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Steel Beams

- Section Classification
- Beam Section Capacity
- Full Lateral Restraint (FLR)

1. Section Classification

1.1 Local Buckling

- Beams can't sustain infinite curvature, at some curvature it fails
- Common failure = local instability (buckling) of plate elements (material fracture is also possible)
- Some beams may fail before reaching yield moment (slender) or plastic moment (some n-c)
- If the beam can reach plastic moment, rotation capacity (R) measures how much this plastic hinge can rotate before failure (can be estimated from a dimensionless moment vs. curvature diagram)

\[
R = \frac{K_1}{K_p} - 1,
\]

where \(K_p = \frac{M_p}{E*I}\)

Distance between curve crossing \(M_p = R\)

\(M_p = \) Plastic moment, \(M_y = \) Yield moment, \(R_{req} = 4\)

![ Moment-curvature behaviour of different types of steel sections (from Zhao et al. 2005) ]

1.2 Section Classification in Different Standards

<table>
<thead>
<tr>
<th>Specification</th>
<th>Section classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eurocode 3</td>
<td>Class 1</td>
</tr>
<tr>
<td>BS 59/30</td>
<td>Plastic</td>
</tr>
<tr>
<td>AS 4100</td>
<td>Compact</td>
</tr>
<tr>
<td>AISC LRFD</td>
<td>Compact</td>
</tr>
</tbody>
</table>

- Compact can attain the plastic moment & have plastic rotation capacity sufficient for plastic design
  \(\lambda_s > \lambda_{sp} \) & rotation capacity \(R > R_{req}\)
- Non-compact sections can reach the yield moment, but cannot reach the plastic moment
  \(\lambda_{sy} < \lambda_s < \lambda_{sp} \) & rotation capacity \(R < R_{req}\)
- Slender sections cannot reach the yield moment due to local buckling
  \(\lambda_s < \lambda_{sy}\)

1.3 Slenderness Limits or Width-to-Thickness ratio

- In AS4100 clear width is used to define element slenderness (Clear width = not including corners)
- EC3 Part 1.1 flat width defines width-to-thickness ratio (Flat width = considers curved corner radii)

For example, the element slenderness (\(\lambda_e\)) in AS4100 or width-to-thickness ratio for flanges and webs in a cold-formed RHS or I-section (dimensions shown in Figure 4) or CHS (circular hollow section) is defined as follows, where \(f_y\) and \(t_w\) are yield stress of the flange and web respectively.

- The slenderness or width-to-thickness ratios are compared w/ limiting values to determine the class
- The origin of slenderness limits was based on the elastic local buckling behaviour of perfect plates
- Material non-linearity (particularly for cold-formed steels), geometric imperfections & residual stresses all affect the local buckling behaviour
- Different slenderness limits are also specified for flanges and webs for the same cross section
1.4 To determine cross section class:
   1. Calculate the element slenderness ($\lambda_e$) for each element in flange & web
   2. Choose element w/ largest ($\lambda_e/\lambda_{ey}$) ratio as critical section slenderness ($\lambda_s$)
   3. Class is:
      - **Compact** if $\lambda_e < \lambda_{ep}$
      - **Non-compact** if $\lambda_{ep} \leq \lambda_e \leq \lambda_{ey}$
      - **Slender** if $\lambda_e > \lambda_{ey}$

$$\lambda_s = \lambda_e, \lambda_{sp} = \lambda_{ep}, \lambda_{sy} = \lambda_{ey} \text{ from critical element w/ largest } \lambda_e/\lambda_{ey}$$

$e$ = critical section, $s$ = whole section

<table>
<thead>
<tr>
<th>Plate stress relieved</th>
<th>Longitudinal edges supported</th>
<th>Residual Stress (RM)</th>
<th>Plasticity Limit ($\sigma_p$)</th>
<th>Yield Limit ($\sigma_y$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat</td>
<td>One</td>
<td>SR</td>
<td>10</td>
<td>16</td>
</tr>
<tr>
<td>(Uniform compression)</td>
<td></td>
<td>HR</td>
<td>9</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LW, CF</td>
<td>8</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HW</td>
<td>8</td>
<td>14</td>
</tr>
<tr>
<td>Flat</td>
<td>One</td>
<td>SR</td>
<td>10</td>
<td>25</td>
</tr>
<tr>
<td>(Uniform compression)</td>
<td></td>
<td>HR</td>
<td>9</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LW, CF</td>
<td>8</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HW</td>
<td>8</td>
<td>22</td>
</tr>
<tr>
<td>Flat</td>
<td>Both</td>
<td>SR</td>
<td>30</td>
<td>45</td>
</tr>
<tr>
<td>(Uniform compression)</td>
<td></td>
<td>HR</td>
<td>30</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LW, CF</td>
<td>30</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HW</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Flat</td>
<td>Both</td>
<td>Any</td>
<td>82</td>
<td>115</td>
</tr>
<tr>
<td>(Compression at one edge, tension at the other)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Circular hollow sections</td>
<td></td>
<td>SR</td>
<td>50</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HR, CF</td>
<td>50</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LW</td>
<td>42</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HW</td>
<td>42</td>
<td>120</td>
</tr>
</tbody>
</table>

**Notes:**
1. SR—stress relieved
   HR—hot rolled or hot finished
   LT—cold formed
   LW—lightly welded longitudinally
   HFW—heavily welded longitudinally
2. Welded members whose compressive residual stresses are less than 45 MPa may be considered to be lightly welded.

Stress relieved, hot welded, hot rolled, cold formed, light welded

Longitudinal edges = boundary conditions of section element
i.e. CHS flange is supported by two webs
I-section web has two boundaries
I-section flange has 1 support

For I-section looking at web, once side in compression and the other in tension hence is bottom category

**How to read table:**
1) look at if element is flat or HS
2) look at boundaries to determine supports
3) look at whether element in tension/comp, or both
4) look at manufacturing process (residual stress)

---

**Example 2**

Determine the class for a light-welded I-section subject to pure bending with the following dimensions and properties:

Overall flange width $b = 200$ mm
Overall depth $d = 600$ mm
Flange thickness $t_f = 16$ mm
Web thickness $t_w = 6$ mm
Web leg length $s = 6$ mm
Yield stress of flange $f_{ye} = 275$ MPa
Yield stress of web $f_{yr} = 275$ MPa

**Solution using AS4100**

The I-section is light-welded (LW).

<table>
<thead>
<tr>
<th>Flange</th>
<th>Web</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slenederness $\lambda_e = \frac{250}{200} = 1.25$</td>
<td>Slenederness $\lambda_w = \frac{250}{200} = 1.25$</td>
</tr>
</tbody>
</table>

Yield slenderness limit $\lambda_{ye} = 15$
Plasticity slenderness limit $\lambda_{sp} = 6.36$ or $6.42$

This I-section is a Non-compact section since $\lambda_{sp} = \lambda_e \leq \lambda_{sy}$

Section slenderness $\lambda_e = 99.29$
Plasticity slenderness limit $\lambda_{sp} = 82$

The web is more critical.

---

**Example 4**

Determine the class for a cold-formed RHS subject to pure bending with the following dimensions:

Overall flange width $b = 50$ mm
Overall depth $d = 75$ mm
Flange thickness $t_f = 2.5$ mm
Web thickness $t_w = 2.5$ mm
Yield stress $f_{ye} = f_{yr} = 350$ MPa

**Solution using AS4100**

<table>
<thead>
<tr>
<th>Flange</th>
<th>Web</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slenederness $\lambda_e = \frac{350}{250} = 1.4$</td>
<td>Slenederness $\lambda_w = \frac{350}{250} = 1.4$</td>
</tr>
</tbody>
</table>

Yield slenderness limit $\lambda_{ye} = 15$
Plasticity slenderness limit $\lambda_{sp} = 33.04$ or $33.06$

This cold-formed RHS is a compact section since $\lambda_e < \lambda_{sy}$

Section slenderness $\lambda_e = 21.24$
Plasticity slenderness limit $\lambda_{sp} = 30$

The flange is more critical.
2. Beam Section Capacity

2.1 Behaviour
- Strength of short beams is influenced by local buckling
- As a member buckles, the section properties change as the section moves closer to the NA (section can carry higher stresses if spread further from NA: capacity when buckled)

Region 1 (Compact), Region 2 (N-C), Region 3 (Slender)

1) Compact section can attain plastic moment
2) Non-compact section are sufficient to reach yield moment but will fail before reaching plastic moment
3) Slender sections governed by local buckling because insufficient to reach yield moment: buckle before yielding

![Diagram](image)

2.2 Section Capacity (from AS4100)

Nominal Section Capacity \( M_s \) = yield stress \( f_y \) * effective section modulus \( Z_e \)

Design section moment capacity \( \phi M_s \) = \( \phi f_y Z_e \) (where \( \phi = 0.9 \))

If \( \phi M_s > M^* \) then section is adequate

- \( Z_e \) = effective section modulus
- \( Z \) = elastic section modulus = \( I/y \) [mm^3]
- \( S \) = Plastic section modulus
- \( Z_c \) = \( Z_e \) for a compact section
- \( \lambda_s \) = section slenderness
- \( \lambda_{sy} \) = section yield slenderness limit
- \( \lambda_{sp} \) = section plasticity slenderness limit

*Note: For cold formed CHS the term \( \sqrt[2]{\lambda_{sy}/\lambda_s} < (2\lambda_{sy}/\lambda_s)^2 \)

Example:

A hot-rolled I-section beam (in m span) is simply supported with a design UDL of 24 kN/m. The beam is fully restrained so that it can achieve its section capacity. The dimensions and properties of the I-section are:

- Flange:
  - Width: 146 mm
  - Thickness: 9.8 mm
  - Flange yield stress: 255 MPa

- Web:
  - Width: 125 mm
  - Thickness: 5.5 mm
  - Web yield stress: 255 MPa

Yield stress of flange = 320 MPa
Yield stress of web = 250 MPa

Is the 1-section adequate if full lateral restraint is provided?

Solution using AS4100

1. Cross-section classification

<table>
<thead>
<tr>
<th>Flange</th>
<th>Cross-section classification</th>
<th>Web</th>
<th>Cross-section classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slenderness ( \lambda_s = \frac{b}{t} )</td>
<td>Flange: 2.35</td>
<td>Web: 2.60</td>
<td></td>
</tr>
<tr>
<td>Yield slenderness limit ( \lambda_{sy} = 15 )</td>
<td>Flange: 15</td>
<td>Web: 15</td>
<td></td>
</tr>
</tbody>
</table>

2. Section capacity

\( M_s = \max(150 \text{ kNm}, 1.5 \times 480 	imes 10^3 \text{ kNm} / 250 \) = 320 kNm

\( \phi M_s = 0.9 \times 320 = 288 \) kNm

The flange is more critical.

Section slenderness \( \lambda_s = 7.25 \)

Plasticity slenderness limit \( \lambda_{sp} = 9 \)

This 1-section is a Compact section since \( \lambda_s < \lambda_{sy} \)

The 1-section is adequate if full lateral restraint is provided.
3. Full Lateral Restraint

3.1 Behaviour

A beam bent about its major axis can cause flexural torsional buckling. Beam deflects downwards, but at some stage buckling occurs over the length of the member, in which the cross section moves laterally (out of the plane of bending) & twists.

The buckling deformations create bending about the minor axis & occur over the entire length of the beam, and it is sometimes called a member buckle, & the associated strength is sometimes called a member strength.

(also called lateral buckling, lateral-torsional buckling, or out-of-plane buckling)

3.2 FLR length

- If full lateral restraint (FLR) is provided to a beam the member capacity of the beam = section capacity (Lateral restraints prevent sideways movement of beam)
- The length below which the section capacity can be achieved is called FLR (Full Lateral Restraint) length in AS4100

\[
L_{FLR} = \begin{cases} 
\sqrt{\frac{250}{r_y}} & \text{if the segment is of equal flanged I-section} \\
\sqrt{\frac{250 \times (800 + 1500 \beta_{m})}{b_y \times (br)\times r_y}} & \text{if the segment is of RHS}
\end{cases}
\]

where \( r_y \) is the radius of gyration about the minor principal axis

\[
r_y = \sqrt{\frac{I_y}{A}}
\]

\( r_y = \text{section modulus about minor axis} \)

Example

A welded I-section beam (6 m span) is simply supported with a design UDL of 24 kN/m. The dimensions and properties of the I-section are:

<table>
<thead>
<tr>
<th>Overall flange width ( b )</th>
<th>146 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall depth ( d )</td>
<td>256 mm</td>
</tr>
<tr>
<td>Flange thickness ( t_f )</td>
<td>10.9 mm</td>
</tr>
<tr>
<td>Web thickness ( t_w )</td>
<td>6.4 mm</td>
</tr>
<tr>
<td>( I_y )</td>
<td>5.66 \times 10^4 mm^4</td>
</tr>
<tr>
<td>Yield stress of flange ( f_y )</td>
<td>320 MPa</td>
</tr>
<tr>
<td>Yield stress of web ( f_w )</td>
<td>200 MPa</td>
</tr>
</tbody>
</table>

**Solution using AS4100**

- Area \( A = 4882 \text{ mm}^2 \)
- Radius of gyration \( r_y = 34.8 \text{ mm} \)
- \( \beta_{m} = -0.8 \) (load on top)

**FLR length**

\[
L_{FLR} = \sqrt{\frac{250}{r_y}} \times \frac{250}{80 + 50\beta_{m}} \\
= 34.8 \times \sqrt{\frac{250}{80 + 50(-0.8)}} \\
= 1230.4 \text{ mm}
\]

Choose spacing = 1200 mm

(bars work well with whole span)
1. Tension Members

1.2 Design Capacity

\[ \Phi N_t = N \]
\[ \phi = 0.9 \]

- \( N_t \) = Nominal section capacity of a tension member
- \( N_t \) is taken as the min \([N_t, N_i]\) because in tension we can either failure by yielding or fracture

\( N_t = A_g f_y \) \text{ and } \( N_t = 0.85 k_t A_n f_u \)

- \( A_g \) = Gross area of cross-section
- \( f_y \) = Yield strength
- \( k_t \) = Correction factor to allow for eccentricity of connections
- \( f_u \) = Ultimate tensile strength
- \( A_n \) = Net area = Gross area – area of holes = \( A_g - A_h \)
  - If holes in line across member, \( A_h = \sum \) (hole diameter * plate thickness)
  - If holes staggered:
    
    \( A_h = \) greater of:
    1. Total hole area along straight ABDE
    2. Total hole area along staggered ABCDE less \( s_p^2 t/4 \cdot s_g \)

  - E.g.: \( A_h = 3d \cdot t - 2(s_p^2 t/4 \cdot s_g) \)

- To find \( k_t \): Tension members in trusses are connected eccentrically to other members or to gusset plates. When in bracing, tension members are often connected eccentrically to the members there bracing: \( \uparrow \) induces BM = \( P \cdot e \), \( \uparrow \) bending stresses, \( \uparrow \) stress on one side of member (hence non-uniform stress distribution) & distortion of bracing/truss

<table>
<thead>
<tr>
<th>Type</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unequal angle, short leg attached</td>
<td>0.75</td>
</tr>
<tr>
<td>Unequal angle, long leg attached</td>
<td>0.85</td>
</tr>
<tr>
<td>Equal angle</td>
<td>0.85</td>
</tr>
<tr>
<td>Channel, back attached</td>
<td>0.85</td>
</tr>
<tr>
<td>T-section &quot;bar&quot; attached</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Note: For I-sections & channels connected by their flanges \( k_t = 0.85 \)

Example: Determine the tensile capacity of a square hollow section (SHS 50x50x3) of Grade C350 \( f_y \) of 350 MPa and \( f_u \) of 430 MPa.

<table>
<thead>
<tr>
<th>Step</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>( A_g = 541 \text{ mm}^2 )</td>
</tr>
<tr>
<td>2</td>
<td>( f_y = 350 \text{ MPa} )</td>
</tr>
<tr>
<td>3</td>
<td>( f_u = 430 \text{ MPa} )</td>
</tr>
<tr>
<td>4</td>
<td>( 0.85f_u = 366 \text{ MPa} &gt; f_y )</td>
</tr>
</tbody>
</table>

Yielding governs: \( \Phi = 0.85 f_u f_y \) as we design against yielding, hence we take lesser ft.

Example: When the grade becomes C450 \( f_y \) of 450 MPa and \( f_u \) of 500 MPa?

<table>
<thead>
<tr>
<th>Step</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>( f_y = 450 \text{ MPa} )</td>
</tr>
<tr>
<td>2</td>
<td>( f_u = 500 \text{ MPa} )</td>
</tr>
<tr>
<td>3</td>
<td>( 0.85f_u = 425 \text{ MPa} &lt; f_y )</td>
</tr>
</tbody>
</table>

Fracture governs.

Nominal capacity:

\( N_i = 0.85 A_g f_y = 451.350 \approx 189 \text{ N} \) For yield use \( A_g \)

Design capacity:

\( \Phi N_i = 0.9 \cdot 189 = 170 \text{ kN} \)

The tensile capacity of the section is 170 kN.
Design of Bolts

- Failure Modes
- Method of Tightening
- Geometry of Bolt
- Design of Bolts
- Connection Capacity

1. Design of Single Bolts

1.1 Failure Modes
- **Bolts in Tension** = Forces are parallel to axis of bolt
- **Bolts in Shear** = Forces are perpendicular to axis of bolt
- Bolts under combined (tension & shear)

- Plate in bearing/tearing (tearing failure = Plate yields, necks above bolt and fails to extreme fibre)
- Plate in shear (Plate shear failure = Plate necks & fails (bolt stays in position)

1.2 Basic Properties
- **Commercial Bolts (or Black, Mild steel)**
  - Grade 4.6
    - Tensile Strength ($f_u$) = 400 MPa
    - Yield Stress ($f_y$) = 240 MPa
- **High Strength Structural Grade**
  - Grade 8.8
    - Tensile Strength ($f_u$) = 830 MPa
    - Yield Stress ($f_y$) = 640 MPa

1.3 Tightening
- **Snug Tight** = hand-tightened for bearing-type connections
  - i.e. edges of holes bear off bolt
- **Tensioned** = tightened w/ wrench to specific tension
  - i.e. develops friction by tightening

1.4 Types of Bolts

<table>
<thead>
<tr>
<th>Comparison: Grades 4.6 and 8.8 bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
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<tr>
<td></td>
</tr>
</tbody>
</table>

- 4.6/S = Commercial bolt, snug tight (bearing)
- 8.8/S = High strength, snug tight (bearing)
- 8.8/TB = HS structural bolt, tightened to specific tension (bearing + friction)
- 8.8/TF = HS structural bolt, tensioned + surface of plies prepared for friction

Simply supported = flexible
Fixed end = rigid
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1. Timber Properties

1.1 Comparison w/ steel
- **Density**: i.e. self weight is different (550kg/m³ vs 7800kg/m³)
- **Long term performance**: termite attacks vs corrosion
- **Timber** = Orthotropic, meaning properties change transversely vs longitudinally whereas steel = isotropic (properties don’t change)
- **Temp/humidity effect**: timber, because moisture absorption whereas thermal stresses are induced in steel
- **Timber can be seasoned** & steel can have different treatments

1.2 Seasoning
Reduce moisture content to produce timber at 15% moisture (seasoned) to minimize in-service shrinkage

1.3 Types of Wood

<table>
<thead>
<tr>
<th>Hardwoods</th>
<th>Softwoods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Broad leaf, high density, dark colour, larger heartwood bands</td>
<td>Needle like leaves, lower density, light colour, large sapwood band</td>
</tr>
<tr>
<td>i.e. Oaks, spotted gum</td>
<td>i.e. Pines, cedars</td>
</tr>
</tbody>
</table>

1.4 Sizes (typical width, thickness & length)
Varies based on whether sawn timber is:
Unseasoned Hardwood or Softwood
Seasoned Hardwood or Softwood

1.5 Strength Group
Timber species are subject to ‘small clear’ (small clears specimen = 600mm long & are 20*20) testing & allocated to a particular strength group based primarily on bending strength & stiffness, defined:
- **S1-S7** for unseasoned timber
- **SD1-SD8** for seasoned (dry) timber
1.6 Structural Grades & Stress Grades

Note:
‘Small clears’ test only gives estimate of timber strength for idealised piece
Also sort timber into structural grades based on defect level, & assign degraded design properties to defected pieces
Structural grades then define defect level that, for a timber w/ known strength assign it to a stress grade whereby the designer can obtain the balance of the design properties from AS1720.1

F-Grade

MGP10-15 & A17
Timber Beam Strength

- Bending Capacity
- Shear Capacity
- Bearing Capacity

1. Bending Capacity

1.1 Bending Capacity

\[ M_d = \phi_k_1 k_4 k_6 k_9 k_{12} f'_b Z \]

- **\( M_d \)** = Design capacity in Bending of unnotched beam (see example on this page, page 7)
- **\( M^* \)** = Moment Action (for simply supported \( M^* = \frac{wxL^2}{8} \))

\[ Z = \frac{bd^2}{6} \text{ or } Z = \frac{bd^4}{6} \]

\( \phi \) = Capacity reduction factor (table 2.1, see page 4)

- **\( k_1 \)** = Factor for load duration
- **\( k_4 \)** = Factor for in-service absorption/desorption of moisture by timber
- **\( k_6 \)** = Factor for temperature/humidity affect
- **\( k_9 \)** = Factor for load-sharing in grid system
- **\( k_{12} \)** = Factor for stability

\( f'_b \) = Bending strength [MPa] (table H2.1 & H3.1, see page 3)

Example 1

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>D</td>
<td>S</td>
<td>D</td>
</tr>
</tbody>
</table>

Diagonal parallel system

Four members are diagonally spaced parallel to each other. Diagonal parallel system

Length of beam, \( L = 3000 \text{mm} \)

Assumed: Beams are bending about their major axis, concrete lateral restraint to compression edge is provided every 1000mm, \( L_{xy} = 1000 \text{mm} \)

What is the maximum allowed UEL in kN/m?

Parameters influencing the bending capacity

- Floor live load (UEL)
- Concrete grade
- Floor thickness
- Height of beam

Solution

\[ M_d = \frac{250}{1000} \times \frac{3000}{1000} \times \frac{3.61}{10} \]

Bending capacity

\[ M_d = 0.05 \times 0.8 \times 1.2 \times 1.0 \times 0.8 \times 0.8 \times 0.8 \text{ kN/m} = 26.6 \text{ kN/m} \]

Maximum allowed UEL

\[ w = \frac{BM_dL^2}{8} = 26.6 \times 0.8 = 21.3 \text{ kN/m} \]
1.2 Bending Factors

**k₁** = Load duration factor (Table G1)

<table>
<thead>
<tr>
<th>Type of load (system)</th>
<th>UNSYS/UNSPRT load combination (Q)</th>
<th>Load duration factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent action (dead load)</td>
<td>1.5Q</td>
<td>0.37</td>
</tr>
<tr>
<td>Permanent and short term imposed actions</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Roof live load—Generalized</td>
<td>1.0Q</td>
<td>0.57</td>
</tr>
<tr>
<td>(b) Roof live load—Concentrated</td>
<td>1.05Q</td>
<td>0.53</td>
</tr>
<tr>
<td>(c) Floor live loads—Distributed</td>
<td>1.0Q</td>
<td>0.57</td>
</tr>
<tr>
<td>(d) Floor live loads—Concentrated</td>
<td>1.05Q</td>
<td>0.53</td>
</tr>
<tr>
<td>Permanent and long term* imposed action</td>
<td>1.5Q</td>
<td>0.57</td>
</tr>
<tr>
<td>Permanent, total and imposed action</td>
<td>1.0Q</td>
<td>1.0</td>
</tr>
<tr>
<td>Permanent and wind action reversal</td>
<td>0.95Q</td>
<td>1.0</td>
</tr>
<tr>
<td>Fire</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* The notation used in this Table is from AS/NZS 1170.4

**k₄** = Partial Seasoning Factor (Table 2.5)

If seasoned/unseasoned, k₄ = 1
If partial seasoned = Use table 2.5

<table>
<thead>
<tr>
<th>Least dimension of member</th>
<th>38 mm or less</th>
<th>50 mm</th>
<th>75 mm</th>
<th>100 mm or more</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value of k₄</td>
<td>1.15</td>
<td>1.10</td>
<td>1.05</td>
<td>1.00</td>
</tr>
</tbody>
</table>

**k₆** = Temperature/humidity factor

High temps over extended times cause embrittlement & ↓ strength of timber. Humidity shrinks/swells timber

- Covered timber under ambient conditions k₆ = 1
- Seasoned timber structures in coastal QLD or regions in North AUS k₆ = 0.9

**k₉** = Strength sharing Factor

Applied to:
- Closely spaced parallel & similar members
- Cross members that provide load-sharing of parallel members

- Parallel members working together, weak members get assistance from stronger members in parallel systems

Achieved by whole-system transferring load (shares load to parallel members) to prevent failure

\[
k₉ = g₃₁ + (g₃₂ - g₃₁) \left[1 - \frac{2s}{L}\right], \text{ but not less than } 1.0
\]

- L = length of beam
- s = spacing of centres
- \( g₃₁ \) = geometric factor for no. of members (n_com) in combined parallel system (from table 2.7)
- \( g₃₂ \) = geometric factor for no. of members (n_com * n_mem) in discrete system (from table 2.7)
- n_com = no. of elements in single group
- n_mem = no. of members that are discretely spaced parallel

Note: k₉ cannot be greater than g₃₂ or less than g₃₁
$k_{12}$ = Stability factor (lateral torsional buckling)

- $k_{12}$ is a function of Material Constant ($\rho_b$) & Slenderness ($S$)
  - $k_{12} < 1$ for slender members
- Slender sections (large depth to breadth ratio) under bending, compression edge buckles causing sideways movement/twisting i.e lateral torsional buckling
- Loads applied in plane: beam had tendency to buckle & go out-of-plane

- **Material Constant ($\rho_b$) allows for:**
  - Initial curvature of member
  - Inelasticity of timber (creep buckling)

### TABLE 3.1

<table>
<thead>
<tr>
<th>Stress grade</th>
<th>Material constant ($\rho_b$)</th>
<th>Seasoned timber</th>
<th>Unseasoned timber</th>
</tr>
</thead>
<tbody>
<tr>
<td>F34</td>
<td>1.12</td>
<td>1.21</td>
<td></td>
</tr>
<tr>
<td>F27</td>
<td>1.08</td>
<td>1.17</td>
<td></td>
</tr>
<tr>
<td>F22</td>
<td>1.05</td>
<td>1.15</td>
<td></td>
</tr>
<tr>
<td>F17</td>
<td>0.98</td>
<td>1.08</td>
<td></td>
</tr>
<tr>
<td>F14</td>
<td>0.98</td>
<td>1.08</td>
<td></td>
</tr>
<tr>
<td>F11</td>
<td>0.98</td>
<td>1.07</td>
<td></td>
</tr>
<tr>
<td>F8</td>
<td>0.89</td>
<td>0.99</td>
<td></td>
</tr>
<tr>
<td>F7</td>
<td>0.86</td>
<td>0.96</td>
<td></td>
</tr>
<tr>
<td>F5</td>
<td>0.82</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td>F4</td>
<td>0.80</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>MGP 15</td>
<td>0.91</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MGP 12</td>
<td>0.85</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MGP 10</td>
<td>0.75</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A17</td>
<td>0.95</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **Slenderness ($S_1$)**

**Beams that bend about their major axis having discrete lateral restraint systems to compression edge**

$$S_1 = 1.25 \frac{d}{b} \left(\frac{L}{d}\right)$$

**Beams that bend about their major axis having discrete lateral restraint systems to tension edge**

$$S_1 = \frac{L}{d} \left(\frac{E}{F_{\text{tens}}}\right)$$

**Beams that bend about their major axis continuous lateral restraint to compression edge**

$$S_1 = 0$$

**Beams that bend about their major axis continuous lateral restraint to tension edge**

$$S_1 = 2.25 \frac{d}{b}$$

**Beams that bend about their major axis discrete lateral restraint systems to tension edge and torsional restraints**

$$S_1 = \frac{L}{d} \left(\frac{E}{F_{\text{tens}}}\right) + 0.4$$

**Continuous restraint along the tension edge**

**Continuous restraint along the compression edge**
1. Shear Capacity

2.1 Shear Capacity

$V_d \geq V^*$

$V_d = \text{Design capacity in Bending of unnotched beam (see example on page 10)}$
$V^* = \text{Shear Action (for simply supported case $V^* = \frac{wL^2}{2}$)}$

$$V = \phi k_1 k_4 k_6 f'_s A_s$$

$\phi = \text{Capacity reduction factor (table 2.1, see page 4)}$
$k_1 = \text{Factor for load duration (see page 8)}$
$k_4 = \text{Factor for in-service absorption/desorption of moisture by timber (see page 8)}$
$k_6 = \text{Factor for temperature/humidity affect (see page 8)}$
$f'_s = \text{Shear strength [MPa] (table H2.1 & H3.1, see page 3)}$

**Note:** Shear strength is small because timber grains are weak in shear (shear splits cells in grain)

$A_s = \text{Shear plane area (temperature/humidity affects will affect plane area)}$

$A_s = \frac{2}{3}b + d$

---

**Example 1**

- Four members are discretely spaced parallel to each other.
- Shear timber (F17 stress grade) used for Category 1 structural floor application in Melbourne.
- Cross section: $b = 120$ mm, $d = 200$ mm
- $L = 0.75$ m
- $L = 3$ m, simply supported (length of beam, $L=3000$ mm)
- Assumed: Shears are bending about their major axis. Discrete lateral restraint to compression edge is provided every 600 mm, $L_y = 600$ mm

What is the maximum allowed UDL in kN/m?

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**Example 2**

Following Example 1

- Under the maximum allowed UDL for bending capacity, check shear capacity.
- $w = 25.4$ kN/m = max UDL found from Eq. 1

**Shear force**

- $V = \frac{wL^2}{2} = 25.4 \times 3 \times 2 = 30.1$ kN
- $f_s = 3.6$ MPa, tensile grade $= F17$
- $A_s = (2/3)(bd) = (2/3) \times (120 \times 200) = 16000$

**Shear capacity**

- $\phi = 0.95$ tensile grade 1 A F17 grade
- $k_1, k_4, k_6$ factors are taken from example 1

- $V_s = k_1 k_2 k_3 k_4 k_6 A_s$

- $V_s = 0.95 \times 0.8 \times 1.0 \times 1.0 \times 3.6 \times 16000 \times 43.4$ kN > $V^*$

- OK