Student ID: ________________ Name: ____________________

LAKEHEAD UNIVERSITY
Department of Civil Engineering

Engineering 3435 – Steel Structures

FINAL EXAMINATION

Times: 3 hours
December 9, 2010 Instructor: Dr. Y Gong

GENERAL NOTES:

- This exam is open book.
- Read all the pages before answering any questions.
- Write your name on the specified place at the top of pages
- All pages of this examination paper must be returned
- Answer each question only in the specified space. You may write on both sides.
- Any assumptions you make must be clearly stated.
- Detailed calculations based on the governing standard are required for all design problems. You will be credited 20% of the mark if you read solution directly from design tables in Handbook.

Problem No.	**Mark Value**	Your Mark
1	20	
2	20	
3	20	
4	20	
5	20	
Total	100	

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1. As shown in the figure, beams and girders are designed as being simply-supported. The cross-section of girder G1 is W460×67 (A= 8570 mm^2) and the steel is G40.21 350W. For concrete f_c'=20 MPa and E_c=20100 MPa. The specified loads are given as:

Occupancy live load: 2.4 kPa

Dead load: concrete slab = 2.8 kPa, steel = 0.25 kPa, finish and partition wall = 1.2 kPa.

Unshored construction is adopted. For girder G1, lateral supports during construction are provided at B1 joints only. For the design of girder G1, check the following:

(a) Construction condition prior to the hardening of the concrete (assume the construction loads during the casting of concrete include the weight of wet concrete and steel only).

(b) Service load condition.

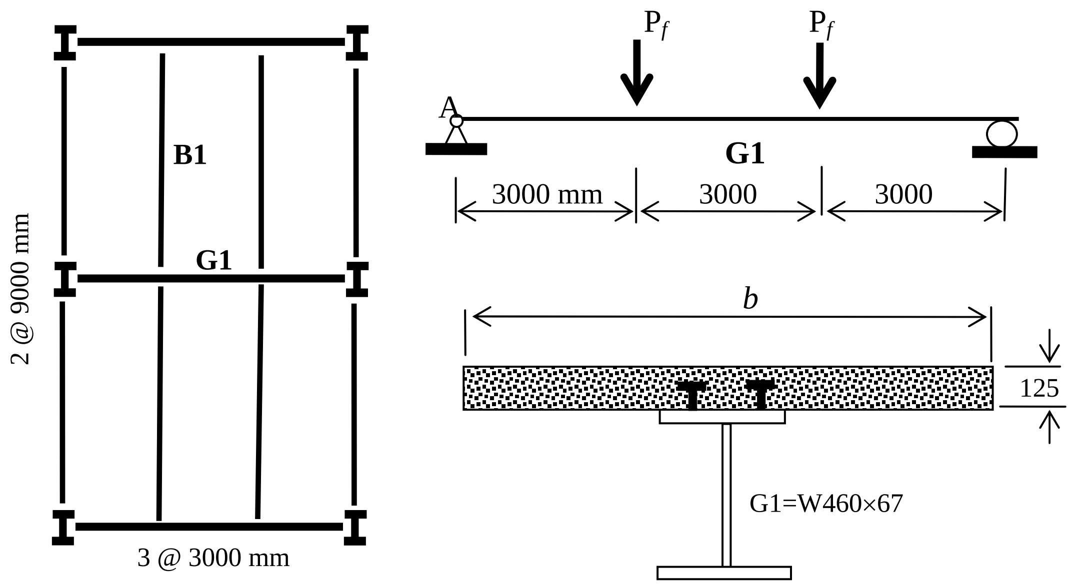

Figure: Composite Girder G1

Solution

(1) Check the construction condition prior to the hardening of the concrete

Step 1: Loads during the construction:

Specified weight of (Slab + steel) = 2.8 + 0.25 = 3.05 KPa

Factored point load at B1 joints P_1= (1.25)(3.05)(9)(3)=103 kN

The maximum factored moment under the factored construction loads is

$$M_f = P_1(3) = 103(3) = 309 \text{ kN-m}$$

Step 2: Compute the capacity of the steel beam

(1) Check section classification of W460×67

Flange: $\dfrac{b_f/2}{t} = \dfrac{190/2}{12.7} = 7.5 < \dfrac{145}{\sqrt{F_y}} = \dfrac{145}{\sqrt{350}} = 7.8$, class 1

Web: $\dfrac{h}{w} = \dfrac{429}{8.5} = 51 < \dfrac{1100}{\sqrt{F_y}} = \dfrac{1100}{\sqrt{350}} = 59$, class 1

Therefore, overall W460×67 is a class 1 section.

(2) Compute M_r

Since lateral supports during construction are provided at B1 joints only, ω_2=1.0 for the middle laterally unsupported segment (which is the governing segment).

The lateral unsupported length L=3000 mm.

$$M_u = \frac{\omega_2 \pi}{L}\sqrt{EI_yGJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

$$= \frac{1.0\pi}{3000}\sqrt{2\times10^5 \times 14.5\times10^6 \times 77000 \times 372\times10^3 + \left(\frac{\pi \times 2\times10^5}{3000}\right)^2 \times 14.5\times10^6 \times 708\times10^9}$$

= 764 × 10^6 N-mm = 764 kN-m

Since M_u = 764 kN-m > 0.67 M_p = 0.67$Z_x F_y$ = 0.67(1470×10^3)(350)/10^6 = 345 kN-m

$$M_r = 1.15\phi M_p\left(1 - \frac{0.28M_p}{M_u}\right) = 1.15(0.9)(515)\left(1 - \frac{0.28\times515}{764}\right) = 432 \text{ kN-m},$$ which is less than

0.9M_p. Thus, M_r = 432 kN-m, which is greater than M_f=309 kN-m, therefore, construction condition is satisfactory.

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(b) Check service load condition

Step 1: Service loads prior to concrete hardening:

Specified weight of (Slab + steel) = 2.8 + 0.25 = 3.05 KPa

Specified point load at B1 joints P_{s1}= (3.05)(9)(3)=82 kN

The service moment prior to concrete hardening is $M_1 = 82(3) = 246$ kN-m

Step 2: Service loads after concrete hardening:

Specified finish and partition weight = 1.2 kPa

Specified live load =2.4 kPa

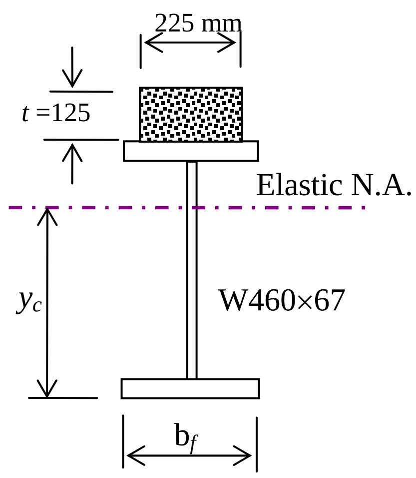
Specified point load at B1 joints P_{s2}=(3.6)(27)=97.2 kN

The service moment after concrete hardening is $M_2 = 97.2(3) = 292$ kN-m

Step 3: To find I_t and S_t of the transformed section under service loads

$$n = \frac{E_s}{E_c} = \frac{2\times10^5}{20.1\times10^3} = 10$$

The transformed section under service load is shown as below:



Assuming elastic neutral axis is in the steel section (concrete is fully effective under such a assumption), then

$$y_c = \frac{8560\times\left(\frac{454}{2}\right) + 125\times225\times\left(454+\frac{125}{2}\right)}{8560+125(225)} = 448 \text{ mm}$$

Since $y_c < d=454$ *mm*, the elastic neutral axis in steel section is confirmed.

The moment of inertia of the transformed section is

$$I_t = 295\times10^6 + 8560\times(448-454/2)^2 + \frac{1}{12}\times225\times125^3 + 225\times125\left(454+\frac{125}{2}-448\right)^2$$

$$= 882\times10^6 \text{ mm}^4$$

The elastic modulus of the composite section referred to steel bottom flange is:

$$S_t = \frac{I_t}{y_c} = \frac{882\times10^6}{448} = 1969\times10^3 \text{ mm}^3$$

$$\frac{M_1}{S_s} + \frac{M_2}{S_t} = \frac{246\times10^6}{1300\times10^3} + \frac{292\times10^6}{1969\times10^3} = 338 \text{ MPa} < F_y = 350 \text{ MPa}$$

Therefore, service condition is satisfactory.

Comments: the maximum normal stress of beam under bending moment M is $\sigma_{\max} = \frac{M}{I} y_{\max}$, where $y_{\max}$ is the distance from extreme fiber to the neutral axis. The equation is rewritten as $\sigma_{\max} = \frac{M}{I/y_{\max}}$, where $S = I/y_{\max}$ is the definition of elastic modulus of the section.

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2. A one-story building structure is shown in figure (a). A typical interior frame is shown in figure (b), assuming that the roof purlins deliver uniformly distributed loads to the frame beam BC. The frame is pin supported at column bases A and D. The members are oriented so that their webs are parallel to the plane of the frame. Columns AB and CD are laterally braced at both ends only in their two principal axis planes. The beam BC are laterally braced such that lateral-torsional buckling failure mode will not govern. The steel is of G40.21 300W. The frame is subjected to combination of the specified wind load W, dead load Q_D and snow load Q_S, as shown in the figure (b). Building importance category is normal.

From first-order analysis, the roof undergoes 0.51 mm drift when subjected to W=1 kN wind load only. The corresponding support reactions are shown in figure (c). The support reactions under 1 kN/m gravity load on the frame beam from first-order analysis are shown in figure (d).

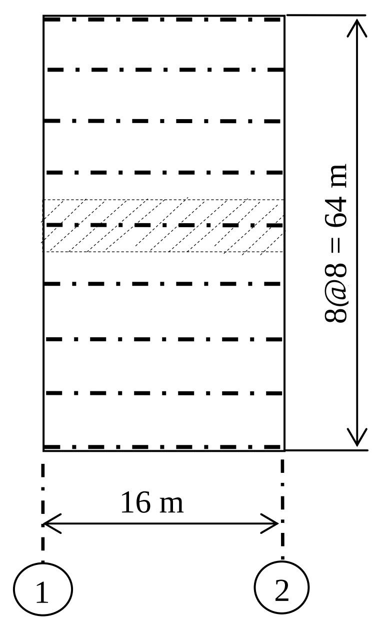

Fig. (a) Roof framing plan

Consider load case (1.25Dead + 1.5Snow + 0.4Wind). For the design of the frame, determine

(1) Notional lateral load according to Clause 8.7.2 of S16-01.

(2) Second-order moments at sections B, C and the mid-span of beam BC when considering P-Δ effect.

(3) Is the beam BC safe in terms of moment resistance? The axial force in the beam is neglected.

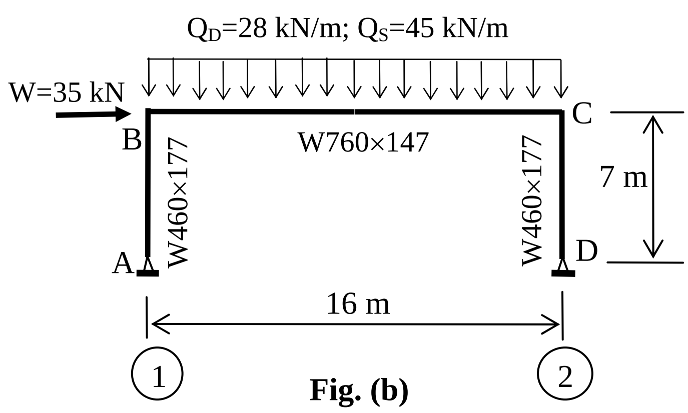

Fig. (b)

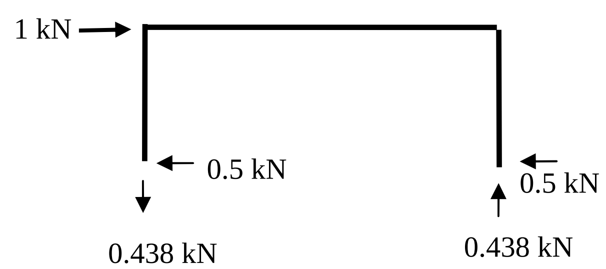

Fig. (c)

Sectional Properties of W760×147:

$A=18700\ mm^2$, $I_x=1660\times10^6\ mm^4$, $I_y=52.9\times10^6\ mm^4$,
$S_x=4410\times10^3\ mm^3$, $Z_x=5100\times10^3\ mm^3$,
$S_y=399\times10^3\ mm^3$, $Z_y=631\times10^3\ mm^3$,
$r_x=298$ mm, $r_y=53.1$ mm
$J=1560\times10^3\ mm^4$, $C_w=7160\times10^9\ mm^6$
d = 753 mm, b = 265 mm, t = 17 mm,
w = 13.2 mm, d–2t = 719 mm

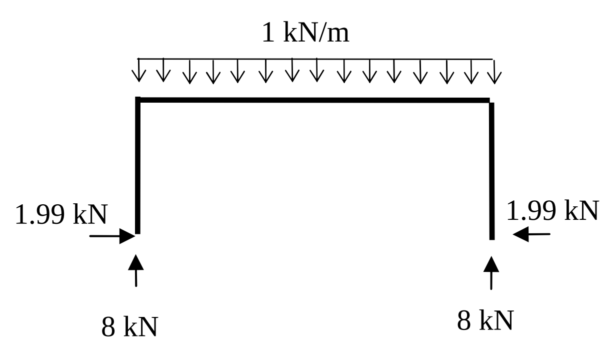

Fig. (d)

Solution:

(1) Load Case: 1.25D + 1.5L + 0.4W

Factored gravity load w_F =1.25(28) + 1.5(45) = 102.5 kN/m
Factored wind load = 0.4(35) = 14 kN
Notional lateral load = 0.005(102.5)(16) = 8.2 kN
Total lateral load = 14 + 8.2 =22.2 kN

(2) Second-order structural analysis using amplification factor method

Step 1: Decomposition of sidesway and nonsidesway components

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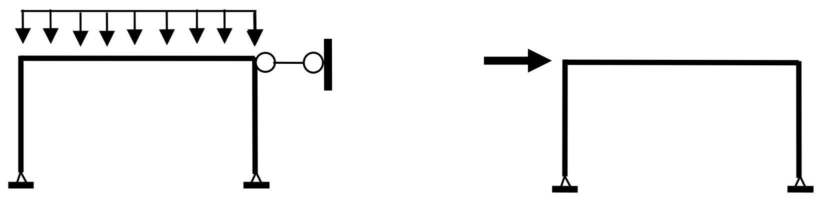
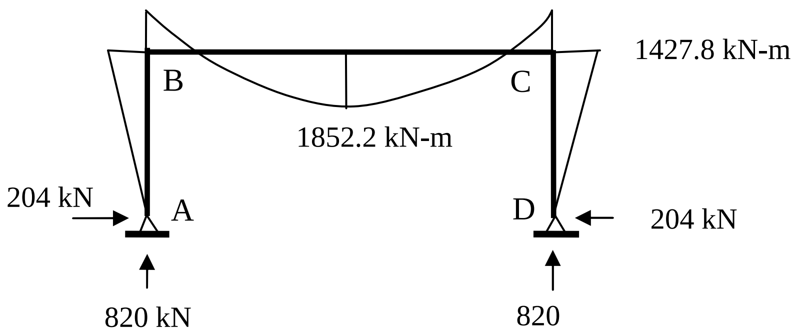
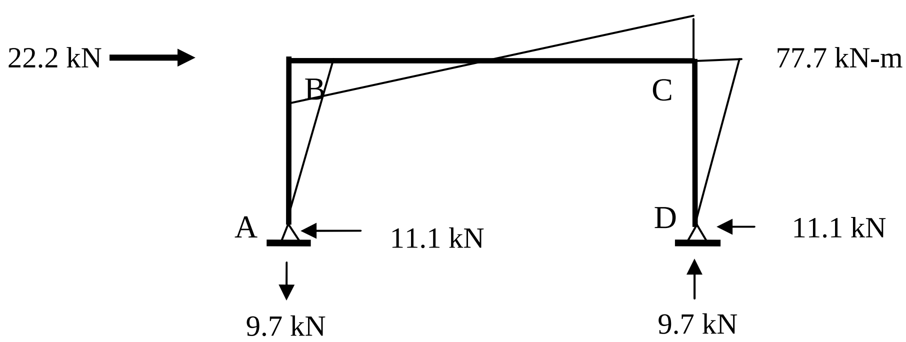


Step 2: Nonsidesway moment M_{fg} under factored gravity load w_F =102.5 kN/m



The reaction of artificial restraint is equal to zero since structural and loading are symmetric.

Step 3: Sidesway moment M_{ft} and first-order lateral deflection Δ_f under lateral load 52.5 kN



The bending moment diagram is drawn on the tension sides of members. The first-order lateral displacement of the frame is equal to 0.51(22.2) = 11.3 mm.

Step 4: The total factored moment of the frame are $M_f = M_{fg} + U_2 \bullet M_{ft}$

$$U_2 = \frac{1}{1 - \dfrac{\sum C_f \bullet \Delta_f}{\sum V_f \bullet h}} = \frac{1}{1 - \dfrac{(102.5 \times 16) \times 11.3}{22.2 \times 7000}} = 1.135$$

At section B: $M_f = M_{fg} + U_2 \bullet M_{ft}$ = 1428 – 1.135×77.7 = 1340 kN-m
At section C: $M_f = M_{fg} + U_2 \bullet M_{ft}$ = 1428 + 1.135×77.7 = 1516 kN-m
At mid-span of beam, M_f=1852 kN-m
The axial force of column AB = 820 – 1.135×9.7 = 810 kN
The axial force of column CD = 820 + 1.135×9.7 = 831 kN

Summary of second-order analysis: Column CD is the most critical member with larger moment and axial force. Although this column is critical for this load case, if the lateral load, including the notional load, were reversed, column AB would become the critical member. Therefore, both columns should be designed based on the same set of axial load and bending moment.

(3) Is the beam BC safe in terms of moment resistance? The axial force in the beam is neglected

Check local buckling

Flange: $\frac{b/2}{t} = \frac{265/2}{17} = 7.8 < \frac{145}{\sqrt{F_y}} = \frac{145}{\sqrt{300}} = 8.4$ class 1

Web: $\frac{h}{w} = \frac{d - 2t}{w} = \frac{719}{13.2} = 54.4 < \frac{1100}{\sqrt{F_y}} = \frac{1100}{\sqrt{300}} = 63.5$, class 1.

Overall, it is a class 1 section (ok).

$M_r = \phi Z_x F_y = 0.9(5100 \times 10^3)(300) = 1530 \times 10^6$ N-mm=1530 kN-m < M_f=1852 kN-m, NOT ok

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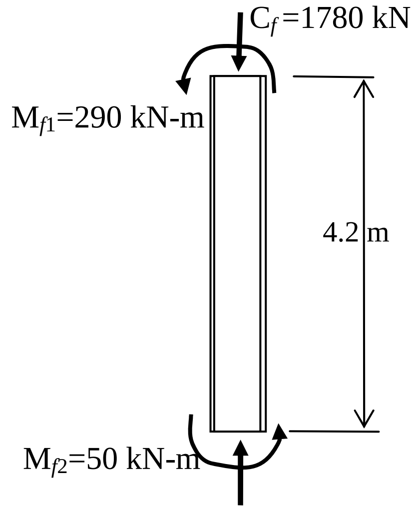
3. A W360×101 member of G40.21 350W steel is subjected to an axial force C_f and end moments M_{f1} and M_{f2} about strong axis under factored loads, as shown in the figure. The member is part of an unbraced moment frame which is similar to the structure (Column CD) in Problem 2. Only ends of the member are braced in both major and minor axis directions. Check the adequacy of the member.



Sectional Properties of W360×101:
F_y=350 MPa, A=12900 mm^2, $I_x=301\times10^6$ mm^4,
$S_x=1690\times10^3$ mm^3, $Z_x=1880\times10^3$ mm^3, r_x=153 mm,
$I_y=50.6\times10^6$ mm^4, $S_y=397\times10^3$ mm^3, $Z_y=605\times10^3$ mm^3,
r_y=62.7 mm, $J=1250\times10^3$ mm^4, $C_w=1450\times10^9$ mm^6,
d=357 mm, b=255 mm, t =18.3 mm,
w =10.5 mm, d–2t =320 mm

Solution:

(1) Check local buckling

flange: $\frac{b/2}{t}=\frac{255/2}{18.3}=7<\frac{145}{\sqrt{F_y}}=\frac{145}{\sqrt{350}}=7.8$ class 1

Web:

$$\frac{h}{w}=\frac{d-2t}{w}=\frac{320}{10.5}=30.5<\frac{1100}{\sqrt{F_y}}\left(1-0.39\frac{C_f}{\phi C_y}\right)=\frac{1100}{\sqrt{350}}\left(1-0.39\frac{1780\times1000}{0.9\times12900\times350}\right)=48.7$$

∴ class 1 web

Overall, it is a class 1 section. Cl.13.8.2 applies.

(2) Check Cross-Sectional strength

Cross-sectional strength never governs for prismatic beam-columns in unbraced frames and need not be checked.

For the purpose of illustration, it is still checked hereafter.

$C_r = \phi A F_y = 0.9(12900)(350) = 4063500$ N = 4064 kN
$M_{rx} = \phi Z_x F_y = 0.9(1880\times10^3)(350) = 592\times10^6$ N-mm = 592 kN-m

κ=50/290=0.17 positive for double curvature, $\omega_{1x} = 0.6 - 0.4\kappa = 0.6 - 0.4(0.17) = 0.532$

$$C_{ex}=\frac{\pi^2 EI_x}{L^2}=\frac{\pi^2(200000)(301\times10^6)}{4200^2}=33680{,}000\ \text{N}=33680\ \text{kN}$$

$$U_{1x}=\frac{\omega_{1x}}{1-\frac{C_f}{C_e}}=\frac{0.532}{1-\frac{1780}{33680}}=0.561,\quad \text{Use } U_{1x}=1.0$$

At the top end, where the moment is maximum,

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}=\frac{1780}{4064}+\frac{0.85(1.0)(290)}{592}=0.85<1.0,\ \text{and}\ \frac{M_{fx}}{M_{rx}}=\frac{290}{592}<1$$

∴ The cross-sectional strength is satisfactory.

(3) Check overall member strength

$(KL)_x / r_x = 4200/153 = 27.5$

$$\lambda=\frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}}=27.5\sqrt{\frac{350}{\pi^2(200000)}}=0.366$$

$C_{rx} = \phi A F_y(1+\lambda^{2n})^{-1/n} = 0.9(12900)(350)(1+0.366^{2.68})^{-0.746} = 3869890$ N = 3870 kN

M_{rx} = 592 kN-m

U_{1x} =1.0 because the structure is an unbraced frame.

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}=\frac{1780}{3870}+\frac{0.85(1.0)(290)}{592}=0.88<1.0$$

∴ the overall member strength is satisfactory

(4) Check lateral-torsional buckling strength

$(KL)_y / r_y = 4200/62.7 = 67$, weak axis governs.

$$\lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}} = 67\sqrt{\frac{350}{\pi^2(200000)}} = 0.892$$

$$C_r = C_{ry} = \phi A F_y (1+\lambda^{2n})^{-1/n} = 0.9(12900)(350)(1+0.892^{2.68})^{-0.746} = 2692554 \text{ N} = 2693 \text{ kN}$$

The value of M_{rx} should consider lateral buckling failure mode. The unbraced length of the member is 4200 mm.

$M_p = Z_x F_y = (1880\times10^3)(350) = 658\times10^6$ N-mm = 658 kN-m

$\omega_2 = 1.75+1.05k+0.3k^2 = 1.94$ where k=0.17.

$$M_u = \frac{\omega_2 \pi}{L}\sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

$$= \frac{1.94\pi}{4200}\sqrt{(2\times10^5)(50.6\times10^6)(77000)(1250\times10^3) + \left(\frac{\pi\times2\times10^5}{4200}\right)^2 (50.6\times10^6)(1450\times10^9)}$$

$= 2345\times10^6$ N-mm = 2345 kN-m

Since M_u = 2345 kN-m > 0.67 M_p = 0.67(658) = 441 kN-m,

$$M_{rx} = 1.15\phi M_p\left(1-\frac{0.28M_p}{M_u}\right) = 1.15(0.9)(658)\left(1-\frac{0.28\times658}{2345}\right) = 627 \text{ kN-m},$$

but M_{rx} should not greater than ϕM_p = 592 kN-m.

Therefore, M_{rx} =592 kN-m.

U_{1x} =1.0 because the structure is a sway frame.

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} = \frac{1780}{2693} + \frac{0.85(1.0)(290)}{592} = 1.08 > 1.0, \text{ and } \frac{M_{fx}}{M_{rx}} = \frac{290}{592} < 1.0$$

$\therefore$ The lateral-torsional buckling strength is not satisfactory

Summary: since the interaction value lateral-torsional member strength is greater than 1.0, the W360×101 is not adequate for the load combination in consideration.

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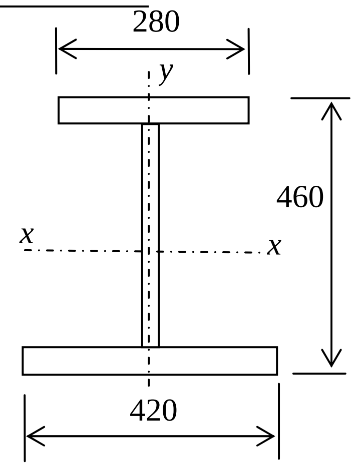
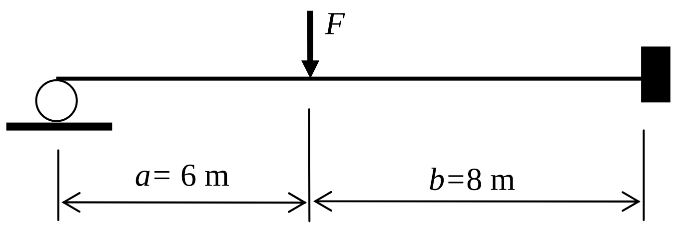


4. (1) A steel beam has one axis of symmetry *y-y* (G40.21 300W steel). The thickness of the flange is 24 mm, while the thickness of web is 14 mm. Compute the plastic modulus of the section about a horizontal axis.

(2) A statically indeterminate beam, as shown in the second figure, is subjected to a concentrated load F. The beam is W920×201 (class 1) and F_y=350 MPa and M_p=2878 kN-m. The beam is fully laterally supported during the loading process. Assume an elastic-perfectly-plastic behavior for the moment-curvature relationship of the cross section. Calculate the load F when the first plastic-hinge forms.



Solution:

(1) **Compute plastic modulus Z_x about *x-x* axis**

Cross section area:

$A = \Sigma A_i$ =flanges + web = 280(24) + 420(24) + (412)(14) = 22568 mm^2

At the plastic limit state, the plastic neutral axis divides the cross-section into two equal area zones.

$A_c = A_t = A/2 = 0.5(22568) = 11284$ mm^2

Define y_p = the distance of plastic neutral axis to the bottom fiber, then,

$11284 = 420\times24 + 14\times(y_p - 24)$

$\therefore y_p = 110$ mm

The distance from the centriod of the compression zone (assume it is the zone below the neutral axis) to the plastic neutral axis is

$$y_c' = \frac{\sum A_i y_i}{\sum A_i} = \frac{420\times24(110-12)+86\times14\times(86/2)}{11284} = 92 \text{ mm}$$

The distance from the centriod of the tensile zone to the plastic neutral axis is

$$y_t' = \frac{280(24)(460-110-12)+14(326)(326/2)}{11284} = 267 \text{ mm}$$

$$Z_x = A_c\left(y_c + y_t\right) = 11284\times(267+92) = 4051\times10^3 \text{ mm}^3$$

(2) Elastic stage:

moment at fixed end is $M_2 = \frac{Fab}{2L^2}(a+L) = \frac{F(6)(8)}{2(14)^2}(6+14) = 2.45F$,

moment at point load is $M_1 = \frac{Fb^2}{2L^3}(a+2L)a = \frac{F8^2}{2\times14^3}(6+2\times14)6 = 2.38F$

therefore, the first plastic-hinge forms at the fix end,

$$M_p = 2.45F\ , \quad F = \frac{M_p}{2.45} = \frac{2878}{2.45} = 1175\,\text{kN}$$

the moment at the point load is $M_{1,e} = 2.38(1175) = 2796$ kN-m

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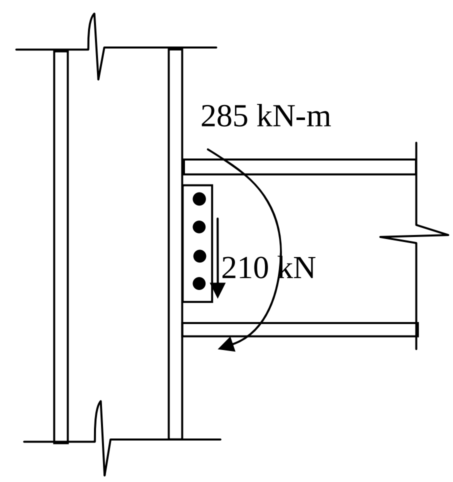
5. An edge beam-to-column rigid connection is shown in the figure, where the factored beam moment and shear force are also shown. Shear forces from the column, above and below the connection, are ignored for simplicity. For web connection, use single plate shop-welded to column flange, field-bolted to beam web with ¾ in A325 bolts (1 in=25.4 mm). For flange connection, the top and bottom flanges of beam are welded directly to the column flanges using complete-joint-penetration groove welds. The column use W310×97, and the beams use W530×85 (**Class 1 in bending**). Use 300W steel (F_y =300 MPa, and F_u =450 MPa) for all members and E49XX electrodes (X_u=490 MPa). Determine:

(a) How many bolts are needed for the web connection? Assume bolt threads are intercepted by the shear plane. The thickness of the connection plate is 7.9 mm.
(b) What is the shear resistance of the column web?
(c) Determine the compressive force which is applied to the column by the compression flange of the beam.
(d) What is the bearing resistance of the column web opposite the compression flange of the beam?



This connection design does not consider seismic loading.

W310×97 column: t_c = 15.4 mm, w_c = 9.9 mm, b = 305 mm, d=308 mm, k_1 =25 mm, T=234 mm

W530×85 beam: t =16.5 mm, w = 10.3 mm, b = 166 mm, d = 535 mm, T=469, k=33, k_1=20 mm, d-2t = 502 mm

Solution:

(a) web connection

thickness of plate = 7.9 mm of F_y=300 MPa and F_u=450 MPa.

Factored shear resistance of one bolt:

$$V_r = 0.6\phi_b n m A_b F_u = (0.6)(0.8)(1)(1\ shear\ plane)(0.25\times 3.1415\times 19.05^2)(825) = 113\times 10^3 = 113\ \text{kN}$$

where F_u = 825 MPa for A325 bolts, see Clause 13.12.1.1.

If thread intercepted, V_r = 0.7(113) = 79 kN

The bolts bear against the beam web in one direction (t=10.3 mm) and the plate in the other direction (t=7.9 mm). Obviously, the latter governs the bearing resistance.

$$B_r = 3\phi_{br} t d n F_u = (3)(0.67)(7.9)(19.05)(1\ bolt)(450) = 136\times 10^3 = 136\ \text{kN}$$

Therefore, one bolt has **V_r = 79 kN**

the number of bolts required is

$$n = \frac{V_f}{V_r} = \frac{210}{79} = 2.7\ \text{bolts.}\quad \text{Use 3 3/4 in A325 bolts.}$$

(b) The panel zone is subjected to shear force generated by the beam moment. i.e., the web of column is subjected to a shear force V_f = (285)/0.535 = 533 kN.

The shear resistance is $V_r = 0.55\phi w d F_y = 0.55(0.9)(9.9\times 308)(300) = 452800$ N = 453 kN.

Therefore, the web stiffener or reinforcement for shear is required [Clause 21.3].

(c) C= (285)/0.535 = 533 kN

(d) Cl.21.3(a), bearing resistance of column web is

$$B_r = \phi_{bi} w_c (t_b + 10 t_c) F_{yc} = 0.8(9.9)(16.5 + 10\times 15.4)300 = 405{,}000\ \text{N} = 405\ \text{kN}.$$

Since compression flange force 533 kN > 405 kN, stiffeners are required opposite the compression flange of the beam for a capacity of F_{st} = 533 – 405 = 128 kN. These stiffeners will prevent buckling of the column web.