CIV2225 – Design of Steel & Timber Structures (Part 1)

Contents

Steel Beams

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

1. Section Classification

1.1 Local Buckling

- Beams cant 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)

  $R = K_1/K_p - 1$, where $K_p = M_p/E*I$

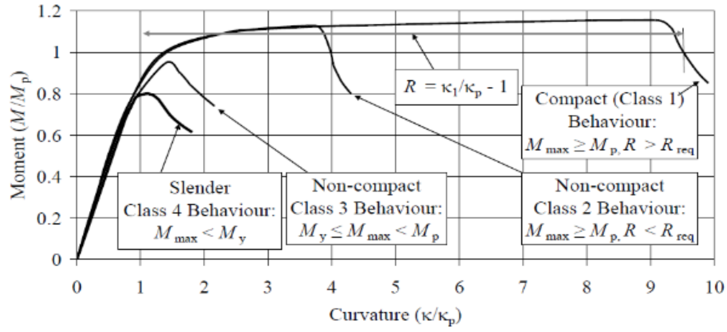


Figure 2 Moment-curvature behaviour of different types of steel sections (from Zhao et al. 2005)

Distance between curve crossing M_p = R

M_p = Plastic moment, M_y = Yield moment, $R_{req} = 4$

1.2 Section Classification in Different Standards

Table 1 Section classification in various design standards

Specification	Section classification			
Eurocode 3	Class 1	Class 2	Class 3	Class 4
BS 5950	Plastic	Compact	Semi-Compact	Slender
AS 4100	Compact	Non-Compact		Slender
AISC LRFD	Compact	Non-Compact		Slender

- **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 (λ_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_{yf} and f_{yw} are yield stress of the flange and web respectively.

(a) Cold-formed RHS (b) I-section

Element slenderness in AS4100

Flange in cold-formed RHS $\lambda_e = (\frac{b-2t_w}{t_f}) \cdot \sqrt{\frac{f_{yf}}{250}}$

Web in cold-formed RHS $\lambda_e = (\frac{d-2t_f}{t_w}) \cdot \sqrt{\frac{f_{yw}}{250}}$

CHS $\lambda_e = (\frac{d}{t}) \cdot \frac{f_y}{250}$

Flange in I-section $\lambda_e = (\frac{b-t_w}{2t_f}) \cdot \sqrt{\frac{f_{yf}}{250}}$

Web in I-section $\lambda_e = (\frac{d-2t_f}{t_w}) \cdot \sqrt{\frac{f_{yw}}{250}}$

Width-to-thickness ratio in EC3 Part 1.1

Flange in cold-formed RHS $(\frac{b-3t_w}{t_f})$

Web in cold-formed RHS $(\frac{d-3t_f}{t_w})$

Flange in I-section $(\frac{b-t_w-2r}{2t_f})$

Web in I-section $(\frac{d-2t_f-2r}{t_w})$

where r is the corner radius for rolled sections or weld leg length for welded sections.

- 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 (λe) for each element in flange & web
2. Choose element w/ largest (λe/λey) ratio as critical section slenderness (λs)
3. Class is:

- Compact if $\lambda_s < \lambda_{sp}$
- Non-compact if $\lambda_{sp} \leq \lambda_s \leq \lambda_{sy}$
- Slender if $\lambda_s > \lambda_{sy}$

λs = λe, λsp = λep, λsy = λey from critical element w/ largest λe/λey
e = critical section, s = whole section

Table 2 Values of plate element slenderness limits for class classification (from AS4100)

Plate element type	Longitudinal edges supported	Residual stresses (see Notes)	Plasticity limit (λ_{ep})	Yield limit (λ_{ey})
Flat	One	SR	10	16
(Uniform compression)		HR	9	16
		LW, CF	8	15
		HW	8	14
Flat	One	SR	10	25
Maximum compression at unsupported edge, zero stress or tension at supported edge)		HR	9	25
		LW, CF	8	22
		HW	8	22
Flat	Both	SR	30	45
(Uniform compression)		HR	30	45
		LW, CF	30	40
		HW	30	35
Flat	Both	Any	82	115
(Compression at one edge, tension at the other)				
Circular hollow sections		SR	50	120
		HR, CF	50	120
		LW	42	120
		HW	42	120

NOTES:

1 SR—stress relieved
HR—hot-rolled or hot-finished
CF—cold formed
LW—lightly welded longitudinally
HW—heavily welded longitudinally

2 Welded members whose compressive residual stresses are less than 40 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)

E.g.

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 = 16mm
Web thickness t_w = 6 mm
Weld leg length s = 6 mm
Yield stress of flange f_{yf} = 275 MPa
Yield stress of web f_{yw} = 275 MPa

Solution using AS4100

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

Flange

$$\text{Slenderness } \lambda_e = \left(\frac{b - t_w}{2t_f}\right)\cdot\sqrt{\frac{f_{yf}}{250}} = \left(\frac{200-6}{2\times16}\right)\sqrt{\frac{275}{250}} = 6.36$$

Yield slenderness limit $\lambda_{ey} = 15$
$(\lambda_e / \lambda_{ey}) = 6.36/15 = 0.42$

Get lamda.e from equation
Get lamda.ey from table 2

Web

$$\text{Slenderness } \lambda_e = \left(\frac{d - 2t_f}{t_w}\right)\cdot\sqrt{\frac{f_{yw}}{250}} = \left(\frac{600-2\times16}{6}\right)\sqrt{\frac{275}{250}} = 99.29$$

Yield slenderness limit $\lambda_{ey} = 115$
$(\lambda_e / \lambda_{ey}) = 99.29/115 = 0.86$

The web is more critical.

Section slenderness λ_s = 99.29

Plasticity slenderness limit λ_{sp} = 82

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

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 $f_{yf} = f_{yw} = f_y$ = 350 MPa

Solution using AS4100

$$\sqrt{\frac{f_y}{250}} = \sqrt{\frac{350}{250}} = 1.18$$

Flange

$$\text{Slenderness } \lambda_e = \left(\frac{b - 2t_w}{t_f}\right)\cdot\sqrt{\frac{f_{yf}}{250}} = \left(\frac{50-2\times2.5}{2.5}\right)\times1.18 = 21.24$$

Yield slenderness limit $\lambda_{ey} = 40$
$(\lambda_e / \lambda_{ey}) = 21.24/40 = 0.53$

Web

$$\text{Slenderness } \lambda_e = \left(\frac{d - 2t_f}{t_w}\right)\cdot\sqrt{\frac{f_{yw}}{250}} = \left(\frac{75-2\times2.5}{2.5}\right)\times1.18 = 33.04$$

Yield slenderness limit $\lambda_{ey} = 115$
$(\lambda_e / \lambda_{ey}) = 33.04/115 = 0.29$

The flange is more critical.

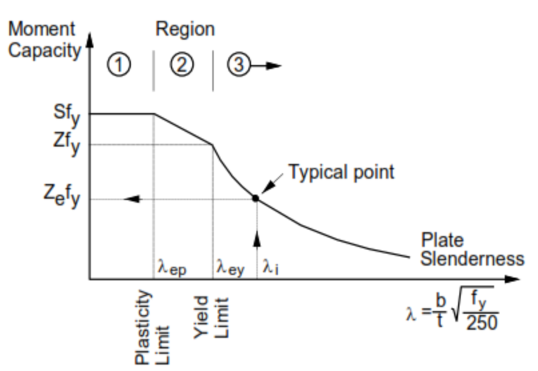
Section slenderness λ_s = 21.24
Plasticity slenderness limit λ_{sp} = 30

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

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 bcas insufficient to reach yield moment ∴ buckle before yielding

2.2 Section Capacity (from AS4100)

$$M_s = f_y Z_e$$

Nominal Section Capacity (M_s) = yield stress (f_y) * effective section modulus (Z_e)
Msx = beam section capacity about major axis

$$\phi M_s = \phi f_y Z_e$$

Design section moment capacity (ϕM_s) = ϕf_y* Z_e $\phi = 0.9$

If ϕM_s > M* then section = adequate

For compact sections

$$Z_e = \min[S, 1.5Z]$$

For non-compact sections

$$Z_e = Z + \left(\frac{\lambda_{sy} - \lambda_s}{\lambda_{sy} - \lambda_{sp}}\right)(Z_c - Z)$$

For slender sections

$$Z_e = Z\left(\frac{\lambda_{sy}}{\lambda_s}\right)$$

For slender circular hollow sections

$$Z_e = \min\left[Z\sqrt{\frac{\lambda_{sy}}{\lambda_s}}; Z\left(\frac{2\lambda_{sy}}{\lambda_s}\right)^2\right]$$

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
λ_s = section slenderness
λ_{sy} = section yield slenderness limit
λ_{ep} = section plasticity slenderness limit

*Note: For cold formed CHS the term sqrt(λsy/λs) < (2λsy/λs)^2

E.g.

2.3 Example

A hot-rolled I-section beam (6 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:

Overall flange width b = 146 mm
Overall depth d = 256 mm
Flange thickness t_f = 10.9 mm
Web thickness t_w = 6.4 mm
Root radius r = 8.9 mm
Radius of gyration r_y = 34.5
Plastic section modulus $S_x = W_{pl} = 486 \times 10^3$ mm³
Elastic section modulus $Z_x = W_{el} = 435 \times 10^3$ mm³
Yield stress of flange f_{yf} = 320 MPa
Yield stress of web f_{yw} = 320 MPa

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

Solution using AS4100

(1). Cross-section classification

Flange

Slenderness $\lambda_e = \left(\frac{b - t_w}{2t_f}\right)\cdot\sqrt{\frac{f_{yf}}{250}} = \left(\frac{146 - 6.4}{2 \times 10.9}\right)\sqrt{\frac{320}{250}} = 7.25$

Yield slenderness limit $\lambda_{ey} = 16$
$(\lambda_e / \lambda_{ey}) = 7.25/16 = 0.45$

Web

Slenderness $\lambda_e = \left(\frac{d - 2t_f}{t_w}\right)\cdot\sqrt{\frac{f_{yw}}{250}} = \left(\frac{256 - 2 \times 10.9}{6.4}\right)\sqrt{\frac{320}{250}} = 41.4$

Yield slenderness limit $\lambda_{ey} = 115$
$(\lambda_e / \lambda_{ey}) = 41.4/115 = 0.36$

The flange is more critical.

Section slenderness λ_s = 7.25
Plasticity slenderness limit λ_{sp} = 9

This I-section is a Compact section since $\lambda_s < \lambda_{sp}$

(2). Section capacity

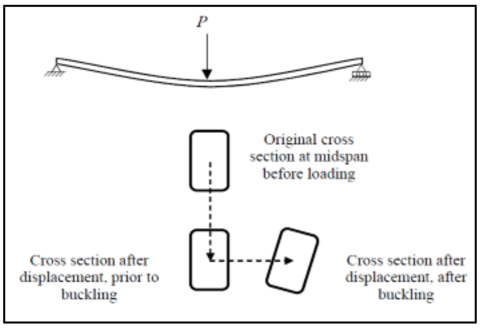
$Z_{ex} = \min[S_x, 1.5Z_x] = \min[486 \times 10^3, 1.5 \times 435 \times 10^3] = 486 \times 10^3 \text{ mm}^3$
$\phi M_s = \phi f_y Z_{ex} = 0.9 \times 320 \times 486 \times 10^3$ N mm = 140 kNm

Action M* = wL²/8 = 24*6²/8 = 108 kNm
If phy*M.s > M* section = adequate
If design moment > moment max = section adequate

The I-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 ∴ 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

Full lateral restraint length (L_{FLR}):

$$L_{FLR} \le r_y \times (80 + 50\beta_m)\sqrt{\frac{250}{f_y}}$$ if the segment is of equal flanged I-section

$$L_{FLR} \le r_y \times (1800 + 1500\beta_m)\left(\frac{b_f}{b_w}\right)\left(\frac{250}{f_y}\right)$$ if the segment is of RHS

where r_y is the radius of gyration about the minor principal axis

$$r_y = \sqrt{\frac{I_y}{A}}$$

I.y = section modulus about minor axis

the ratio β_m shall be taken as one of the following as appropriate:

- -1.0 conservative
- -0.8 for segments with transverse loads; or
- the ratio of the smaller to the larger end moments in the length L, (positive when the segment is bent in reverse curvature and negative when bent in single curvature) for segments without traverse loads.

E.g.

1.3 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:

Overall flange width b = 146 mm
Overall depth d = 256 mm
Flange thickness t_f = 10.9 mm
Web thickness t_w = 6.4 mm

$I_y \approx 5.66 \times 10^6$ mm⁴

Yield stress of flange f_{yf} = 320 MPa
Yield stress of web f_{yw} = 320 MPa

What is the FLR length?

Solution using AS4100

Area A ≈ 4882 mm^2
Radius of gyration $r_y \approx$ 34.8 mm
β_m = - 0.8 bcas UDL transfers load on top

FLR length

$L_{FLR} = r_y (80+50\beta_m) \sqrt{(250/f_y)}$
= 34.8 x (80 + 50 (-0.8)) √(250/320)
= 1230.4 mm

Choose spacing = 1200 mm
bcas works well with whole span

Tension Members, Base Plates & Combined actions

- Tension Members
- Base Plates
- Combined Actions

1. Tension Members

1.2 Design Capacity

$$N^* \le \Phi N_t$$

$\phi = 0.9$

N_t = nominal section capacity of a tension member

N_t is taken as the min[N_t,N_t] bcas in tension we can either failure by yielding or fracture

$$N_t = A_g\, f_y \quad \text{and} \quad N_t = 0.85\, k_t\, A_n\, f_u$$

A_g = Gross area of cross-section

f_y = Yield strength

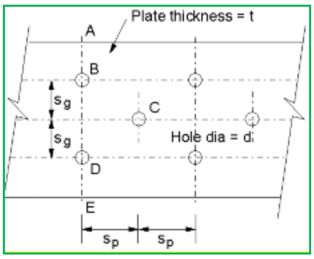
k_t = correction factor to allow for eccentricity of connections

f_u = ultimate tensile strength

A_n = net area = gross area – area of holes = $A_g - A_h$

if holes are in line across member A_h = ∑(hole dimeters * plate thickness)

if holes are staggered:



A_h = greater of:

(i) total hole area along straight ABDE

(ii) total hole area along staggered ABCDE less $s_p{}^2*t/4*s_g$

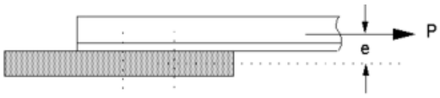
e.g. $A_h = 3*d*t - 2(s_p{}^2*t/4*s_g)$

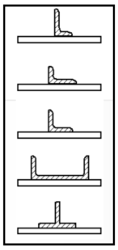
- To find k_t:

Tension members in trusses are connected eccentrically to other members or to gusset plates

When in bracing, tensions members are often connected eccentrically to the members there bracing

∴ induces BM = P*e, ↑ bending stresses, ↑ stress on one side of member (hence non-uniform stress distribution) & distortion of bracing/truss





unequal angle, short leg attached	0.75
unequal angle, long leg attached	0.85
equal angle	0.85
channel, back attached	0.85
T-section "bar" attached	0.90

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

E.g.

1.4 Example

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

What happens when the Grade becomes C450 (f_y of 450 MPa and f_u of 500 MPa)?

Solution

check fracture v yielding

$A_g = 541\ mm^2$

$f_y = 350$ MPa

$f_u = 430$ MPa

$0.85 f_u = 0.85 \times 430 = 366\ MPa > f_y$

Yielding governs — as .85fu>fy we design against yielding bcas we take lesser Nt

Nominal capacity

$N_t = A_g \cdot f_y = 541 \times 0.350 = 189\ kN$ For yield use Ag

Design capacity

$\phi N_t = 0.9 \times 189 = 170\ kN$

The tensile capacity of the section is 170 kN.

What happens when the Grade becomes C450 (f_y of 450 MPa and f_u of 500 MPa)?

f_y = 450 Mpa

f_u = 500 Mpa

$0.85 f_u$ = 425 Mpa < f_y Fracture governs

Nominal capacity

$N_t = 0.85\, k_t\, A_n\, f_u = 0.85 \times 1.0 \times 541 \times 0.500 = 230\ kN$

Design capacity

$\Phi N_t = 0.9 \times 230 = 207\ kN$

The tensile capacity of the section is 207 kN.

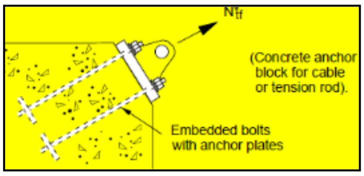
Design of Bolts

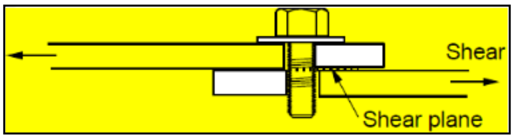
- Failure Modes
- Method of Tightening
- Geometry of Bolt
- Design of Bolts
- Connection Capacity

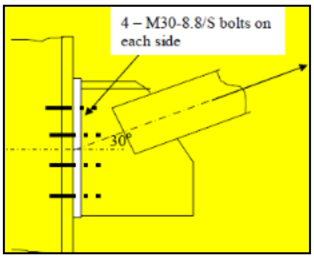
1. Design of Single Bolts

1.1 Failure Modes

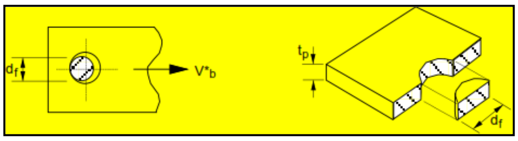
- Bolts in Tension = Forces are parallel to axis of bolt
- Bolts in Shear = Force are perpendicular to axis of bolt
- Bolts under combined (tension & Shear)

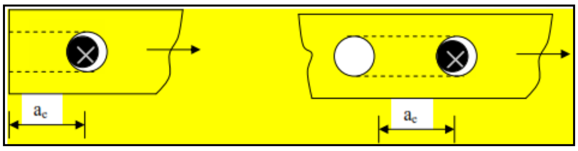






- Plate in bearing/tearing (tearing failure = Plate yields, necks above bolt and fails to extreme fibre)
- Plate in shear (Plate shear failure = Plate necks & fails (bolt stays in position)





1.2 Basic Properties

- Commercial Bolts (or Black, Mild steel)

  Grade 4.6

  Tensile Strength (f_{uf}) = 400 MPa

  Yield Stress (f_{yf}) = 240 MPa

- High Strength Structural Grade

  Grade 8.8

  Tensile Strength (f_{uf}) = 830 MPa

  Yield Stress (f_{yf}) = 640 MPa

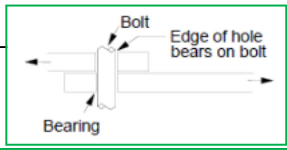
1.3 Tightening

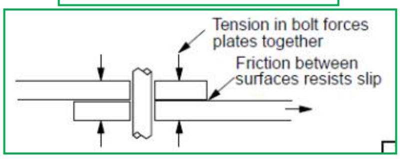
- **Snug Tight** = hand-tightened for bearing-type connections

  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

Comparison: Grades 4.6 and 8.8 bolts

	Grade 4.6	Grade 8.8/S	Grade 8.8/TB	Grade 8.8/TF
Strength	Lower	Higher (about 2 times of Grade 4.6)		
Cost	Lower	Higher (about 30% higher than Grade 4.6)		
Joint type	Flexible	Flexible	Rigid	Rigid
Installation requirement	Snug tight	Snug tight	Full tensioning	Full tensioning
Special requirement				Slip of joint prevented; e.g. under cyclic loading; higher cost for surface preparation

- 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

CIV2225 – Design of Steel & Timber Structures (Part 2)

Contents

Timber Properties

- Comparison w/ Steel
- Types of Wood
- Sizes (width, thickness, length)
- Strength groups
- Structural Grades & Stress Grades
- Design Capacity

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, bcas 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
∴ ↑ dimensional stability of product
i.e timber at 15% is in equilibrium w/ environment & can predict its behaviour in service

- Cells in trees are like pipes, the moisture in cell walls cause swelling
- These cells form the grain, where:
  Push Parallel to grain gives strength
  Push perpendicular to grain splits cells

- **Seasoning Processes (drying)**
  Air seasoning = ambient air circulates around timber to remove moisture
  Inexpensive, very slow and requires large storage space
  Kiln seasoning = Energy given for rapid drying by circulating heated air in a furnance
  Fast, costly but can dry to 12% moisture
  Solar Kiln seasoning = Controlled air movement & temp, offering faster drying than
  air & cheaper than kiln seasoning

1.3 Types of Wood

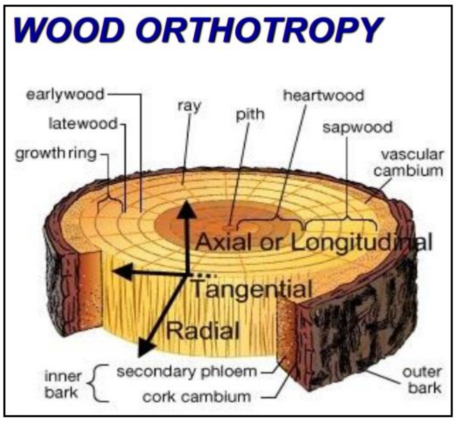
Hardwoods	Softwoods
Broad leaf, high density, dark colour, larger heartwood bands i.e Oaks, spotted gum	Needle like leaves, lower density, light colour, large sapwoodband i.e Pines, cedars

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

E.g.

F-Grade

TABLE H2.1

CHARACTERISTIC VALUES FOR DESIGN—F-GRADES—BENDING AND SHEAR FOR BEAMS, TENSION, COMPRESSION AND ELASTIC MODULI PARALLEL TO GRAIN

Stress grade	Characteristic values, MPa							
	Bending	Tension parallel to grain		Shear in beam	Compression parallel to grain	Short duration average modulus of elasticity* parallel to the grain, MPa	Short duration average modulus of rigidity, MPa
		Hardwood	Softwood				
	(f'_b)	(f'_t)		(f'_s)	(f'_c)	(E)	(G)
F34	84	51	42	6.1	63	21 500	1 430
F27	67	42	34	5.1	51	18 500	1 230
F22	55	34	29	4.2	42	16 000	1 070
F17	42	25	22	3.6	34	14 000	930
F14	36	22	19	3.3	27	12 000	800
F11	31	18	15	2.8	22	10 500	700
F8	22	13	12	2.2	18	9 100	610
F7	18	11	8.9	1.9	13	7 900	530
F5	14	9	7.3	1.6	11	6 900	460
F4	12	7	5.8	1.3	8.6	6 100	410

TABLE H2.2

CHARACTERISTIC VALUES FOR DESIGN RELATED TO STRENGTH GROUP

For strength group, see Table H2.3 and H2.4

F-graded timber

Strength Group		Characteristic values, MPa			
		Bearing		Shear at joint details	perpendicular to grain
Unseasoned	Seasoned	Perpendicular to grain (f'_p)	Parallel to grain (f'_l)	(f'_{sj})	(f'_{tp})
	SD1	26	76	10	0.8
	SD2	23	67	8.4	0.8
	SD3	19	59	7.3	0.6
	SD4	17	51	6.1	0.6
	SD5	13	40	5.4	0.5
	SD6	10	30	4.2	0.5
	SD7	8.6	23	3.8	0.4
	SD8	6.8	20	3.3	0.4
S1		17	51	6.1	0.8
S2		13	40	5.4	0.8
S3		10	30	4.2	0.6
S4		8.6	23	3.8	0.6
S5		6.8	20	3.3	0.5
S6		5.5	17	2.8	0.5
S7		4.4	13	2.2	0.4

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MGP10-15 & A17

TABLE H3.1

CHARACTERISTIC VALUES FOR DESIGN—MGP10, MGP12, MGP15 & A17 STRESS GRADES

Stress grade	Section size		Characteristic values, MPa											Design density	Joint group
	Depth	Breadth	Bending	Tension parallel to grain	Compression parallel to grain	Shear in beams	Average modulus of elasticity (see Note1) parallel to grain	Average modulus of rigidity	Bearing		Shear at joint details	Tension perpendicular to grain		
									Perpendicular to grain	Parallel to grain				
	mm	mm	(f'_b)	(f'_t)	(f'_c)	(f'_s)	(E)	(G)	(f'_p)	(f'_l)	(f'_{sj})	(f'_{tp})	(kg/m³)	
MGP 10	70 to 140	35 and 45	17	7.7	18	2.6	10 000	670	10	30	4.2	0.5	500	JD5 (see Note 2)
	190		16	7.1	18	2.5								
	240		15	6.6	17	2.4								
	290		14	6.1	16	2.3								
MGP 12	70 to 140	35 and 45	28	12	24	3.5	12 700	850	10	30	4.2	0.5	540	JD4
	190		25	12	23	3.3								
	240		24	11	22	3.2								
	290		22	9.9	22	3.1								
MGP 15	70 to 140	35 and 45	39	18	30	4.3	15 200	1 010	10	30	4.2	0.5	570	JD4
	190		36	17	29	4.1								
	240		33	16	28	4.0								
	290		31	14	27	3.8								
A17	70 to 120	35	45	26	40	5.1	16 000	930	17	50	6.0	0.6	650	JD3
		45	40	24	35	4.5								
	140, 190	35	45	24	35	4.5								
		45	40	21	32	4.0								
	240, 290	35	40	18	27	3.6								
		45	40	17	25	3.3								

Timber Beam Strength

- Bending Capacity
- Shear Capacity
- Bearing Capacity

1. Bending Capacity

1.1 Bending Capacity

$$M_d \geq M^*$$

M_d = Design capacity in Bending of unnotched beam (see example on this page, page 7)

M^* = Moment Action (for simply supported $M^* = \frac{w*L^2}{8}$)

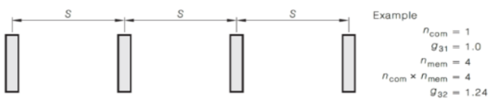
$$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f'_b\, Z$$

ϕ = Capacity reduction factor (table 2.1, see page 4)
k1 = Factor for load duration
k4 = Factor for in-service absorption/desorption of moisture by timber
k6 = Factor for temperature/humidity affect
k9 = Factor for load-sharing in grid system
k12 = Factor for stability
f'_b = Bending strength [MPa] (table H2.1 & H3.1, see page 3)
Z = Elastic section modulus

$Z_x = \frac{bd^2}{6}$ or $Z_y = \frac{db^2}{6}$

E.g.

Example 1



(b) Discrete parallel system

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

What is the maximum allowed UDL in kN/m?

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Solution

Capacity factor
$\phi = 0.95$

Material Property
$f'_b = 42$ MPa

Geometric Sectional Property
$Z = bd^2/6 = 120 \times 200^2/6 = 0.8 \times 10^6\ mm^3$

Parameters influencing the bending capacity

Floor live load (UDL)
$k_1 = 0.8$

b>100mm
$k_4 = 1.0$

Melbourne
$k_6 = 1.0$

s/L = 0.25
$k_9 = (g_{31}+g_{32})/2 = (1+1.24)/2 = 1.12$

F17, $\rho_b = 0.98$
$S_1 = 1.25 \times (200/120) \times (600/200)^{0.5} = 3.61$
$\rho_b S_1 = 0.98 \times 3.61 = 3.54 < 10$
$k_{12} = 1.0$

3.2.1 Bending strength

3.2.1.1 *Design capacity*

The design capacity in bending (M_d) of unnotched beams, for the strength limit state, shall satisfy the following:

$$M_d \geq M^* \quad \text{. . . 3.2(1)}$$

where

$$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f'_b\, Z \quad \text{. . . 3.2(2)}$$

bending capacity

$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f'_b\, Z$

$M_d = 0.95 \times 0.8 \times 1.0 \times 1.0 \times 1.12 \times 1.0 \times 42 \times 0.8 \times 10^6$ Nmm = 28.6 kNm

Action
$M^* = wL^2/8 \leq M_d$

Maximum allowed UDL

$w < 8M_d/L^2 = 8 \times 28.6 / 9 = 25.4$ kN/m

1.2 Bending Factors

k1 = Load duration factor (Table G1)

TABLE G1 k_1

LOAD DURATION FACTORS FOR TYPICAL LOAD COMBINATIONS FOR STRENGTH LIMIT STATE

Type of load (action)	AS/NZS 1170.0 specified load combination*	Load duration factor	
		Solid timber	Joints
Permanent action (dead load)	1.35 G	0.57	0.57
Permanent and short term imposed actions			
(a) Roof live load—Distributed	1.2 G + 1.5 Q	0.94	0.77
(b) Roof live load—Concentrated		0.97	0.86
(c) Floor live loads— Distributed		0.80	0.69
(d) Floor live loads— Concentrated		0.94	0.77
Permanent and long-term† imposed action	1.2 $G + 1.5\ \psi_l Q$	0.57	0.57
Permanent, wind and imposed action	1.2 $G + W_u + \psi_c Q$	1.00	1.14
Permanent and wind action reversal	0.9 $G + W_u$	1.00	1.14
Permanent, earthquake and imposed action	$G + E_u + \psi_c Q$	1.00	1.14
Fire	$G + \psi_c\ Q$	0.94	0.77

* The notation used in this Table is drawn from AS/NZS 1170.0.

† Long-term in this context is the terminology in AS/NZS 1170.0 for the quasi-permanent component of imposed action.

k4 = Partial Seasoning Factor (Table 2.5)

If seasoned/unseasoned, k4 = 1

If partial seasoned = Use table 2.5

TABLE 2.5

PARTIAL SEASONING FACTOR (k_4)

Least dimension of member	38 mm or less	50 mm	75 mm	100 mm or more
Value of k_4	1.15	1.10	1.05	1.00

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

$$k_9 = g_{31} + (g_{32} - g_{31})\left[1 - \frac{2s}{L}\right], \text{ but not less than } 1.0$$

L = length of beam

s = spacing of centres

g_{31} = geometric factor for no. of members (n_{com}) in combined parallel system (from table 2.7)

g_{32} = 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

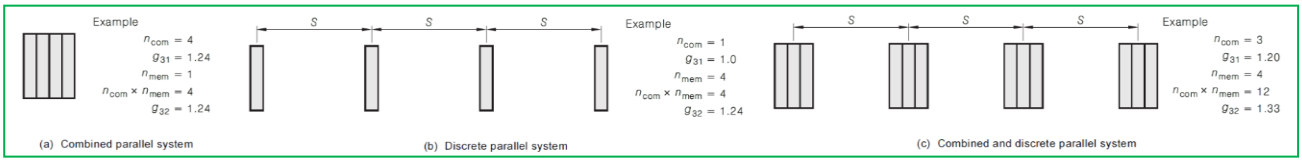
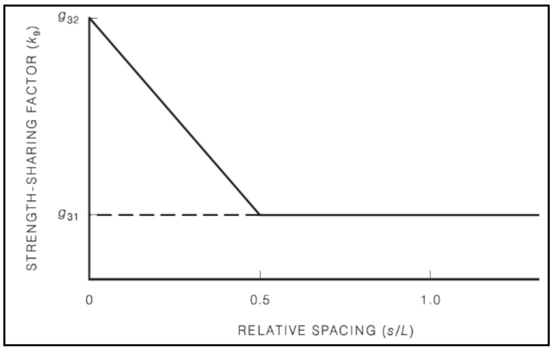


TABLE 2.7

GEOMETRIC FACTORS FOR PARALLEL SYSTEMS

Number of members in combined parallel (n_{com})	g_{31}	Total number of members in parallel system ($n_{com} \times n_{mem}$)	g_{32}
1	1.00	1	1.00
2	1.14	2	1.14
3	1.20	3	1.20
4	1.24	4	1.24
5	1.26	5	1.26
6	1.28	6	1.28
7	1.30	7	1.30
8	1.31	8	1.31
9	1.32	9	1.32
10 or more	1.33	10 or more	1.33



Note: k9 cannot be greater than g32 or less than g31

k12 = Stability factor (lateral torsional buckling)

- k12 is a function of Material Constant (ρ_b) & Slenderness (S)
  k12 < 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 tendancy to buckle & go out-of-plane

- **Material Constant (ρ_b table 3.1) allows for:**
  Initial curvature of member
  Inelasticity of timber (creep buckling)

TABLE 3.1

MATERIAL CONSTANT (ρ_b) FOR SAWN TIMBER BEAMS

Stress grade	Material constant (ρ_b)	
	Seasoned timber	Unseasoned timber
F34	1.12	1.21
F27	1.08	1.17
F22	1.05	1.15
F17	0.98	1.08
F14	0.98	1.08
F11	0.98	1.07
F8	0.89	0.99
F7	0.86	0.96
F5	0.82	0.91
F4	0.80	0.90
MGP 15	0.91	—
MGP 12	0.85	—
MGP 10	0.75	—
A17	0.95	—

(a) For $\rho_b S_1 \leq 10$—

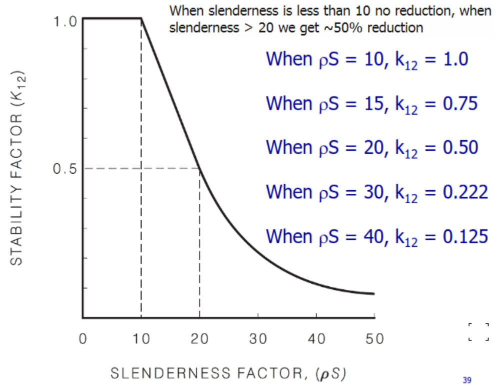
$$k_{12} = 1.0$$

(b) For $10 \leq \rho_b S_1 \leq 20$—

$$k_{12} = 1.5 - 0.05\,\rho_b S_1$$

(c) For $\rho_b S_1 \geq 20$—

$$k_{12} = \frac{200}{(\rho_b S_1)^2}$$



- **Slenderness (S_1)**

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

$$S_1 = 1.25\frac{d}{b}\left(\frac{L_{ay}}{d}\right)^{0.5}$$

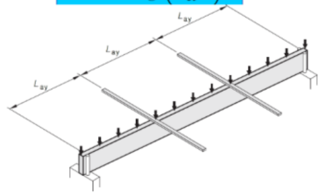


FIGURE 3.2 DISCRETE RESTRAINTS TO THE COMPRESSION EDGE

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

$$S_1 = \left(\frac{d}{b}\right)^{1.35}\left(\frac{L_{ay}}{d}\right)^{0.25}$$

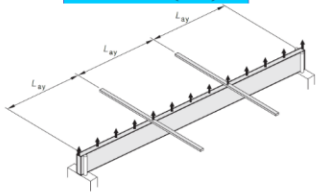


FIGURE 3.3 DISCRETE RESTRAINTS TO THE TENSION EDGE

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

$$S_1 = 0$$

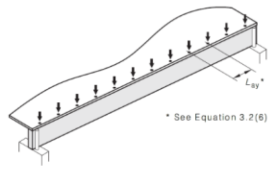


FIGURE 3.4 CONTINUOUS RESTRAINT ALONG THE COMPRESSION EDGE

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

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

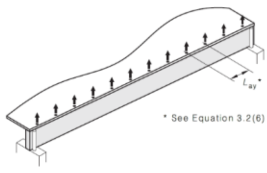


FIGURE 3.5 CONTINUOUS RESTRAINT ALONG THE TENSION EDGE

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

$$S_1 = \frac{1.5\,d/b}{\left[\left(\frac{\pi d}{L_{a\phi}}\right)^2 + 0.4\right]}$$

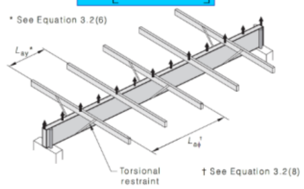


FIGURE 3.6 CONTINUOUS RESTRAINT ALONG THE TENSION EDGE COMBINED WITH DISCRETE TORSIONAL RESTRAINTS

1. Shear Capacity

2.1 Shear Capacity

$$V_d \geq V^*$$

V_d = Design capacity in Bending of unnotched beam (see example on page 10)

V^* = Shear Action (for simply supported case $V^* = \frac{w*L}{2}$)

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

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

k1 = Factor for load duration (see page 8)

k4 = Factor for in-service absorption/desorption of moisture by timber (see page 8)

k6 = Factor for temperature/humidity affect (see page 8)

f'_s = Shear strength [MPa] (table H2.1 & H3.1, see page 3)

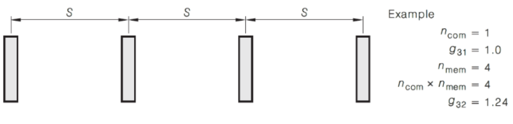
Note: Shear strength is small bcas timber grains are weak in shear (shear splits cells in grain)

A_s = Shear plane area (temperature/humidity affects will affect plane area)

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

E.g.

Example 1



(b) Discrete parallel system

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

What is the maximum allowed UDL in kN/m?

40

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 E.g. 1

Shear force
$V^* = wL/2 = 25.4 \times 3 / 2 = 38.1$ kN

$f'_s = 3.6$ MPa bcas grade = F17

$A_s = (2/3)(bd) = (2/3) \times (120 \times 200) = 16000$

Shear capacity — phy = 0.95 bcas told category 1 & F17 grade

$V_d = \phi k_1 k_4 k_6 f'_s A_s$ — k1, k4, k6 factors are taken from example 1; k1 due to floor live load, k1 = 0.8

$V_d = 0.95 \times 0.8 \times 1.0 \times 1.0 \times 3.6 \times 16000$ N = 43.4 kN > V^* OK