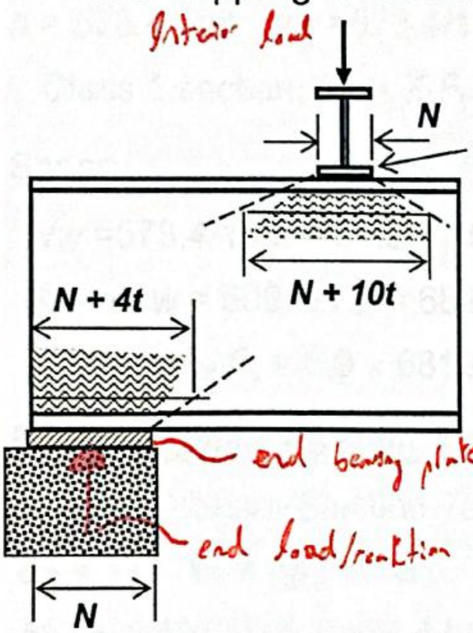
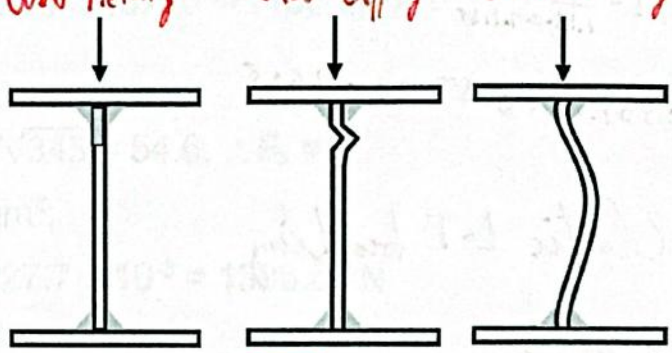
Concentrated Loads and Support Reactions

Special consideration should be given to the local compression in the web at the end, intermediate supports and locations of large concentrated loads. Concentrated loads may be introduced directly into a beam web by means of a bolted or welded connection, and the loaded beam may in turn transmit its end reactions to columns or girder by means of similar connections. The large compressive force in the web may lead to

3 Possible failure modes:

- Web Crippling or Yielding





if $\frac{h}{w} > 439\sqrt{\frac{5.34}{F_y}} \approx \frac{1014}{\sqrt{F_y}}$, Need stiffeners

Impacted by
- bearing width
- t of flange
- w thickness

Web Crippling and Yielding 14.3.2

Factored bearing resistance of the web:

(a) for interior loads (concentrated load applied at a distance from the member end greater than the member depth), the smaller of

(i) $B_r = \phi_{bi}\, w\,(N + 10t)\, F_y$ Web yielding

(ii) $B_r = 1.45\, \phi_{bi}\, w^2\,(F_y E)^{1/2}$ Web crippling

does not consider both sides. To be safe, use stiffener!!

(b) for end reactions, the smaller of

(i) $B_r = \phi_{be}\, w\,(N + 4t)\, F_y$

(ii) $B_r = 0.6\, \phi_{be}\, w^2\,(F_y E)^{1/2}$ Not function of N!!!

ϕ_{bi} = 0.8 and ϕ_{be} = 0.75; N = bearing length, w = web thickness, and t = flange thickness why different?

Wherever the bearing resistance of the web is exceeded, bearing stiffeners shall be used (see Clause 14.4).

Example 4-8 (see CivE 310 Example 4-7 for the solution) **:** A W610×113 simply supported beam of G40.21-M 350W steel spans 7000 mm. and subjected an uniformly distributed factored load 75 kN/m.

What is the factored shear resistance of the beam? Calculate the response ratios on shear and bending (assume the beam is laterally unsupported).

Solution: W610×113: b=228 mm, d=608 mm; w=11.2 mm; t =17.3 mm

$b/2t$ = 228 / (2×17.3) = 6.6 < $145/\sqrt{345} = 7.8$; Class 1 flange.

h = 573.4 mm; h/w =573.4/11.2 = 51.2 < $1100/\sqrt{345} = 51.2$; Class 1 web.

∴ Class 1 section; $M_p = Z_x F_y$ = 3290×345×10^{-3}=1135 MPa

Shear: 13.4.1.1

h/w =573.4/11.2 = 51.2 < $1014/\sqrt{345} = 54.6$; ∴ $\boldsymbol{F_s}$ **=**

$A_w = d \times w$ = 608×11.2 = 6810 mm^2;

∴ $V_r = \phi A_w F_s$ = 0.9 × 6810 × 227.7 × 10^{-3} = 1395.6 kN

Bending (unbraced span = 7m): 13.6.1

Use HSC Beam Slection Table: W610×113 (350W steel), L = 7000 mm

- Due uniform load, assume 1

ω_2 = 1.0, M_r = 481 ~~MPa~~ kN·m ; kN·m $M_u \geq 0.67 M_P$

M_r / ϕ = 481 / 0.9 = 534.4 kN-m < 0.67M_p = 760.5 ~~MPa~~.

∴ Elastic L-T buckling ?.? $M_u < 0.67\, M_p$

Combined Shear and Bending: Inelastic L-T buckling? Possible . Look at Req! 14.6

As F_s = 0.66F_y = 227.7 MPa, shear resistance reduction applies

M_f = 75×7^2 / 8 = 459 kN-m, M_r = 481 kN-m

Shear resistance reduction factor:

$$\left[2.20 - 1.6\, M_f / M_r\right] = 2.20 - 1.6 \times 459/481 = 0.67 < 1$$

V_r = 0.67× 1395.6 = 935 kN **< 0.6$\boldsymbol{\phi A_w F_y}$ = 1268.7 kN;** ∴ $\boldsymbol{V_r}$ **= 1268.7 kN**

Response ratios:

V_f = 0.5×75×7 = 263 kN; M_f = 459 kN-m

V_f / V_r = 263 / 1268.7 = 0.21 ; M_f / M_r = 457 / 481 = 0.95 ← Controls

If the beam it is laterally braced,

$M_r = \phi M_p$ = 0.9×1135 = 1021.5; M_f / M_r = 457 / 1021.5 = 0.45 ← Controls

Check for inelastic

if inelastic L-T buckling, class 1 or 2, $M_u > 0.67 M_p$

$$M_r = 1.15 \phi M_p \left[1 - \frac{0.28 M_p}{M_u} \right]$$

$$M_u = \frac{0.28 M_p}{1 - \frac{M_r}{1.15 \phi M_p}}$$

$$= \frac{0.28 \times 1135.5}{1 - \frac{483}{1.15 \cdot 0.9 \cdot 1135}}$$

$$= 538.1 < 0.67 M_p = 760.8$$

∴ Elastic L-T buckling

$M_r = \omega_2 M_r$ beam selection table

Example 4-9: Check the bearing resistance for the beam given in Example 4-8 to see if end-bearing stiffeners are required. The end-bearing length of the beam is 200 mm.

Solution :

End Reaction $B_f = V_f = 263$ kN

Web yielding of end support: 14.3.2

$B_r = \phi_{be} w(N + 4t)F_y = 0.75 \times 11.2 \times (200 + 4 \times 17.3) \times 345 \times 10^{-3} = \underline{780.1}$ kN

Web Crippling of end support:

$B_r = \phi_{be} w^2 \sqrt{F_y E} = 0.6 \times 0.75 \times 11.2^2 \sqrt{345 \times 2 \times 10^5} \times 10^{-3} = 468.9$ kN

$\therefore$ **B_r =** 468.9 > R_f = 263 kN ; $\therefore$ No stiffeners reqd

$B_f / B_r = 263 / 468.9 = 0.56$

Alternative method (HSC Beam Load Table):

HSC 12$^{ed.}$ Page 5-40.

Web yielding of end support:

R = 490 kN for 100 mm bearing length

G = 29. 0 kN for additional 10 mm bearing length

$\therefore$ For 200 mm bearing length:

B_r = R + 10 G = 490 + 10×29 = 780 kN

Web crippling of end support:

B_r' = 469 kN

Example 4-10 Design the parallel row of simply supported steel wide flange beams (G40.21–M 350W, F_y = 350MPa) of 10 m length, and spaced at 6 m apart, to carry a specified dead load of 2.0 kPa (self-weight of the beam is not included), and a specified live load of 2.5 kPa. Assuming:

1. The deck transmits the loads directly to the beams. The [deck is continuously attached to the above beams] thus providing full braced condition. ? deck span = 6m ? Also, not fully braced
2. The deck which is supported by purlins spaced at 2.5 m transmits the loads. The deck is continuously welded to the purlins only, and the [purlins are welded to the beam] ? Not practical
3. The deck which is supported by purlins spaced at 2.5 m transmits the loads. The [deck is continuously welded to the purlins only], and the purlins are not attached to the web of the beams. However, the beam is laterally braced at the ends .

(

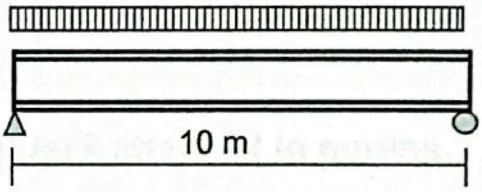
①

weld in line w/ applied force

weld not in same direction as force

① > ② bc more ductile

1. <u>Loads on the beam</u>



Dead Load = 2 kPa × 6 m = 12 kN / m
Self-weight = 1 kN / m (assume)

Live load including tributary area reduction

$$\text{Live Load} = \overset{\text{kPa}}{2.5}\left[0.3+\sqrt{9.8/\underbrace{(10\times 6)}_{\text{tributary area of the beam}}}\right]\times 6 = 10.6\ \text{kN/m}$$

∴ Factored load on beam = 1.25 Dead Load + 1.5 Live Load
= 1.25 × 13 + 1.5 × 10.6 = 32.15 kN / m

12 kN/m + 1 kN/m

Maximum Bending Moment: M_f = $\frac{wL^2}{8} = 32.15\times\left(\frac{10^2}{8}\right) = 402$ kN·m

Beam Selection Tables (HSC p5-96) 350W: Try W460 × 60; $F_y = 345$ MPa

Fully braced ∴ $L \le L_u$ =1970 mm; M_r = 397 kN-m (slightly less than M_f)

<u>Detailed Calculation:</u>

Self weight of W460 × 60 = 0.584 kN / m < 1 kN/m
Revised Factored moment due to loads

M_f = $\left[1.25(12+0.584)+1.5E6\right]\times\frac{10^2}{8} = 395.4$ kNm

W460 × 60 is a Class 1 section (see HSC Table 5-1 p.5-7)

F_y for G40.21 – M350W for shape Group 1 = 350 MPa (see HSC p.6-9)

Alternatively, check for classification: W460 × 60 (see HSC p6- 48)

$$\frac{b}{t}\sqrt{F_y} = \frac{(153/2)\times\sqrt{350}}{13.3} = 107.6 < 145$$

$$\frac{h}{w}\sqrt{F_y} = \frac{428\times\sqrt{350}}{8.0} = 1002 < 1100$$

$\therefore$ W460 × 60 is a Class 1 section

Since the deck is continuously attached to the beam, the beam can be considered fully braced $\therefore L_u = 0$ lateral, torsional? (assumed)

$\therefore$ **M_r =** $\phi M_p = \phi Z_x F_y = 0.9 \cdot 1280 \cdot 350 \cdot 10^{-3} = 403$ kNm 13.5.(a)

$\therefore M_r >$ Effects of factored loads (395.4 kN-m); O.K.

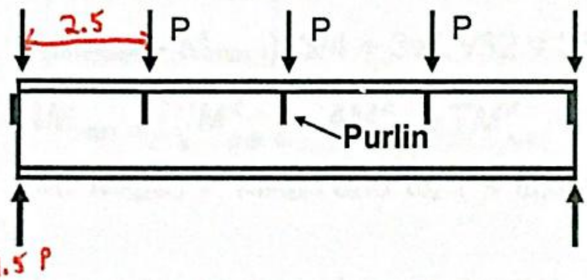
Deflection Check $I_{required} > W\times C_d \times B_d$ Service LL only

W = 10.6 kN / m × 10 m = 106 kN; C_d = 2.34 { for L = 10m, L/Δ = 360 } CivE 310 Chapter 4

B_d = 1.0 $\therefore I_{required} > 248 \times 10^6$ mm^4; I_x for W460×60 is 255×10^6 mm^4.

$\therefore$ W460 × 60 is satisfactory.

2. <u>Purlins transfer loads to beam and Deck attached to purlins</u>



Live Loads:

Note that only the interior three purlins transmit load to the beam. Therefore, tributary area for the beam 6 m × 2.5 m × 3 = 45 m^2

Factored Point Load:

$$P_f = 1.25\times \underset{kPa}{\overset{DL}{2}} \times(\underset{m}{6}\times \underset{m}{2.5})+1.5\times 2.5\times\overset{LLRF}{\left[0.3+\sqrt{\frac{9.8}{45}}\right]}\times\overset{T_a}{(6\times2.5)} = 80.6\,\text{kN}$$

Factored uniformly distributed load (assumed self-weight of the beam):

w = $1.25 \cdot 0.7 = 0.875$ kN/m

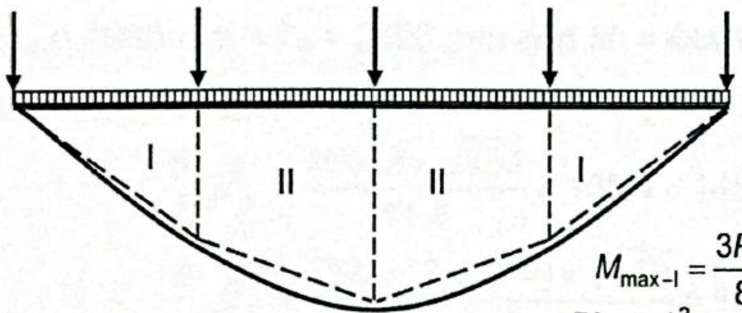
Maximum factored moment: → Self weight

$M_f = PL/2 + wL^2/8 = 80.6\times10/2 + 0.875\times10^2/8 = 403+10.94=414$ kN-m

$\therefore$ Choose a section with $M_r >$ 414 kN - m at unbraced length of 2.5 m.

$\frac{1.5}{2}PL - \frac{PL}{4}$

Bending Coefficient ω_2:



$$M_{\text{max-I}} = \frac{3PL}{8} + \frac{3wL^2}{32} = 310.45\,\text{kN-m}$$

$$M_{\text{max-II}} = \frac{PL}{2} + \frac{wL^2}{8} = 414\,\text{kN-m}$$

Segment I:

$M_{max\text{-}I}$=3PL/8+3wL²/32=310.45kNm; $M_{a\text{-}I}$=(3PL/8)/4+3wL²/32=78.12kNm

$M_{b\text{-}I}$=(3PL/8)/2+wL²/8=155.93kNm; $M_{c\text{-}I}$=3(3PL/8)/4+3wL²/32=233.35 kNm

$$\omega_{2\text{-I}} = 4M_{\text{max-I}} \Big/ \sqrt{M^2_{\text{max-I}} + 4M^2_{a\text{-I}} + 7M^2_{b\text{-I}} + 4M^2_{c\text{-I}}} = 1.74$$

Segment II:

$M_{max\text{-}II}$ = PL/2 + wL²/8 = 414 kN-m;

$M_{a\text{-}II}$ = $M_{max\text{-}I}$ + ($M_{max\text{-}II}$ - $M_{max\text{-}I}$)/4 + 3wL²/32 = 336.85 kN-m;

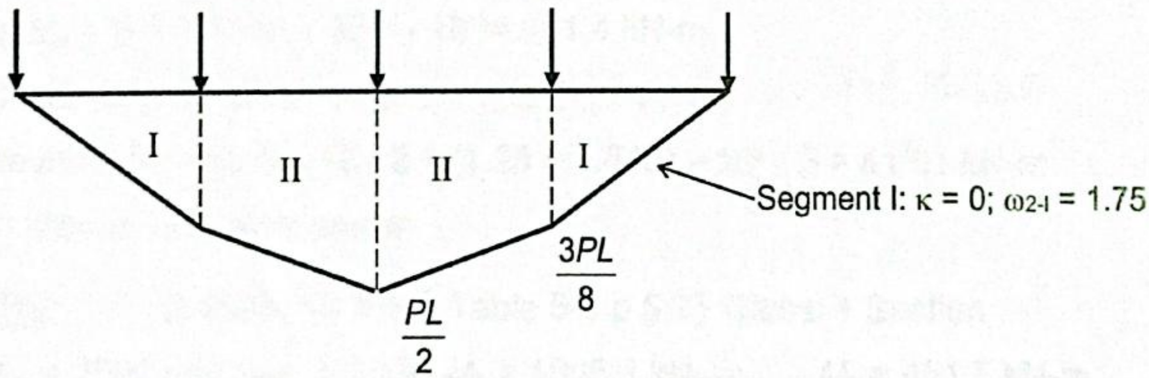
$M_{b\text{-}II}$ = $M_{max\text{-}I}$ + ($M_{max\text{-}II}$ - $M_{max\text{-}I}$)/2 + wL²/8 = 362.91 kN-m;

$M_{c\text{-}II}$ = $M_{max\text{-}I}$ + ($M_{max\text{-}II}$ - $M_{max\text{-}I}$)×3/4 + 3wL²/32 = 388.63 kN-m;

$$\omega_{2\text{-II}} = 4M_{\text{max-II}} \Big/ \sqrt{M^2_{\text{max-II}} + 4M^2_{a\text{-II}} + 7M^2_{b\text{-II}} + 4M^2_{c\text{-II}}} = 1.13$$

Considering that $M_{max\text{-}II}$ > $M_{max\text{-}I}$ and $\omega_{2\text{-}II}$ < $\omega_{2\text{-}I}$, segment II is the most critical section.

Alternatively, ω_2 can be evaluated approximately as following,



$$\text{Segment II}: \kappa = -\frac{3PL/8}{4PL/8} = -\frac{3}{4}; \quad \omega_{2\text{-II}} = 1.75 + 1.05 \times (-0.75) + 0.3 \times (0.75)^2 = 1.13$$

∴ Segment II $\omega_{2\text{-}II}$ = 1.13 governs ← in practice, self weight can be neglected

Try W530 × 66 (see HSC): self-weight =0.645 kN/m < 0.7 kN/m

L_{u_cr} = 1980 mm < L_u = 2500 mm and M_r = 444 kN-m (ω_2 = 1.0)

∴ L-T buckling control. Cannot take 1.13 × 444 = 502 as shown at e) that Inelastic LT buckling controls

Detail Calculations:

$$\frac{b}{t}\sqrt{F_y} = \frac{165/2 \times \sqrt{350}}{11.4} = 135.4 < 145$$

$$\frac{h}{w}\sqrt{F_y} = \frac{(525 - 2 \times 11.4) \times \sqrt{350}}{8.9} = 1055.7 < 1100$$

∴ W530 × 66 is a class 1 section.

Since only purlins provide lateral support, L_u = 2500 mm 13.6.1.a)

e) $$M_u = \frac{\omega_2 \cdot \pi}{L_u} \cdot \sqrt{EI_y \cdot G \cdot J + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w}$$

$$= \frac{1.13 \times \pi}{2500} \cdot \sqrt{\begin{matrix} 200 \times 10^3 \times 8.57 \times 10^6 \times 77 \times 10^3 \times 320 \times 10^3 \\ + \left[\left(\frac{\pi \times 200 \times 10^3}{2500}\right)^2 \times 8.57 \times 10^6 \times 565 \times 10^9\right] \end{matrix}} = 837.8\ \text{kN-m}$$

$0.67M_p = 0.67 \times Z \times F_y = 0.67 \times 1560 \times 0.35 = 365.8$ kN-m

∴ $M_u > 0.67M_p$

∴ $M_r = 1.15\phi\, M_p\,(1 - 0.28M_p / M_u)$ but < $\phi\, M_p$

$$M_r = 1.15 \times 0.9 \times 1560 \times 350\left(1 - \frac{0.28 \times 1560 \times 350}{837.8 \times 10^3}\right) = 462\ \text{kN-m}$$

$\phi\, M_p = 0.9 \times 1560 \times 350 \times 10^{-3} = 491.4$ kN-m

∴ $\boldsymbol{M_r}$ **=** 462 kN·m < 502. NO BEAM SELECTION TABLE

revised $M_f = 80.6 \times 10 / 2 + (1.25 \times 0.645) \times 10^2 / 8 = 413.1$ kN-m

∴ about 12% over design

Try: W460 × 60 (HSC Table 5-1 p.5-7) Class: 1 Section

L_u = 2500 mm; ω_2 = 1.13; M_u = 1005.3 kN-m; M_p = 451.5 kN-m

$0.67M_p < M_u$ ∴ M_r = 408.5 kN-m.

revised $\boldsymbol{M_f}$ **=** $80.6 \times \frac{10}{2} + (1.25 \times 0.590) \times \frac{10^2}{8} = 412.2$ kNm

∴ about 1% under design => OK in engineering practices

Deflection Check

$$W = 2.5 \times \left[0.3 + \sqrt{\frac{9.8}{45}}\right] \times 6 \times 2.5 = 28.75\,\text{kN}$$

$C_d = 2.34$ {for $L = 10$ m, $L / \Delta = 360$}

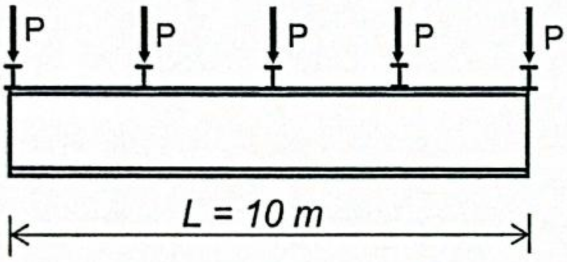
$B_d = 3.8$

$\therefore\ I_{req} = 28.75 \times 2.34 \times 3.8 = 255.6 \times 10^6\ \text{mm}^4$

I for W460 × 60 is $255 \times 10^6\ \text{mm}^4$

$\therefore$ Satisfactory.

3. Purlins are not connected to the web of the beam



Moment diagram for this situation is same as Case 2 except self-weight moment $\therefore\ M_f = 414$ kN-m

However, unbraced length = ? ; ω_2 = ?

For loads applied at the level of the top flange, in lieu of a more accurate analysis, M_u may be determined using $\omega_2 = 1.0$ and using an effective length, for pinned-ended beams, equal to 1.2L and, for all other cases, 1.4L.

L_u = $1.2 \times 2.5\text{m} = 3.0\text{m}$; $\omega_2 = 1.0$

$\therefore$ Choose a section $M_r > 414$ kN-m at $L_u = 3$ m

Try: 1 W410 × 74 (HSC Beam Selection Table) Class: 1 Section

Beam Selection Table: $L_u = 3000$ mm; $\omega_2 = 1.0$;

$\therefore\ M_r = 440.0$ kN-m; $M_f / M_r = 414/440 = 0.94 < 1.0$; OK

Deflection Check

As far as deflection is concerned there is no difference between Case 2 and Case 3.

$\therefore I_{required} \geq 28.75 \times 2.34 \times 3.8 = 255.6 \times 10^6$ mm^4

I_x for W410×74 is 275×10^6 mm$^4 > I_{required}$ O.K.

Shear Check

Bearing Failure Possibility (need to know end bearing length N)

Web Crippling Possibility (need to know end bearing length N)