University of California, San Diego
Faculty of Engineering

DESIGN OF BEAM COLUMNS

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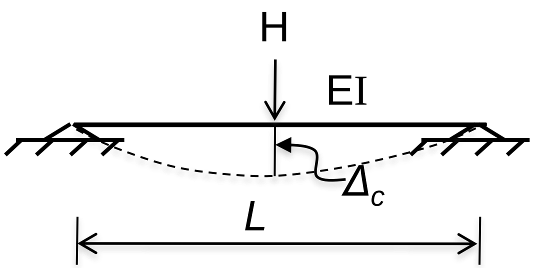
CONTENTS

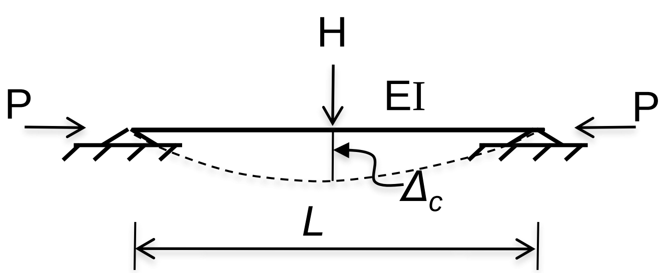
INTRODUCTION

Consider a simply supported beam with a midspan point load:



$$\Delta_c = \frac{HL^3}{48EI}$$

What if the beam is now subjected to axial compression?



$$\Delta_c \rangle \frac{HL^3}{48EI}$$

The resulting momnet will be amplified by the axial load:

$$M = \frac{HL}{4} + P\Delta_c, \quad where \quad \Delta c \rangle \frac{HL^3}{48EI}$$

INTRODUCTION

A member subjected to combined bending and axial load is called a beam column.

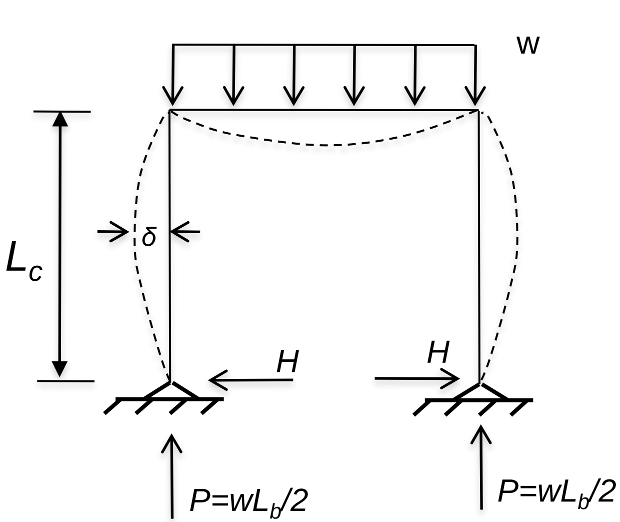
The amplification of the moment due to the axial load is called 2nd order effects.

$$M = \underbrace{\frac{HL}{4}}_{\text{First order moment}} + \underbrace{P\Delta_c}_{\text{2nd order moment}}$$

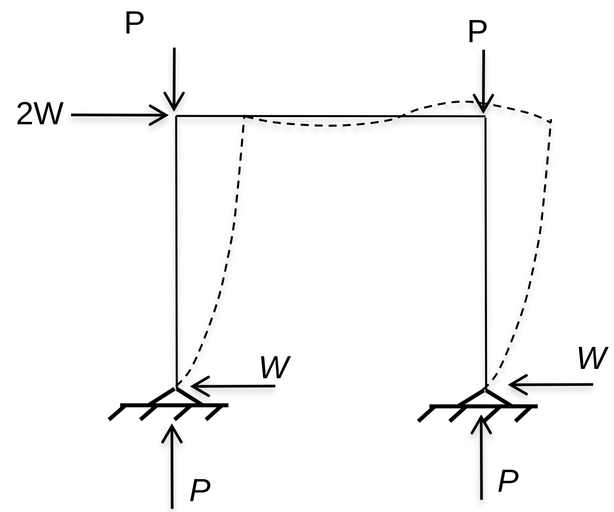
Engineers are encouraged to perform an exact 2nd order analyses to determine 2nd order effects. This is now getting much more possible with advanced software programs.

If a computer program is used to calculate 2nd order moments it is important to consider "both" types of moment amplifications.

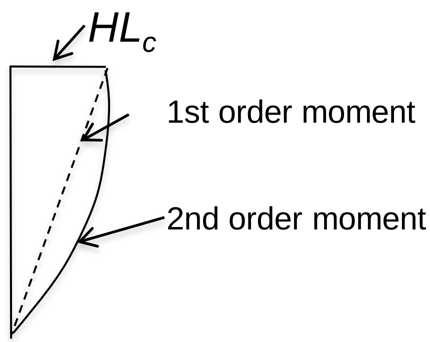
MOMENT AMPLIFICATION



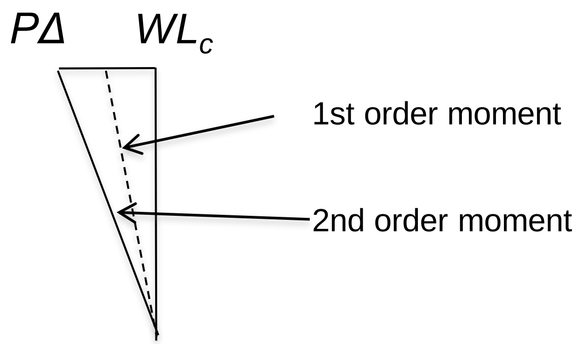
No Sway Moments: Pδ



No Sway Moments: PΔ



Maximum moment can occur ar the end or away from the end if Pδ is large.



Maximum moment occurs at column ends.

MOMENT AMPLIFICATION

A Second order analysis considers both Pδ and PΔ.

The AISC Specification provides amplification factors which can be used with the moments from a 1st order analysis. The approximate 2nd order effects are covered in Chapter C Section C2.

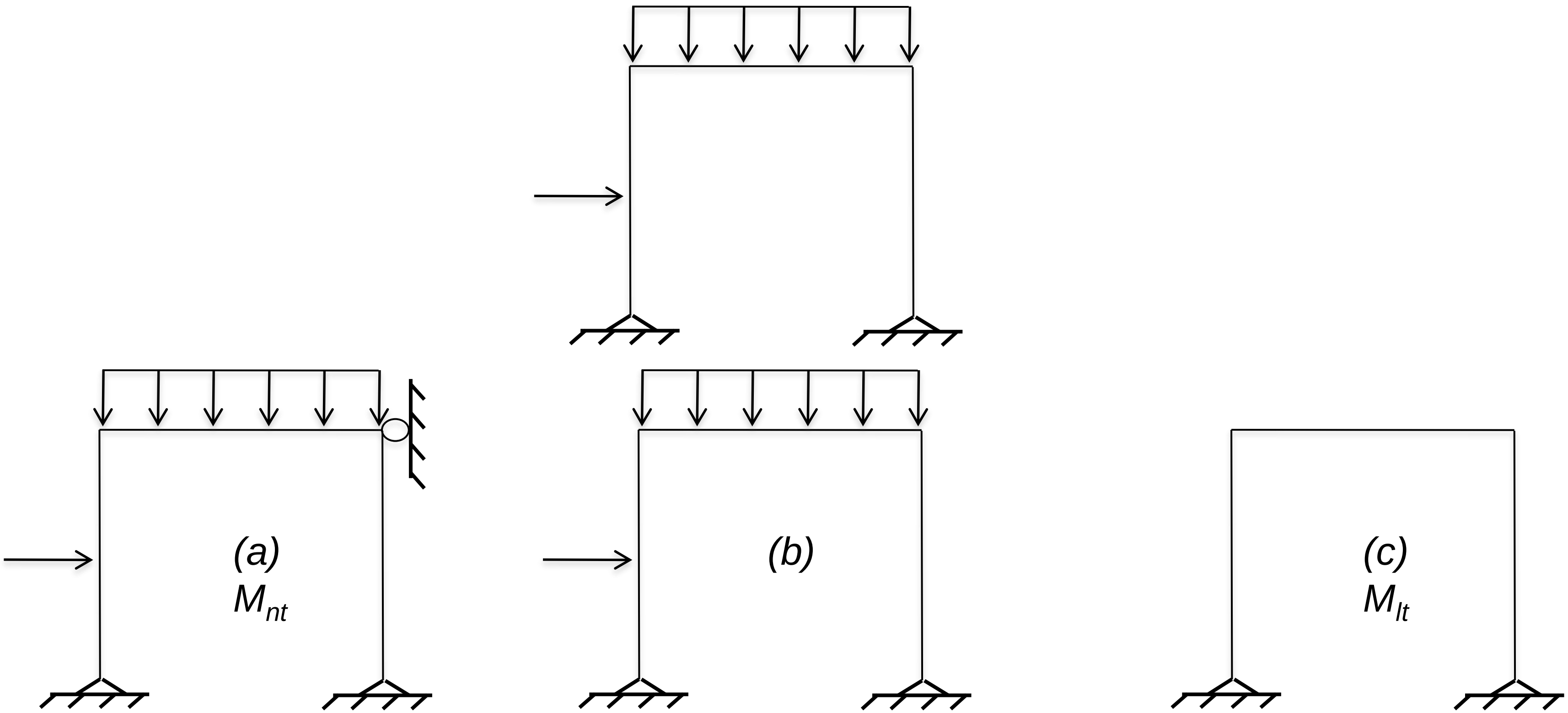
$$M_r = B_1 M_{nt} + B_2 M_{lt}$$

$$P_r = P_{nt} + B_2 P_{lt}$$

(*Pδ*) M_{nt} is the required flexural strength in the member assuming there is no lateral translation of the frame.

(PΔ) M_{lt} is the required flexural strength in the member as a result of the lateral translation of the frame only.

MOMENT AMPLIFICATION



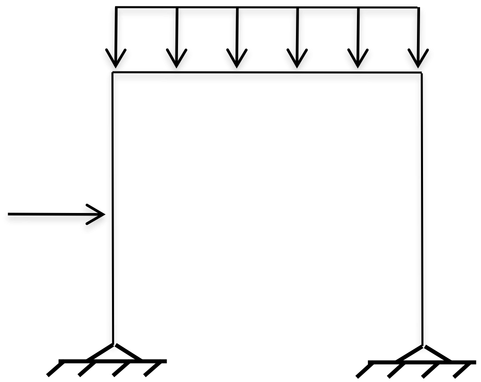
Perform a 1st order analyses on the frame preventing lateral transformation. This gives M_{nt}.

Perform a 1st order analyses on the frame allowing lateral transformation.

Subtract the moments from (a) from the moments in (b). This gives M_{lt}.

$$M_r = B_1 M_{nt} + B_2 M_{lt}$$

MOMENT AMPLIFICATION



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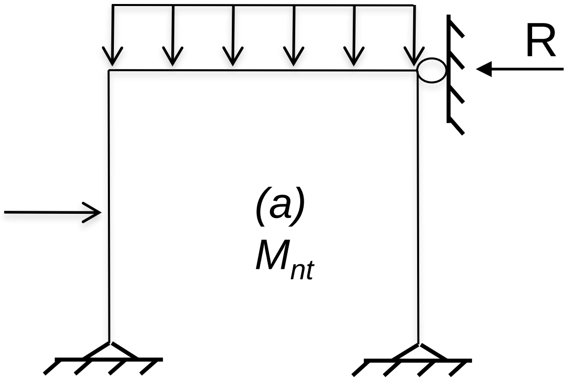

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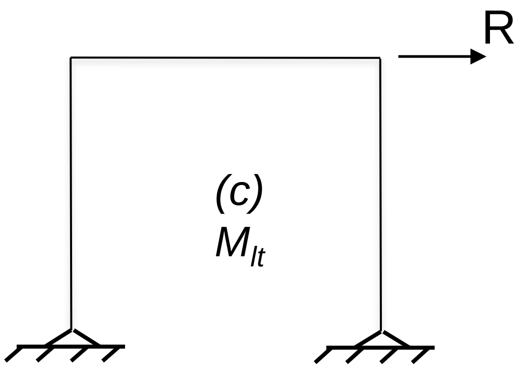
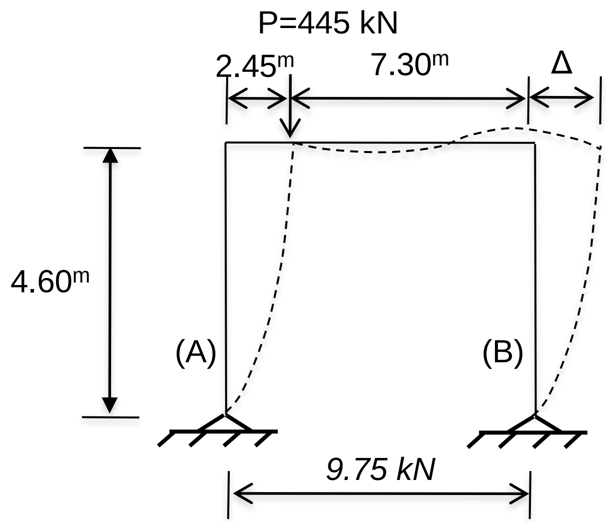


Perform a 1st order analyses on the frame preventing lateral transformation. This gives M_{nt}.

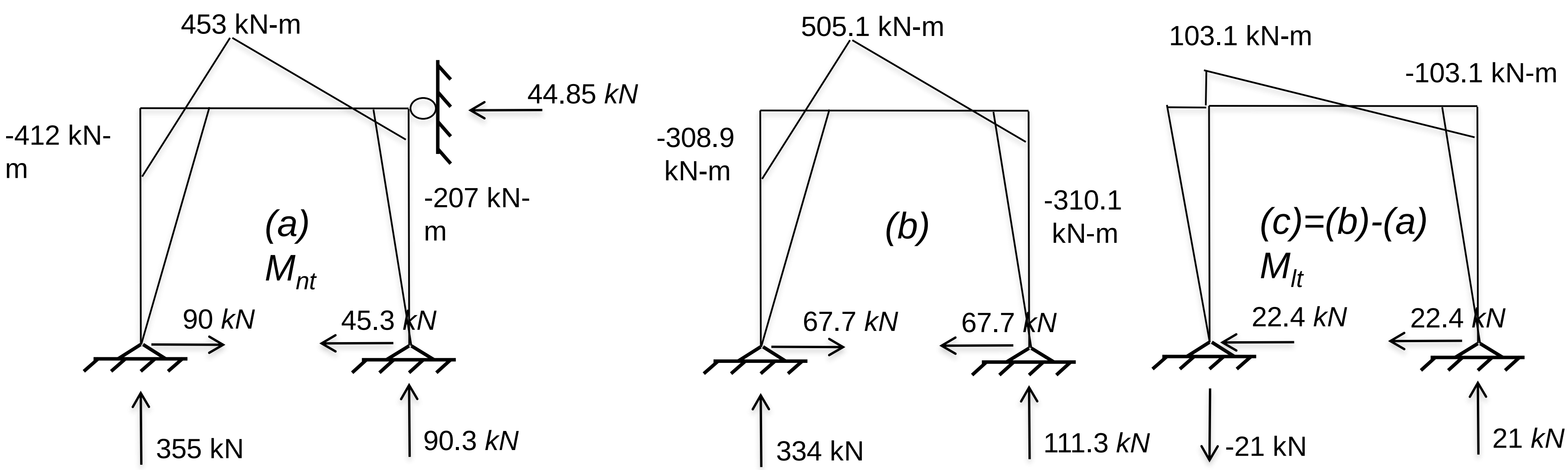
Compute M_{lt} directly.

$$M_r = B_1 M_{nt} + B_2 M_{lt}$$

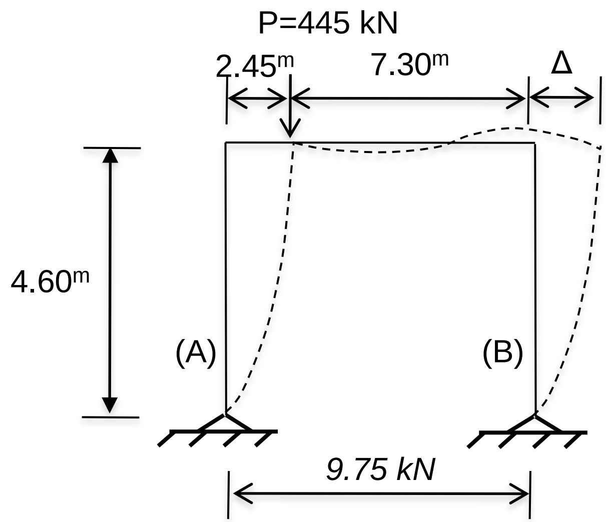
EXAMPLE



Columns and beams are: W460×82
$I_x = 370 \times 10^6$ mm4
$r_x = 189$ mm
$A = 10400$ mm^2
Δ= 20mm from 1sr order analyses



EXAMPLE



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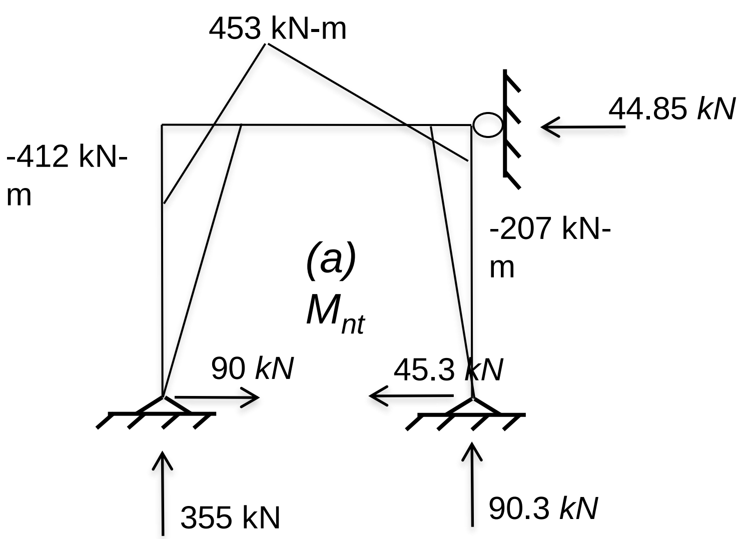
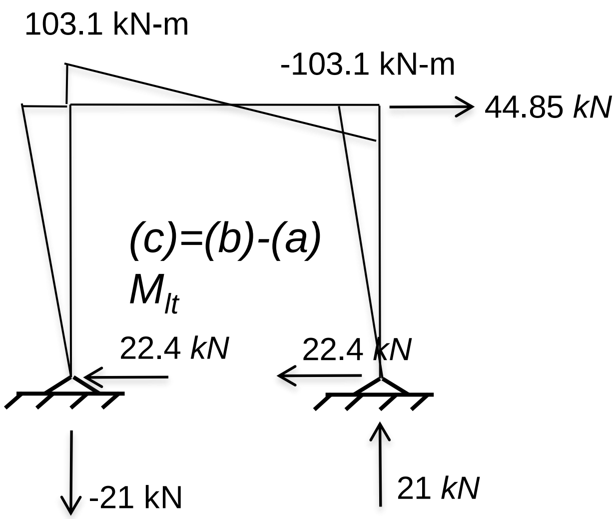




EXAMPLE

Calculate B_2: PΔ Moment. $$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{estory}}} \geq 1 (A-8-6)$$

Calculate P_{e2} for Column A and use it for both columns:
m=1; P/A=(445^{kN}x7.30^{m}/9.75^{m})/10400^{mm}=32 MPa; τ=1.0

$$G_T = \frac{1.0 \sum \left(\frac{I_x}{L_c} \right)_{W460x82}}{1.0 \sum \left(\frac{I_x}{L_b} \right)_{W460x82}} = \frac{\left(\frac{370x10^6}{4.6^m} \right)}{\left(\frac{370x10^6}{9.75^m} \right)} = 2.12$$

k_x=2.7

$$G_B = pinned = \infty$$

$$P_{e2} = \frac{\pi^2 EI}{(kL)^2} = \frac{3.14^2 x200000^{N/mm^2} x370x10^6{}_{mm4}}{\left(2.7x4600^{mm}\right)^2} = 4842kN$$

EXAMPLE

Calculate B_2: PΔ Moment.

$$B_2 = \frac{1}{1-\dfrac{\alpha P_{story}}{P_{e\,story}}} = \frac{1}{1-\dfrac{1.0x445^{kN}}{4842^{kN}x2}} = 1.05$$

EXAMPLE

Calculate B_1: Pδ Moment.

Calculate P_{e1} for Column A and Column B with worst k factor, k=1.0:

$$P_{e1} = \frac{\pi^2 EI}{(kL)^2} = \frac{3.14^2 x 200000^{N/mm^2} x 370 x 10^6{}_{mm4}}{\left(1.0 x 4600^{mm}\right)^2} = 34516\, kN$$

Calculate C_m using Eq. C2-4, AISC-05.

$$C_m = 0.6 - 0.4(M_1 / M_2)$$

M_1/M_2 ratio is the ratio of the moments at each end of the column. M_1 is the smallest moment, M_2 is the larger moment. If the column is under double curvature M_1/M_2 is (+), if the column is under single curvature the ratio is (-).

EXAMPLE

Calculate C_m using Eq. C2-4, AISC-05.

$$C_m = 0.6 - 0.4(M_1 / M_2)$$

$$M_1 = 0 \Rightarrow C_m = 0.6$$

$$B_1 = \frac{0.6}{1 - 1.0 P_r / P_{e1}} \geq 1.0$$

EXAMPLE

Calculate B_1 for Column A:

$$B_1 = \frac{0.6}{1 - 1.0x\dfrac{445kN\dfrac{7.3^m}{9.75^m}}{34516^{kN}}} = 0.606 \leq 1.0;\ therefore B_1 = 1.0$$

Calculate B_1 for Column B:

$$B_1 = \frac{0.6}{1 - 1.0x\dfrac{445kN\dfrac{2.45^m}{9.75^m}}{34516^{kN}}} = 0.602 \leq 1.0;\ therefore B_1 = 1.0$$

EXAMPLE

2nd order Moments:

$$Column A \Rightarrow M_u = 1.0x412^{kN-m} + 1.05x(-103.1)^{kN-m} = 303.7kN-m$$

$$Column B \Rightarrow M_u = 1.0x207^{kN-m} + 1.05x(103.1)^{kN-m} = 315.3kN-m$$

2nd order Axial Forces:

$$Column A \Rightarrow P_u = 355.1^{kN} + 1.05x(-21)^{kN} = 333.1kN$$

$$Column B \Rightarrow P_u = 90.3^{kN} + 1.05x(21)^{kN} = 112.3kN-m$$

AXIAL FORCE-MOMENT INTERACTION

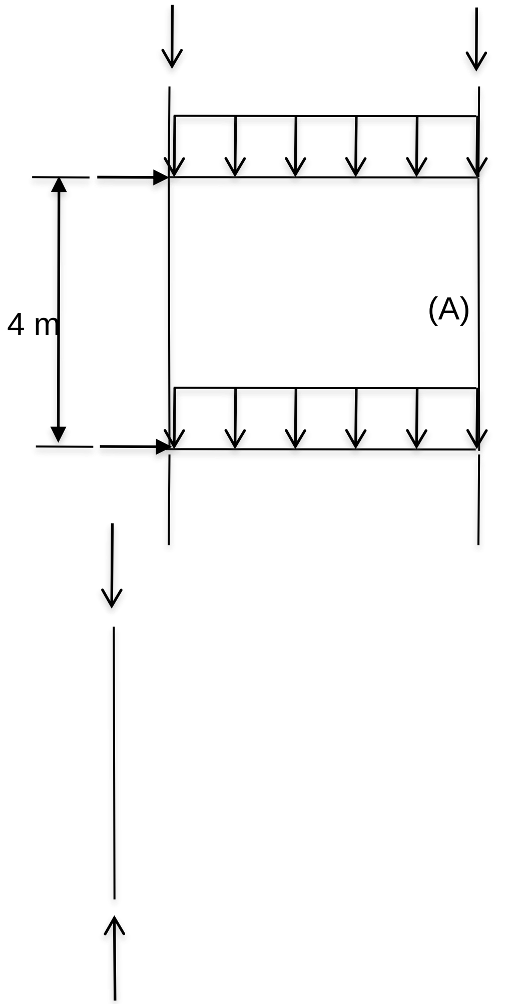
Once the axial force and 2nd order moments are calculated, use the axial force-moment interaction formula to evaluate the capacity. These equations are in Chapter H on p. 16.1-70. Use the same equations for tension and compression.

$$(a) For \frac{P_r}{P_c} \geq 0.2 \Rightarrow \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_c x} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

$$(b) For \frac{P_r}{P_c} \rangle 0.2 \Rightarrow \frac{P_r}{2P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

$M_c=\Phi_b M_n$=available flexural strength
M_r= required flexural strength using LRFD load combinations
x= subscript relating to strong axis bending
y= subscript relating to weak axis bending
P_r=required axial compressive strength using LRFD load combinations
$P_c=\Phi_c P_n$=design axial compressive strength

EXAMPLE



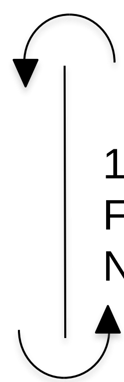
P_r = 1450 kN=$(1250^{kN})_{nt}+(200^{kN})_{lt}$

Consider a portion of a multistory frame.

Beams are: W410 × 149
I_x=619 × 10^6 mm^4

Unbraced in plane and braced each story out-of-plane.

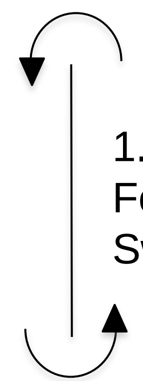
61.1 kN-m



40.7 kN-m

1.2D+0.5L
For B_1 Factor
No-Sway

298.7 kN-m



312.2 kN-m

1.3W
For B_2 Factor
Sway

EXAMPLE

Try: W360 × 110
I_x=331 × 10^6 mm^4
I_y=55.7 × 10^6 mm^4
r_x=154 mm
r_x=63.1 mm
A=14000 mm^2
Z_x=2060000 mm^3

m=1.0; P/A=1450000^N/14000^{mm2}=103.6 MPa; τ=1.0 (Table-3.1)

In-Plane: Sway

$$G_T = G_B = \frac{1.0\sum\left(\frac{I_x}{L_c}\right)_{W360x110}}{1.0\sum\left(\frac{I_x}{L_b}\right)_{W410x149}} = \frac{2x\left(\frac{331x10^6}{4.0^m}\right)}{\left(\frac{6190x10^6}{6.0^m}\right)} = 1.6 \Rightarrow k_x = 1.5$$

Out-of-Plane: No Sway

k_y=1.0

$$\frac{k_y L}{r_y} = \frac{1.0x4000^{mm}}{63.1^{mm}} = 63.4$$

Controls

$$\frac{k_x L}{r_x} = \frac{1.5x4000^{mm}}{154^{mm}} = 39.0$$

EXAMPLE

Use Column Load Tables with kL_y=4 m. p 3-21; ΦP_n=3060 kN.

$$\frac{P_r}{P_c}=\frac{P_u}{\phi P_n}=\frac{1450^{kN}}{3060^{kN}}=0.47\geq 0.2\Rightarrow Therforeuse\ Eq.H1-1a$$

$$\frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)=\frac{P_u}{\phi P_n}+\frac{8}{9}\frac{M_u}{\phi M_n}\leq 1.0$$

$$M_r=M_u=B_1 M_{nt}+B_2 M_{lt}$$

Calculate B_1:

$$B_1=\frac{C_m}{1-\alpha P_r/P_{e1}}\geq 1.0$$

Double curvature (+)

$$C_m=0.6-0.4(M_1/M_2)$$

$$C_m=0.6-0.4(40.7^{kN-m}/61.1^{kN-m})=0.33$$

EXAMPLE

$$P_{e1} = \frac{\pi^2 EI}{(kL)^2} = \frac{3.14^2 x 200000^{N/mm^2} x 331 x 10^{6}{}_{mm4}}{\left(1.0 x 4000^{mm}\right)^2} = 40836\, kN$$

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} = \frac{0.33}{1 - 1.0 \dfrac{1450^{\,kN}}{40836^{\,kN}}} = 0.342 \leq 1.0 \Rightarrow B_1 = 1.0$$

Calculate B_2:

$$B_2 = \frac{1}{1 - \dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}}$$

EXAMPLE

$$P_{e2} = \frac{\pi^2 EI}{(kL)^2} = \frac{3.14^2 x 200000^{N/mm^2} x 331 x 10^{6}{}^{mm4}}{\left(1.5 x 4000^{mm}\right)^2} = 18149\ kN$$

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} = \frac{1}{1 - 1.0\,\frac{2x1450^{kN}}{2x18149^{kN}}} = 1.074$$

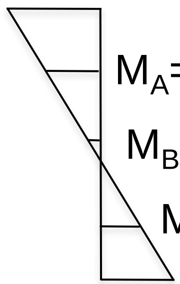
$$(M_u)_{top} = 1.0 x 61.1^{kN-m} + 1.074\, x (298.7)^{kN-m} = 381.9\ kN - m$$

$$(M_u)_{bottom} = 1.0 x 40.7^{kN-m} + 1.074\, x (312.2)^{kN-m} = 376.0\ kN - m$$

EXAMPLE

Calculate C_b factor for the column

381.9 kN-m



M_A=194.8 kN-m

M_B=2.8 kN-m

M_C=-189.81 kN-m

-376.0 kN-m

$$C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3M_A + 4M_B + 3M_C}$$

$$C_b = \frac{12.5x382^{kN-m}}{2.5x382^{kN-m} + 3x194.8^{kN-m} + 4x2.85^{kN-m} + 3x189.1^{kN-m}} = 2.26$$

EXAMPLE

W360 × 110: L_r=8.47 m >L_b=4 m; therefore check $C_b \Phi M_r$.

$$\phi M_r = 457\ kN-m;\ \phi M_p = 640\ kN-m$$

$$C_b \phi M_r = 2.26 x 457\ kN-m = 1033\ kN-m \rangle\ \phi M_p = 640\ kN-m$$

$$\phi M_n = \phi M_p = 640\ kN-m$$

Calculate $P_r=P_{nt}+B_2P_{lt}$

$$P_r = P_{nt} + B_2 P_{lt} = 1.0 x 1250^{kN} + 1.074 x 200^{kN} = 1465\ kN$$

EXAMPLE

Use the interaction equation to see if the beam-column W360 $\times$ 110 is adequate or not.

$$\frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)=\frac{P_u}{\phi P_n}+\frac{8}{9}\frac{M_u}{\phi M_n}\leq 1.0$$

$$\frac{1465^{kN}}{3060^{kN}}+\frac{8}{9}\frac{382^{kN-m}}{640^{kN-m}}=1.00\leq 1.0\ OK$$

Use W360 $\times$ 110. If W360 $\times$ 110 was inadequate use a bigger beam size

DESIGN

We have discussed the design of beam columns using the interaction equations given in Chapter H.

$$(a)\ For\ \frac{P_r}{P_c} \geq 0.2 \Rightarrow \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_c x} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

$$(b)\ For\ \frac{P_r}{P_c} \rangle 0.2 \Rightarrow \frac{P_r}{2P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_c x} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

The P_r/P_c or $P_u/\Phi P_n$ ratio that is used is the larger ratio for strong axis or weak axis buckling.

The examples that we have considered so far have been for symmetrical loading. If we have unsymmetrical loading we can use the ΣP concept $(\Sigma P_u/\Phi)\Sigma P_n$.

It is important that we check a column for buckling in both directions and use the larger ratio of P_r/P_c.

DESIGN

Trial Size

We can get a trial size to use by calculating an "equivalent" load for the member:

If the member behaves more like a column:

$$P_{uEQ} = P_u + M_x(2/d) + M_y(7.5/b_f)$$

Use column load tables.

If the member behaves more like a beam:

$$M_{uEQ} = P_u d/2 + M_x + M_y$$

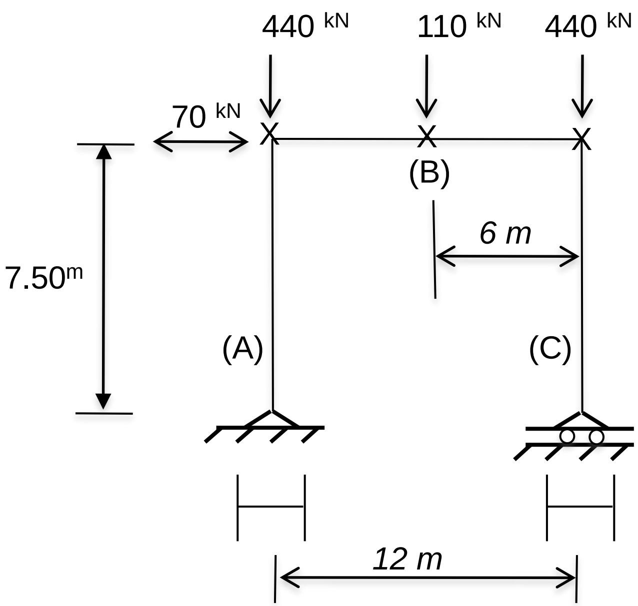
Use Z tables.

This is a judgement call in which you look at the relative magnitudes of the column contribution from axial force and moment.
ie. If $M_x(2/d)$ is >>P_u, it would be obviously be more efficient to select a deeper "beam" type section than going with a stocky "column" type section.

EXAMPLE

Design Column A and C and Beam B

F_y=345MPa

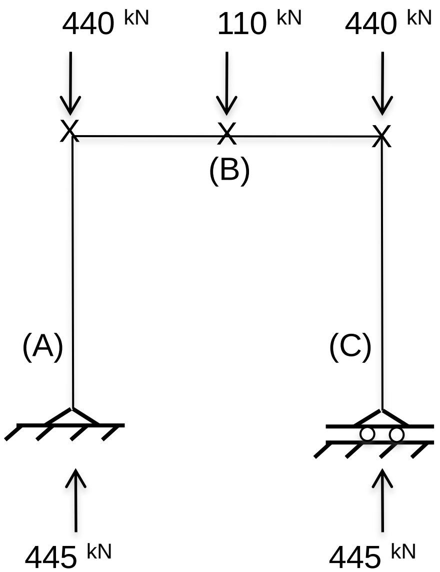


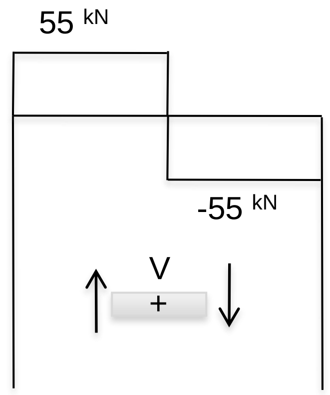
Columns are braced at top and bottom out of plane. Factored loads are shown. Use same size members for columns A and C.

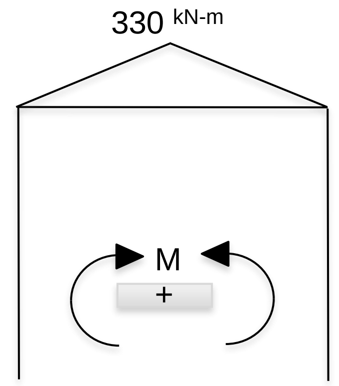
x: brace locations

EXAMPLE

Shear and Bending Moment diagrams under gravity loads:

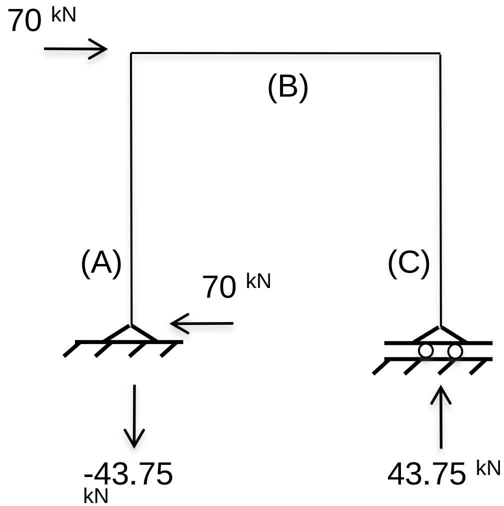


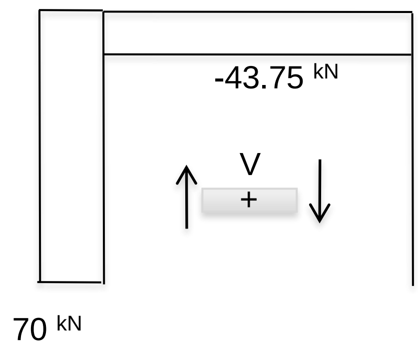


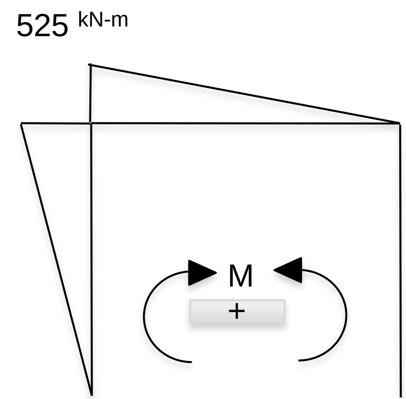


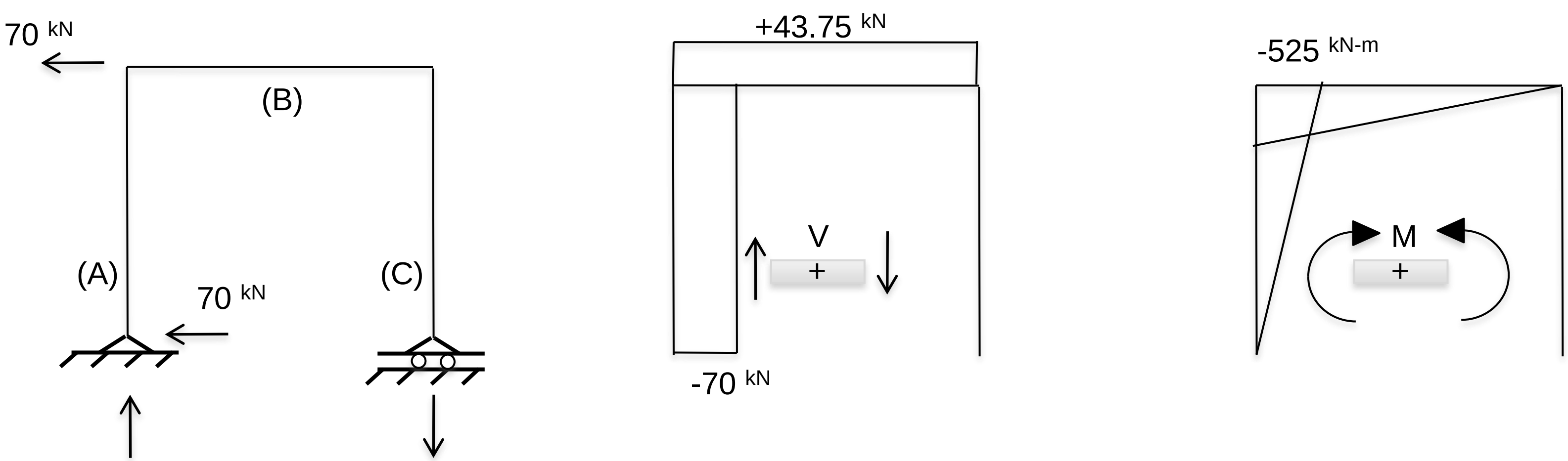
EXAMPLE

Shear and Bending Moment diagrams under wind loads:



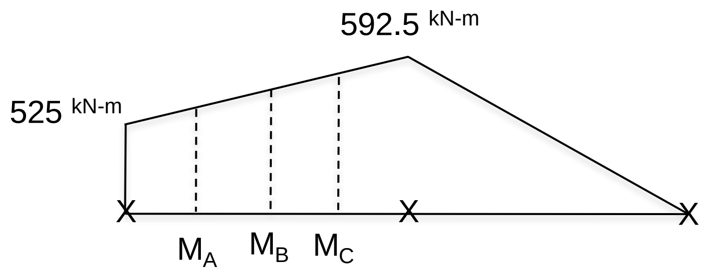






EXAMPLE

Design the beam.



C_b=?

M_A=541.9 kN-m
M_B=558.8 kN-m
M_C=575.6 kN-m

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C}$$

$$C_b = \frac{12.5x592.5^{kN-m}}{2.5x592.5^{kN-m} + 3x541.9^{kN-m} + 4x558.8^{kN-m} + 3x575.6^{kN-m}} = 1.05$$

EXAMPLE

Design the beam.

$$M_u = 592.5\ kN-m \qquad\qquad L_b = 6\ m$$

<u>Try a W610×82</u> (Z-Tables p.4-19)

$$\phi_b M_p = 683\ kN-m \qquad L_r = 3.91\ m \qquad L_p = 1.45\ m$$

LTB Check: L_b>>L_r, this section clearly wont work.

<u>Try a W530×92</u> (Z-Tables p.4-19)

$$\phi_b M_p = 733\ kN-m \qquad L_r = 5.07\ m \qquad L_p = 1.90\ m$$

$$\phi_b M_r = 514\ kN-m$$

LTB Check: p. 4-154 or 4-156; ΦM_o=300 kN-m; Wont work

EXAMPLE

Design the beam.

$$M_u = 592.5\ kN - m \qquad L_b = 6\ m$$

Try a W610 × 101 (Z-Tables p.4-19)

$$\phi_b M_p = 900\ kN - m \qquad L_r = 5.33\ m \qquad L_p = 2.02\ m$$

Try a W610 × 113 (Z-Tables p.4-19)

$$\phi_b M_p = 1020\ kN - m \qquad L_r = 5.51\ m \qquad L_p = 2.07\ m$$

$$\phi_b M_r = 715\ kN - m$$

LTB Check: p. 4-154; ΦM_o=602; $C_b \Phi M_o$=632 kN-m.

USE W610 × 113

EXAMPLE

Design the beam.

Shear should not be a problem but let us check it anyway.

$$V_u = 98.75\ kN - m$$

$$\frac{h}{t_w} = 48.9 \leq 2.45\sqrt{\frac{E}{F_{yw}}}$$

$$L_p = 2.02\ m$$

$$\phi V_n = 0.9x0.6xA_w F_{yw} = 0.9x0.6x608^{mm} x11.2^{mm} x345^{MPa} = 1268\ kN$$

$$\phi V_n = 1268\ kN \geq Vu = 98.75\ kN$$

Displacements should also be checked. However, since live loads are not given we will assume deflections are OK.

EXAMPLE

Design the columns.

Column A is more critical than column C. Same Pu but column A has also bending.

Does the member behave more like a colum or a beam?

$$P_{uEQ} = P_u + M_x(2/d) + M_y(7.5/b_f)$$

Use average.

$$P_{uEQ} = \frac{990^{kN}}{2} + 525^{kN-m}\frac{2}{0.31^{m}} = 495^{kN} + 3387^{kN}$$

Assume a W310 is used.

Bending dominates. Choose a section that is more efficient in bending. Obtain an equivalent moment in the beam column.

EXAMPLE

Assume a W460 is used.

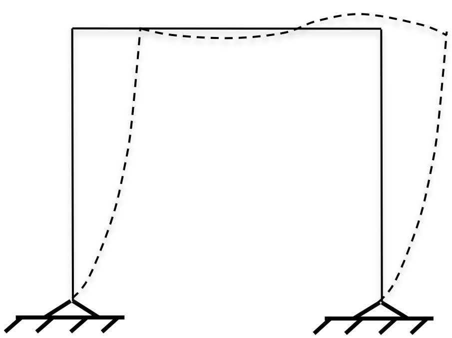
Design the columns.

$$M_{uEQ} = \frac{P_u d}{2} + M_u = \frac{(995^{kN}/2)x0.46^{m}}{2} + 525^{kN-m}$$

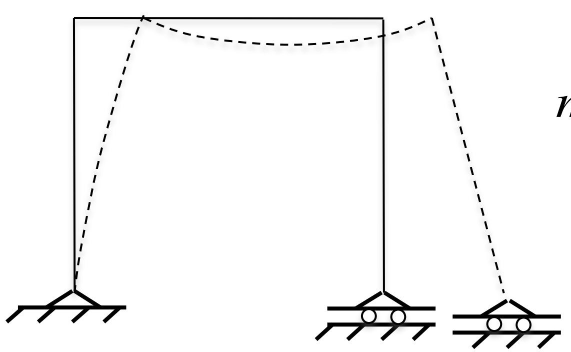
$$M_{uEQ} = 639.5\ kN - m$$

Look at possible "beam" type sections with $\Phi M_{preq'd}$=640 MPa.

However, before spending a lot of time checking a particular section for this beam-column, consider the stifness of beam (B) that provides the restraint.



Assumed stiffness: 6EI/L.



Actual stiffness: 3EI/L.

$$m = \frac{2EI/L}{6EI/L} = 0.33$$

EXAMPLE

Design the columns.

The M_{EQ} that is calculated does not reflect the low beam stiffness that will affect ΦP_n. However, we can use this knowledge to slightly increase M_{EQ} as we select a trial member size.

$$\phi M_p = 639.5\ kN - m \Rightarrow Check\ \ Z - Tables$$

$$W610x82 - \phi M_p = 683\ kN - m$$

$$W530x92 - \phi M_p = 733\ kN - m$$

Try W530×92.

$$\phi M_p = 683\ kN - m \qquad I_y = 23.8x10^6\ mm^4 \qquad I_x = 552x10^6\ mm^4$$

$$A = 11800\ mm^2 \qquad r_x = 216\ mm \qquad r_y = 44.9\ mm$$

EXAMPLE

Design the columns.
Calculate the available column strength.

No Sway Mode: kLy=1.0; kLx can be assumed to be 1.0 due to low stiffness of the beam.

$$kL_y = 7500\ mm \Rightarrow \frac{kL_y}{r_y} = \frac{7500^{mm}}{44.9^{mm}} = 167$$

Table 3-345 p.6-148.

$$\frac{kL_y}{r_y} = 167 \Rightarrow \phi F_{cr} = 52.8\ MPa$$

$$\phi P_n = 11800^{mm2} x 52.8^{MPa} = 622.5\ kN$$

EXAMPLE

Design the columns.
Check the interaction Eq.

$$(a)\ For\ \frac{P_r}{P_c} \geq 0.2 \Rightarrow \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_c x} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

$$(b)\ For\ \frac{P_r}{P_c} \rangle 0.2 \Rightarrow \frac{P_r}{2P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_c x} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

Calculate Pr.

$$P_r = P_{nt} + B_2 P_{lt}$$

$$P_{nt} = 495\ kN; P_{lt} = 43.75\ kN$$

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}}$$

EXAMPLE

Design the columns.

$$\sum P_{e2} = 2x\frac{\pi^2 x200000^{MPa} x1010x10^{6mm4}}{(3.5x7500^{mm})^2} = 5786\ kN$$

$$\sum P_{nt} = \sum P_u = 440^{kN} + 110^{kN} + 440^{kN} = 990\ kN$$

$$B_2 = \frac{1}{1 - \dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}} = \frac{1}{1 - 1.0\dfrac{990^{kN}}{5786^{kN}}} = 1.21$$

$$P_r = P_{nt} + B_2 P_{lt} = 495^{kN} + 1.21x43.75^{kN} = 548\ kN$$

EXAMPLE

Design the columns.

$$P_r = P_{nt} + B_2 P_{lt} = 495^{kN} + 1.21x43.75^{kN} = 548\ kN$$

$$\frac{P_r}{P_c} = \frac{P_u}{\phi P_n} = \frac{548^{kN}}{622.5^{kN}} = 0.88$$

This ratio is very big and there is no room for bending moment.

The problem is related to the relatively narrow flanges (b_f=209 mm). Lets try W530×150 with a b_f=312 mm.
ΦM_p= 1290 kN-m; A=19200 mm2; I_x= 1010×106 mm4
I_y= 103×106 mm4; r_x=229 mm; r_y=73.2 mm.

<u>No Sway Mode</u>: kLy/ry=(7500mm/73.2mm)=102
ΦF_{cr}= 137 Mpa
ΦP_n= 137MPax19200mm2=2630 kN
$P_u/\Phi P_n$=495kN/2630kN=0.19 (Reasonable.)

EXAMPLE

Design the columns.

<u>Sway Mode</u>: Check in-plane.
Inelasticity: P/A=495kN/19200mm2=25.8 MPa; τ=1.0

$$G_T = \frac{\tau \sum \left(\frac{I_x}{L_c} \right)_{W530x150}}{m \sum \left(\frac{I_x}{L_b} \right)_{W610x113}} = \frac{1.0 \left(\frac{1010x10^6}{7.5^m} \right)}{\left(\frac{875x10^6}{12.0^m} \right)} = 5.54 \rightarrow K_x = 3.5$$

$$G_B = pinned = \infty \rightarrow K_x = 3.5$$

$$\frac{kL_x}{r_x} = \frac{3.5x7500^{mm}}{229^{mm}} = 115 \Rightarrow \phi F_{cr} = 111.3\ MPa$$

$$\phi P_n = \phi F_{cr} A = 111.3^{MPa} x19200^{mm2} = 2136\ kN \Rightarrow \frac{P_u}{\phi P_n} = \frac{2x548^{kN}}{2x2136^{kN}} = 0.25$$

EXAMPLE

Design the columns.

Use the larger Pu/ΦPn ratio: 0.23

$$\frac{\sum P_u}{\sum \phi P_n} = 0.25 \rangle 0.2 \Rightarrow Use\ Eq.\ H1-1a$$

$$(a)\ For\ \frac{P_r}{P_c} \geq 0.2 \Rightarrow \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_c x} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + 0\right) \leq 1.0$$

$$M_{rx} = M_{ux} = B_1 M_{nt} + B_2 M_{lt}$$

EXAMPLE

Design the columns.

$$M_{rx} = M_{ux} = B_1 M_{nt} + B_2 M_{lt}$$

$$M_{nt} = 0 \Rightarrow No\ moment\ at\ columns\ due\ to\ gravity\ loads.$$

$$B_2 = \frac{1}{1 - \dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}}$$

$$\sum P_{e2} = 2x \frac{\pi^2 x 200000^{MPa} x 1010 x 10^{6^{mm4}}}{(3.5x7500^{mm})^2} = 5786\ kN$$

$$\sum P_{nt} = \sum P_u = 440^{kN} + 110^{kN} + 440^{kN} = 990\ kN$$

EXAMPLE

Design the columns.

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} = \frac{1}{1 - 1.0\,\frac{990^{kN}}{5786^{kN}}} = 1.21$$

$$M_{rx} = M_{ux} = B_1 M_{nt} + B_2 M_{lt} = 0 + 1.21x525^{\,kN-m} = 635.2\ kN - m$$

EXAMPLE

Design the columns.
Calculate the available bending strength of the “beam-column.”

1.2x525 $^{kN\text{-}m}$=635.3 kN-m

W530 × 150; ΦMp=1290 MPa
Cb=1.67; Lb= 7500 m; ΦMr=924 kN-m
Lp=3.1 m; Lr= 8.37 m
Lb<Lr; Therefor check C_bΦMr.
C_bΦMr=1.67x924kN-m=1543 MPa>ΦMp=1290 MPa
ΦMn=ΦMp=1290 MPa. (The section is compact.)

Calculate P_r.

$$P_r = P_{nt} + B_2 P_{lt} = 495^{kN} + 1.21x43.8^{kN} = 548\ kN$$

EXAMPLE

Design the columns.

$$(a)\ For\ \frac{P_r}{P_c} \geq 0.2 \Rightarrow \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_c x} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

$$\frac{P_r}{P_c} + \frac{8}{9}\frac{M_{rx}}{M_{cx}} = \frac{548^{kN}}{2136_{kN}} + \frac{8}{9}\frac{635.3^{kN-m}}{1290^{kN-m}} = 0.69 \langle 1.0$$

The differenece is too much. Decrease the size of the “beam-column” keeping in mind that b_f>200 mm.

Try W460×113 with a b_f = 280 mm. This section will do better. Do similar calculations one more time.

Need to check shear in the “beam-column”.

Also need to check Beam (B) for increased 2nd order moments.