3]  $F_u = 450 \text{ MPa}$ [§13.1]  $\Phi_{br} = 0.8$ **3.2 Gusset Plate**

Max Tensile Force = 348 kN

**Slenderness Limit Check**

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

**Gross Area Yielding**

$$[\S 13.2] T_r = \Phi A_g F_y = 0.9(5890)(350) = 1855 \text{ kN}$$

**Selection of Number of Bolts**

$$[\S 13.12.1.2] V_r = 0.6 \Phi_b n m A_b F_u$$

Iteration 1: let  $n = 4$ , select M16 bolts

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

### **Net Section Fracture Check**

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

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

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

### **Block Shear**

Determine proper bolt spacing  $r$

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

$$[\S 22.3.2, \text{Table 6}] \text{Min Edge} = 28 \text{ mm}$$

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

$$[\S 22.3.4] \text{Min End} = 1.5d_b = 1.5(16) = 24 \text{ mm}$$

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

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

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

### **Bolt Bearing**

$$[\S 13.12.1.2] B_r = 3\phi_{br} n t d F_u = 3(0.8)(4)(7.2)(16)(450) = 497 \text{ kN}$$

### **Bolt Tearout**

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

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

#### **Gross Area Yielding**

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

### **Net Section Fracture Check**

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

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

### **3.3 Base Plate**

**Governing Load Combination:**  $C_f = -1339 \text{ kN}$

#### **Variables:**

$$d = 206 \text{ mm}, b = 204 \text{ mm}, t = 12.6 \text{ mm}, w = 7.9 \text{ mm} \quad b_{cl} = b/2 = 204/2 = 102 \text{ mm}, h = d - 2t = 180.8 \text{ mm}$$

#### **Dimensions:**

$$[\S 25.3.1] A = \frac{C_f}{B_r}; B_r = 0.85\phi f'c$$

$$A = \frac{1339 \times 10^3}{0.85 \times 0.65 \times 20} = 121,176 \text{ mm}^2$$

$$\text{Choose } C = 380 \text{ mm}, B = 330 \text{ mm}, \text{Area} = 125,400 \text{ mm}^2 > 121,176 \text{ mm}^2$$

Plate thickness:

$$m = (C - 0.95d)/2 = 92.15 \text{ mm} \quad n = (B - 0.80b)/2 = 83.4 \text{ mm} \quad n < m, \text{ use } n \text{ for design}$$

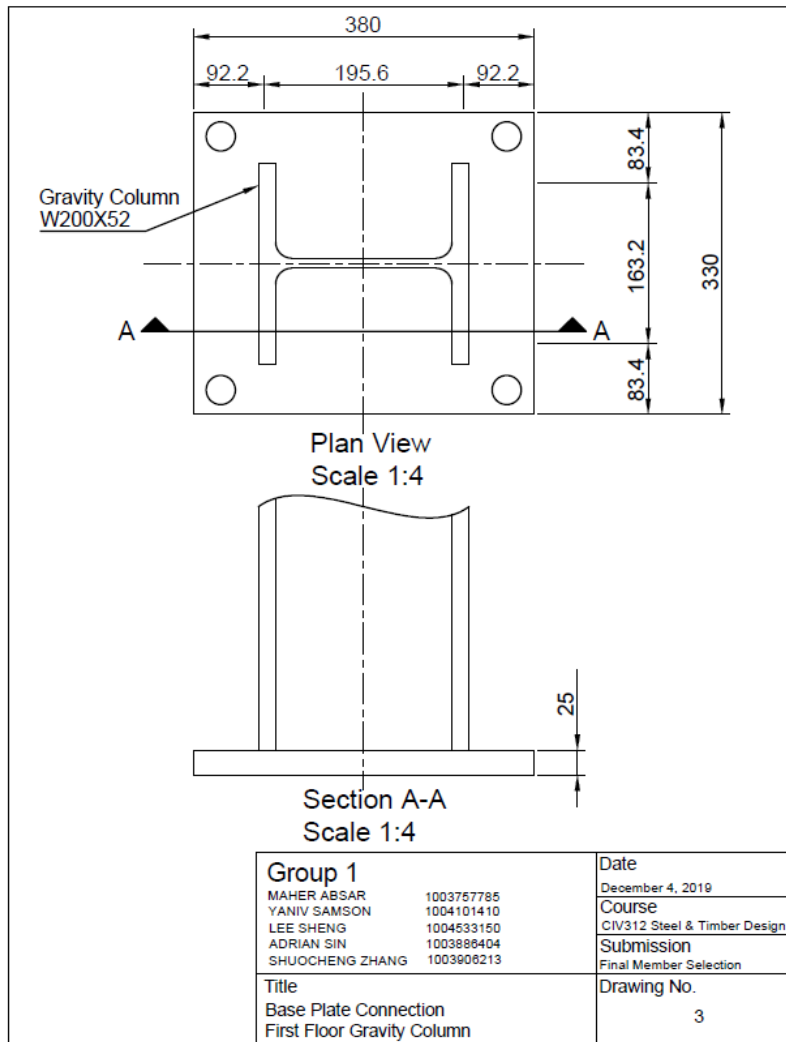
$$\text{Plate thickness required} = t_p = \sqrt{\frac{2C_f n^2}{BC\phi F_y}} = \sqrt{\frac{2 \times 1339 \times 10^3 \times 83.4^2}{330 \times 380 \times 0.9 \times 300}} = 23.4 \text{ mm}$$

$$n/5 = 16.7 \text{ mm} < 23.4 \text{ mm} \quad \text{OK} \quad \text{Use } 25 \text{ mm } (25 \text{ mm} < 65 \text{ mm}, F_y = 300 \text{ MPa})$$

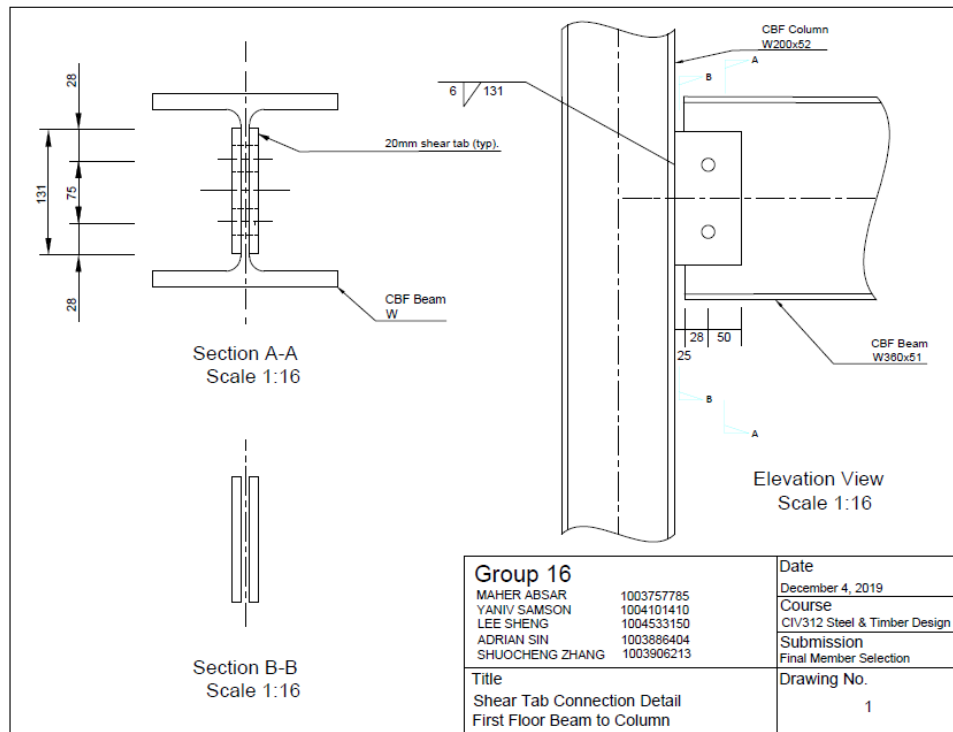
Use PL 25x380x330

#### 4.0 Connection Drawings

##### 4.1 Base Plate



## 4.2 Shear Tab



## 4.3 Gusset Plate

