INTRODUCTION
TO
STRUCTURAL STEEL

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Preface

0.1 Standards

This book refers to the following standards.

- AS/NZS 1170.0:2002 Structural design actions
- NZS 3404.1&2:1997 Steel structures standard
- AS/NZS 3679.1&2:2016 Structural steel
- AS/NZS 3678:2016 Structural steel — Hot-rolled plates, floorplates and slabs
- AS/NZS 1163:2016 Cold–formed structural steel hollow sections
- AS/NZS 4855:2007 Welding consumables — Covered electrodes for manual metal arc welding of non-alloy and fine grain steels — Classification
- AS/NZS 1252.1:2016 High–strength steel fastener assemblies for structural engineering — Bolts, nuts and washers
- ANSI/AISC 360-16 Specification for Structural Steel Buildings

Due to various factors, this book is not aimed to cover everything in structural steel design. It is in general a good practice to have additional references at hand. Readers shall frequently refer to those standards for additional figures, tables, equations, etc.

0.2 Additional Resources

A number of distributors provide section properties. The most comprehensive ones are given here. Liberty Steel provides the manual for specifications of hot rolled structural steel sections manufactured in accordance with AS/NZS 3679.1&2:2016 Structural steel.

- Liberty Steel Hot Rolled and Structural Steel Product
  This book is attached at the end of this document.

Fletcher Easysteel provides design charts of some section types.

- Structural Steel Properties & Design Charts Book

AustubeMills provides design tables for hollow sections.

- Design Capacity Tables for Structural Steel Hollow Sections[1]

The Australian Steel Institute provides design capacity tables for structural steel (Australian Steel Institute, 2016).

- DESIGN CAPACITY TABLES FOR STRUCTURAL STEEL, VOL. 1: OPEN SECTIONS

Readers shall frequently refer to those documents for properties that could be used in design.

The following YouTube channels provide engineering related contents of good quality.

- The Efficient Engineer[2]

0.3 To Use This Book

If you are reading the digital copy of this book, you can check the additional links and worksheets provided.

Links coloured like this Gregory MacRae are made available in the document. These are not necessary for the course, but they may be of interest to readers. Readers are welcome to report any invalid links in this book.

Worksheets prepared in SMath Studio[3] for some examples are provided in a digital folder named as 'WORKSHEET'. Readers may change parameters within the example if they wish. It must be noted that

- Assignments require handwritten submissions, but worksheets can be used to check the hand calculation.
- The worksheets have been configured for the particular examples only. If readers wish to use the worksheets for actual design, they need to ensure that all relevant code clauses are satisfied.

[1] https://www.austubemills.com.au/en-au/resource-centre/resources/design-capacity-tables-for-structural-steel-hollow/
[2] https://www.youtube.com/channel/UCXAS_Ekkq0iFJ9dSUIkcAkw
[3] https://en.smath.com/view/SMathStudio/summary

Readers need to install it so that by clicking the 'Worksheet' link, the corresponding worksheet can be automatically opened. Not all PDF readers support such a functionality. The feature–rich PDF-XChange Viewer[4] is recommended. Please use the following one to check if the current one can work properly.

Example 0.1 *Worksheet ← Click this link to test.*

0.4 Additional Resources

The digital version of this book, along with other relevant documents, diagrams and worksheets, is available online via `https://github.com/TLCFEM/introduction-to-structural-steel`.

[4] `https://www.tracker-software.com/product/pdf-xchange-viewer`

Part I

Design

1.1 Design Process

The typical design process is shown in Fig. 1.1.

1.2 Design Approaches

1.2.1 Design for Strength

The ultimate requirement can be summarised by the following inequality:

$$\begin{array}{c}\textbf{FORCE DEMAND}\\\textbf{(ACTIONS, LOADS)}\end{array} \leqslant \begin{array}{c}\textbf{STRENGTH}\\\textbf{(CAPACITY, RESISTANCE)}\end{array} \tag{1.1}$$

There are two common design methods for this strength requirement.

ASD

It stands for **Working Stress (or Allowable Stress) Design (ASD)** (e.g., NZ Steel Codes pre-1984).

$$\sum \begin{array}{c}\textbf{WORKING}\\\textbf{FORCES}\end{array} \leqslant \begin{array}{c}\textbf{PERMISSIBLE}\\\textbf{STRENGTH}\end{array},$$

$$\sum E_i \leqslant \frac{\text{Ultimate Strength}}{\text{Safety Factor}} = \frac{R}{\text{SF}}.$$

The safety factor, SF is approximately equal to 1.7 and it allows for variation in loading and material strength. ASD was popular in NZ until the mid 1980s. In the USA, and some other countries, it is still used, especially for timber, and for some steel structures. It assumes that all types of loading have the same weighting. It cannot handle unusual loads well.

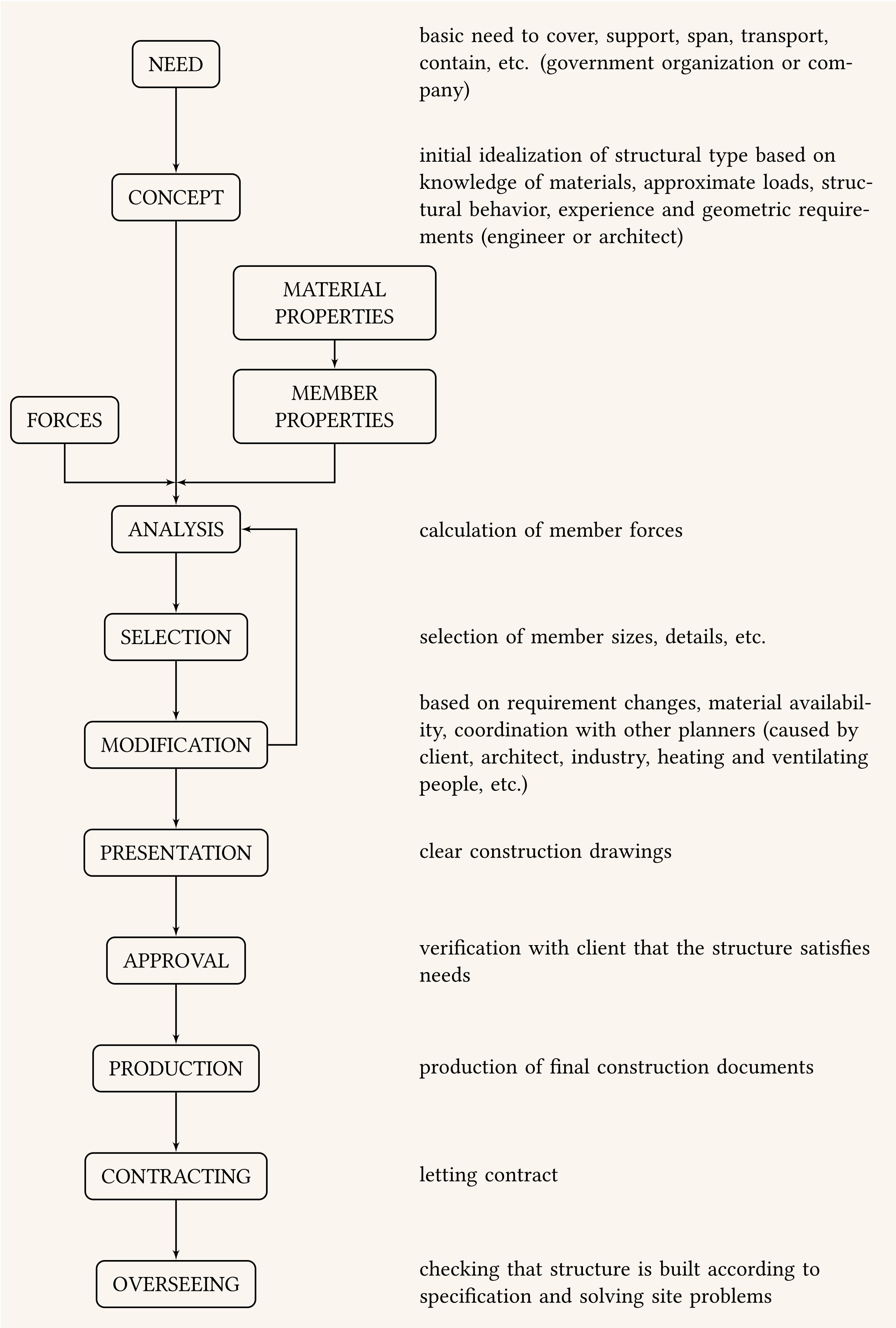


Figure 1.1: Typical design process

LSD, LRFD

It stands for **Limit State Design, a.k.a., Load and Resistance Factor Design (LRFD)** (e.g., many NZ Steel Codes post-1984).

A structure should be designed in such a way that it does not reach a **critical limit state**.

structural state $\leqslant$ limit state

A critical limit state for a structure may be governed by the following aspects:

- safety
  - strength
  - stability
- serviceability
  - deformation
  - vibration

NZ limit state design codes are written in a LRFD format recognizing the probabilistic variations in both applied forces and capacities.

The many advantages of LRFD are well–expressed by Beedle, whose listing is the basis of the following:

1. LRFD is another 'tool' for structural engineers to use in steel design. Why not have the same tools (variable overload factors and resistance factors) available for steel design as are available for concrete design?
2. Adoption of LRFD is not mandatory but provides a flexibility of options to the designer. The marketplace will dictate whether or not LRFD will become the sole method.
3. ASD is an approximate way to account for what LRFD does in a more rational way. The use of plastic design concepts in ASD has made ASD such that it no longer may be called an 'elastic design' method.
4. The rationality of LRFD has always been attractive, and becomes an incentive permitting the better and more economical use of material for some load combinations and structural configurations. It will also likely lead to having safer structures in view of the arbitrary practice under ASD of combining dead and live loads and treating them the same.
5. Using multiple load factor combinations should lead to economy.
6. LRFD will facilitate the input of new information on loads and load variations as such information becomes available. Considerable knowledge of the resistance of steel structures is available. On the other hand our knowledge of loads and their variation is much less. Separating the loading from the resistance allows one to be changed without the other if that should be desired.
7. Changes in overload factors and resistance factors ϕ are much easier to make than to change the allowable stress in ASD.

8. LRFD makes design in all materials more compatible. The variability of loads is actually unrelated to the material used in the design. Future specifications not in the limit states format for any material will put that material at a disadvantage in design.

9. LRFD provides the framework to handle unusual loads that may not be covered by the specification. The design may have uncertainty relating to the resistance of the structure, in which case the resistance factors may be modified. On the other hand, the uncertainty may relate to the loads and different overload factors may be used.

10. Future adjustments in the calibration of the method can be made without much complication. Calibration for LRFD was done for an *average* situation but might be adjusted in the future.

11. Economy is likely to result for low live load to dead load ratios. For high live load to dead load ratios there will be diseconomy but a low amount.

12. Safer structures may result under LRFD because the method should lead to a better awareness of structural behaviour.

LRFD equation has the following typical form:

$$\sum \gamma_i E_i \leqslant \phi R_n, \tag{1.2}$$

$$\textbf{Forces} \leqslant \textbf{Resistance}, \tag{1.3}$$

where

$$\begin{aligned} \gamma_i &= \text{load factor for specific action } E_i \\ E_i &= \text{load/action due to specific loading condition } i \text{ (e.g., wind, live load)} \\ \gamma_i E_i &= \text{factored load for specific loading type } i \\ \sum \gamma_i E_i &= \text{total factored loads} \\ \phi &= \text{resistance factor or strength reduction factor} \\ R_n &= \text{nominal resistance } \textbf{or} \text{ ideal resistance against a particular limit state (e.g., yield, fracture)} \\ \phi R_n &= \text{dependable resistance} \end{aligned}$$

The factor ϕ reflects the likely variation in

- material stress–strain characteristics,
- cross–section properties,
- structural deterioration due to corrosion or fatigue,
- consequences of reaching limit state.

For example, for steel members, $\phi = 0.9$ for tension, compression bending, shear and combined actions. The value of ϕ is different for bolts, pins or welded connections (NZS 3404.1&2:1997 Table 3.3(1)).

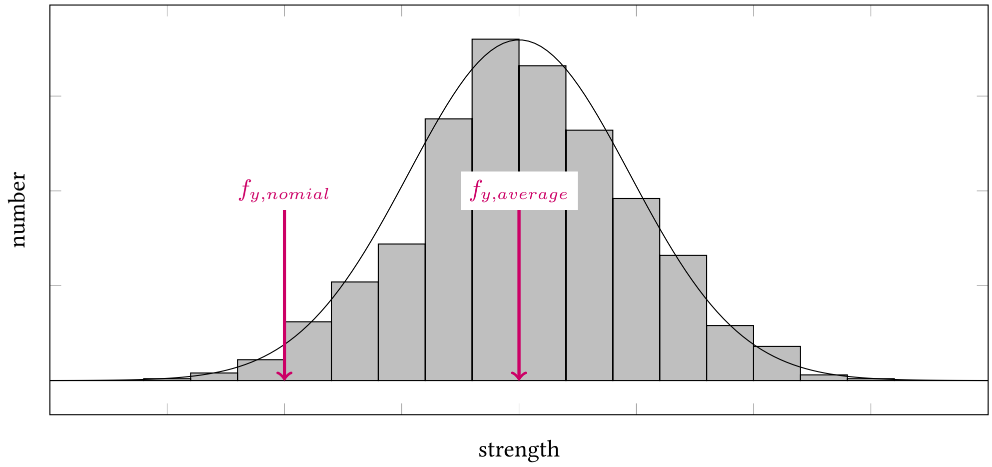


Figure 1.2: Univariate Gaussian distribution

The factor γ_i allows for

- possibility of overload,
- accuracy of analysis,
- load duration.

It has values of up to 1.5.

Factors γ_i and ϕ are chosen to have a low and uniform probability of failure for all loads.

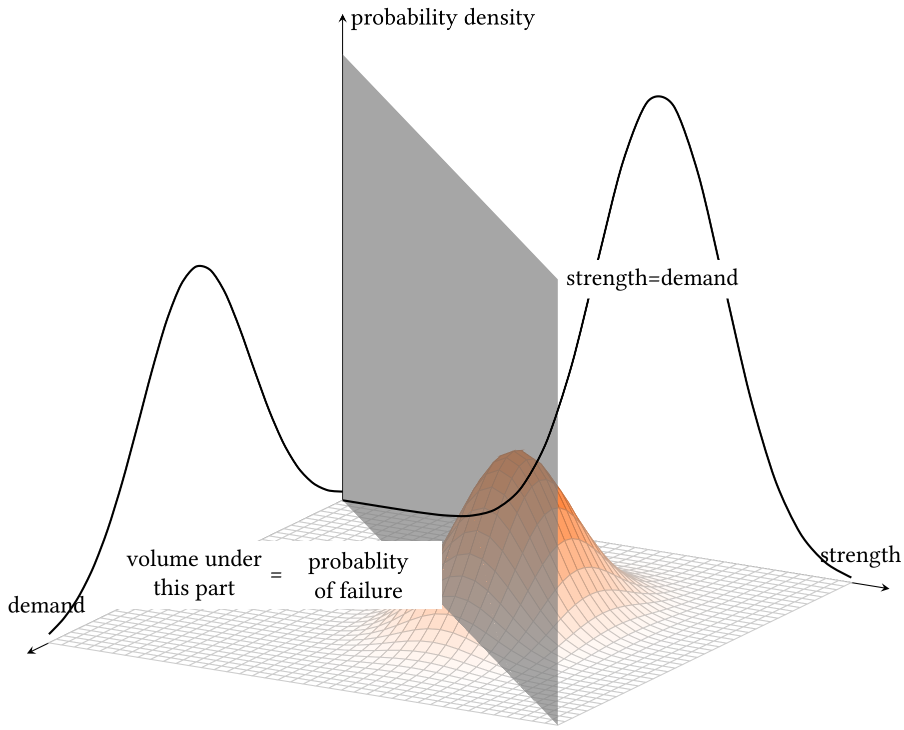


Figure 1.3: Bivariate Gaussian distribution

The basic combinations for the ultimate limit states ($\sum \gamma_i E_i$) used in checking strength shall be as follows (AS/NZS 1170.0:2002 § 4.2.2):

- $1.35G$ — permanent action only
- $1.2G + 1.5Q$ — permanent and imposed action
- $1.2G + 1.5\psi_l Q$ — permanent and long-term imposed action
- $1.2G + W_u + \psi_c Q$ — permanent, wind and imposed action
- $0.9G + W_u$ — permanent and wind action reversal
- $G + E_u + \psi_E Q$ — permanent, earthquake and imposed action
- $1.2G + S_u + \psi_c Q$ — permanent action, actions given in AS/NZS 1170.0:2002 § 4.2.3 and imposed action

The maximum combination will govern the design. The live load factor ψ depends on the load duration as shown below (AS/NZS 1170.0:2002 Table 4.1).

Table 1.1: Short-term ψ_s, long-term ψ_l, combination ψ_c and earthquake ψ_E factors

	ψ_s	ψ_l	ψ_c	ψ_E
distributed imposed action, Q				
floors				
residential and domestic	0.7	0.4	0.4	0.3
offices	0.7	0.4	0.4	0.3
parking	0.7	0.4	0.4	0.3
retail	0.7	0.4	0.4	0.3
storage	1.0	0.6	0.6	0.6
other	1.0	0.6	0.6	0.6
roofs				
roofs used for floor type activities	0.7	0.4	0.4	0.3
all other roofs	0.7	0.0	0.0	0.0
concentrated imposed actions (including balusrades), Q				
floors	1.0	0.6	as of	0.3
floors of domenstic housing	1.0	0.4	distributed	0.3
roofs used for floor type activities	1.0	0.6	floor actions	0.3
all other roofs	1.0	0.0	0.0	0.0
balustrades	1.0	0.0	0.0	0.0
long-term installed machinery, tare weight	1.0	1.0	1.2	1.0

1.2.2 Design for Serviceability

Serviceability checks are **always** carried out under service (or working) loads, ($\sum E_i$) using a combination of: G, $\psi_s Q$, $\psi_l Q$, W_s, E_s and other appropriate values.

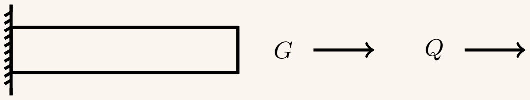
The methods are the same for Allowable Stress Design (ASD) and Limit State Design (LRFD). Suggested serviceability limit state criteria are shown in Table 1.2. For relevant notes, readers shall refer to AS/NZS 1170.0:2002 Table C1.

Table 1.2: Suggested serviceability limit state criteria

Element	Phenomenon controlled	Serviceability parameter	Applied action	Element response (see Notes 1 and 2)
Roof cladding				
Metal roof cladding	Indentation	Residual deformation	$Q = 1\,\mathrm{kN}$	Span/600 but $< 0.5\,\mathrm{mm}$
	De-coupling	Mid-span deflection	$G + \psi_s Q$	Span/120
Concrete or ceramic roof cladding	Cracking	Mid-span deflection	$G + \psi_s Q$	Span/400
Roof–supporting elements				
Roof members (trusses, rafters, etc.)	Sag	Mid-span deflection	$G + \psi_l Q$	Span/300
Roof elements supporting brittle claddings	Cracking	Mid-span deflection	$G + \psi_s Q$ or W_s	Span/400
Ceiling and ceiling supports				
Ceilings with matt or gloss paint finish	Ripple	Mid-span deflection	G	Span/500 (seet Note 3)
Ceilings with textured finish	Ripple	Mid-span deflection	G	Span/300
Suspended ceilings	Ripple	Mid-span deflection	G	Span/360
Ceiling support framing	Sag	Mid-span deflection	G	Span/360
Ceilings with plaster finish	Cracking	Mid-span deflection	$G + \psi_s Q$ or W_s	Span/200
Wall elements				
Columns	Side sway	Deflection at top	W_s	Height/500
Portal frames (frame racking action)	Roof damage	Deflection at top	W_s or E_s	Spacing/200 (see Note 4)
Lintel beams (vertical sag)	Doors/windows jam	Mid-span deflection	W_s	Span/240 but $< 12\,\mathrm{mm}$ (see Note 5)
Walls — General (face loaded)	Discerned movement	Mid-height deflection	W_s	Height/150
	Impact: soft body (neighbours notice)	Mid-height deflection	$Q = 0.7\,\mathrm{kN}$	Height/200 but $< 12\,\mathrm{mm}$ (see Note 6)
	Supported elements rattle	Mid-height deflection	W_s	Height/1000
Walls — Specific claddings (see Note 7):				
Brittle cladding (ceramic) face loaded	Cracking	Mid-height deflection	W_s	Height/500
Masonry walls (in plane)	Noticeable cracking	Deflection at top	W_s or E_s	Height/600
Masonry walls (face loading)	Noticeable cracking	Deflection at top	W_s or E_s	Height/400
Plaster/gypsum walls (in plane)	Lining damage	Mid-height deflection	W_s	Height/300
Plaster/gypsum walls (face loading)	Lining damage	Mid-height deflection	W_s or E_s	Height/200
Movable partitions (soft body impact)	System damage	Deflection at top	$Q = 0.7\,\mathrm{kN}$	Height/160
Glazing systems	Bowing	Mid-span deflection	W_s	Span/400
Windows, facades, curtain walls	Facade damage	Mid-span deflection	W_s or E_s	Span/250
Fixed glazing systems	Glass damage	Deflection	W_s or E_s	2×glass clearance (see Note 3)
Floors and floor supports				
Beams where line-of-sight is along invert	Sag	Mid-span deflection	$G + \psi_l Q$	Span/500 (seet Notes 8 and 9)
Beams where line-of-sight is across soffit	Sag	Mid-span deflection	$G + \psi_l Q$	Span/250
Flooring	Ripple	Mid-span deflection	$G + \psi_l Q$	Span/300
Floor joists/beams	Sag	Mid-span deflection	$G + \psi_l Q$	Span/300
Floors	Vibration	Static mid-span deflection	$Q = 1\,\mathrm{kN}$	$< 1\,\mathrm{mm}$ to $2\,\mathrm{mm}$ (see Note 10)
Normal floor systems	Noticeable sag	Mid-span deflection	$G + \psi_l Q$	Span/400
Specialist floor systems	Noticeable sag	Mid-span deflection	$G + \psi_l Q$	Span/600
Floors — Side-sway (acceleration)	Sway	Acceleration at floor	$W_s(P = 5)$	$< 0.01g$ (see Note 11)
Floors — Supporting masonry walls	Wall cracking	Mid-span deflection	$G + \psi_l Q$	Span/500
Floors — Supporting plaster lined walls	Cracks in lining	Mid-span deflection	$G + \psi_l Q$	Span/300
Floors supporting existing masonry walls — Underpinning floors	Wall cracking	Mid-span deflection	$G + \psi_l Q$	Span/750
Floors — For access for working by operators and maintenance	Sag	Mid-span deflection	$Q = 1\,\mathrm{kN}$	Span/250
Handrails — Post and rail system	Side sway	Mid-span system deflection	$Q = 1.5\,\mathrm{kN\,m^{-1}}$	Height/60+Span/240

Example 1.1 *Worksheet Rod Under Tension*

Assume the safety factor is $\mathrm{SF} = 1.7$*, a rod in tension is required to carry dead and live loads of* $G = 25\,\mathrm{kN}$ *and* $Q = 30\,\mathrm{kN}$ *respectively. The steel yield stress is* $f_y = 300\,\mathrm{MPa}$*. What diameter* d *should it have?*



Solution 1.1

The resistance against axial tension yield is

$$R = A_g f_y = \frac{\pi d^2}{4} f_y.$$

- **ASD**
  Allowable force: $R/\mathrm{SF} = A_g f_y/1.7$. Working forces: $\sum E_i = G + Q = 55\,\mathrm{kN}$.
  ASD requires:

$$\begin{aligned} \sum E_i &\leqslant R/\mathrm{SF} \\ 55\,\mathrm{kN} &\leqslant \frac{\pi d^2}{4 \times 1.7} f_y \\ d &\geqslant \sqrt{\frac{55\,\mathrm{kN} \times 4 \times 1.7}{300\,\mathrm{MPa} \times \pi}} = 19.9\,\mathrm{mm} \end{aligned}$$

- **LRFD**
  LRFD requires:

$$\begin{aligned} \gamma_G E_G + \gamma_Q E_Q &\leqslant \phi R \\ 1.2 \times 25\,\mathrm{kN} + 1.5 \times 30\,\mathrm{kN} &\leqslant 0.9 \times \frac{\pi d^2}{4} f_y \\ d &\geqslant \sqrt{\frac{75\,\mathrm{kN} \times 4}{0.9 \times 300\,\mathrm{MPa} \times \pi}} = 18.8\,\mathrm{mm} \end{aligned}$$

If rods come in 2 mm size increments, then in both cases a ϕ 20 mm rod is okay.

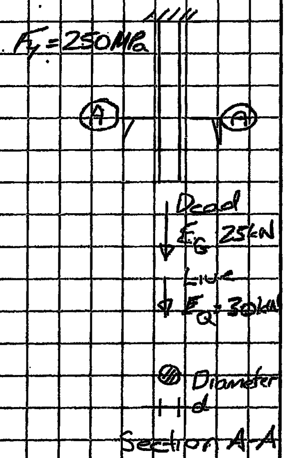
For handwritten homework, please follow the convention as shown in the example.

Problem

A rod in tension is required to carry dead and live loads of 25 kN and 30 kN respectively.

The steel yield stress is 250MPa.

What diameter should it have?



Solution

The resistance against axial tension yield, $R = A_g F_y$ ← yield stress; ← total area

$$= \frac{\pi d^2}{4} F_y$$

i) ASD

Allowable force $= R/SF$ $\quad SF = 1.70$ [Given]

$$= A_g F_y / 1.70$$

Working forces $\Sigma E_i = 55$ kN

ASD Requirement

$$\Sigma E_i < R/SF$$

$$55 \text{ kN} < 250 \text{ MPa} \times \frac{\pi d^2}{4} / 1.7$$

$$\Rightarrow d \geq \sqrt{\frac{4 \times 55{,}000\text{N} \times 1.7}{250{,}000{,}000\text{Pa} \times \pi}}$$

$$\geq 21.8 \text{ mm}$$

ii) LRFD: $\gamma_G E_G + \gamma_Q E_Q < \phi R$, $\phi = 0.90$, $\gamma_G = 1.2$, $\gamma_Q = 1.5$ [Given]

$$1.2 \times 25\text{kN} + 1.5 \times 30\text{kN} \leq 0.9 \times \pi d^2/4 \times 250\text{MPa}$$

$$\Rightarrow d \geq 20.6\text{mm}$$

If rods come in 2mm size increments, then in both cases a 22mm ϕ rod is OK.

22mm ϕ rod $F_y = 250$MPa

Figure 1.4: Sample handwritten solution

1.3 Calculation Accuracy

Estimates of likely maximum forces (or actions) and resistances are generally very approximate in design. Also, the load factors and resistance factors are not provided with great precision. For this reason, using more than three significant digits is seldom necessary and three significant digits is generally used. (Care must be taken though because in some cases, such as when similar size numbers are subtracted more accuracy is required.)

Because the computed result contains a lot of uncertainty, it may not be necessary to follow the calculation results too rigidly. However, because we may need to defend our calculations in a court of law, we must make sure that our recommendations, based on our calculations are totally defensible.

Gregory MacRae suggests the following guidelines:

- If the demand is greater than the capacity by **less than 1%**, then the difference is usually within the range of usual round-off errors. It can usually be argued that greater than nominal material strengths and strain hardening will guarantee that the demand is greater than the capacity. A note of this sort should be made in the calculations.
- If the demand is greater than the capacity by **less than 5%**, then the design should only be accepted if it can be argued that the design is satisfactory for some reason which is not considered in the calculations used in the design approach. The reason for accepting the design should be stated in the calculations, and the engineer should be able to produce more refined calculations to justify the decision if required at a later date.
- If the demand is greater than the capacity by **more than 5%**, then the design should not be accepted, or a revised set of calculations should be carried out to show that the design works.

It should be noted that different groups have different approaches to accuracy and it is prudent to check the policy of the company you are working for.

Material

2.1 Sustainability and Steel Structures

The issue of sustainability is becoming important in both NZ and worldwide policy and its consideration will affect choices made in the future.

The World Commission on Environment and Development in their 1986 report 'Our Common Future' called for: *a form of sustainable development which meet the needs of the present without compromising the needs of future generations to meet their own needs.*

Sustainability is commonly measured by the triple bottom line — social, economic, and environmental priorities. For buildings, this is important because:

- social
  - we spend about 90 % of our time in buildings,
  - buildings affect our life quality,
- economic
  - buildings affect our productivity,
  - construction is about 10 % GDP and employs many people,
- environmental
  - construction consumes many resources,
  - about one half of all energy is consumed in buildings,
  - construction and demolition generate huge amounts of waste each year.

Life cycle assessment (LCA) is a tool commonly used to assess environmental impacts of the built environment. The UK Building Research Establishment (BRE) has developed an environmental assessment methodology (EAM) for structures (BREEAM). A typical energy life cycle is given (from SSC) in Fig. 2.1.

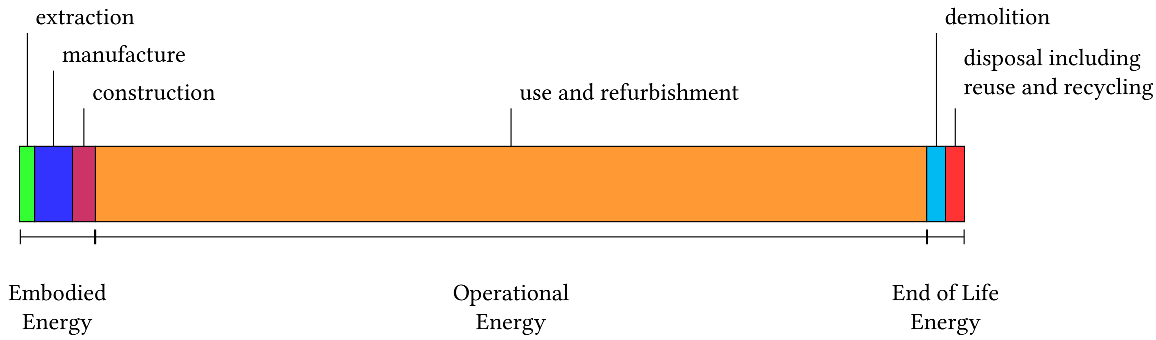


Figure 2.1: Energy life cycle for an office building over 60 years

It may be seen that the embodied energy of the structure is only a small part of the total cost and that operational energy is the most significant. Embodied energy and end-of-life energy effects are minimized in structures with long lives.

Greater energy efficiency in buildings is generally achieved by a combination of the following measures:

- reducing primary heat losses through the building envelope, and cooling loads,
- introducing energy saving measures in the operation of the building, e.g., energy efficient electrical appliances,
- installing energy creation systems, such as photovoltaic panels and combined heat and power plants
- improving natural lighting.

In the future, it is expected that structures will be built using not only one material, but a combination of materials. The REI Flagship store (Seattle)[1] is a good example of this.

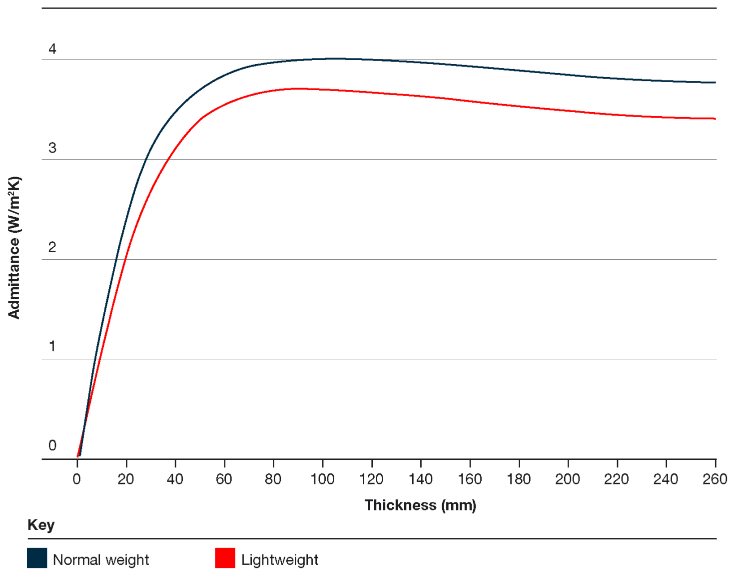


Figure 2.2: Admittance of normal and lightweight concrete remains unchanged beyond depths of 100 mm (https://www.steelconstruction.info/File:CSD126_N7.jpg)

[1] https://www.rei.com/stores/seattle.html

A typical slab in a steel building (75 mm to 100 mm thick) provides almost the maximum admittance (i.e., ability of the building element to store and release heat) in a naturally ventilated building as shown below. Adding concrete mass solely for passive 'fabric energy storage' is therefore not effective (SSC).

Steel structures are known for:

- A high structural efficiency, i.e., a small amount of material provides considerable strength & stiffness.
- 'Lean' construction, i.e., highly planned construction with little waste.
- Flexibility and adaptability. For example, long spans mean that internal configurations may easily be changed without any major work; extensions are also relatively easy; web openings or cellular beams allow easy service integration. This extends the building life and allows greater value to be extracted from the resources invested in it.
- Durability and Maintenance. Steel components are durable and require little maintenance in a well designed environment.
- Recyclability. Steel is almost 100% recyclable so it does not fill the waste dumps reducing end of life impacts. Much is currently recycled.

A 2006 BRANZ study indicated that whole of life costs for steel, timber and concrete options for two particular government buildings were within 5 % of each other. The study mentions that some recycling advantages of steel and concrete buildings, which may affect the final decision about what type of building should be used, were not included.

2.2 Structural Steel

Structural steel is **an alloy of icon and carbon plus small amount of other elements, e.g., silicon (Si), manganese (Mn), magnesium (Mg), copper (Cu).** Check this video: Understanding Metals[2].

Common types include:

- **carbon steels** — typical $f_y = 200\,\mathrm{MPa}$ to $350\,\mathrm{MPa}$
  The name comes from the reality that they contain a very small amount of other alloying elements. More carbon causes
  - high strength
  - lower ductility
  - less weldability
- **high strength low alloy (HSLA) steels** — typical $f_y = 280\,\mathrm{MPa}$ to $500\,\mathrm{MPa}$
  They provide better mechanical properties or greater resistance to corrosion than carbon steels.
- **alloy steels** — typical $f_y = 550\,\mathrm{MPa}$ to $800\,\mathrm{MPa}$
  They are alloyed with a variety of elements in total amounts from 1 % to 50 % by weight to improve its mechanical properties.

[2] https://www.youtube.com/watch?v=PaGJwOPg2kU

- heat treated (heated to 900 °C) and cooled rapidly in water (quenched) to make a hard, strong and brittle structure), then it is reheated to 620 °C and cooled slowly (tempered) reducing strength and increasing toughness and ductility
- no distinct yield point, 0.2 % offset strain is used

2.2.1 Material Properties

The following typical values are used for material properties of structural steel.

Young's modulus (elastic modulus)	E	200 GPa
shear modulus	G	$\approx$ 80 GPa
Poisson's ratio	ν	≈ 0.3
density	ρ	$7850\,\mathrm{kg/m^3}$
coefficient of thermal expansion at 20 °C	α	$11.7 \times 10^{-6}\,°\mathrm{C}^{-1}$

2.2.2 Material Attributes

Attributes	Benefits
High Strength	long spans in bridges and buildings
Uniformity	quality is generally consistent strength does not change with loading direction
Linear Elasticity	previous cycles of elastic loading generally make no difference to behaviour stiffness may be calculated (e.g., EI, EA) creep is not generally a problem strength does not generally change with time
Ductility	can reach high strain before fracture local yielding at stress concentration causes spreading of load and avoids fracture deforms slowly over time when overloaded
Toughness	absorbs a large amount of energy
Connectability	solid connections with welds or bolts
Prefabrication Ability	can be erected fast with high quality good for fast–track construction, standardization
High Strength-to-Cost Ratio	economy
UV Resistant	does not generally become brittle with time
Recyclable	

Steel, like concrete, timber and composite materials is affected by environmental factors. Unprotected steel may corrode in a hostile environment, while concrete may have 'concrete cancer', timber may rot or be attacked by bugs, and composite materials may be UV sensitive. Because corrosion occurs on the surface of steel, it is often observed easily and treated. All of steel, concrete, timber and composite materials are affected by fatigue and fire too.

2.2.3 Typical Mechanical Behaviour

The typical mechnical response of structural steel shown in Fig. 2.3 consists of three stages: elastic range, yield plateau and strain hardening range. Since the upper yield stress is not reliable, the lower yield stress conservatively used as the design yield stress.

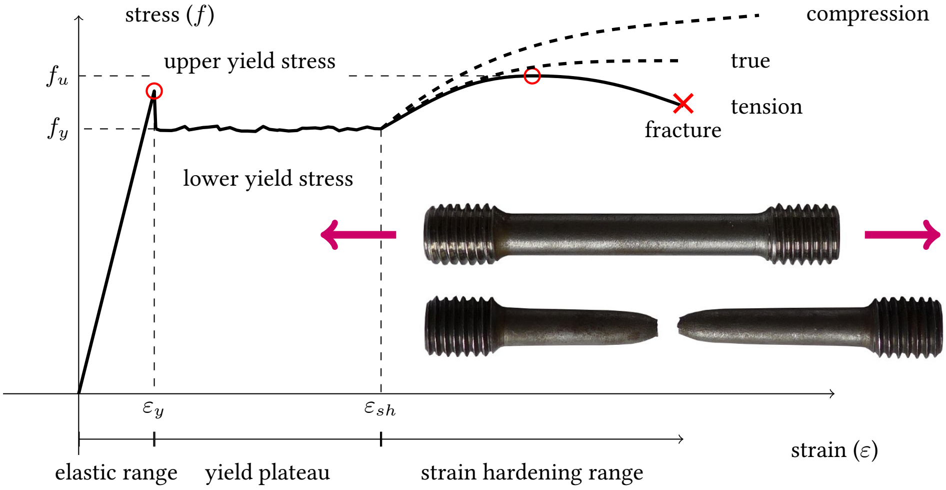


Figure 2.3: Typical strain–stress response of steel

Depending on different strain/stress measures, the response may be different. Sometimes engineers use true strain and true stress (instead of engineering strain and stress) to characterise material response. The difference could be significant when deformation is large, interested readers can check this video: Understanding True Stress and True Strain[3].

[3]https://www.youtube.com/watch?v=AkX6JqlWRqc

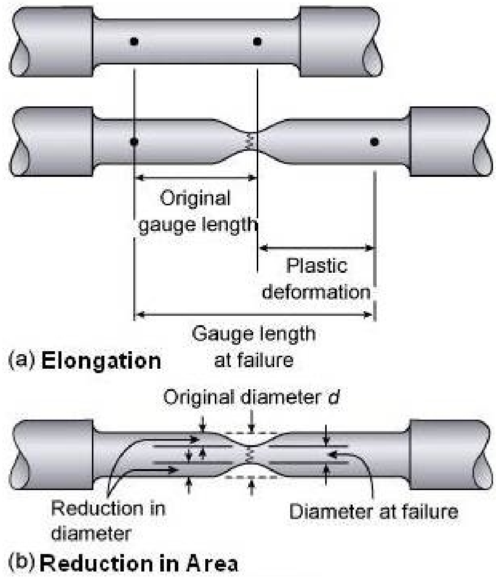


Figure 2.4: Nominal and true area

For idealised response, often the upper yield stress is ignored, resulting in the following idealised curves as shown in Fig. 2.5. For alloy steel, since there is no explicit yielding point, often it is set to the stress corresponds to $0.2\,\%$ offset strain.

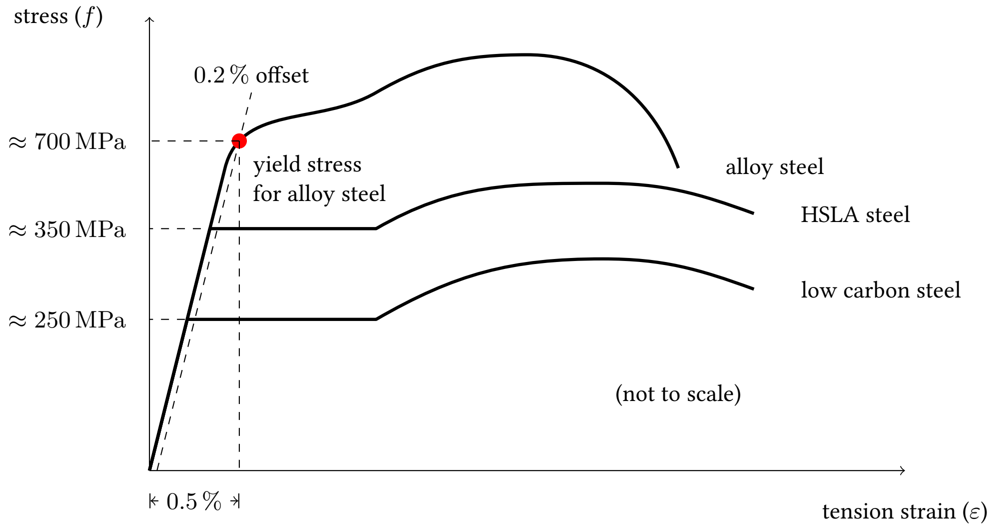


Figure 2.5: Idealised strain–stress responses of various types of steel

2.2.4 Common Grades

The most common type of Australasian structural steel grades for hot–rolled structural steel bars and sections (including universal sections, taper flange beams, angles and channels) are given below. All of these grades are weldable and come with subgrades of L0 and L15 indicating low (27 J at $0\,°C$) and high (27 J at $-15\,°C$) Charpy V-notch[4] toughnesses respectively. For a UB or UC section, coupons to determine the yield stress of a section are to be taken from the flange.

[4]https://www.youtube.com/watch?v=tpGhqQvftAo

Table 2.1: Tensile test requirements for flats and sections

grade	minimum yield stress f_y (MPa) thickness (mm) < 11	$\geqslant 11$ and $\leqslant 17$	> 17 and < 40	$\geqslant 40$	minimum tensile strength f_u MPa	minimum elongation %
300, 300L0	320	300	280	280	440	22
300L15, 300S0	320	300	280	280	440	25
350, 350L0	360	340	340	330	480	20
350S0, 350L15	360	340	340	330	480	25

Please refer to AS/NZS 3679.1&2:2016 Table 14 for notes and details.

Table 2.2: Tensile test requirements for cold–formed hollow sections

grade	minimum yield stress f_y MPa	minimum tensile strength f_u MPa
C250, C250L0	250	320
C350, C350L0	350	430
C450, C450L0	450	500

Please refer to AS/NZS 1163:2016 Table 7 for notes and details.

Table 2.3: Tensile test requirements for plate and floor plate

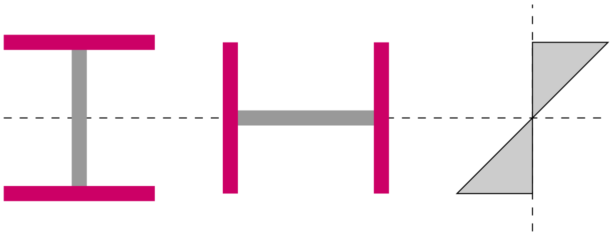
grade	minimum f_y (MPa) thickness t (mm)								minimum f_u (MPa) thickness t (mm)		
	$\leqslant 8$	> 8 $\leqslant 12$	> 12 $\leqslant 20$	> 20 $\leqslant 32$	> 32 $\leqslant 50$	> 50 $\leqslant 80$	> 80 $\leqslant 150$	> 150 $\leqslant 200$	$\leqslant 20$	> 20 $\leqslant 150$	> 150 $\leqslant 200$
200	200	200							300	300	290
250	280	260	250	250	250	240	230	220	410	410	400
300	320	310	300	280	280	270	260	250	430	430	420
350	360	360	350	340	340	340	330	320	450	450	450
400	400	400	380	360	360	360			480	480	
450	450	450	450	420	400				520	500	
WR350	340	340	340	340	340	340			450	450	

Please refer to AS/NZS 3678:2016 Table 8 for notes and details.

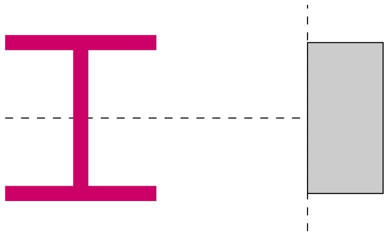
2.3 Design Yield Stress for Sections

For members of uniform thickness (such as plates, angles, etc.), the yield stress may be obtained directly for the appropriate material grade. However, a number of different member types do not have uniform thickness. This includes I-sections, C-sections, etc., where the thickness of flanges is generally greater than that of webs. While the different strengths of the different components (e.g., webs, flanges.) could be used in the calculations, this is usually too cumbersome for routine design, and the increase in accuracy is generally small. It is therefore reasonable to use the following rules:

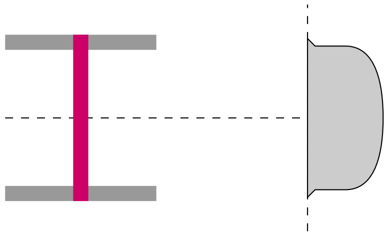
- For bending (flexural) strength (about both strong and weak axes, see sketches below), use $f_y = f_{y,flange}$ as flanges are the main contributors to resistance.



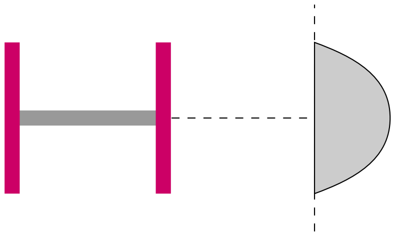
- For axial strength, use $f_y = f_{y,flange}$ as both web and flanges contribute to resistance but $f_{y,flange}$ is often smaller, which leads to conservative design.



- For shear strength associated with strong–axis bending, use $f_y = f_{y,web}$ as web is the main contributor to resistance.



- For shear strength associated with weak–axis bending, use $f_y = f_{y,flange}$ as flanges are the main contributors to resistance.



2.4 Structural Shapes

Common types include:

- hot–formed
  Steel is formed into the shape of the section while it is still hot. Many kinds of working, including rolling, forging, extrusion and drawing, can be done with hot metal.
- cold–formed
  Thin plate steel is shaped by cold–working processes carried out near room temperature, such as rolling, pressing, stamping, bending, etc.

- built–up
  Steel is formed by welding together plates to form various section shapes.

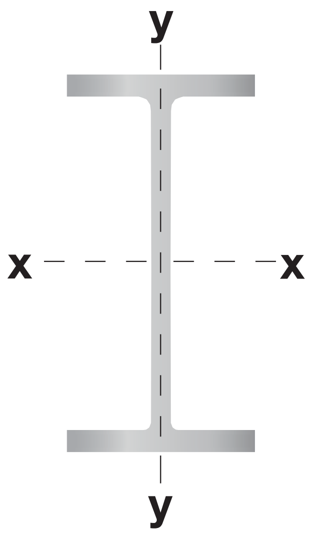
We will not use cold–formed sections in this course. For availability, readers can refer to specification manual provided by Liberty Steel.

2.4.1 Hot–Rolled Products

1. Universal Beams (UB) AS/NZS 3679.1&2:2016

   Different names may be used in other codes. For example, I-Beam — NZ/AU/UK; H-Section — Japan; WF-Beam — USA; IPE, HE, HL, HD — EU. A typical designation consists of three components.

460	**UB**	**74.6**
approx. depth (mm)	universal beam	linear density ($\mathrm{kg\,m^{-1}}$)

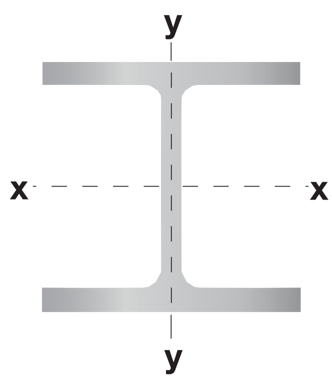
   Linear density is the mass per unit length. The depth is greater than the flange width.

   - high flexural strength to area ratio
   - high flexural stiffness to area ratio
   - high moment capacity due to high lever arm
   - flanges and webs are easy to bolt (flat)
   - thickness of flange/web selected to limit buckling

2. Universal Columns (UC) AS/NZS 3679.1&2:2016

   Similar to UB, a typical designation consists of three components.

250	**UC**	**72.9**
approx. depth (mm)	universal column	linear density ($\mathrm{kg\,m^{-1}}$)

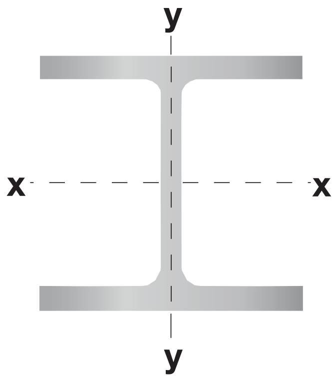
   The geometry of UC sections differ from UB sections with wider flanges in order to limit lateral buckling. The depth is similar to the flange width.

   The radius of gyration $r_y = \sqrt{\dfrac{I_y}{A}}$ about weak axis is often larger than that of UB sections.

3. Universal Bearing Piles (UBP), a.k.a., H sections

A typical designation:

310	**UBP**	**149**
approx. depth (mm)	universal bearing piles	linear density ($\mathrm{kg\,m^{-1}}$)

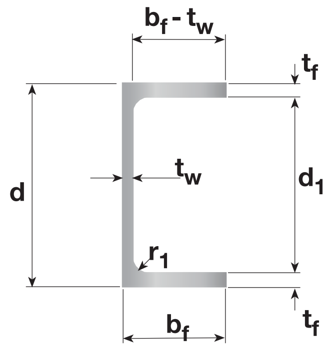


UBP sections are similar to UC sections but have uniform thickness for both web and flanges.

4. Parallel Flange Channels (PFC)

A typical designation:

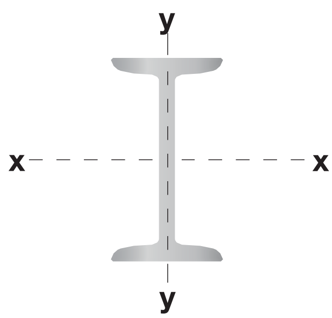
200	**PFC**
section depth (mm)	parallel flange channel



5. Tapered Flange Beams (TFB)

Sometimes TFB is also called Rolled Steel Joist (RSJ), M, S sections. A typical designation:

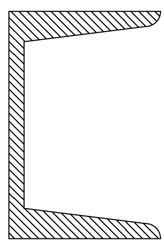
125	**TFB**
section depth (mm)	tapered flange beam



TFBs are traditional sections. However, there are deemed as inefficient structural member. Sloping washers are needed for flange connections.

6. Taper Flange Channels (TFC)

Traditionally often used for framing around door opening, or for built-up lattices members. Seldom used now.

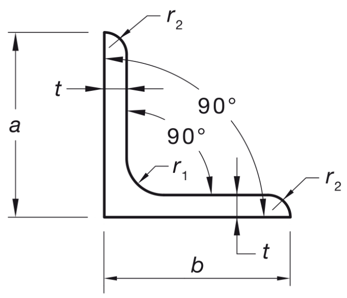


7. Equal Angles (EA)

Angles are commonly used as tension bracing or for built–up members.
A typical designation:

125	×	**125**	×	**16**	**EA**
a (mm)		b (mm)		t (mm)	

It is also denoted as L125×125×16 or 125×16EA.

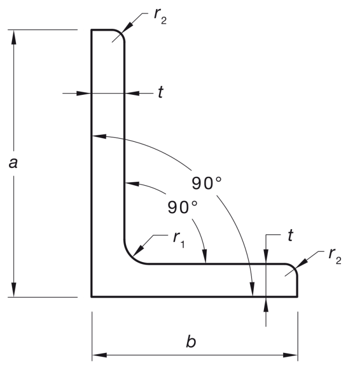


8. Unequal Angles (UA)

Angles are commonly used as tension bracing or for built–up members.
A typical designation:

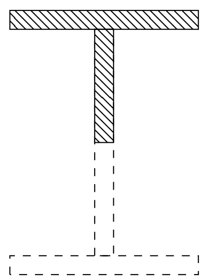
125	×	**75**	×	**12**	**UA**
a (mm)		b (mm)		t (mm)	

It is also denoted as L125×75×12UA.



9. Structural Tees (T)

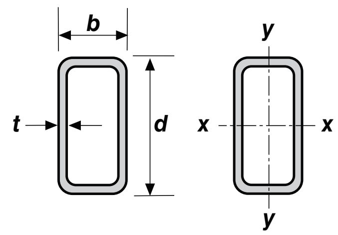
They can be made by splitting UC or UB sections and used for chord members in steel trusses, flanges in plate girders or hangers.



10. Rectangular Hollow Sections (RHS)

A typical designation:

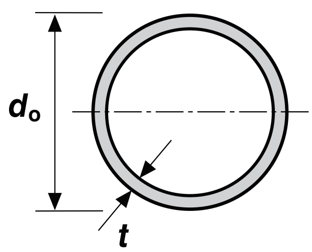
75	×	**25**	×	**2.5**	**RHS**
d (mm)		b (mm)		t (mm)	



11. Circular Hollow Sections (CHS)

A typical designation:

165.1	×	**3.0**	**CHS**
d_0 (mm)		t (mm)	



12. Plates

    Common thicknesses include: 5 mm, 6 mm, 8 mm, 10 mm, 12 mm, 16 mm, 20 mm, 25 mm. Common widths include: 20 mm, 25 mm, 32 mm, 40 mm, 50 mm, 65 mm, 75 mm, 90 mm, 100 mm, 110 mm, 130 mm, 150 mm.

    e.g., 12×200×300 plate.

13. Bars

    Flat bars, commonly also referred to as ‘flats’, commonly have widths ranging from 10 mm to 90 mm.

    e.g., 12×24×300 bar.

Sections may also be described in terms of being non-symmetric, singly symmetric or doubly symmetric. For example,

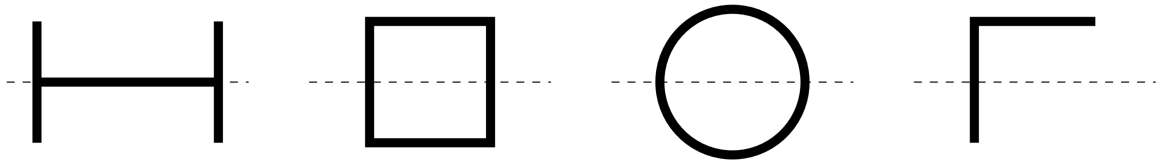


Figure 2.6: Examples of non-symmetric, singly symmetric and doubly symmetric sections

2.4.2 Cold–Formed Products

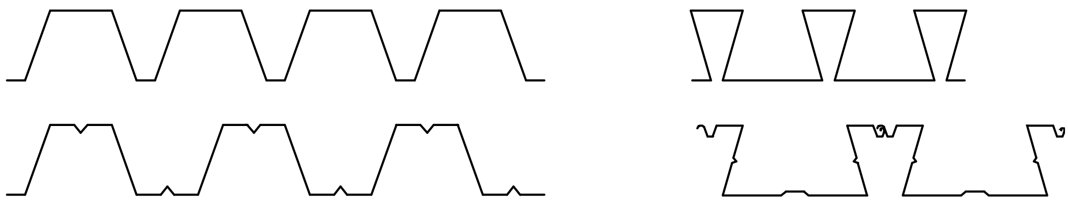


Figure 2.7: Profiled sheets and linear trays (Dubina et al., 2012)

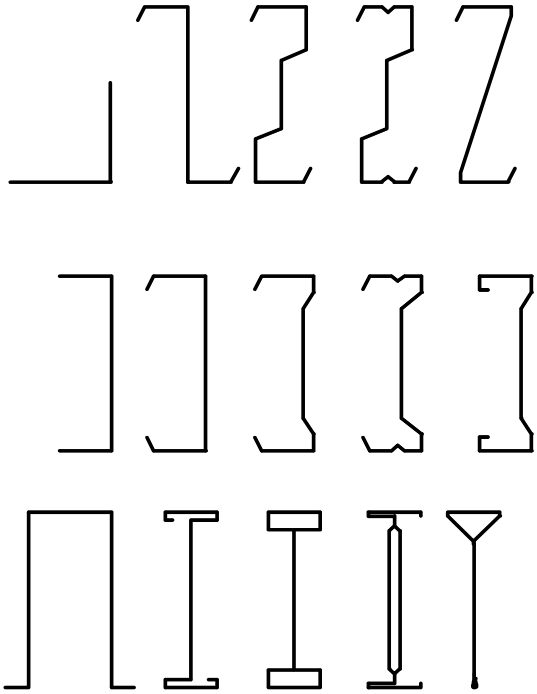


Figure 2.8: Single open sections (Dubina et al., 2012)

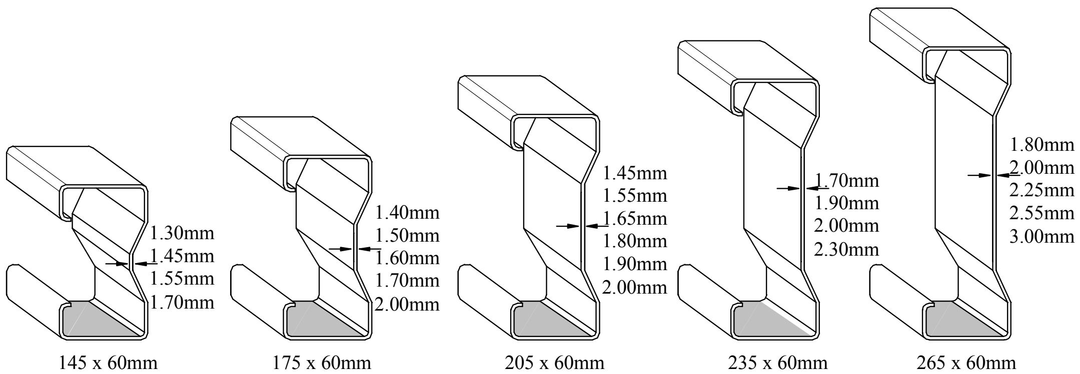


Figure 2.9: Channel sections (Dubina et al., 2012)

2.4.3 Standard Welded Products

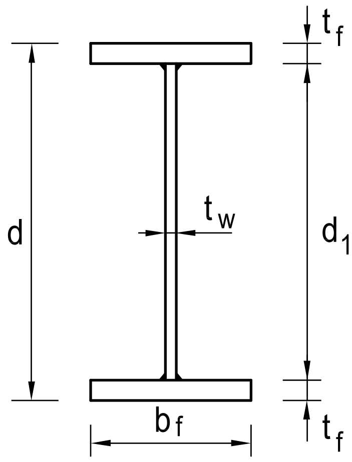
Welded sections are often welded together plates. They can be welded on either one side only or both sides.

1. Welded Beams (WB)
   A typical designation:

900	**WB**	**218**
d	welded	linear
height	beam	density
(mm)		($\mathrm{kg\,m^{-1}}$)

2. Welded Columns (WC)
   A typical designation:

400	**WC**	**270**
d	welded	linear
height	column	density
(mm)		($\mathrm{kg\,m^{-1}}$)



Non-prismatic sections can be used to reduce cost and improve efficiency, e.g.,

Figure 2.10: Non-prismatic sections by plates welded together

2.5 Steel Availability

Some general guidance is available from mills which may be relevant to Australasia (e.g., Liberty Steel Hot Rolled and Structural Steel Product).

SCNZ recommends chartered distributor for sources compliant structural steel. Distributors have limited members so it is best to call the check availability.

2.6 Standard Tolerance

Sections, as well as members, must satisfy code dimension requirements (NZS 3404.1&2:1997 § 14.4). Deformations must arise as parts of members cool at different rates and residual stresses are developed.

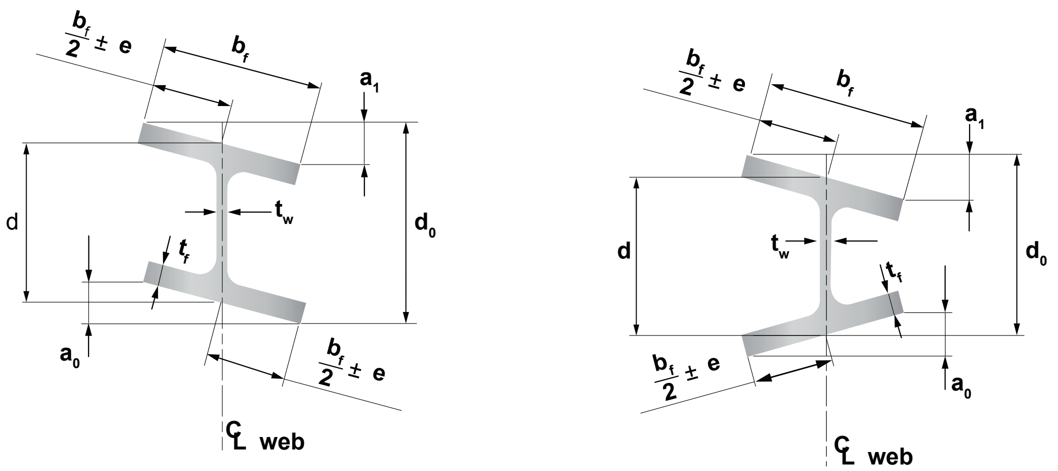


Figure 2.11: Tolerances on a cross section

Typical tolerance for camber and sweep is $L/1000$ (NZS 3404.1&2:1997 Table 14.4.5). For I sections with a flange width less than $150\,\mathrm{mm}$, the tolerance of sweep can be as large as $L/500$.

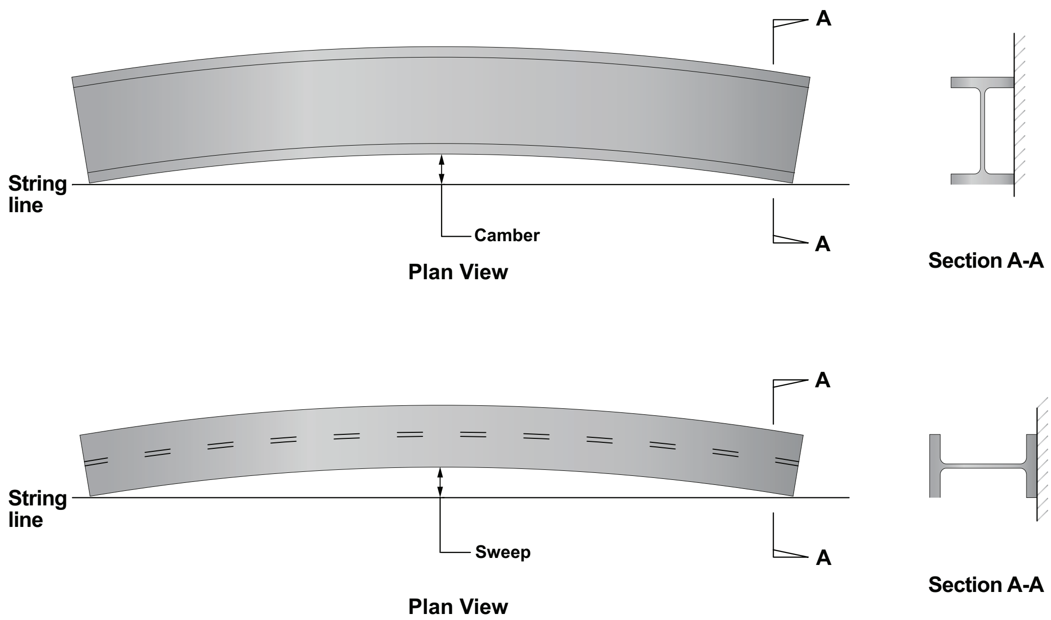


Figure 2.12: Measurement of camber and sweep

2.7 Undesirable Steel Behaviour

2.7.1 Brittle Fracture

Brittle fracture affects strength.

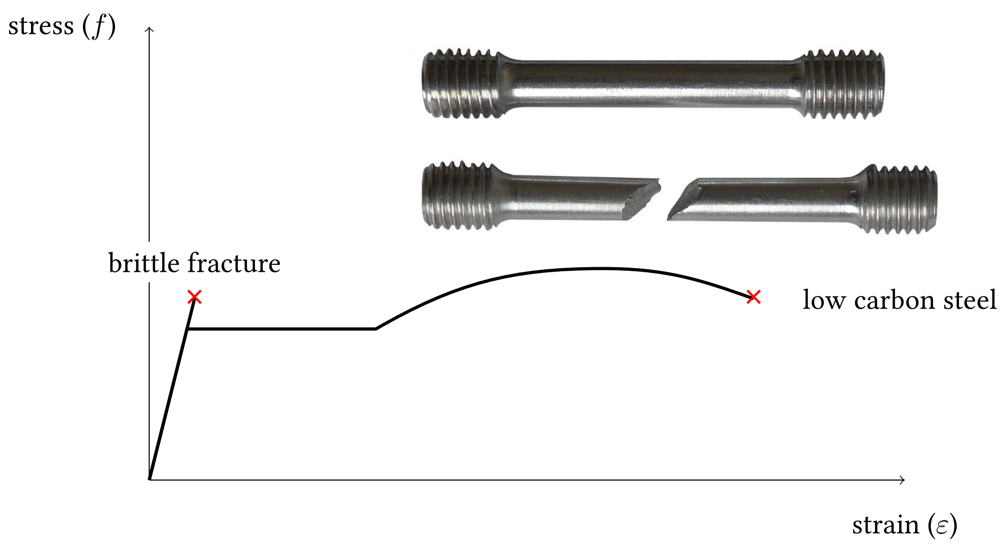


Figure 2.13: Brittle fracture

2.7.2 Buckling

Structural members may fail under buckling, the material strength is not fully utilised under buckling. To learn more, check this video: Understanding Buckling[5].

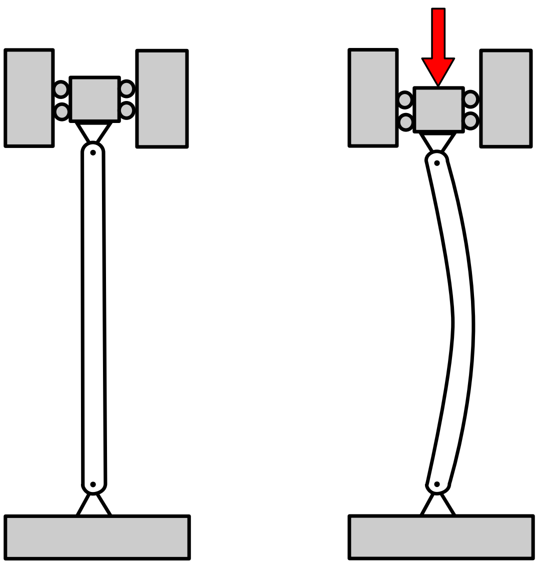


Figure 2.14: Buckling

[5] https://www.youtube.com/watch?v=21G7LA2DcGQ

2.7.3 Excessive Deformation/Vibration

It affects serviceability.

- bouncy floor
- ponding

Figure 2.15: Ponding on roof

2.8 Standard Gauge

The following tables are generated according to tables from Table 10.2-1 to Table 10.2-5 provided by ASI design capacity tables (Australian Steel Institute, 2016). Please refer to ASI manual for notes.

Table 2.4: Gauge lines for universal sections

Section	Flange s_{gf} M20		Flange s_{gf} M24		Web s_{gw} M20			Web s_{gw} M24		
610UB	140	90	140	90	140	90	70	140	90	70
530UB	140	90	140	90	140	90	70	140	90	70
460UB	90	140	90		90	70	140	90	70	140
410UB	90	70	90		90	70	140	90	70	140
360UB,310UB	90	70	90		90	70	140	90	70	140
310UB32.0	70		b		90	70	140	90	70	140
250UB	70	90	b		70	90	140	70	90	
250UB25.7	70*		b		70	90		70	90	
200UB	70		b		70	90		70	90	
200UB18.2	60*		b		70	90		70		
180UB	50**		b		70			70		
150UB	c		b		60*			b		
310UC	140	90	140	90	90	70	140	90	70	140
250UC	140	90	140	90	90	70	140	90	70	
200UC	140	90	140	90	90	70		90	70	
150UC	90	70	90		60*			b		
100UC	60*		b		c			b		
Preference	1	2	1	2	1	2	3	1	2	3

* Gauge listed are for M16 bolts.
** Gauge listed are for M12 bolts.
b indicates that the flange or web cannot accommodate this size of bolt.
c indicates that the flange or web cannot accommodate two lines of bolts with a gauge of 50 mm or more for M12 or larger bolts.
All dimensions are in mm.

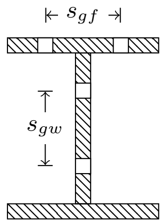
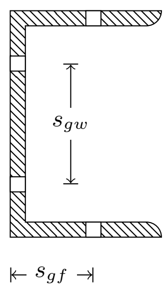


Table 2.5: Gauge lines for TFB and PFC

Section	Flange s_{gf} M16	Flange s_{gf} M20	Flange s_{gf} M24	Web s_{gw} M16			Web s_{gw} M20			Web s_{gw} M24		
125TFB	b,b1	b	b	50			50			c		
100TFB	b,b1	b	b	c			c			c		
380PFC	55	55	55	140	90	70	140	90	70	140	90	70
300PFC	55	55	55	140	90	70	140	90	70	140	90	70
250PFC	55	55	55	140	90	70	140	90	70	140	90	70
230PFC	45	45	45	140	90	70	90	70		90	70	
200PFC	45	45	45	90	70		90	70		90	70	
180PFC	45	45	45	70			70			c		
150PFC	45	45	45	50			50			c		
125PFC	35	35	b	50			c			c		
100PFC	30	b	b	c			c			c		
75PFC	b,b1	b	b	c			c			c		
Preference	1	1	1	1	2	3	1	2	3	1	2	3

b indicates that the flange cannot accommodate this size of bolt.
b1 indicates that the flange cannot accommodate M12 bolt.
c indicates that the web cannot accommodate two lines of bolts with a gauge of 50 mm or more.
All dimensions are in mm.



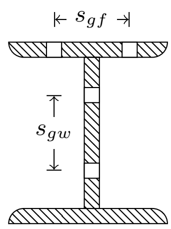


Table 2.6: Gauge lines for structural Tees cut from universal sections

Section	Flange s_{gf} M20		Flange s_{gf} M24		Web s_{gw} M20			Web s_{gw} M24		
305BT	140	90	140	90	140	90	70	140	90	70
265BT	140	90	140	90	140	90	70	140	90	70
230BT	90	140	90		90	70	140	90	70	
205BT	90	70	90		90	70		90	70	
180BT	90	70	90		90	70		70		
155BT	90	70	90		70					
155BT16.0	70		b		70					
125BT	70	90	b		50**			b		
125BT12.9	70*		b		50**			b		
100BT	70		b		c			b		
100BT9.1	60*		b		c			b		
90BT	50**		b		c			b		
75BT	c		b		c			b		
155CT	140	90	140	90	60			b		
125CT	140	90	140	90	50**			b		
100CT	140	90	140	90	c			b		
75CT	90	70	90		c			b		
50CT	60*		b		c			b		
Preference	1	2	1	2	1	2	3	1	2	3

* Gauge listed are for M16 bolts.
** Gauge listed are for M12 bolts.
b indicates that the flange or web cannot accommodate this size of bolt.
c indicates that the flange or web cannot accommodate two lines of bolts with a gauge of 50 mm or more for M12 or larger bolts.
All dimensions are in mm.

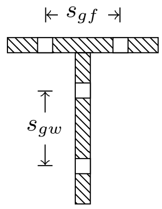


Table 2.7: Gauge lines for welded sections

Section	M20 s_{gf1}		M20 s_{gf2}		M20 s_{gw}			M24 s_{gf1}		M24 s_{gf2}	M24 s_{gw}		
1200WB455-392	140	90	280	420	140	90	70	140	90	280	140	90	70
1200WB342-278	140	90	280		140	90	70	140	90	280	140	90	70
1200WB249	140	90	b		140	90	70	140	90	b	140	90	70
1000WB322-258	140	90	280		140	90	70	140	90	280	140	90	70
1000WB215	140	90	b		140	90	70	140	90	b	140	90	70
900WB282-218	140	90	280		140	90	70	140	90	280	140	90	70
900WB175	140	90	b		140	90	70	140	90	b	140	90	70
800WB	140	90	b		140	90	70	140	90	b	140	90	70
700WB	140	90	b		140	90	70	140	90	b	140	90	70
500WC	140		280	420	140	90	70	140		280	140	90	70
400WC	140		280		140	90	70	140		280	140	90	70
350WC	140		280		140	90	70	140		b	140	90	70
Preference	1	2	1	2	1	2	3	1	2	1	1	2	3

b indicates that the flange cannot accommodate this size of bolt.
All dimensions are in mm.

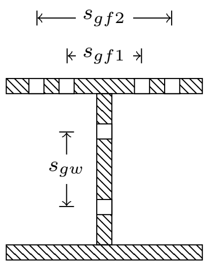
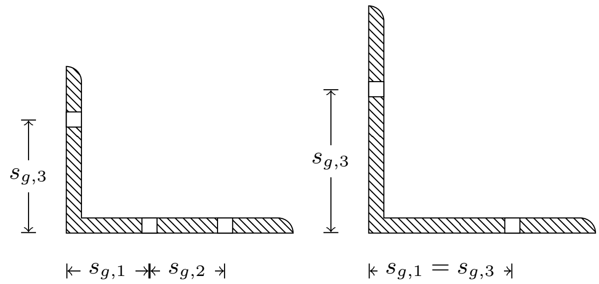


Table 2.8: Gauge lines for angles

Nominal leg length	s_{g1}	s_{g2}	s_{g3}	Bolt
200	65(75)	70(75)	100	M24
150	50(55)	60(55)	75	M24
125	45	50	65(62)	M24
100			50	M20
90			45	M20
75			40(38)	M20
65			35(32)	M16
55			30(28)	M16
50			30(25)	M16
45			25(22)	M12
40			25(20)	M12



2.9 Special Considerations Affecting Steel Properties

- **corrosion**
  Corrosion is an electrochemical process resulting in reduction of the steel cross sectional area.

  The following treatments can be applied to protect steel from corrosion:

  - weathering steels: special alloy steels corrode and the corrosion forms a protective coating reducing the rate of future corrosion
  - separating steel from the atmosphere (e.g., painting)
  - galvanizing with a more reactive metal (e.g., zinc)

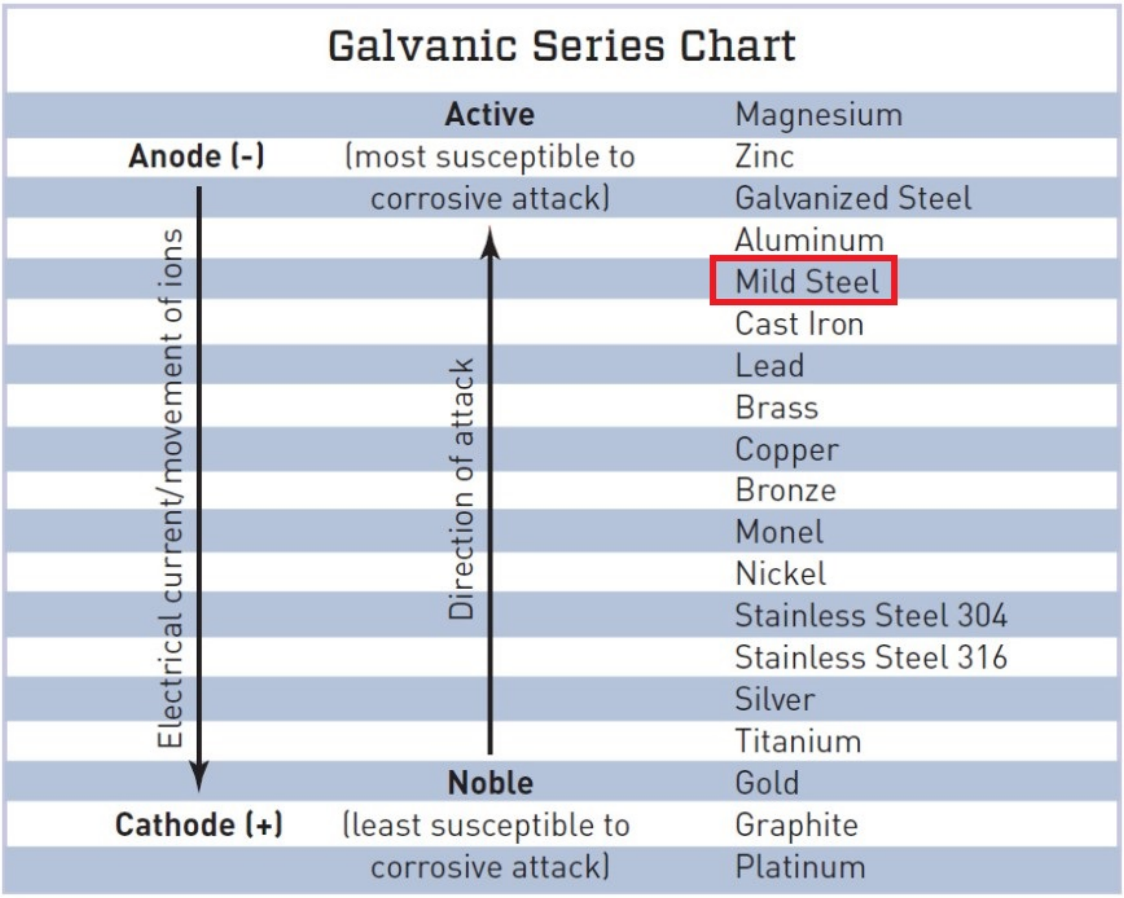


Figure 2.16: Galvanic activity (https://www.jlconline.com/how-to/exteriors/separating-galvanic-metals_o)

- **temperature**
  Metals become brittle under low temperature and ductile/softening under high temperature. It is necessary to provide an environment where the steel does not get too hot nor too cold by, for example:
  - provide adequate ventilation, reduce fuel, e.g., parking structures
  - cover steel surface by using concrete, timber, plaster, chemical foam, mineral fibres, special paints, etc.

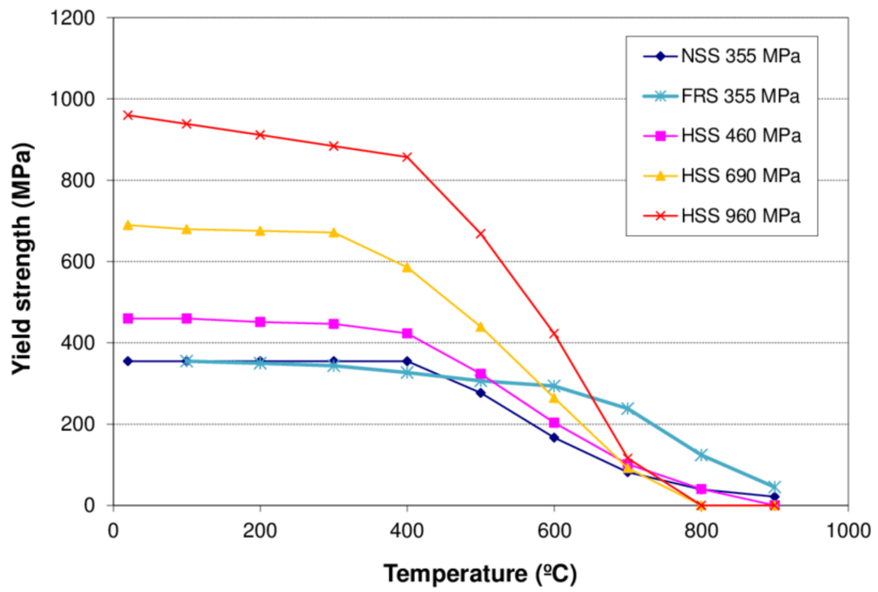
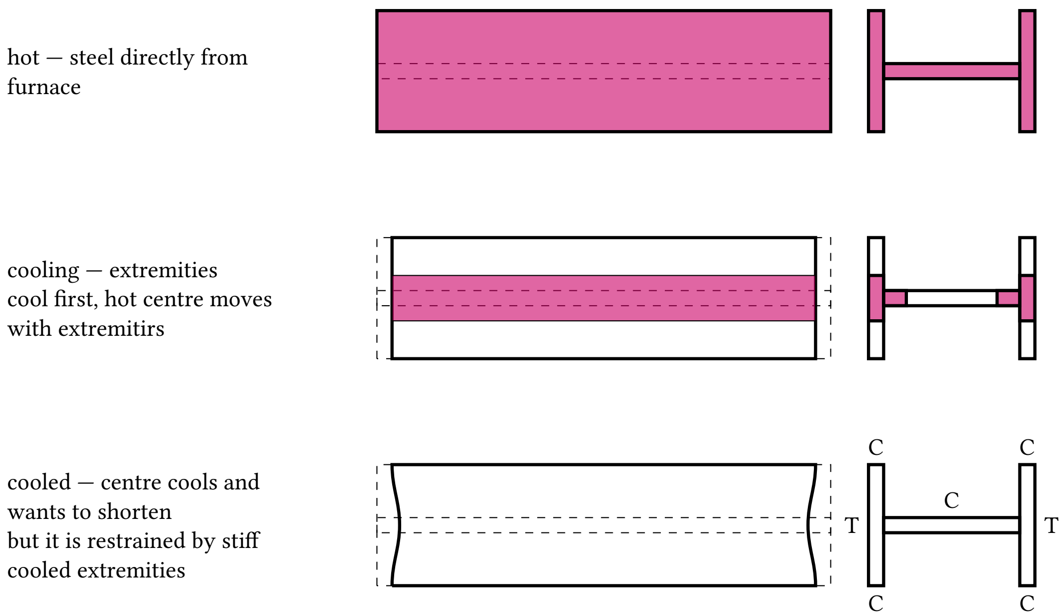


Figure 2.17: Steel strength decreases with temperature (Espinos et al., 2015)

- **residual stress**
  Residual stresses may occur in steel members/elements as a result of nonuniform cooling of the section. It may occur due to rolling or welding. For the section shown below, it will cool fastest at the extremities, where there is less additional steel to maintain its temperature. Once the extremities cool, they become solid. Then, as the core (away from the extremities) cools, it tries to shorten. As it does this it pulls on the extremities, so that they shorten too. However, as the extremities are solid, they do not shorten. Instead, they become subject to a compressive force as they resist the shortening. Also, the core remains in tension as it is unable to cause the extremities to shorten. Therefore, in general, extremities are in compression, while the core is in tension. Welded sections may have significantly different behaviour.



This puts extremities into compression and central regions into tension.

Figure 2.18: Residual stress from cooling member

Although the section as whole is in equilibrium, due to residual stresses, some regions are in compression while some in tension. When loaded, the section yields non-uniformly. The tension force displacement curve of a member becomes rounded as the stress approaches the yield value, but the peak tensile strength does not change. However, in the case of compression force, the rounded stress–strain curve implies a lower member axial stiffness. Since the buckling load is related to the stiffness, the compressive strength can decrease. This affects members of intermediate slenderness (as stocky members do not buckle, and slender members buckle before the residual stresses affect the response).

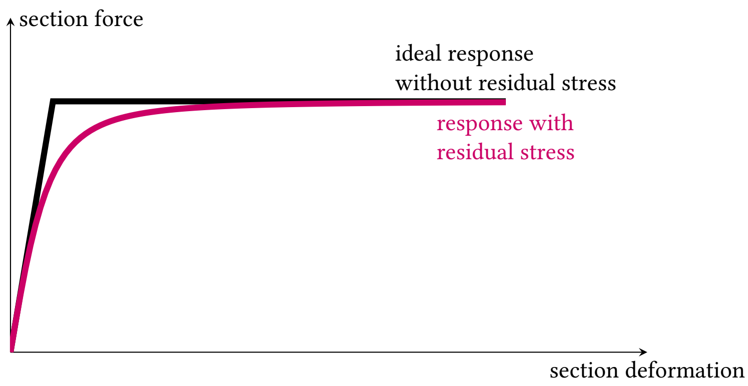


Figure 2.19: Stress–strain curve considering residual stress

Depending on different geometries/configurations, different residual stress patterns may appear as shown in the following figure.

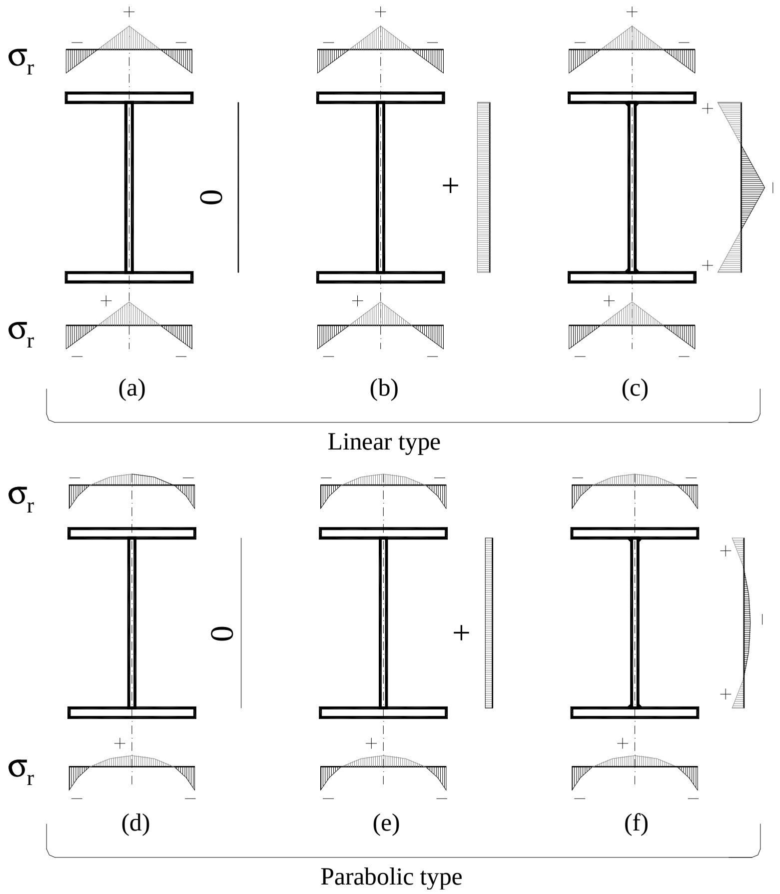


Figure 2.20: Residual stress patterns of hot-rolled sections (https://www.lajss.org/index.php/LAJSS/article/view/176)

- **steel detailing**
  Rapid changes in cross section shape can cause stress concentrations.

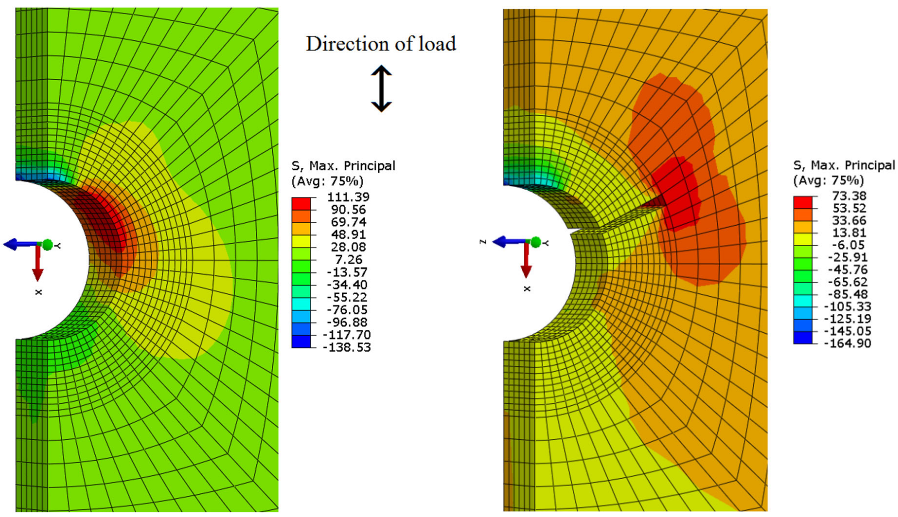


Figure 2.21: Stress concentration due to sharp change of geometry (Katsivalis et al., 2018)

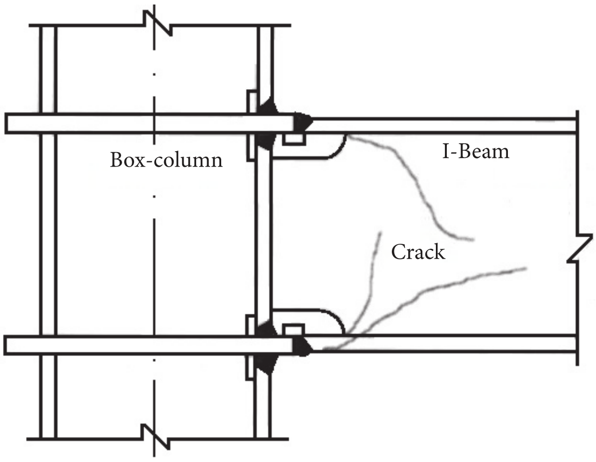


Figure 2.22: Crack in connections (Qu et al., 2017)

- **steel quality**
  Poor quality steel is more likely to be brittle. Brittle behaviour is possible as a result of:
  - large dimension ($t > 20\,\text{mm}$)
  - defects (inclusions, air pockets, etc.) in steel
  - weak lamellar bonds

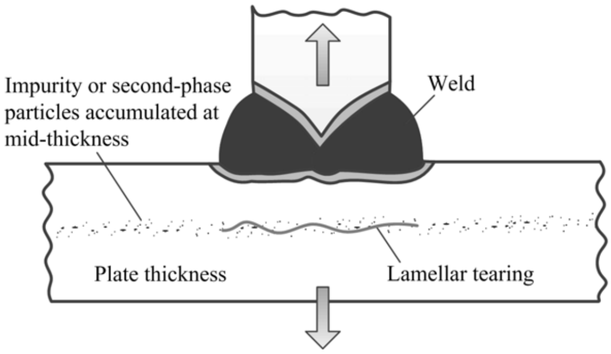


Figure 2.23: Illustration of lamellar tearing in thick steel plates (http://sainsmechanical.blogspot.com/2011/12/hot-working-of-metals.html)

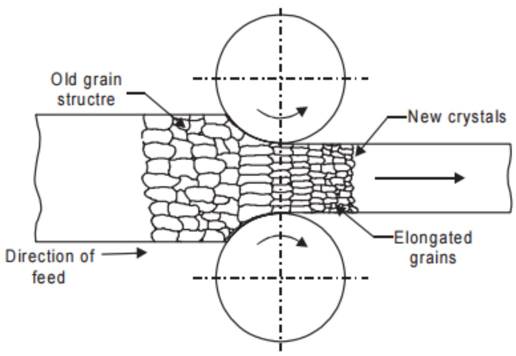


Figure 2.24: Gain reshaped due to rolling

- **cold working**
  Possibility of fracture is increased after cold working, thus one shall be careful if reusing steels.

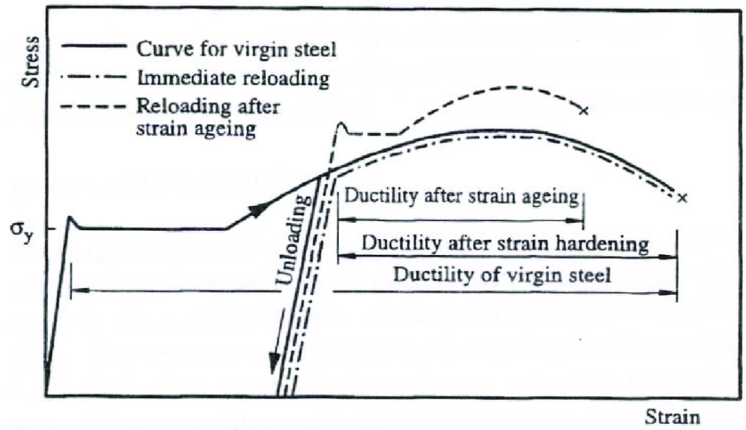


Figure 2.25: Strain hardening by cold working

- **strain ageing**
  Strain aged steel is stronger but more brittle.

  Steel is initially loaded to point A for some time then unloaded. When reloaded, it can reach point B and follow the corresponding path as shown in the figure.

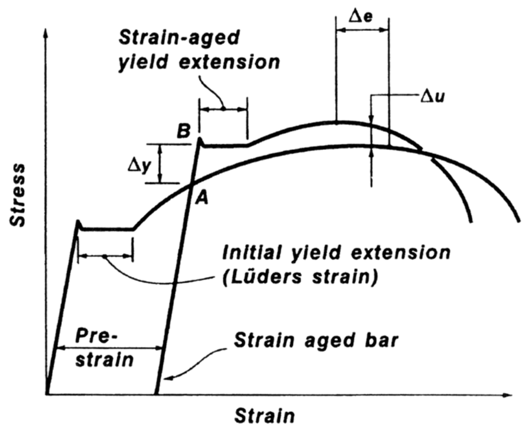


Figure 2.26: Strain ageing

- **repeated loading**
  As the material sustains many elastic load reversals it may lead to brittle failure characterised by fatigue. Fatigue fracture is dependent on magnitude of stress reversal as well as the number of cycles applied. It is most significant when there are sharp discontinuities in the material shape. It is not usually a concern in buildings, but can be a problem due to traffic on bridges.

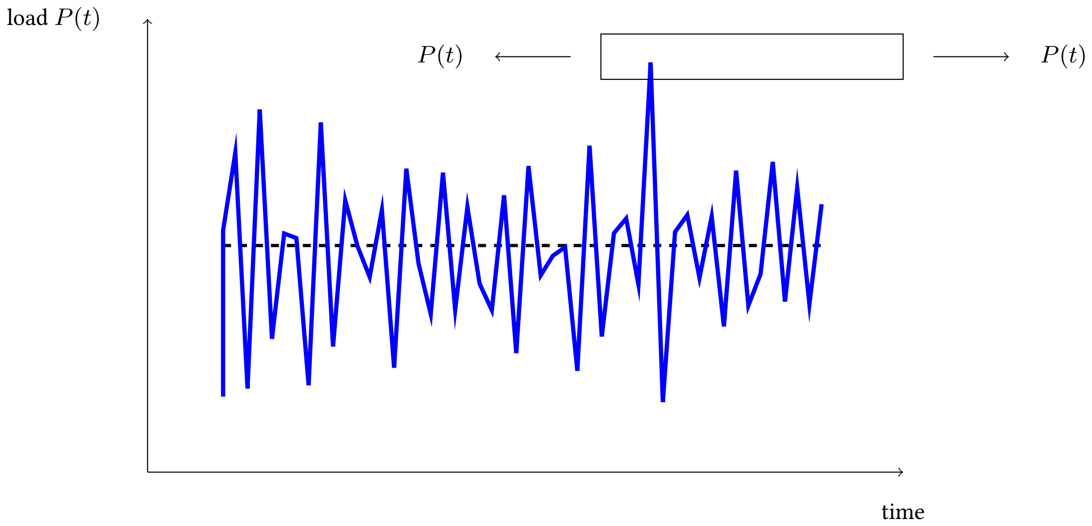


Figure 2.27: Cyclic loads

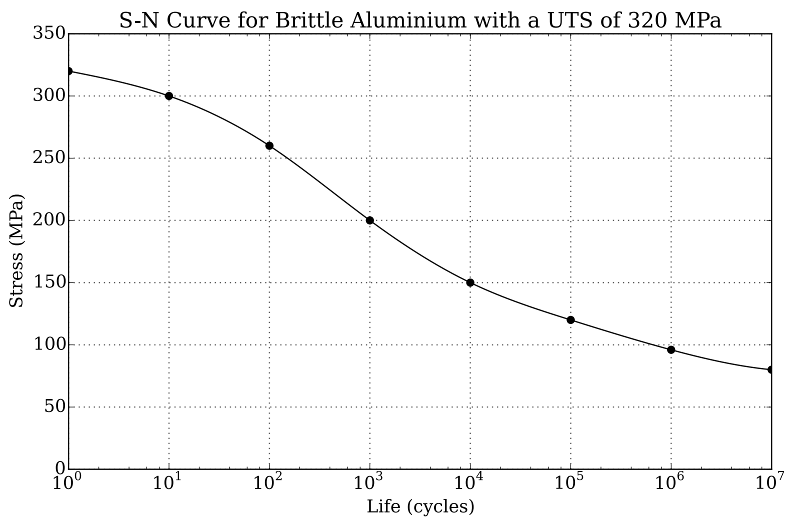


Figure 2.28: Stress–life curve for a brittle aluminium (`https://upload.wikimedia.org/wikipedia/commons/d/d2/BrittleAluminium320MPa_S-N_Curve.svg`)

- **impact loading**

  High speed loading can lead to more brittle failure. This is often indicated by Charpy V-notch[6] test.

  High impact resistance is important for structures subject to earthquake shaking.

  The mass is released from specified height. The angle θ the mass moves to after breaking specimen is recorded as a measure of ductility. Often low θ, indicating more energy is absorbed by the specimen, is desired.

[6] https://www.youtube.com/watch?v=tpGhqQvftAo

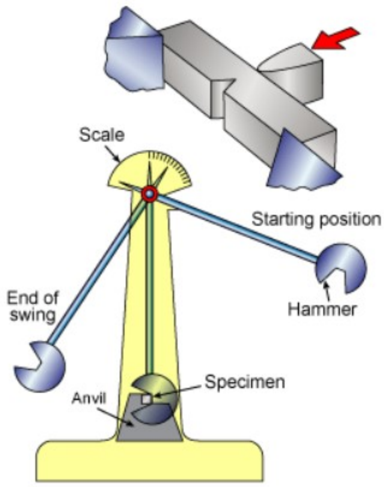


Figure 2.29: Charpy V-notch test apparatus (https://www.totalmateria.com/page.aspx?ID=CheckArticle&site=kts&NM=534)

- **triaxial loading**
  Depending on loading direction, triaxial loading conditions may lead to an increase or decrease in strength. An increase in strength is generally associated with a decrease in ductility.

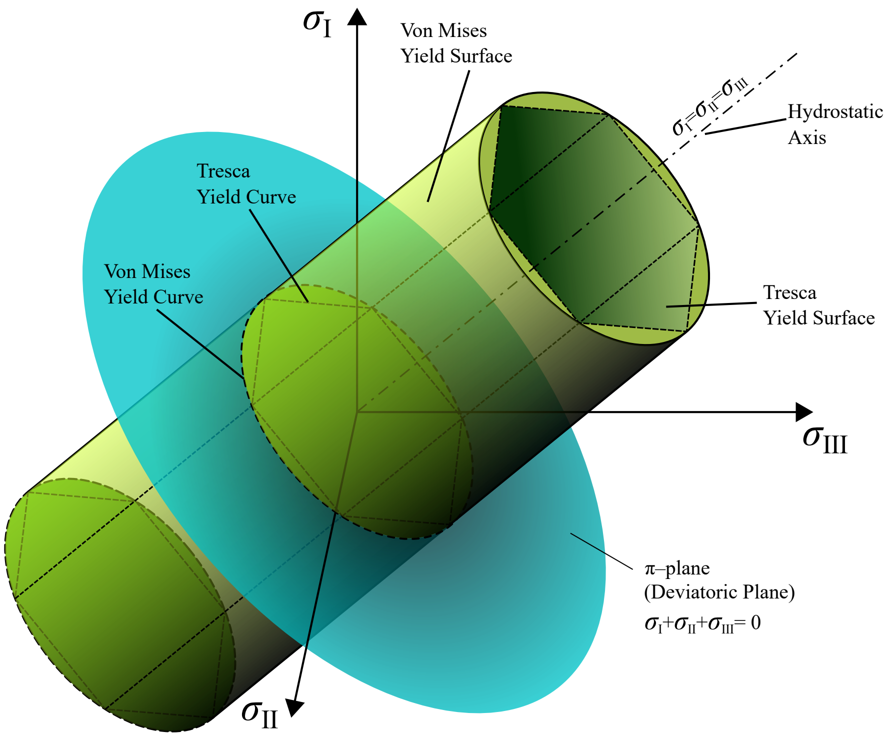
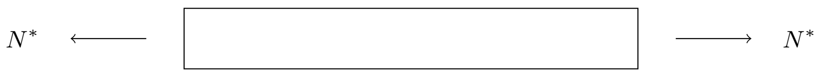


Figure 2.30: von Mises yielding criterion (https://upload.wikimedia.org/wikipedia/commons/c/cc/Yield_surfaces.svg)

Tension Members

Tension members are commonly used as bracing in trusses and elsewhere. They may be wires, rods, bars, structural shapes, etc.



3.1 Strength Design Concept

The LRFD equation possesses the following form (NZS 3404.1&2:1997 § 7.1):

$$N^* \leqslant \phi N_t \tag{3.1}$$

where

N^* = factored summation of forces
N_t = nominal (ideal) tensile strength
ϕ = strength reduction factor, for tension $\phi_t = 0.9$

Failure is considered to occur as a result of either:

- **Excessive elongation**: yield over the **gross** area A_g
  In this case, N_t is determined by the gross area:

  $$N_t = A_g f_y \tag{3.2}$$

  where
  A_g = gross cross section area
  f_y = yield strength

- **Sudden strength decrease**: facture over the **net** area A_n
  In this case, N_t is determined by the net area:

$$N_t = 0.85 k_{te} A_n f_u \tag{3.3}$$

where

k_{te} = the correction factor for distribution of forces
A_n = net cross section area
f_u = tensile (ultimate) strength

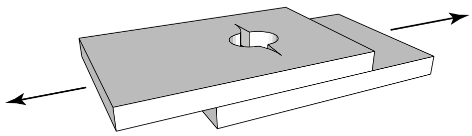


Figure 3.1: Fracture over net area

Thus, to design a member in tension, one shall use the following criterion.

$$N^* \leqslant \min\left(\phi A_g f_y,\ \phi 0.85 k_{te} A_n f_u\right). \tag{3.4}$$

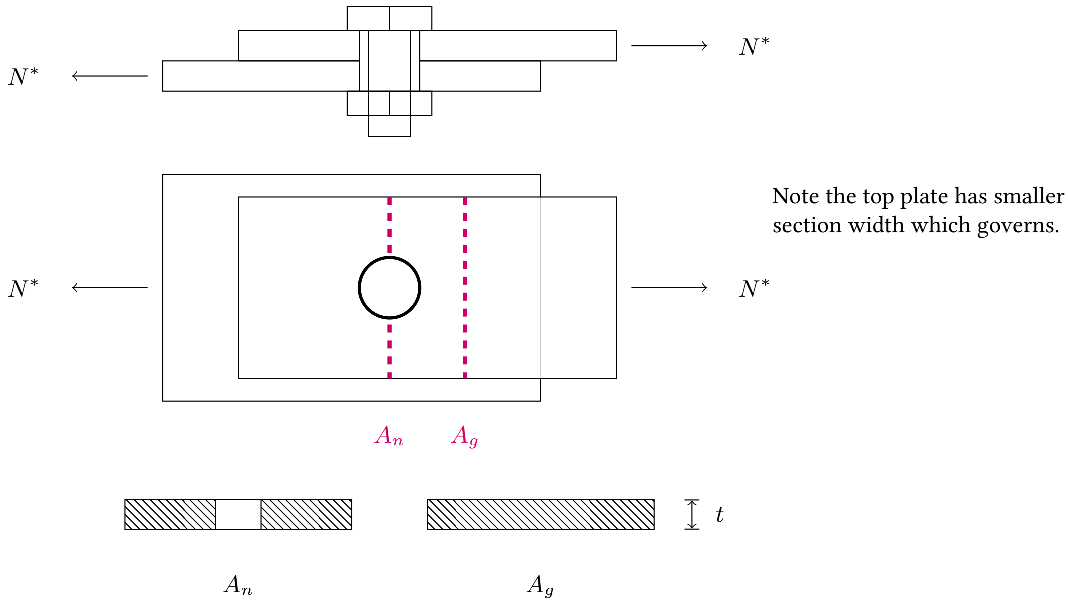


Figure 3.2: Definitions of gross and net areas

3.1.1 Definition of Net Area

The **net** area of the section considers the **gross** areas of holes used in the member. For bolted members, the standard hole size d_h is a function of the bolt diameter d_f (NZS 3404.1&2:1997 § 14.3.5.2.1).

$$d_h = \begin{cases} d_f + 2\,\mathrm{mm}, & d_f \leqslant 24\,\mathrm{mm}, \\ d_f + 3\,\mathrm{mm}, & d_f > 24\,\mathrm{mm}. \end{cases} \tag{3.5}$$

Unstaggered Holes

According to NZS 3404.1&2:1997 § 9.1.10.2, for holes that are not staggered, the area to be deducted shall be the **maximum** summation of the areas of the holes in any cross sections that are perpendicular to the direction of the design action. For a cross section with n holes of size $d_{h,i}$ of a plate of thickness t, the net area can be computed as

$$A_n = A_g - \sum_{i=1}^{n} d_{h,i} t_i. \tag{3.6}$$

The calculated net area is indeed the **minimum** 'net' area.

Staggered Holes

According to NZS 3404.1&2:1997 § 9.1.10.3, for staggered holes, the area to be deducted shall be the **greater** of:

1. the deduction of non-staggered holes,
2. the summation of the areas of all holes in any zig-zag line extending progressively across the member or part of the member.

For a zig-zag line with n holes and m diagonal line segments, the net area shall be computed as follows.

$$A_n = A_g - \sum_{i=1}^{n} d_{h,i} t_i + \sum_{j=1}^{m} \frac{s_{p,j}^2 t_j}{4 s_{g,j}}, \tag{3.7}$$

in which

s_p = staggered pitch, distance between hole centers measured **parallel** to the force direction
s_g = gauge length, distance between hole centers measured **perpendicular** to the force direction

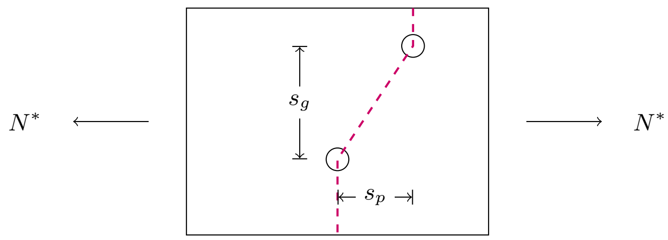
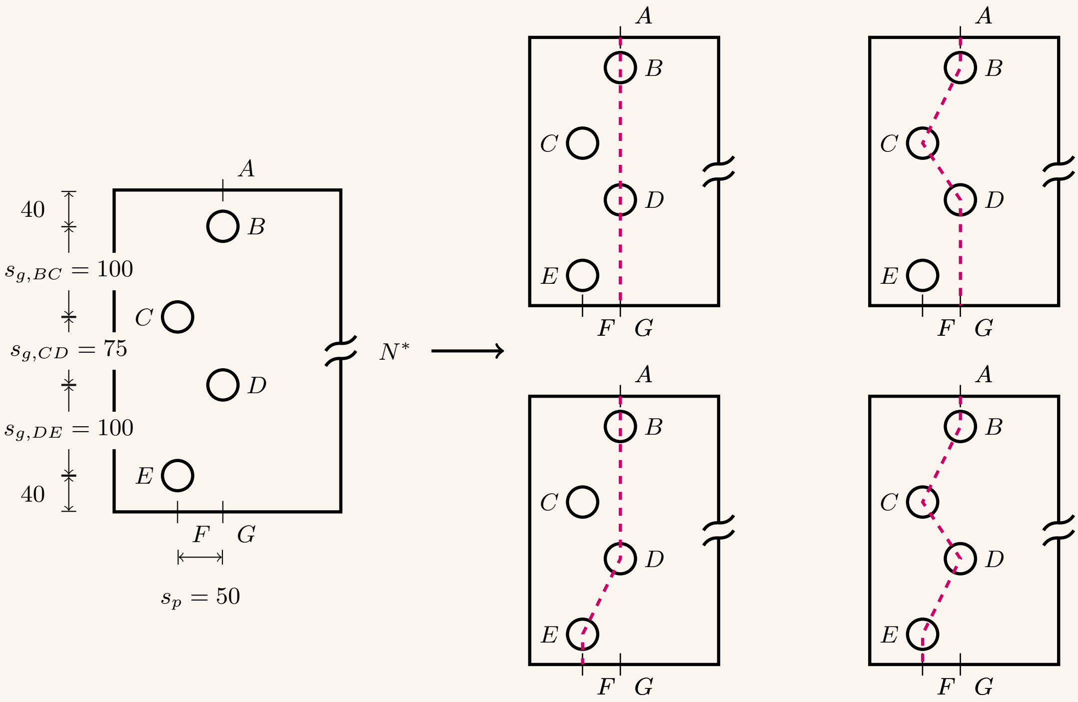


Figure 3.3: Definition of gauge length and pitch

Eq. (3.6) is the simplified case of Eq. (3.7) with no diagonal line segments. For staggered layouts, the third term in Eq. (3.7) must be used. For any given bolt layouts, the governing net area shall be the **minimum** of all staggered and unstaggered cases. This is due to the fact that in a staggered layout, depending on pitch and gauge, an unstaggered section may have smaller net area than a staggered section.

If all holes have the same size $d_{h,i} = d_h$, and the plate thickness is uniform $t_i = t_j = t$, then it is possible to express the net area as $A_n = Lt$ with $L = b - nd_h + \sum_{j=1}^{m} \frac{s_{p,j}^2}{4s_{g,j}}$. The main task is to find the length of critical fracture line L.

Example 3.1 *For the plate shown, assume the thickness is $t = 12\,\text{mm}$ and the diameter of bolts is $20\,\text{mm}$, to compute the critial fracture line, both staggered and unstaggered cases need to be considered.*



Solution 3.1

The standard hole size $d_h = 20\,\text{mm} + 2\,\text{mm} = 22\,\text{mm}$. The width $b = 355\,\text{mm}$. The four cases considered are:

- Line ABDG — Unstaggered

$$L_{ABDG} = b - 2d_h = 355\,\text{mm} - 2 \times 22\,\text{mm} = 311\,\text{mm}.$$

- Line ABDEF — Staggered
  There are three bolts and one diagonal line DE, for which $s_p = 50\,\text{mm}$ and $s_g = 100\,\text{mm}$.

$$\begin{aligned} L_{ABDEF} &= b - 3d_h + \frac{s_p^2}{4s_g} \\ &= 355\,\text{mm} - 3 \times 22\,\text{mm} + \frac{(50\,\text{mm})^2}{4 \times 100\,\text{mm}} = 295.3\,\text{mm}. \end{aligned}$$

- Line ABCDG — Staggered
  There are three bolts and two diagonal lines. For BC, $s_p = 50\,\text{mm}$ and $s_g = 100\,\text{mm}$. For

CD, $s_p = 50\,\text{mm}$ and $s_g = 75\,\text{mm}$.

$$L_{ABCDG} = b - 3d_h + \sum \frac{s_p^2}{4s_g}$$
$$= 355\,\text{mm} - 3 \times 22\,\text{mm} + \frac{(50\,\text{mm})^2}{4 \times 100\,\text{mm}} + \frac{(50\,\text{mm})^2}{4 \times 75\,\text{mm}} = 303.6\,\text{mm}.$$

- Line ABCDEF — Staggered
  There are four bolts and three diagonal lines: BC, CD and DE.

$$L_{ABCDEF} = b - 4d_h + \sum \frac{s_p^2}{4s_g}$$
$$= 355\,\text{mm} - 4 \times 22\,\text{mm} + \frac{2 \times (50\,\text{mm})^2}{4 \times 100\,\text{mm}} + \frac{(50\,\text{mm})^2}{4 \times 75\,\text{mm}} = 287.8\,\text{mm}.$$

The governing case leads to the **minimum** net area $A_n = L_{ABCDEF}t = 287.8\,\text{mm} \times 12\,\text{mm} = 3454\,\text{mm}^2$. In this example, since the plate is resisting N^* applied on the right side, there is **no** need to consider cases such as ABCEF in which bolt at D has provided some resistance. The design of bolted connection will be introduced in Chapter 6.

Angle Connection

For angle connection, the gauge s_g shall be taken as the sum of the back marks to each hole, less the leg thickness (NZS 3404.1&2:1997 Fig. 9.1.10.3(2)).

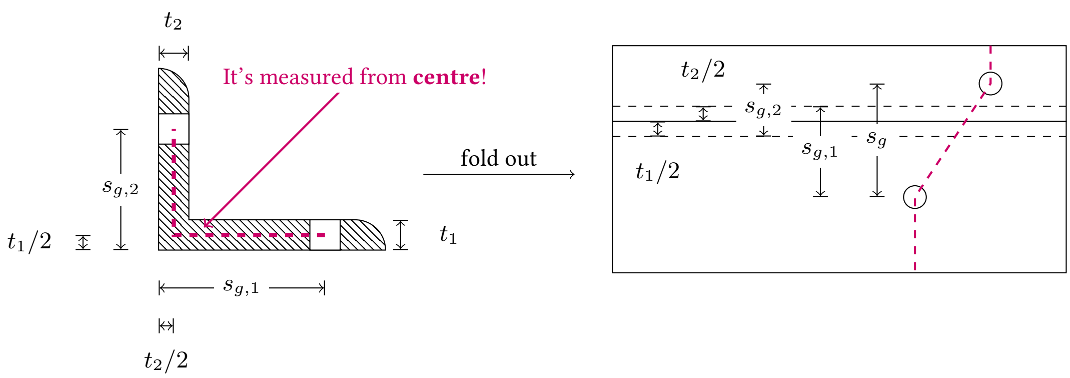


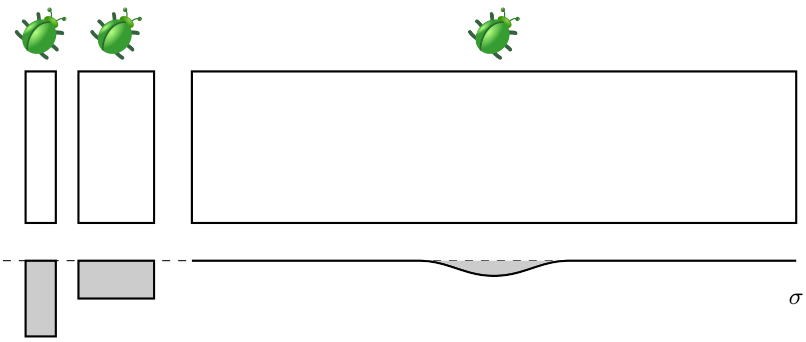
Figure 3.4: Illustration of gauge of staggered layout in an angle

Essentially, the distance measured at the **centre** of thickness is evaluated. That is,

$$s_g = s_{g,1} + s_{g,2} - \frac{t_1 + t_2}{2}. \tag{3.8}$$

3.1.2 Determination of Distribution Factor

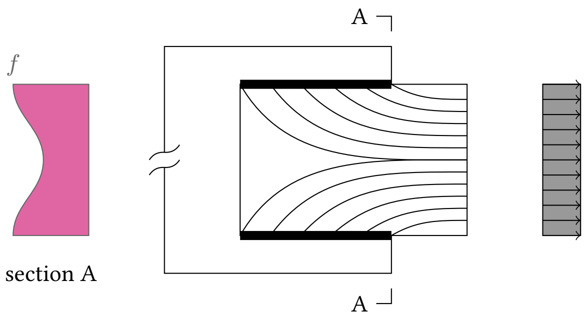
A correction factor k_{te} is used to account for the concept of shear lag, or plane sections not remaining plane. This concept may be seen from the following examples.



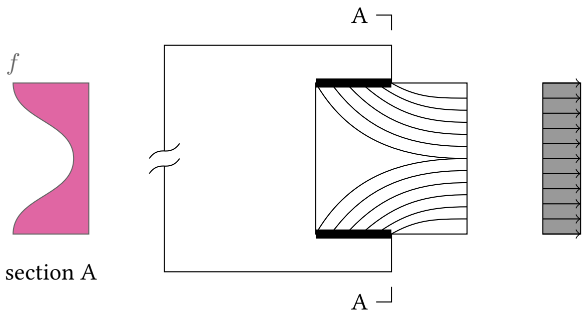
In other connections, the same effect can be seen. Factors affecting shear lag are

- proportion of section connected;

  It may be seen that the stress flow lines change direction and move together concentrating stress.



- length of weld or number of connects, more shear lag is expected with fewer connectors;



- change in direction of stress due to section shape, e.g., angle connected with fillet welds on both sides of one leg.

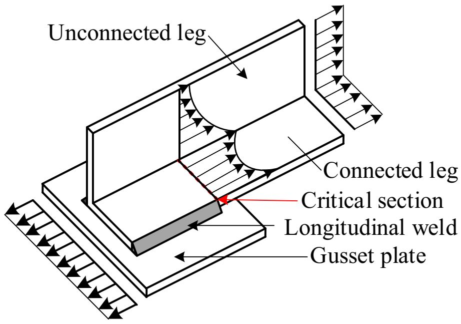


Figure 3.5: Shear lag in welded angle specimen (Dhanuskar and Gupta, 2021)

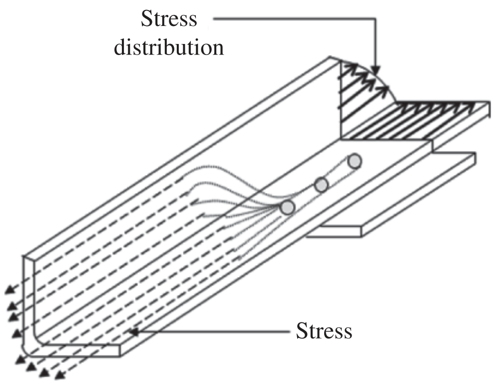


Figure 3.6: Shear lag in bolted angle specimen

The determination of k_{te} is explained in NZS 3404.1&2:1997 § 7.3.

Some codes define an effective net area $A_e = k_{te}A_n$. We will use the AISC recommendations, because they are clearer, and more reasonable than the AS/NZS approach. They are given in Fig. 3.7 (ANSI/AISC 360-16 Table D3.1), where $U = k_{te}$.

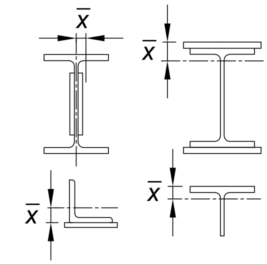
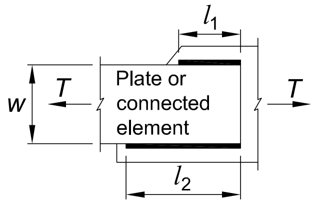
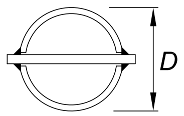
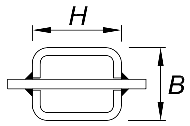
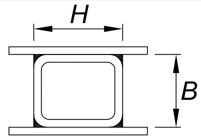
3.2 Tension Member Slenderness Limitations

Most codes do not have any requirements for slenderness. However, undesired flutter (or vibration) of slender tension members have been observed in some situations. The AISC-LRFD code makes the following recommendations (ANSI/AISC 360-16 § D1): For members designed on the basis of tension, the slenderness ratio, L/r, preferably **should not exceed 300**. This suggestion does not apply to rods or hangers in tension.

Example 3.2 *Worksheet Axial Tension Example — Slender UC*

Choose a $8\,\mathrm{m}$ *long UC section to carry* $G = 880\,\mathrm{kN}$ *and* $Q = 1200\,\mathrm{kN}$ *respectively with Grade 300 steel.*

Solution 3.2

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).		$U = 1.0$	–
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)		$U = 1 - \frac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	–
4[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.		$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$	
5	Round HSS with a single concentric gusset plate through slots in the HSS.		$l \geq 1.3D$, $U = 1.0$ $D \leq l < 1.3D$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{D}{\pi}$	
6	Rectangular HSS.	with a single concentric gusset plate	$l \geq H$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2}{4(B+H)}$	
7	W-, M-, S- or HP-shapes, or tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used.)	with flange connected with three or more fasteners per line in the direction of loading	$b_f \geq \frac{2}{3}d$, $U = 0.90$ $b_f < \frac{2}{3}d$, $U = 0.85$	–
		with web connected with four or more fasteners per line in the direction of loading	$U = 0.70$	–
8	Single and double angles. (If U is calculated per Case 2, the larger value is permitted to be used.)	with four or more fasteners per line in the direction of loading	$U = 0.80$	–
		with three fasteners per line in the direction of loading (with fewer than three fasteners per line in the direction of loading, use Case 2)	$U = 0.60$	–

B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); D = outside diameter of round HSS, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm); d = depth of section, in. (mm); for tees, d = depth of the section from which the tee was cut, in. (mm); l = length of connection, in. (mm); w = width of plate, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm).

[a] $l = \frac{l_1 + l_2}{2}$, where l_1 and l_2 shall not be less than 4 times the weld size.

Figure 3.7: Shear lag factors for connections to tension members

Assume $f_y = 280\,\text{MPa}$, the factored tension force is given by the critical load combination,

$$\begin{aligned} N_t^* = \sum \gamma_i E_i &= 1.2G + 1.5Q \\ &= 1.2 \times 880\,\text{kN} + 1.5 \times 1200\,\text{kN} \\ &= 2856\,\text{kN}. \end{aligned}$$

LRFD requires the following for material yielding on the gross area.

$$\begin{aligned} N_t^* &\leqslant \phi N_t \\ &\leqslant \phi A_g f_y, \\ A_g &\geqslant \frac{N_t^*}{\phi f_y} = 11\,330\,\text{mm}^2. \end{aligned}$$

Try 250UC89.5,

$$A_g = 11\,400\,\text{mm}^2 > 11\,330\,\text{mm}^2.$$

Check slenderness requirement, considering the minimum r value:

$$\frac{L}{r_{min}} = \frac{L}{r_y} = \frac{8000}{65.5} = 122.7 < 300, \qquad \text{Okay.}$$

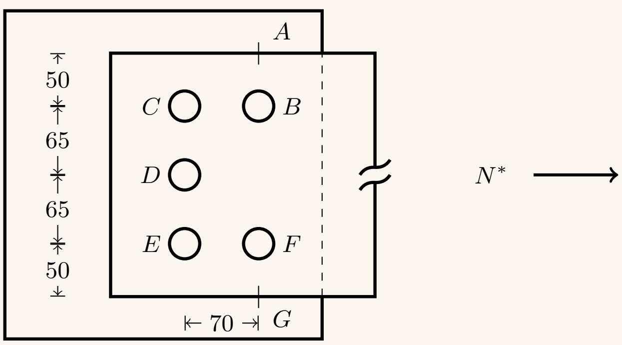
Since $t_f = 17.3\,\text{mm} > 17\,\text{mm}$, the assumed yield stress $f_y = 280\,\text{MPa}$ is correct, see, Table 2.1. Thus, a 250UC89.5 section works. Alternatively, the next section, 310UC96.8, also works but its heavier.

Note:

- The critical loading actions need to be combined for ultimate limit states (left hand side of the equation).
- Remember slenderness may impose additional requirements.

Example 3.3 *Worksheet Axial Tension Example — Staggered Bolt Connection*

Assuming Grade 300 steel and ϕ 12 mm bolts, the thickness of top plate is 12 mm, what is the maximum factored force (or design strength) in tension of the top plate shown below?



Solution 3.3

A member subjected to an axial tension force shall satisfy:

$$N^* \leqslant \min\left(\phi A_g f_y,\ \phi 0.85 k_{te} A_n f_u\right).$$

In this case, $k_{te} = 1$. The overall width b is

$$b = 2 \times 50\,\mathrm{mm} + 2 \times 65\,\mathrm{mm} = 230\,\mathrm{mm}.$$

The standard hole size is

$$d_h = 12\,\mathrm{mm} + 2\,\mathrm{mm} = 14\,\mathrm{mm}.$$

Because $t = 12\,\mathrm{mm} > 11\,\mathrm{mm}$, for Grade 300 steel, $f_y = 310\,\mathrm{MPa}$ and $f_u = 430\,\mathrm{MPa}$.

For gross area,

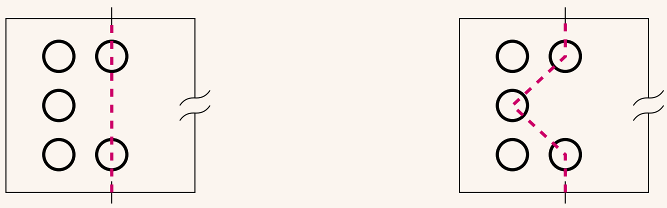
$$A_g f_y = b t f_y = 230\,\mathrm{mm} \times 12\,\mathrm{mm} \times 310\,\mathrm{MPa} = 855.6\,\mathrm{kN}.$$

For net area, find critical fracture line first. When fracture line is $ABFG$,

$$L_{ABFG} = b - 2d = 230\,\mathrm{mm} - 2 \times 14\,\mathrm{mm} = 202\,\mathrm{mm}. \qquad \rightarrow \qquad \text{critical}$$

When fracture line is $ABDFG$,

$$L_{ABDFG} = 230\,\mathrm{mm} + 2 \times \frac{(70\,\mathrm{mm})^2}{4 \times 65\,\mathrm{mm}} - 3 \times 14\,\mathrm{mm} = 225.7\,\mathrm{mm}.$$



Thus,

$$A_n = L_{ABFG} t = 202\,\mathrm{mm} \times 12\,\mathrm{mm} = 2424\,\mathrm{mm}^2.$$

Then,

$$0.85 k_{te} A_n f_u = 0.85 \times 1 \times 2424\,\mathrm{mm}^2 \times 430\,\mathrm{MPa} = 886\,\mathrm{kN}.$$

The maximum factored force is then

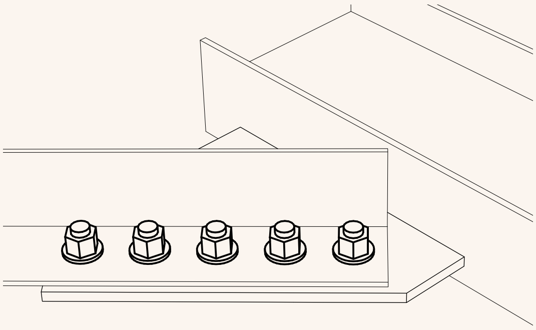
$$N^* \leqslant \min\left(0.9 \times 855.6\,\mathrm{kN},\ 0.9 \times 886\,\mathrm{kN}\right) = 770\,\mathrm{kN}.$$

$N^* = 770\,\mathrm{kN}$

For connected members, both failure mechanisms need to be calculated. This applies to not only bolted connection with area reduction, but also other types of connections.

Example 3.4 *Worksheet Axial Tension Example — Angle Connection*

Use Grade 300 steel, design a $2.7\,\mathrm{m}$ *single angle tension member connected to a plate with one leg (long leg if UA) using 5 M20 (*$\phi 20\,\mathrm{mm}$*) class 8.8 bolts in a line, with one at each section. Let* $G = 130\,\mathrm{kN}$ *and* $Q = 190\,\mathrm{kN}$.



Solution 3.4

The critical combination is

$$N_t^* = \max(1.35G,\ 1.2G + 1.5Q) = 441\,\mathrm{kN}.$$

Assume $f_y = 320\,\mathrm{MPa}$, the minimum gross area is

$$A_g \geqslant \frac{N_t^*}{\phi f_y} = \frac{441\,\mathrm{kN}}{0.9 \times 320\,\mathrm{MPa}} = 1531\,\mathrm{mm}^2.$$

According to case 8 in Fig. 3.7, $k_{te} = 0.8$, the minimum net area is

$$A_n \geqslant \frac{N_t^*}{\phi 0.85 k_{te} f_u} = \frac{441\,\mathrm{kN}}{0.9 \times 0.85 \times 0.8 \times 440\,\mathrm{MPa}} = 1638\,\mathrm{mm}^2.$$

Thus the net area governs. Try the following sections.

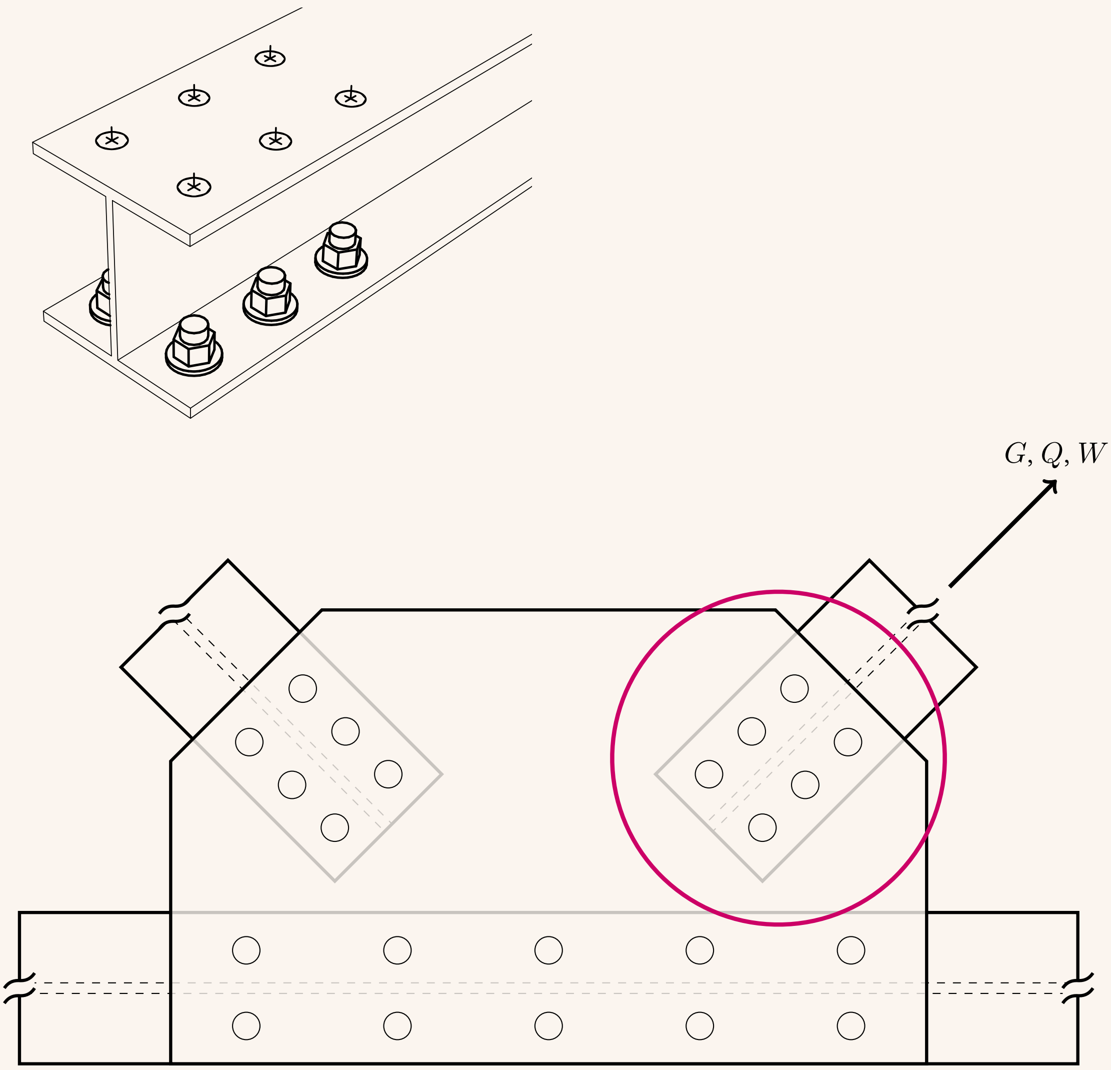
Section	A_g	t	r_{min}	A_n	
125×75×10UA	1810	9.5	16.2	1601	N.G.
100×100×10EA	1810	9.5	19.6	1601	N.G.
125×90×8UA	1820	7.8	19.7	1648.4	Okay
125×125×8EA	1900	7.8	24.8	1728.4	Okay
125×90×10UA	2200	9.5	19.6	1991	Okay
100×100×12EA	2260	12	19.5	1996	Okay

Thus use 125×90×8UA. The thickness is smaller than $11\,\mathrm{mm}$, thus the assumed yield stress is appropriate for Grade 300 steel.

Use 125×90×8UA

Example 3.5 *Worksheet Axial Tension Example — Bolted Flange UC*

Find the size of UC member carrying axial loads $G = 270\,\text{kN}$, $Q = 400\,\text{kN}$ and $W = 45\,\text{kN}$ using Grade 300 steel. The spacing of bolts is $80\,\text{mm}$ along the direction of axial force. Note bolted connections apply to both sides of the member.



Solution 3.5

The demand can be obtained via combinations.

$$\begin{aligned} N_t^* &= \max\left(1.35G,\ 1.2G + 1.5Q,\ 1.2G + \psi_c Q + W\right) \\ &= \max\left(364.5\,\text{kN},\ 924\,\text{kN},\ 529\,\text{kN}\right) \\ &= 924\,\text{kN}. \end{aligned}$$

Assume $f_y = 320\,\text{MPa}$, for yielding on gross area,

$$A_g \geqslant \frac{N_t^*}{\phi f_y} = \frac{924\,\text{kN}}{0.9 \times 320\,\text{MPa}} = 3208\,\text{mm}^2.$$

Try 150UC30.0, $A_g = 3860\,\text{mm}^2$ and $t_f = 9.4\,\text{mm}$. Since for each net section, there are 2 bolts per flange, totalling 4, the net area is

$$A_n = A_g - 4t_f d_h = 3860\,\text{mm}^2 - 4 \times 9.4\,\text{mm} \times 22\,\text{mm} = 3033\,\text{mm}^2.$$

Since there are three bolts in a line and $b_f > \frac{2}{3}d$, from Fig. 3.7, $k_{te} = 0.9$ according to case 7. Thus for fracture on net area,

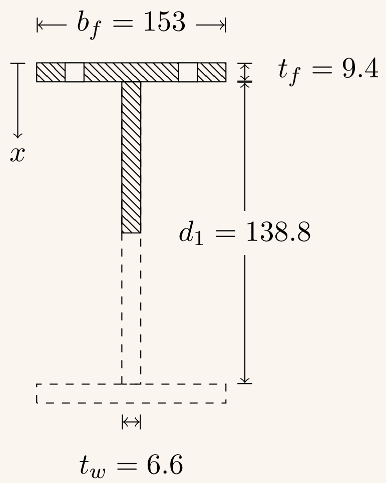
$$\phi 0.85 k_{te} A_n f_y = 0.9 \times 0.85 \times 0.9 \times 3033\,\text{mm}^2 \times 440\,\text{MPa} = 919\,\text{kN}.$$

The difference is only 0.5 %. Considering the ultimate strength would be greater, 150UC30.0 is okay.

Use case 2 in Table 3.7 to recheck the example, from which,

$$k_{te} = 1 - \frac{\bar{x}}{l}.$$

Since bolts are in both flanges, axial forces from half section goes to bolts on each side. Consider half section only, find eccentricity $\bar{x}$,



$$\begin{aligned}\bar{x} &= \frac{\sum x_i A_i}{\sum A_i} \\ &= \frac{(b_f - t_w)\cdot t_f \cdot t_f/2 + t_w \cdot d/2 \cdot d/4}{A_g/2} \\ &= \frac{(153 - 6.6)\cdot 9.4 \cdot 4.7 + 6.6 \cdot 78.8 \cdot 39.4}{1930}\,\text{mm} \\ &= 13.97\,\text{mm}.\end{aligned}$$

The total length of connection is $l = 2 \times 80\,\text{mm} = 160\,\text{mm}$,

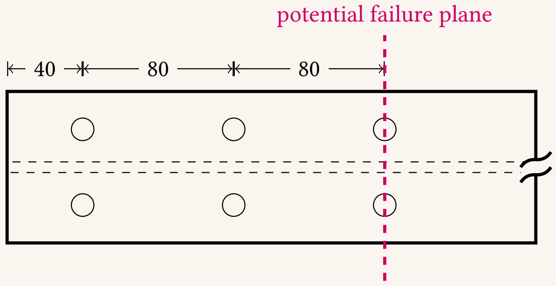
$$\begin{aligned}k_{te} &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{13.97\,\text{mm}}{160\,\text{mm}} \\ &= 0.913.\end{aligned}$$

Thus,

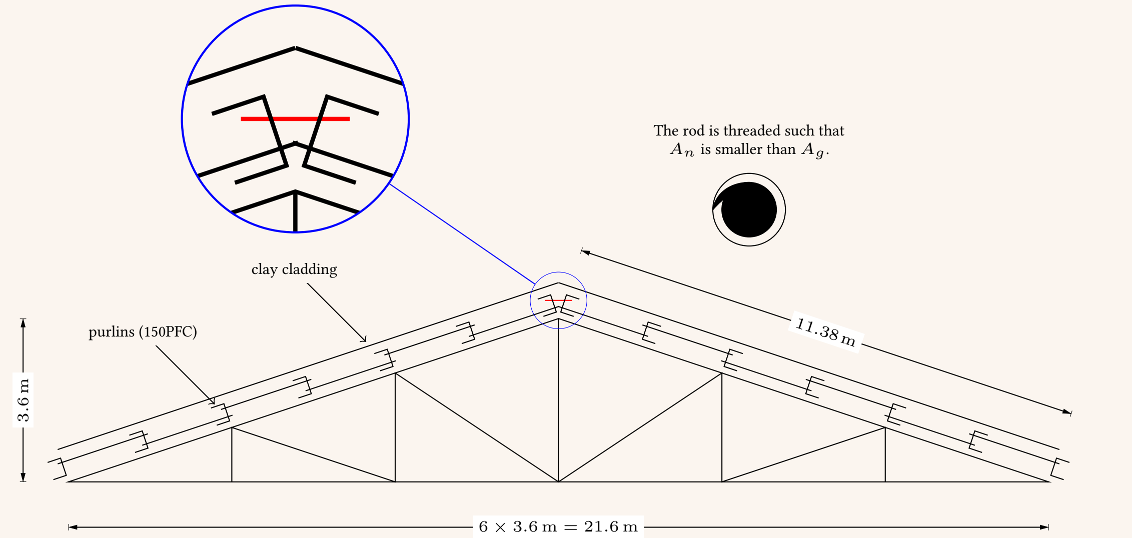
$$\phi 0.85 k_{te} A_n f_u = 0.9 \times 0.85 \times 0.913 \times 3033\,\text{mm}^2 \times 440\,\text{MPa} = 932\,\text{kN} > 924\,\text{kN}.$$

The general method (case 2) is less conservative than the specific one (case 7) in this case.

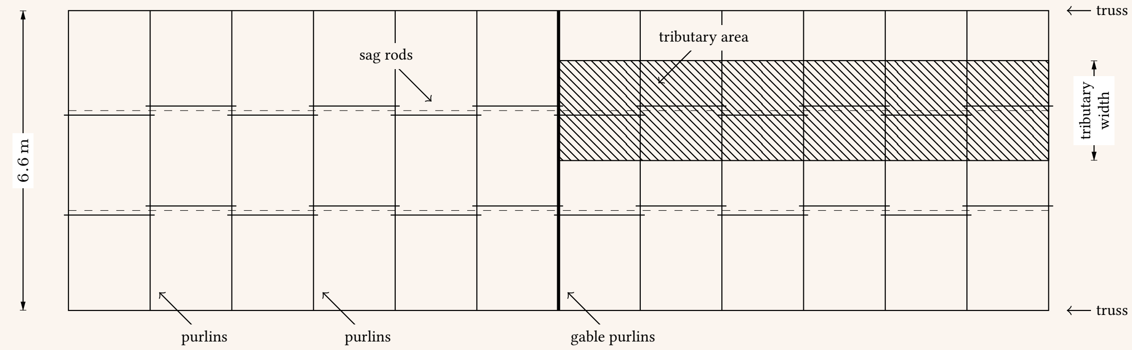
Use 150UC30.0



Example 3.6 *Sag rods, threaded over their whole length, are to be designed to support purlins parallel to the roof surface as shown below. This is because purlins are strong in bending about their major axis, but they are weak about their minor axis. To prevent possible rod damage during construction, use a minimum diameter of* 16 mm *and to prevent flutter, use* $L/d < 500$*. Assume Grade 300 steel.*



Elevation



Plan

Solution 3.6

The rods are to be threaded such that $A_n = 0.75A_g = 0.75 \times \dfrac{\pi d^2}{4}$. Since they are threaded over their whole length we only need to consider fracture,

$$\phi N_t = 0.9 \times 0.95 \times k_{te} A_n f_u \geqslant N_t^*.$$

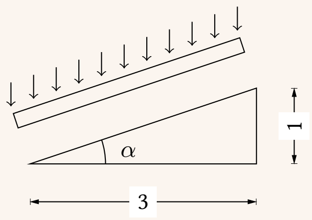
This gives

$$\begin{aligned} d(\text{in m}) &\geqslant \sqrt{\frac{4 \times N_t^*}{0.9 \times 0.85 \times k_{te} \times f_u \times 0.75 \times \pi}} \\ &= \sqrt{\frac{4 \times N_t^*}{0.9 \times 0.85 \times 1 \times 440\,\text{MPa} \times 0.75 \times \pi}} = 7.102 \times 10^{-5} \sqrt{N_t^*(\text{in N})}. \end{aligned}$$

We proceed to find N^* on the member.

1. Clay roofing: 800 Pa on flat roof, i.e., on flat roof, weight is 800 Pa. On sloping roof, the weight per horizontal distance is

$$\frac{800\,\text{Pa}}{\cos\alpha} = \frac{\sqrt{10}}{3} \times 800\,\text{Pa} = 843.3\,\text{Pa}.$$



2. Snow pressure: 1600 Pa (vertical)

3. Purlin equivalent force: 150PFC has a linear density of $17.7\,\text{kg}\,\text{m}^{-1}$, the horizontal spacing is 1.8 m, this gives

$$\frac{17.7\,\text{kg}\,\text{m}^{-1} \times 9.81\,\text{N}\,\text{kg}^{-1}}{1.8\,\text{m}} = 96.5\,\text{Pa}.$$

Collecting those terms, the design action

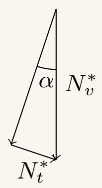
$$\begin{aligned} N_{v,Pa}^* &= 1.2G + S \\ &= 1.2 \times (843.3\,\text{Pa} + 96.5\,\text{Pa}) + 1600\,\text{Pa} = 2728\,\text{Pa} \end{aligned}$$

The equivalent total vertical force acting on sag rod is computed by accounting for tributary area. This leads to

$$N_{v,kN}^* = 2728\,\text{Pa} \times 10.8\,\text{m} \times 6.6\,\text{m}/3 = 64.8\,\text{kN}.$$

The critical tension force in sag rod is

$$N_t^* = N_{v,kN}^* \sin\alpha = 64.8\,\text{kN} \times \frac{1}{\sqrt{10}} = 20.5\,\text{kN}.$$



Thus the minimum diameter is

$$d = 7.102 \times 10^{-5}\sqrt{20500} = 1.017 \times 10^{-2}\,\text{m} = 10.17\,\text{mm} \leq 16\,\text{mm}.$$

Try $\phi 16\,\text{mm}$ rod

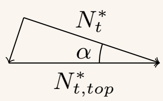
Check flutter.

$$\frac{L}{d} = \frac{10.8\,\text{m}/6 \times \frac{\sqrt{10}}{3}}{16\,\text{mm}} = 118.6 < 500 \tag{3.9}$$

Check force in top tie rod.

$$N_{t,top}^* = N_t^* \times \cos\alpha = 21.6\,\text{kN}.$$

The minimum diameter becomes $10.44\,\text{mm}$. Thus the chosen $16\,\text{mm}$ rod is still okay.



Use Grade 300 $\phi 16\,\text{mm}$ rod

Compression Members

Compression members include columns, truss members, braces, etc. Unlike tension members, their compressive strength is affected by multiple factors which are summarised below. They usually have more stocky sections than tension members to prevent premature strength loss due to buckling.

4.1 Factors Affecting Compressive Strength

4.1.1 Elastic Buckling

Consider the elastic member buckling in axial compression below.

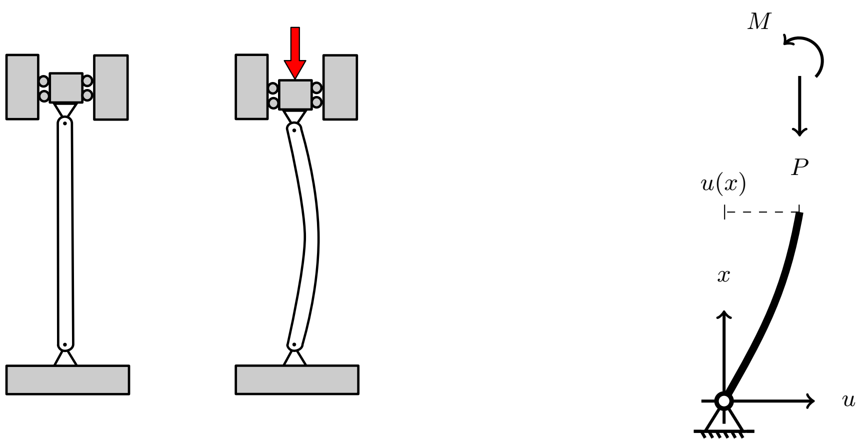


Figure 4.1: Elastic buckling of a pinned–pinned column

From the free body diagram, the deflection $u(x)$ of an elastic column of length L under axial load P is related to the bending moment M such that

$$Pu = M \qquad \longrightarrow \qquad M = -EI\frac{\mathrm{d}^2u}{\mathrm{d}x^2} \qquad \longrightarrow \qquad EI\frac{\mathrm{d}^2u}{\mathrm{d}x^2} + Pu = 0.$$

This is a homogeneous second order ODE. Techniques learnt in *EMTH 210* and *ENCN 304* can be applied to solve it subject to different boundary conditions. The general solution can be found as

$$u(x) = A\cos(\eta x) + B\sin(\eta x)$$

where $\eta = \sqrt{\dfrac{P}{EI}}$ (the conventional λ is retained to define other quantity), A and B are two constants.

For pinned–pinned member, $u(0) = u(L) = 0$, η can be found as

$$\eta = \frac{n\pi}{L}$$

with $n = 0, 1, 2, 3, \cdots$. Thus,

$$P_n = \frac{n^2\pi^2EI}{L^2}.$$

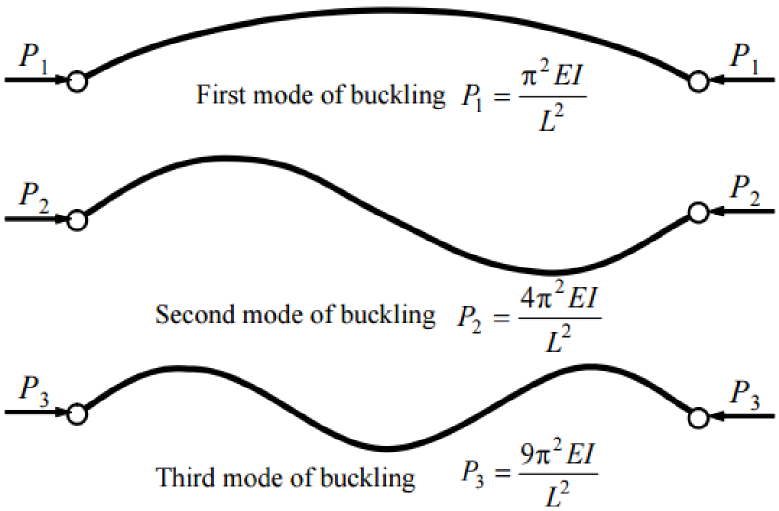


Figure 4.2: First three modes of buckling loads (https://en.wikipedia.org/wiki/Euler%27s_critical_load)

For non-trivial solution, the buckling load P_{cr} would be the lowest possible load, which is

$$P_{cr} = P_1 = \frac{\pi^2EI}{L^2}.$$

For other boundary conditions, similar procedures can be carried out.

In general, the buckling strength N_{om} of a column is expressed as

$$N_{om} = P_{cr} = \frac{\pi^2EI}{(k_eL)^2} = \frac{\pi^2EI}{L_e^2},$$

where k_e is the effective length factor with can be obtained by the above procedure for different boundary conditions, $L_e = k_e L$ is known as the effective length. It is the distance between buckling points of inflection for a flexural member. The following are a few examples.

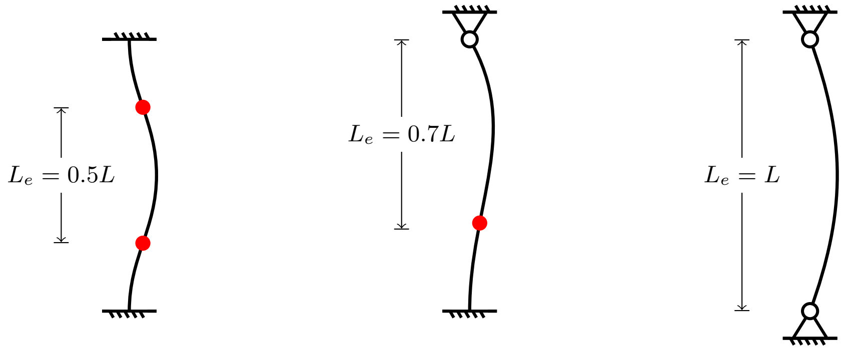


Figure 4.3: Illustration of effective length

Dividing N_{om} by cross section area, we define buckling stress f_{om} as

$$f_{om} = \frac{N_{om}}{A} = \underbrace{E}_{\text{material}} \cdot \underbrace{\frac{I}{A}}_{\text{section}} \cdot \underbrace{\frac{\pi^2}{(k_e L)^2}}_{\text{length}}.$$

Noting that the radius of gyration r is defined as $r = \sqrt{I/A}$, then

$$f_{om} = \frac{\pi^2 E}{(k_e L/r)^2}.$$

Given elastic modulus E is a constant, f_{om} is essentially a function of $k_e L/r$, we define the slenderness ratio λ as

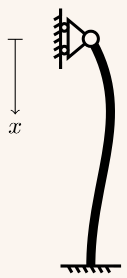
$$\lambda = \frac{k_e L}{r},$$

so that

$$f_{om} = \frac{\pi^2 E}{\lambda^2}.$$

Example 4.1 *Theoretical Buckling*

Find the theoretical buckling load of a column with bottom end fixed and top end pinned. The length of column is L. The flexural rigidity is EI.



Solution 4.1

The problem is equivalent to find the particular solution for the ODE

$$EI\frac{\mathrm{d}^2u}{\mathrm{d}x^2} + Pu = 0.$$

subjected to boundary conditions

$$\begin{aligned}
u(0) &= 0, & \longrightarrow \quad & \text{displacement is zero at top,}\\
u(L) &= 0, & \longrightarrow \quad & \text{displacement is zero at bottom,}\\
\frac{\mathrm{d}^2u}{\mathrm{d}x^2}(0) &= 0, & \longrightarrow \quad & \text{moment is zero at top,}\\
\frac{\mathrm{d}u}{\mathrm{d}x}(L) &= 0, & \longrightarrow \quad & \text{rotation is zero at bottom.}
\end{aligned}$$

The critical load P is the smallest value of the solved solution.

Since in this particular problem, second order derivative appears in BC, more advanced techniques (casting to a fourth order ODE) are needed to find the general solution, which can eventually be expressed as

$$u(x) = A\cos(\eta x) + B\sin(\eta x) + Cx + D.$$

Using $u(0) = 0$ and $\frac{\mathrm{d}^2u}{\mathrm{d}x^2}(0) = 0$,

$$\begin{aligned}
u(0) &= A + D = 0, & \longrightarrow \quad & A = -D,\\
\frac{\mathrm{d}^2u}{\mathrm{d}x^2}(0) &= -A\eta^2 = 0, & \longrightarrow \quad & A = D = 0.
\end{aligned}$$

Since $\eta \neq 0$ for non-trivial solution. The general solution after apply two BCs can be expressed as

$$u(x) = B\sin(\eta x) + Cx.$$

Now apply $\frac{\mathrm{d}u}{\mathrm{d}x}(L) = 0$,

$$\frac{\mathrm{d}u}{\mathrm{d}x}(L) = B\eta\cos(\eta L) + C = 0, \qquad \longrightarrow \qquad C = -B\eta\cos(\eta L).$$

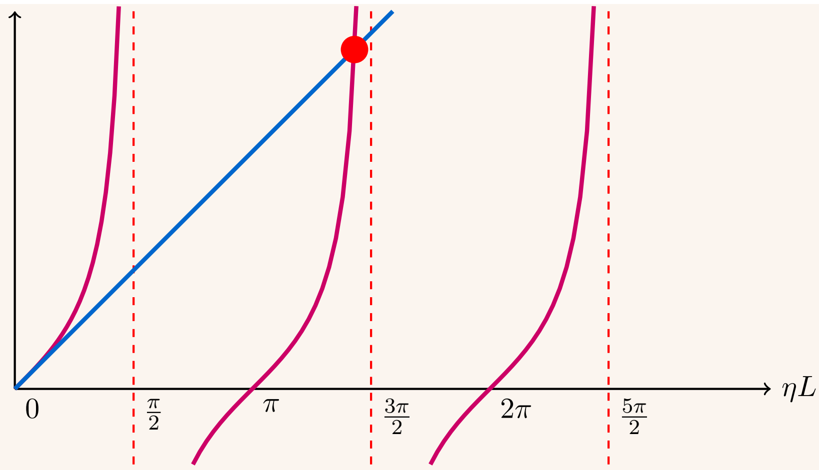
Apply the last BC,

$$u(L) = B\sin(\eta L) + CL = 0, \qquad \longrightarrow \qquad B\left(\sin(\eta L) - \eta L\cos(\eta L)\right) = 0.$$

In order to solve for non-trivial solution, B cannot be zero otherwise all A, B, C and D would be zeros, leads to trivial solution. Thus,

$$\sin(\eta L) - \eta L\cos(\eta L) = 0, \qquad \longrightarrow \qquad \tan(\eta L) = \eta L.$$

This is a transcendental equation. The approximate solution can be found via numerical, analytical approximations, or graphical methods.



The first non-trivial solution is $\eta L \approx 4.4934$. This gives

$$P_{cr} = \frac{4.4934^2 EI}{L^2} = \frac{\pi^2 EI}{(0.699L)^2}, \qquad \longrightarrow \qquad k_e = 0.699.$$

For this case, k_e is often taken as 0.7.

For elastic material, the critical stress f_{cr} is simply f_{om}, which approaches infinity when λ approaches zero (i.e., very short columns). If one plots f_{cr} with regard to λ, the following graph can be obtained.

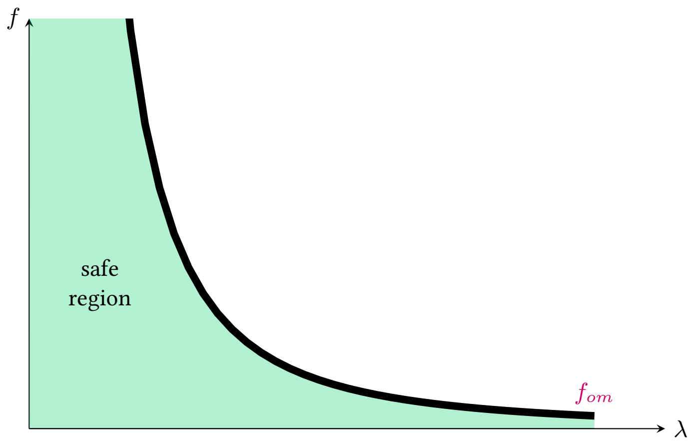


Figure 4.4: Theoretical governing region for f_{cr} for elastic material

4.1.2 Compressive Strength

However, for real life situations, no matter how small λ is, a f_{om} greater than yield strength f_y is meaningless. The yield strength f_y is often considered as a material property that does not vary with λ. The critical stress f_{cr} shall be the minimum of f_y and f_{om}. If one draws f_{cr} against λ, the following graph can be obtained. It affects short members.

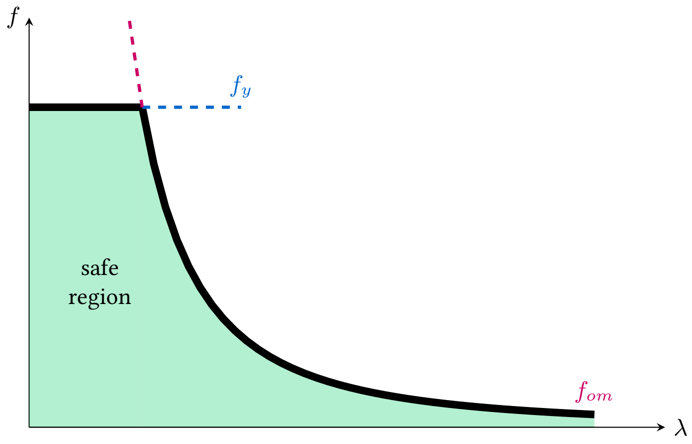


Figure 4.5: Theoretical governing region for f_{cr}

4.1.3 Local Buckling

Section can buckle under complex stress conditions. The buckling of web/flanges would further reduce section compression capacity. If often depends on flange/web slenderness (b_f/t_f and d/t_w). It affects short members.

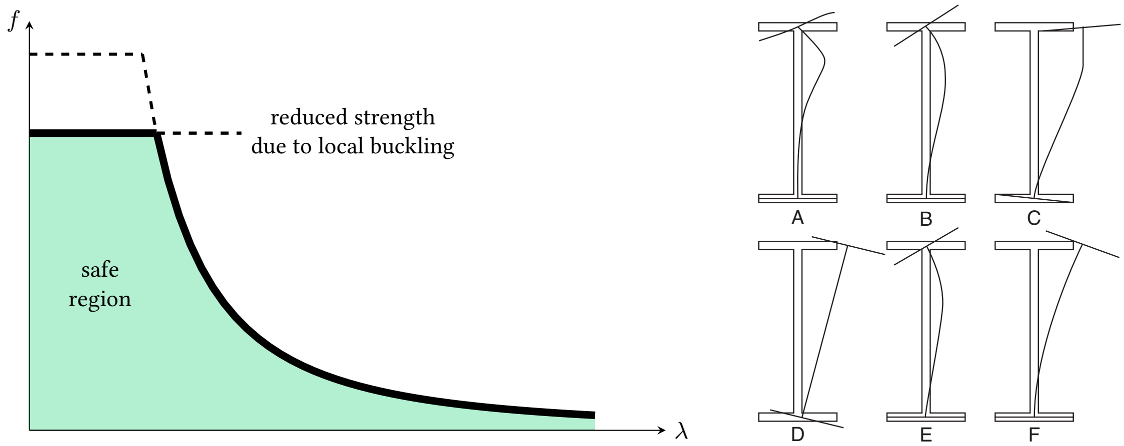


Figure 4.6: Governing region considering local buckling

4.1.4 Residual Stress

All members contain residual stresses due to uneven cooling. These result in earlier yielding and a reduction in elastic modulus.

Due to residual stresses, the tangent stiffness E_T decreases gradually so that $E_T < E$. Recall $f_{om} = \frac{\pi^2 E}{\lambda^2}$, when E decreases, f_{om} would decease accordingly as $\frac{\pi^2 E_T}{\lambda^2} < \frac{\pi^2 E}{\lambda^2}$.

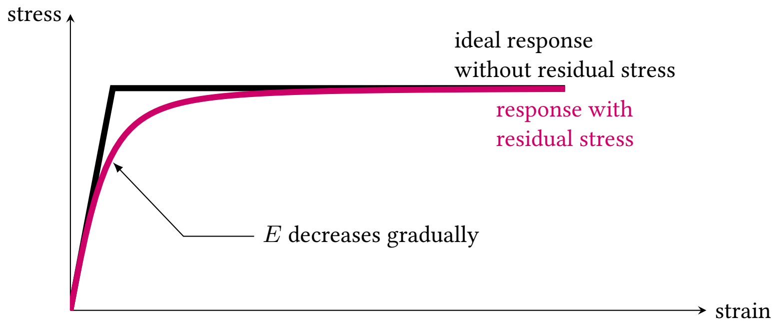


Figure 4.7: Stress–strain curve considering residual stress

The effect is most significant on members of moderate slenderness, where the strength is reduced.

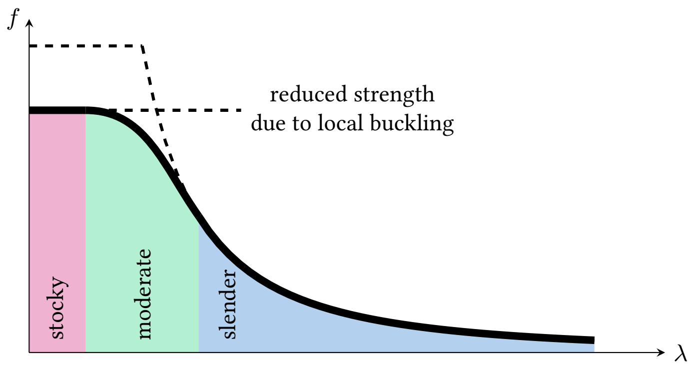


Figure 4.8: Governing region considering residual stress

4.1.5 Initial Out-of-Straightness and Eccentrical Loading

An eccentricity $e = L/1500$ is commonly assumed. This is less than the maximum out-of-straightness camber of $L/1000$. The eccentricity introduces non-zero section moment ever if the member is axially loaded. This additional moment would lower theoretical critical buckling force.For straight members, the loading behaviour follows $O \rightarrow A \rightarrow B$ but the actual behaviour due to eccentricity is the dashed line.

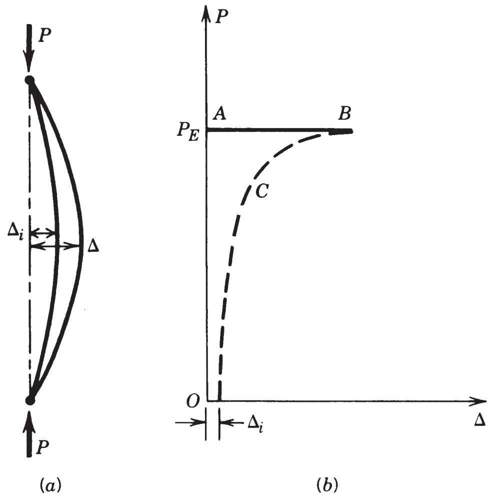


Figure 4.9: Effect of initial out-of-straightness on column behaviour

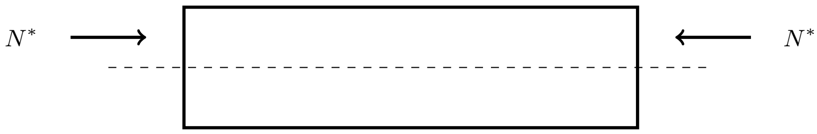


Figure 4.10: Eccentrical loading

The eccentrical loading has a similar effect to out-of-straightness and thus can be taken into account in a similar way. Both out-of-straightness and eccentrical loading affect slender members in a similar way.

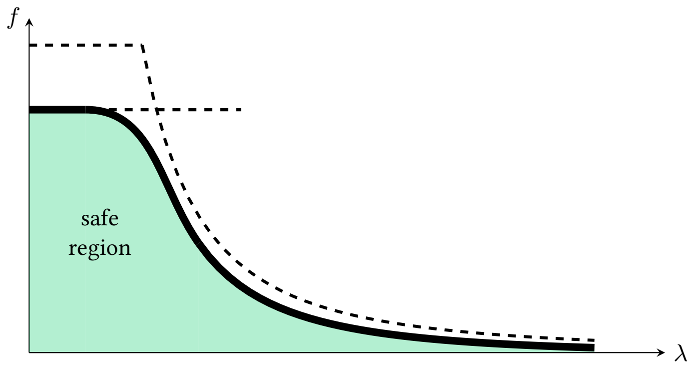


Figure 4.11: Governing region considering initial OOS and eccentric loading

» 4.2 Strength Design Concept

The LRFD equation for columns has a similar form NZS 3404.1&2:1997 § 6.1 to that of tension members.

$$N^* \leqslant \phi N_c = \phi \alpha_c N_s, \tag{4.1}$$

where

ϕ = strength reduction factor, 0.9
N_s = nominal section capacity
N_c = nominal member capacity

The design member strength ϕN_c requires calculation of the section capacity N_s, and the member slenderness factor α_c.

4.2.1 Section Capacity

The nominal section capacity N_s is determined as:

$$N_s = k_f A_n f_y, \tag{4.2}$$

where

k_f = form factor
A_n = net area
f_y = yield stress

The form factor k_f accounts for local buckling. It is defined to be the ratio between the effective area A_e and gross area A_g.

$$k_f = \frac{A_e}{A_g},$$

where $A_e = \sum b_{e,i} t_i$ and $b_{e,i}$ and t_i are the effective width and thickness of each **individual** element. For a flat plate with thickness of t and **clear** width of b (outstand from the face of the supporting plate element, or between the faces of the supporting plate elements), the effective width b_e can be computed as

$$b_e = \min\left(b,\ \lambda_{ey} t \sqrt{\frac{250\,\mathrm{MPa}}{f_y}}\right), \tag{4.3}$$

in which λ_{ey} is the yield slenderness limit. For an I-section, values of b_i and t_i are shown below. There are 7 elements in total, 4 belong to flange which are one-edge supported while 1 belongs to web which is both-edge supported. Note the corresponding $b_{e,i}$ may be smaller than the physical length b_i.

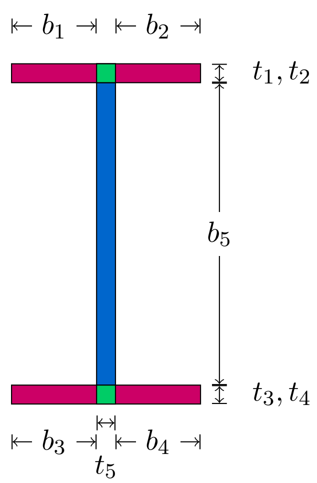


Figure 4.12: Five flat elements in a typical I section

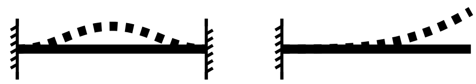
If the slenderness for any element, $\lambda_{e,i}$, exceeds the yield limit $\lambda_{ey,i}$, the excess width is neglected when calculated the effective width $b_{e,i}$ for that element. Some universal beam sections have k_f values slightly less than one. For flat plates, NZS 3404.1&2:1997 Table 6.2.4 gives the following limits shown in Fig. 4.13.

An element, which has the same slenderness as the yield limit λ_{ey}, should just reach the yield stress before local buckling occurs. Elements which have higher levels of residual stresses from rolling and welding will reach the yield stress earlier and therefore have lower values of λ_{ey}.

The allowable slenderness of stiffened elements is much greater than that for unstiffened elements.

longitudinal edges supported	residual stresses	λ_{ey}
one	SR	16
	HR	16
	LW, CF	15
	HW	14
both	SR	45
	HR	45
	LW, CF	40
	HW	35

SR — stress relived
HR — hot-rolled
CF — cold-formed
LW — lightly welded longitudinally
HW — heavily welded longitudinally



both edges supported stiffened, S

one edge supported unstiffened, U

The following illustrations show one and both edges supported for common sections.

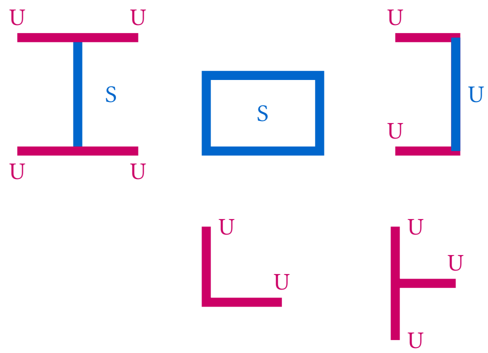
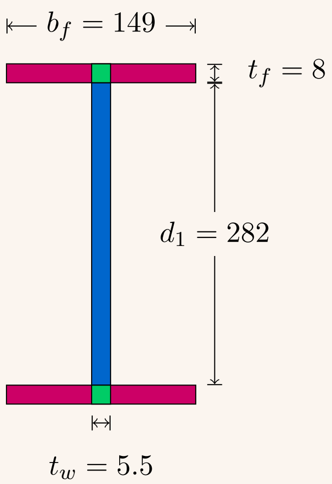


Figure 4.13: Yield slenderness limits of flat plate elements

Often k_f is provided with section parameters in the product specification manual.

Example 4.2 *Assume Grade 300 steel, find the section capacity N_s of a 310UB32.0 section.*

Solution 4.2



This is a hot-rolled section. Now compute the effective area A_e. For flanges, only one side is supported. Thus, $\lambda_{ey,f} = 16$.

$$b_{e,f} = 16 \times 8\,\text{mm} \times \sqrt{\frac{250}{320}} = 113.14\,\text{mm}.$$

This is greater than the clear width $b = \dfrac{b_f - t_w}{2} = 71.75\,\text{mm}$, hence $b_{e,f} = b = 71.75\,\text{mm}$. Hence for flange alone, $k_f = 1$. For web, both sides are supported. Thus, $\lambda_{ey,w} = 45$.

$$b_{e,w} = 45 \times 5.5\,\text{mm} \times \sqrt{\frac{250}{320}} = 218.76\,\text{mm}.$$

This is smaller than $d_1 = 282\,\text{mm}$. Hence for web alone, $k_f < 1$.

The effective area is then

$$A_e = \sum b_{e,i}t_i = \underbrace{218.76\,\mathrm{mm} \times 5.5\,\mathrm{mm}}_{\text{web, blue}} + \underbrace{4 \times 71.75\,\mathrm{mm} \times 8\,\mathrm{mm}}_{\text{flange, red}}$$
$$+ \underbrace{2 \times 5.5\,\mathrm{mm} \times 8\,\mathrm{mm}}_{\text{connection, green}} = 3587.18\,\mathrm{mm}^2.$$

The gross area is

$$A_g = 2 \times b_f \times t_f + d_1 \times t_w = 3935\,\mathrm{mm}^2.$$

This value is smaller than the value listed in Liberty catalogue ($4080\,\mathrm{mm}^2$) because the transition radii are not considered in the computation here.

The form factor $k_f = A_e/A_g = 0.912$, which is smaller than the value listed (see the table in Liberty catalogue) of 0.915. Since there is no area reduction, $A_n = A_g$.

*** If the area of transition regions is taken as the difference between two gross areas, that is $4080\,\mathrm{mm}^2 - 3935\,\mathrm{mm}^2 = 145\,\mathrm{mm}^2$, adding it to both numerator and denominator, k_f can be computed as

$$k_f = \frac{A_e}{A_g} = \frac{3587.18\,\mathrm{mm}^2 + 145\,\mathrm{mm}^2}{4080\,\mathrm{mm}^2} = 0.915,$$

which agrees with the value shown in tables.

To compute section capacity, we use the listed values such that

$$N_s = k_f A_g f_y = 0.915 \times 4080\,\mathrm{mm}^2 \times 320\,\mathrm{MPa} = 1195\,\mathrm{kN}.$$

4.2.2 Member Capacity

Before we introduce how to calculate **member** capacity, the concept of modified slenderness ratio λ_n shall be discussed.

Since buckling plays a vital role in compression members, we can no more use the full length to compute slenderness ratio, instead, the effective length L_e shall be used.

$$L_e = k_e L, \tag{4.4}$$

where k_e is the member effective length factor (NZS 3404.1&2:1997 Fig. 4.8.3.2).

The modified slenderness ratio λ_n is defined as

$$\lambda_n = \lambda\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{L_e}{r}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}}. \tag{4.5}$$

If $f_y = 250\,\mathrm{MPa}$ and $k_f = 1$, then

$$\lambda_n = \lambda = \frac{L_e}{r}.$$

The nominal member capacity N_s is associated with the nominal section capacity N_c via a reduction factor α_c:

$$N_c = \alpha_c N_s, \tag{4.6}$$

in which,

$$\alpha_c = \xi\left(1-\sqrt{1-\left(\frac{90}{\xi\lambda'}\right)^2}\right), \qquad \xi = \frac{\left(\frac{\lambda'}{90}\right)^2+1+\eta}{2\left(\frac{\lambda'}{90}\right)^2},$$

with

$$\lambda' = \lambda_n + \alpha_a\alpha_b, \qquad \eta = \max\left(0.00326\left(\lambda'-13.5\right),\ 0\right), \qquad \alpha_a = \frac{2100\left(\lambda_n-13.5\right)}{\lambda_n^2-15.3\lambda_n+2050},$$

and α_b is the compression member section constant which are shown in Table 4.1 that characterises sectional residual stress effects (also refer to NZS 3404.1&2:1997 Table 6.3.3(1)). For a given α_b, α_c is a function of λ_n, it can be plotted as shown in Fig. 4.14.

Table 4.1: Values of α_b

section type	k_f	α_b
UB and UC sections, hot-rolled ($t_f \leqslant 40$ mm), Box sections, welded	$\leqslant 1.0$	0.0
UB and UC sections, hot-rolled ($t_f > 40$ mm)	$\leqslant 1.0$	1.0
RHS and CHS, hot-formed	$= 1.0$	-1.0
RHS and CHS, cold-formed (stress relieved)	< 1.0	-0.5
RHS and CHS, cold-formed (non-stress relieved)	$\leqslant 1.0$	-0.5
H and I sections, welded from flame cut plate ($t_f \leqslant 40$ mm)	$= 1.0$	0.0
	< 1.0	0.5
H and I sections, welded from flame cut plate ($t_f > 40$ mm)	$= 1.0$	0.0
	< 1.0	1.0
H and I sections, welded from as-rolled plate ($t_f \leqslant 40$ mm)	$\leqslant 1.0$	0.5
H and I sections, welded from as-rolled plate ($t_f > 40$ mm)	$\leqslant 1.0$	1.0
Tee sections, flame cut from universal sections, Angles, Channels, hot-rolled	$= 1.0$	0.5
	< 1.0	1.0

It can be inferred that since α_c accounts for buckling, residual stress, OOS, etc., it has a shape resembles the theoretical curve as seen previously.

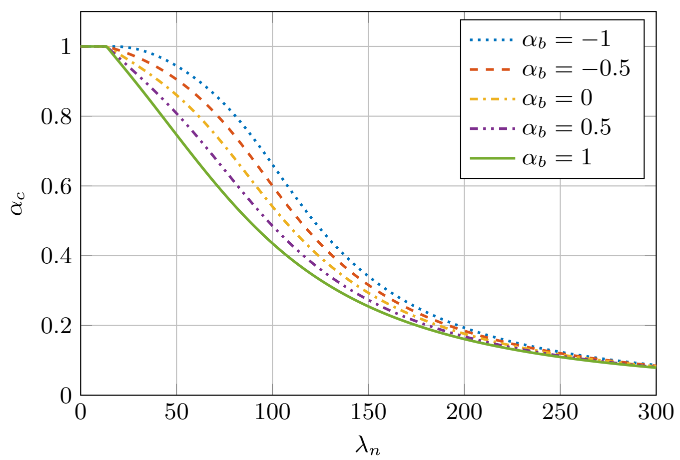


Figure 4.14: α_c as a function of λ_n

Performing such a computation is somehow tedious, in practice, engineers can refer to the pre-compiled table NZS 3404.1&2:1997 Table 6.3.3(2), as well as Table 4.2, to find the proper value to be used.

Table 4.2: Values of α_c as function of α_b and λ_n

	α_b						α_b						α_b				
λ_n	-1	-0.5	0	0.5	1	λ_n	-1	-0.5	0	0.5	1	λ_n	-1	-0.5	0	0.5	1
0	1.000	1.000	1.000	1.000	1.000												
2	1.000	1.000	1.000	1.000	1.000	102	0.646	0.586	0.528	0.474	0.426	202	0.190	0.181	0.173	0.165	0.158
4	1.000	1.000	1.000	1.000	1.000	104	0.630	0.571	0.515	0.463	0.416	204	0.186	0.178	0.170	0.163	0.156
6	1.000	1.000	1.000	1.000	1.000	106	0.615	0.557	0.502	0.452	0.407	206	0.182	0.174	0.167	0.160	0.153
8	1.000	1.000	1.000	1.000	1.000	108	0.599	0.542	0.489	0.441	0.398	208	0.179	0.171	0.164	0.157	0.151
10	1.000	1.000	1.000	1.000	1.000	110	0.584	0.528	0.477	0.431	0.389	210	0.176	0.168	0.161	0.154	0.148
12	1.000	1.000	1.000	1.000	1.000	112	0.568	0.515	0.465	0.421	0.381	212	0.172	0.165	0.158	0.152	0.146
14	1.000	0.999	0.998	0.997	0.997	114	0.553	0.501	0.454	0.411	0.372	214	0.169	0.162	0.156	0.149	0.144
16	1.000	0.996	0.992	0.987	0.983	116	0.539	0.488	0.442	0.401	0.364	216	0.166	0.159	0.153	0.147	0.141
18	1.000	0.992	0.985	0.977	0.970	118	0.524	0.476	0.431	0.392	0.356	218	0.163	0.156	0.150	0.145	0.139
20	1.000	0.989	0.978	0.967	0.956	120	0.510	0.463	0.421	0.383	0.348	220	0.160	0.154	0.148	0.142	0.137
22	0.999	0.985	0.971	0.957	0.943	122	0.496	0.451	0.410	0.374	0.341	222	0.157	0.151	0.145	0.140	0.135
24	0.998	0.981	0.965	0.947	0.929	124	0.483	0.439	0.400	0.365	0.334	224	0.154	0.148	0.143	0.138	0.133
26	0.996	0.977	0.958	0.937	0.916	126	0.470	0.428	0.390	0.357	0.327	226	0.152	0.146	0.141	0.136	0.131
28	0.994	0.973	0.951	0.927	0.902	128	0.457	0.417	0.381	0.349	0.320	228	0.149	0.143	0.138	0.133	0.129
30	0.991	0.968	0.943	0.917	0.888	130	0.445	0.406	0.372	0.341	0.313	230	0.146	0.141	0.136	0.131	0.127
32	0.988	0.963	0.936	0.907	0.874	132	0.433	0.396	0.363	0.333	0.306	232	0.144	0.139	0.134	0.129	0.125
34	0.985	0.958	0.928	0.896	0.860	134	0.421	0.386	0.354	0.325	0.300	234	0.141	0.136	0.132	0.127	0.123
36	0.981	0.952	0.921	0.886	0.846	136	0.410	0.376	0.346	0.318	0.294	236	0.139	0.134	0.130	0.125	0.121
38	0.977	0.946	0.913	0.875	0.832	138	0.399	0.367	0.337	0.311	0.288	238	0.137	0.132	0.128	0.123	0.119
40	0.973	0.940	0.905	0.865	0.818	140	0.389	0.357	0.330	0.304	0.282	240	0.134	0.130	0.126	0.122	0.118
42	0.968	0.934	0.896	0.854	0.804	142	0.378	0.349	0.322	0.298	0.276	242	0.132	0.128	0.124	0.120	0.116
44	0.962	0.927	0.888	0.842	0.789	144	0.369	0.340	0.314	0.291	0.270	244	0.130	0.126	0.122	0.118	0.114
46	0.957	0.920	0.879	0.831	0.775	146	0.359	0.332	0.307	0.285	0.265	246	0.128	0.124	0.120	0.116	0.113
48	0.951	0.913	0.870	0.820	0.761	148	0.350	0.324	0.300	0.279	0.260	248	0.126	0.122	0.118	0.115	0.111
50	0.944	0.905	0.861	0.808	0.747	150	0.341	0.316	0.293	0.273	0.255	250	0.124	0.120	0.116	0.113	0.110
52	0.938	0.897	0.851	0.796	0.732	152	0.332	0.309	0.287	0.267	0.250	252	0.122	0.118	0.115	0.111	0.108
54	0.931	0.889	0.841	0.784	0.718	154	0.324	0.301	0.281	0.262	0.245	254	0.120	0.116	0.113	0.110	0.107
56	0.923	0.880	0.830	0.771	0.704	156	0.316	0.294	0.274	0.256	0.240	256	0.118	0.115	0.111	0.108	0.105
58	0.915	0.871	0.820	0.759	0.690	158	0.308	0.287	0.268	0.251	0.235	258	0.116	0.113	0.110	0.107	0.104
60	0.907	0.862	0.809	0.746	0.676	160	0.301	0.281	0.263	0.246	0.231	260	0.115	0.111	0.108	0.105	0.102
62	0.899	0.852	0.797	0.733	0.662	162	0.294	0.274	0.257	0.241	0.226	262	0.113	0.110	0.107	0.104	0.101
64	0.890	0.842	0.785	0.720	0.649	164	0.287	0.268	0.252	0.236	0.222	264	0.111	0.108	0.105	0.102	0.099
66	0.881	0.832	0.773	0.707	0.635	166	0.280	0.262	0.246	0.231	0.218	266	0.110	0.106	0.104	0.101	0.098
68	0.871	0.821	0.761	0.694	0.622	168	0.274	0.257	0.241	0.227	0.214	268	0.108	0.105	0.102	0.099	0.097
70	0.861	0.809	0.748	0.680	0.609	170	0.267	0.251	0.236	0.222	0.210	270	0.106	0.103	0.101	0.098	0.096
72	0.851	0.797	0.735	0.667	0.596	172	0.261	0.246	0.231	0.218	0.206	272	0.105	0.102	0.099	0.097	0.094
74	0.840	0.785	0.722	0.653	0.583	174	0.255	0.240	0.227	0.214	0.202	274	0.103	0.101	0.098	0.095	0.093
76	0.829	0.772	0.708	0.639	0.570	176	0.250	0.235	0.222	0.210	0.199	276	0.102	0.099	0.097	0.094	0.092
78	0.817	0.759	0.695	0.626	0.558	178	0.244	0.230	0.218	0.206	0.195	278	0.100	0.098	0.095	0.093	0.091
80	0.805	0.746	0.681	0.612	0.545	180	0.239	0.225	0.213	0.202	0.192	280	0.099	0.096	0.094	0.092	0.089
82	0.792	0.732	0.667	0.599	0.533	182	0.233	0.221	0.209	0.198	0.188	282	0.097	0.095	0.093	0.091	0.088
84	0.779	0.718	0.653	0.586	0.522	184	0.228	0.216	0.205	0.195	0.185	284	0.096	0.094	0.092	0.089	0.087
86	0.766	0.704	0.638	0.572	0.510	186	0.224	0.212	0.201	0.191	0.182	286	0.095	0.092	0.090	0.088	0.086
88	0.752	0.689	0.624	0.559	0.499	188	0.219	0.208	0.197	0.187	0.178	288	0.093	0.091	0.089	0.087	0.085
90	0.737	0.675	0.610	0.547	0.487	190	0.214	0.203	0.193	0.184	0.175	290	0.092	0.090	0.088	0.086	0.084
92	0.723	0.660	0.596	0.534	0.477	192	0.210	0.199	0.190	0.181	0.172	292	0.091	0.089	0.087	0.085	0.083
94	0.708	0.645	0.582	0.521	0.466	194	0.206	0.196	0.186	0.178	0.169	294	0.090	0.088	0.086	0.084	0.082
96	0.692	0.630	0.568	0.509	0.456	196	0.201	0.192	0.183	0.174	0.167	296	0.089	0.087	0.085	0.083	0.081
98	0.677	0.615	0.554	0.497	0.445	198	0.197	0.188	0.179	0.171	0.164	298	0.087	0.085	0.084	0.082	0.080
100	0.661	0.600	0.541	0.485	0.435	200	0.194	0.185	0.176	0.168	0.161	300	0.086	0.084	0.082	0.081	0.079

The most conservative curve is that with $\alpha_b = 1$. The European codes, like NZS 3404.1&2:1997 Steel structures standard, have five curves for α_b. The Canadian code has 3 curves for different residual stress conditions, while the US code has just one curve for all residual stress conditions.

Example 4.3 *What is the maximum stress that can be applied to a hot-rolled compact Grade 300 column (no holes) with $t_f < 11\,\mathrm{mm}$ and $\lambda = 100$?*

Solution 4.3

Since it is compact section, shear lag is often not a problem. It can be safely assumed that $k_f = 1$. Since there are no holes, $A_n = A_g$. From Table 4.1, $\alpha_b = 0$ for hot-rolled section. For $t_f < 11\,\mathrm{mm}$, assume $f_y = 320\,\mathrm{MPa}$ for Grade 300 steel.

The modified slenderness ratio

$$\lambda_n = \lambda\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = 100 \times \sqrt{1} \times \sqrt{\frac{320}{250}} = 113.1.$$

α_c can be computed (or found from Table 4.2, or Fig. 4.14) as 0.459. Thus, the maximum stress is

$$f_{max} = \frac{N^*}{A_n} \leqslant \frac{\phi N_c}{A_n} = \phi\alpha_c k_f f_y = 0.9 \times 0.459 \times 1 \times 320\,\mathrm{MPa} = 132.2\,\mathrm{MPa}.$$

Example 4.4 *A $10\,\mathrm{m}$ long 310UB32.0 beam is braced laterally (pinned) at its ends for buckling about its strong axis, and braced (pinned) at the ends and at the* ***quarter*** *points for buckling about its weak axis. Find the maximum axial compressive force it can carry.*

Solution 4.4

Both strong axis and weak axis need to be considered. For a 310UB32 section, the following properties can be found: $\alpha_b = 0$, $A_n = A_g = 4080\,\mathrm{mm}^2$, $f_y = 320\,\mathrm{MPa}$, $k_f = 0.915$, $r_x = 124\,\mathrm{mm}$, $r_y = 32.9\,\mathrm{mm}$.

- strong axis (x-axis)
  Since it is pinned at both ends, $L_e = 10\,\mathrm{m}$, the modified slenderness ratio is

  $$\lambda_n = \frac{L_e}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{10\,\mathrm{m}}{124\,\mathrm{mm}} \times \sqrt{0.915} \times \sqrt{\frac{320}{250}} = 87.3.$$

  Then, $\alpha_c = 0.629$. The maximum compressive force is then

  $$\phi N_{c,x} = \phi\alpha_c k_f A_n f_y = 0.9 \times 0.629 \times 0.915 \times 4080\,\mathrm{mm}^2 \times 320\,\mathrm{MPa} = 676.6\,\mathrm{kN}.$$

  This value is also given in Table 4.3.

- weak axis (y-axis)
  Since it is pinned at quarter points, $L_e = 0.25 \times 10\,\mathrm{m} = 2.5\,\mathrm{m}$ the modified slenderness ratio is

  $$\lambda_n = \frac{L_e}{r_y}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{2.5\,\mathrm{m}}{32.9\,\mathrm{mm}} \times \sqrt{0.915} \times \sqrt{\frac{320}{250}} = 82.2.$$

Then, $\alpha_c = 0.665$. The maximum compressive force is then

$$\phi N_{c,y} = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.665 \times 0.915 \times 4080\,\text{mm}^2 \times 320\,\text{MPa} = 715.0\,\text{kN}.$$

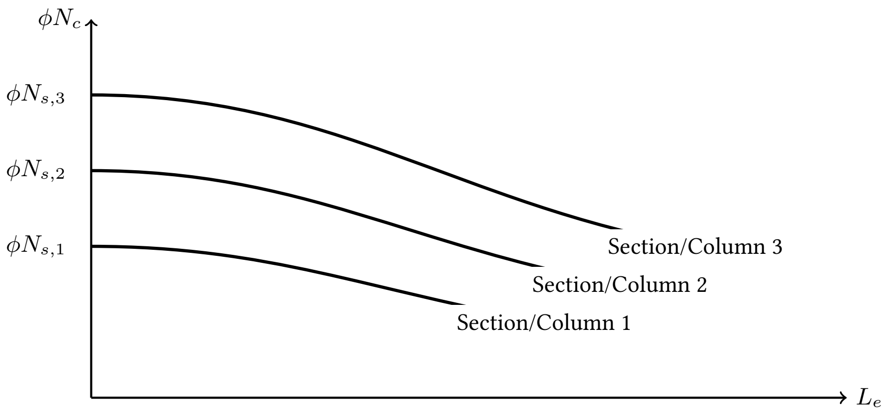
This value is also given in Table 4.4.

The governing maximum compressive force shall be the minimum of two, thus $\phi N_c = 676.6\,\text{kN}$. In practice, **always** check both axes for capacity.

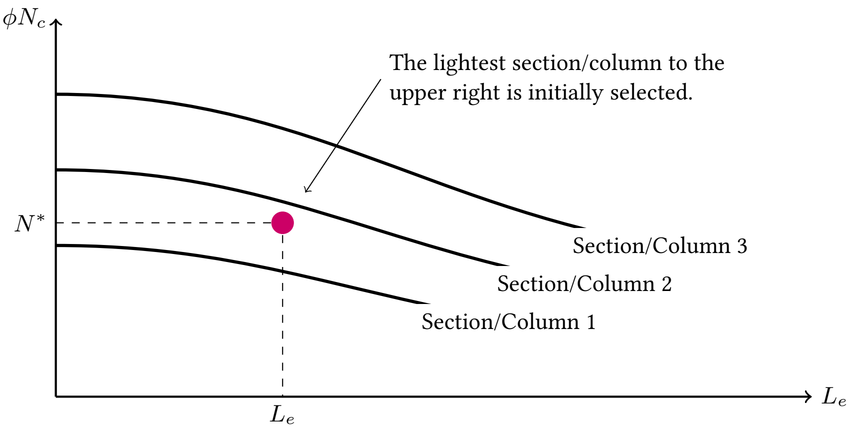
Note, if $L_e = 0\,\text{m}$, then $\phi N_c = \phi N_s = 0.9 \times 0.915 \times 4080\,\text{mm}^2 \times 320\,\text{MPa} = 1075.2\,\text{kN}$, which are given by both tables for strong- and weak-axis buckling.

4.2.3 Development and Use of Design Chart/Table

By varying the effective length L_e, for a typical section, the corresponding member section capacities can be computed and compiled into design charts/tables. A typical design chart may look like the following.



To use those charts, for a given N^* and a given L_e, locate this point on the chart and pick the lightest section/column above.



Always check both axes for capacity.

4.3 Design Procedure

Any column that satisfies all strength and serviceability criteria will be a satisfactory design.

The most economical satisfactory design for a given situation will be dictated by fabrication costs for the whole frame and by architectural considerations. A rolled section will usually be cheaper than a fabricated section. A tubular section will usually require a smaller area than an I section, but a RHS will be more expensive to buy than an I section and more expensive to connect. Fabricated tubes are relatively expensive to make, but may be used for heavier loads.

In general, for a column of the given cross section, the lightest column is likely to be the most economic because it uses the least steel material.

Design of compression members may be carried out by the following approaches.

Trial and Error

A number of section sizes are chosen, and evaluated to determine the best (safe and economical) solution. The following steps may be useful for rolled shapes:

1. compute the factored compression force N^*
2. assume flange yield stress f_y (e.g., $280\,\mathrm{MPa}$, $300\,\mathrm{MPa}$.) and form factor $k_f = 1$
3. assume a proper gyration radius r (around $50\,\mathrm{mm}$) and compute the modified slenderness ratio λ_n
4. compute α_c
5. compute the minimum net area $A_{n,min} = \dfrac{N^*}{\phi \alpha_c k_f f_y}$
6. select a section
7. update k_f and λ_n according to section properties
8. compute ϕN_c
   (a) if $N^* < \phi N_c$, check if a smaller section would work in order to obtain the most economical section
   (b) if $N^* > \phi N_c$, select a larger section and repeat the previous steps

Tables and Charts

Design capacity tables and charts may be developed for specific sections to speed up the design. Given that effective length L_e may affect member capacity, for all designations, it is possible to calculate and compile the corresponding capacities under different L_e values. Fig. 4.15 and Fig. 4.16 are two examples. Similar charts/tables can be seen in design books and other references.

Alternatively, by using property table, similar design tables can be generated. Table 4.3, Table 4.4, Table 4.5 and Table 4.6 are examples.

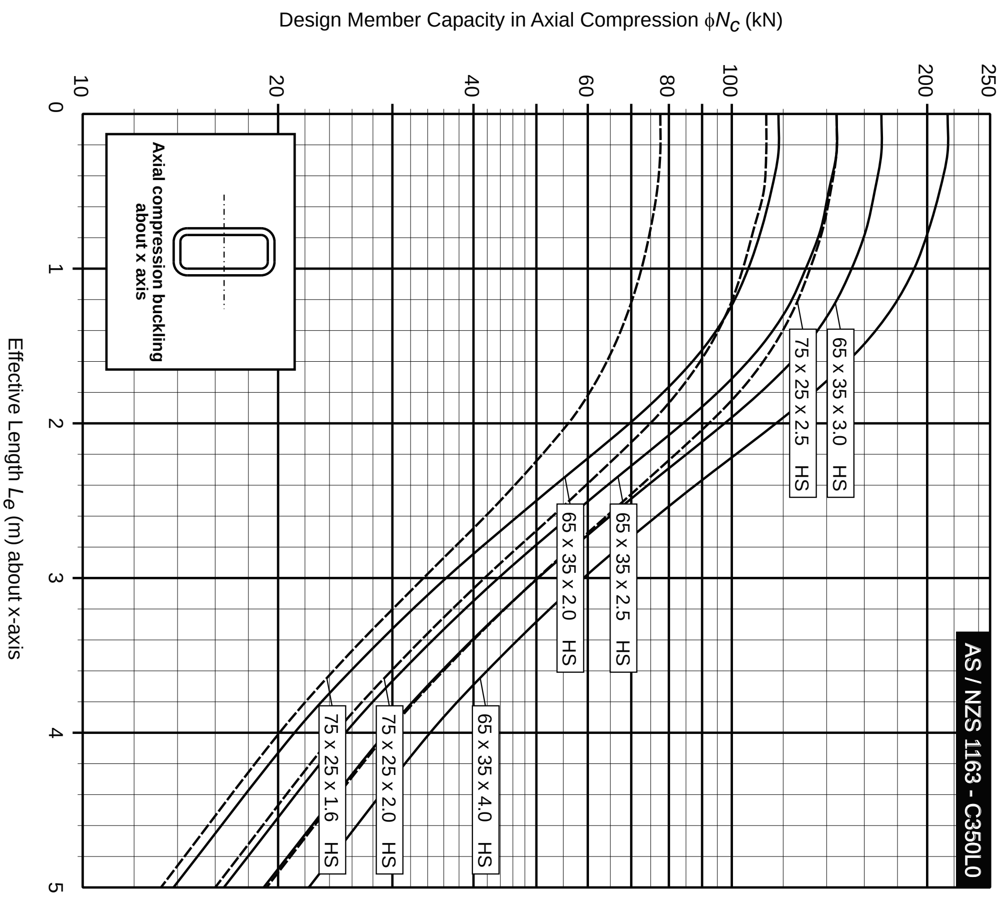
GRADE 300 STEEL

Designation		ϕN_s	Design Load Capacities for Axial Compression ϕN_c (kN) for Effective Length (L_e) in metres																			Percent holes to affect net Area, A_n
	kg/m	0	0.5	1	1.5	2	2.25	2.5	2.75	3	3.25	3.5	3.75	4	4.25	4.5	4.75	5	6	7	8	9	10	
310UC	137	4410	4410	4410	4310	4210	4150	4100	4040	3980	3920	3850	3780	3700	3630	3540	3460	3370	2960	2540	2150	1810	1540	25%
310UC	96.8	3320	3320	3310	3230	3150	3110	3060	3020	2970	2910	2860	2800	2740	2670	2600	2530	2460	2130	1800	1500	1260	1060	20%
250UC	89.5	2870	2870	2850	2770	2680	2640	2590	2540	2490	2430	2370	2310	2240	2170	2090	2020	1930	1610	1310	1070	884	738	25%
250UC	72.9	2510	2510	2480	2410	2330	2290	2250	2200	2150	2100	2040	1980	1910	1840	1770	1700	1620	1330	1070	871	715	595	20%
200UC	59.5	2060	2060	2010	1930	1850	1800	1750	1690	1630	1560	1490	1410	1330	1260	1180	1110	1040	794	617	489	396	327	20%
200UC	52.2	1810	1810	1760	1690	1620	1570	1530	1480	1420	1360	1300	1230	1160	1100	1030	965	903	691	537	426	345	284	20%
200UC	46.2	1600	1600	1560	1500	1430	1390	1350	1300	1250	1200	1140	1080	1020	960	900	842	787	601	466	370	299	246	20%
150UC	37.2	1290	1280	1220	1150	1060	1010	951	889	824	759	696	637	582	532	487	447	411	300	227	177	142	117	20%
150UC	30.0	1110	1110	1050	986	904	855	801	744	685	627	572	521	474	432	395	361	331	241	182	142	114	93.2	14%
150UC	23.4	867	861	815	760	691	650	605	558	511	465	422	382	347	315	287	263	241	174	131	102	82.1	67.2	14%
100UC	14.8	544	527	478	410	323	280	242	210	182	160	140	124	111	99.3	89.4	80.9	73.5	52.2	38.9	30.1	24.0	19.6	14%

GRADE 350 STEEL

Designation		ϕN_s	Design Load Capacities for Axial Compression ϕN_c (kN) for Effective Length (L_e) in metres																			Percent holes to affect net Area, A_n
	kg/m	0	0.5	1	1.5	2	2.25	2.5	2.75	3	3.25	3.5	3.75	4	4.25	4.5	4.75	5	6	7	8	9	10	
310UC	283	10700	10700	10700	10500	10200	10100	9920	9780	9620	9460	9290	9110	8920	8720	8510	8290	8050	7040	5990	5030	4230	3570	19%
310UC	198	7740	7740	7710	7520	7330	7220	7120	7000	6880	6760	6630	6490	6340	6180	6010	5830	5650	4870	4090	3400	2840	2390	17%
310UC	137	5360	5350	5330	5200	5050	4980	4900	4820	4740	4650	4550	4450	4340	4230	4100	3980	3850	3290	2740	2270	1890	1590	17%
310UC	96.8	3760	3760	3740	3650	3550	3490	3440	3380	3320	3250	3180	3110	3030	2940	2860	2760	2670	2270	1880	1550	1290	1080	17%
250UC	89.5	3490	3490	3440	3330	3220	3150	3090	3020	2940	2860	2770	2680	2580	2470	2370	2260	2150	1730	1380	1110	909	754	17%
250UC	72.9	2850	2850	2800	2710	2620	2570	2510	2450	2390	2320	2250	2170	2090	2000	1910	1820	1730	1390	1110	890	728	604	17%
200UC	59.5	2340	2340	2270	2170	2060	2000	1940	1860	1780	1700	1610	1520	1420	1330	1240	1160	1080	814	628	496	401	330	17%
200UC	52.2	2050	2050	1980	1900	1810	1750	1690	1630	1560	1480	1400	1320	1240	1160	1080	1010	938	709	546	432	348	287	17%
200UC	46.2	1920	1920	1860	1770	1680	1620	1570	1500	1430	1350	1270	1190	1110	1040	963	894	830	622	478	377	304	250	12%
150UC	37.2	1460	1450	1370	1280	1170	1110	1030	955	877	801	728	662	602	548	499	457	419	304	229	179	143	117	17%
150UC	30.0	1250	1240	1180	1090	991	930	863	793	723	656	594	538	488	443	403	368	337	244	184	143	115	93.7	12%
150UC	23.4	975	966	910	842	756	704	649	592	536	484	437	394	356	322	293	267	244	176	133	103	82.5	67.5	12%
100UC	14.8	612	590	530	444	340	291	250	215	186	162	142	126	112	100	90.2	81.6	74.1	52.5	39.1	30.2	24.1	19.6	12%

Figure 4.15: Design load capacity table for members subject to axial compression buckling about weak axis



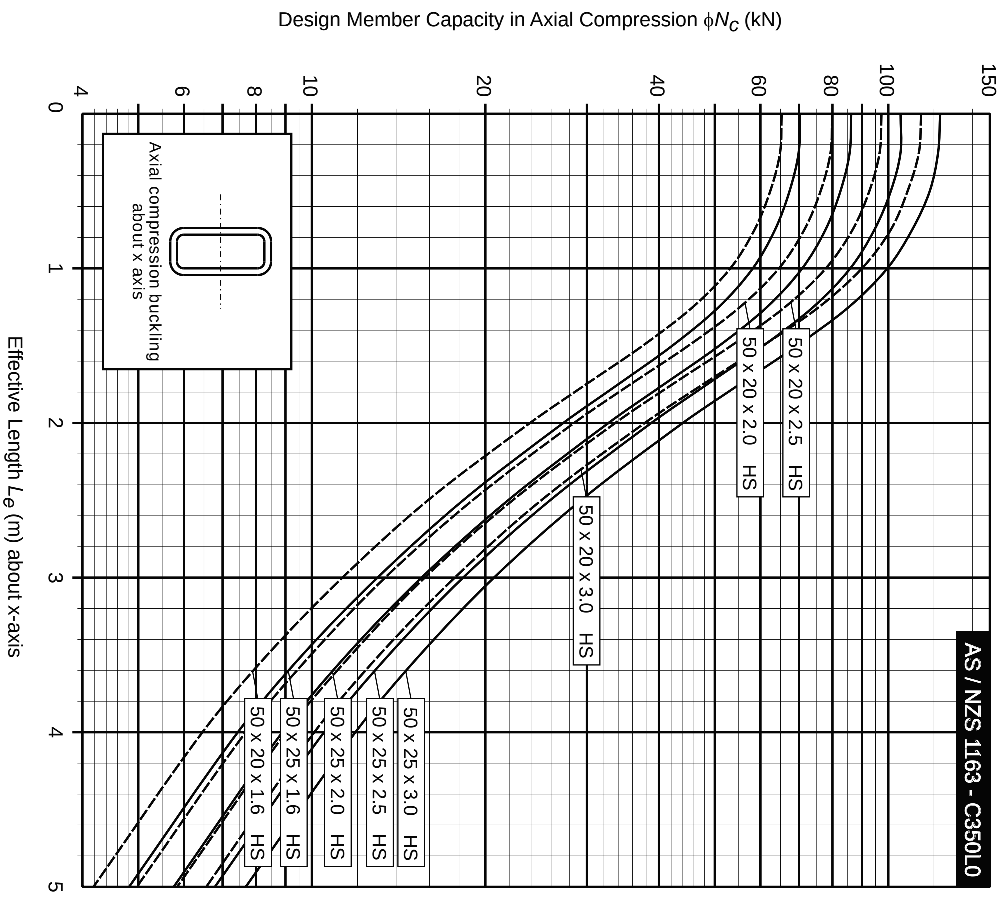


Figure 4.16: Design capacity chart for members subject to axial compression

Table 4.3: Design load capacity table for UB members subject to axial compression buckling about strong axis (manually generated)

Grade 300 Steel UB Section Subject to Axial Compression **Strong** Axis Buckling

	ϕN_s (kN)	ϕN_c (kN)																							
L_e (m)	0	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9	9.5	10	11	12	13	14	15	16	17	18
610UB125	3830.4	3830.4	3830.4	3830.4	3817.6	3791.1	3764.5	3737.7	3710.7	3683.3	3655.5	3627.2	3598.3	3568.8	3538.7	3507.7	3475.9	3443.2	3374.7	3301.6	3223.5	3139.9	3050.6	2955.7	2855.5	2750.5
113	3383.6	3383.6	3383.6	3383.6	3372.4	3349.0	3325.6	3301.9	3278.0	3253.8	3229.3	3204.3	3178.9	3152.8	3126.2	3098.9	3070.8	3042.0	2981.5	2917.0	2848.1	2774.4	2695.6	2611.9	2523.5	2430.9
101	3116.9	3116.9	3116.9	3116.9	3102.0	3079.8	3057.5	3034.9	3012.2	2989.1	2965.7	2941.8	2917.4	2892.5	2866.9	2840.6	2813.5	2785.6	2727.0	2664.4	2597.3	2525.4	2448.5	2366.9	2280.9	2191.2
530UB92.4	2956.6	2956.6	2956.6	2945.9	2921.8	2897.7	2873.3	2848.6	2823.5	2797.8	2771.6	2744.7	2717.0	2688.4	2658.8	2628.2	2596.5	2563.5	2493.5	2417.8	2336.0	2248.3	2155.2	2057.6	1957.0	1855.1
82.0	2557.2	2557.2	2557.2	2547.3	2526.5	2505.5	2484.3	2462.8	2440.9	2418.6	2395.8	2372.4	2348.3	2323.4	2297.7	2271.0	2243.4	2214.6	2153.6	2087.6	2016.2	1939.7	1858.6	1773.6	1686.1	1597.5
460UB82.1	2775.5	2775.5	2767.0	2740.3	2713.3	2686.1	2658.3	2629.9	2600.8	2570.7	2539.6	2507.2	2473.5	2438.4	2401.6	2363.0	2322.6	2280.3	2189.7	2091.2	1985.8	1875.4	1762.2	1649.1	1538.4	1432.3
74.6	2436.7	2436.7	2431.2	2408.1	2384.8	2361.3	2337.4	2313.0	2287.9	2262.1	2235.4	2207.7	2178.9	2148.8	2117.4	2084.5	2050.1	2014.1	1936.9	1853.0	1763.1	1668.4	1571.0	1473.0	1376.7	1283.8
67.1	2135.9	2135.9	2131.4	2111.2	2090.9	2070.3	2049.4	2028.1	2006.2	1983.7	1960.4	1936.2	1911.1	1884.9	1857.5	1828.9	1798.9	1767.5	1700.3	1627.2	1548.8	1466.3	1381.2	1295.6	1211.3	1129.9
410UB59.7	1934.9	1934.9	1920.1	1899.6	1878.9	1857.8	1836.3	1814.1	1791.1	1767.2	1742.4	1716.3	1689.0	1660.3	1630.0	1598.2	1564.7	1529.6	1454.4	1373.5	1288.6	1202.0	1116.1	1033.2	954.6	881.5
53.7	1811.7	1811.7	1794.3	1774.3	1754.1	1733.5	1712.4	1690.6	1667.9	1644.4	1619.7	1593.8	1566.5	1537.7	1507.4	1475.5	1441.8	1406.5	1331.1	1250.6	1167.1	1083.1	1001.0	922.7	849.6	782.2
360UB56.7	1947.0	1939.4	1915.4	1891.3	1866.6	1841.2	1814.9	1787.5	1758.8	1728.6	1696.6	1662.7	1626.7	1588.5	1548.1	1505.3	1460.5	1413.7	1315.6	1214.5	1114.2	1018.1	928.4	846.3	772.1	705.5
50.7	1682.3	1676.5	1656.1	1635.4	1614.3	1592.7	1570.3	1546.9	1522.5	1496.8	1469.6	1440.9	1410.3	1378.0	1343.7	1307.5	1269.5	1229.7	1146.3	1060.0	973.9	891.1	813.5	742.2	677.6	619.5
44.7	1532.0	1524.7	1505.5	1486.1	1466.2	1445.8	1424.6	1402.5	1379.3	1354.7	1328.8	1301.2	1271.9	1240.7	1207.8	1173.0	1136.5	1098.4	1019.2	938.0	858.2	782.3	712.0	648.0	590.4	539.0
310UB46.2	1586.7	1569.3	1546.9	1524.0	1500.4	1475.9	1450.1	1422.9	1393.9	1362.9	1329.8	1294.4	1256.5	1216.3	1173.8	1129.4	1083.5	1036.6	942.4	851.2	766.3	689.3	620.7	560.2	507.1	460.4
40.4	1428.5	1411.2	1390.6	1369.5	1347.7	1325.0	1301.2	1275.8	1248.9	1220.0	1189.1	1155.9	1120.5	1082.9	1043.2	1001.9	959.4	916.2	829.9	747.3	671.0	602.4	541.7	488.3	441.5	400.6
32.0	1075.2	1061.0	1045.1	1028.9	1012.1	994.5	976.0	956.4	935.4	912.8	888.7	862.8	835.1	805.7	774.8	742.7	709.8	676.6	610.7	548.3	491.1	440.1	395.1	355.8	321.4	291.4
250UB37.3	1368.0	1333.8	1309.1	1283.3	1256.1	1227.0	1195.5	1161.4	1124.2	1083.8	1040.2	993.7	944.9	894.5	843.7	793.4	744.4	697.4	611.1	536.0	471.7	417.1	370.7	331.1	297.3	268.3
31.4	1154.9	1123.7	1102.1	1079.5	1055.6	1029.9	1002.1	971.8	938.6	902.6	863.8	822.6	779.6	735.6	691.5	648.3	606.6	567.0	494.9	432.9	380.2	335.7	298.0	266.0	238.6	215.2
25.7	893.7	870.6	854.3	837.2	819.1	799.8	778.8	756.0	731.2	704.2	675.1	644.1	611.6	578.2	544.7	511.5	479.4	448.7	392.5	343.9	302.4	267.2	237.4	212.0	190.3	171.6
200UB29.8	1100.2	1053.8	1028.1	1000.5	970.3	936.9	899.9	858.9	814.0	766.0	716.0	665.5	615.9	568.5	524.0	482.8	445.1	410.9	351.8	303.5	263.9	231.3	204.1	181.4	162.2	145.8
25.4	930.2	889.2	866.8	842.7	816.2	786.9	754.2	718.1	678.6	636.5	593.1	549.5	507.2	467.0	429.5	395.1	363.8	335.4	286.7	247.0	214.6	187.9	165.8	147.3	131.7	118.3
22.3	826.6	790.2	770.3	748.9	725.5	699.4	670.5	638.4	603.4	566.1	527.6	488.9	451.3	415.6	382.3	351.7	323.8	298.6	255.2	219.9	191.1	167.4	147.7	131.2	117.2	105.4
18.2	661.5	630.5	614.0	596.1	576.4	554.4	529.9	502.8	473.3	442.0	410.1	378.5	348.1	319.6	293.2	269.2	247.4	227.8	194.3	167.2	145.1	127.0	112.0	99.4	88.8	79.8
180UB22.2	812.2	763.9	740.0	713.5	683.7	649.9	612.0	570.5	526.8	482.6	439.7	399.5	362.5	329.2	299.4	272.9	249.4	228.6	193.6	165.7	143.3	125.0	110.0	97.5	87.0	78.1
18.1	662.4	622.0	602.2	580.1	555.1	526.8	495.1	460.5	424.2	387.7	352.6	319.8	289.8	262.9	238.9	217.6	198.8	182.1	154.1	131.9	114.0	99.4	87.5	77.5	69.1	62.1
16.1	587.5	551.1	533.3	513.5	491.0	465.5	436.9	405.8	373.3	340.7	309.5	280.4	253.9	230.2	209.1	190.4	173.8	159.2	134.7	115.2	99.6	86.9	76.4	67.7	60.4	54.2
150UB18.0	662.4	609.9	585.1	556.8	524.0	486.7	445.9	403.7	362.4	323.8	289.0	258.2	231.4	208.0	187.6	170.0	154.5	141.0	118.7	101.1	87.2	75.9	66.6	58.9	52.5	47.1
14.0	512.6	470.0	450.0	426.9	400.2	369.8	337.0	303.4	271.0	241.3	214.7	191.4	171.2	153.7	138.5	125.4	113.9	103.9	87.3	74.4	64.1	55.7	48.9	43.3	38.6	34.6

Table 4.4: Design load capacity table for UB members subject to axial compression buckling about weak axis (manually generated)

Grade 300 Steel UB Section Subject to Axial Compression **Weak** Axis Buckling																											
	ϕN_s (kN)	ϕN_c (kN)																									
L_e (m)	0	2	2.25	2.5	2.75	3	3.25	3.5	3.75	4	4.25	4.5	4.75	5	5.25	5.5	5.75	6	6.25	6.5	6.75	7	7.25	7.5	7.75	8
610UB125	3830.4	3440.5	3353.6	3259.2	3156.6	3045.1	2924.7	2796.1	2661.0	2521.5	2380.2	2240.0	2103.2	1971.7	1846.8	1729.5	1619.9	1518.1	1423.9	1336.9	1256.6	1182.5	1114.3	1051.3	993.1	939.4
113	3383.6	3035.9	2958.4	2874.3	2782.6	2683.1	2575.6	2460.9	2340.5	2216.3	2090.9	1966.5	1845.4	1729.2	1619.0	1515.6	1419.1	1329.6	1246.8	1170.4	1099.9	1034.9	975.1	919.8	868.9	821.8
101	3116.9	2774.8	2698.6	2615.4	2524.6	2425.8	2319.4	2206.5	2089.0	1969.1	1849.4	1732.3	1619.6	1512.7	1412.4	1319.0	1232.5	1152.8	1079.4	1011.9	949.9	893.0	840.6	792.4	748.1	707.1
530UB92.4	2956.6	2585.7	2502.7	2411.3	2311.2	2202.5	2086.6	1965.6	1842.5	1720.2	1601.5	1488.3	1382.0	1283.3	1192.2	1108.7	1032.3	962.6	899.0	841.0	788.0	739.5	695.2	654.5	617.1	582.8
82.0	2557.2	2230.5	2157.3	2076.7	1988.3	1892.4	1790.4	1684.2	1576.6	1470.0	1367.0	1269.1	1177.5	1092.6	1014.4	942.9	877.6	818.0	763.7	714.2	669.1	627.8	590.0	555.4	523.7	494.5
460UB82.1	2775.5	2370.1	2278.2	2176.6	2065.3	1946.0	1821.4	1695.0	1570.6	1451.0	1338.6	1234.3	1138.7	1051.6	972.6	901.0	836.3	777.7	724.7	676.5	632.8	593.0	556.7	523.6	493.2	465.3
74.6	2436.7	2084.8	2005.2	1917.0	1820.6	1716.9	1608.5	1498.3	1389.5	1284.7	1185.9	1094.1	1009.8	932.9	863.0	799.7	742.4	690.5	643.5	600.9	562.1	526.8	494.6	465.2	438.2	413.5
67.1	2135.9	1827.2	1757.3	1679.9	1595.3	1504.3	1409.2	1312.6	1217.1	1125.3	1038.7	958.2	884.4	817.0	755.8	700.3	650.2	604.7	563.5	526.2	492.2	461.3	433.1	407.4	383.8	362.1
410UB59.7	1934.9	1632.3	1563.2	1486.8	1403.4	1314.8	1223.4	1132.2	1043.8	960.2	882.5	811.4	746.6	688.1	635.3	587.7	544.9	506.2	471.3	439.7	411.0	384.9	361.2	339.5	319.7	301.6
53.7	1811.7	1504.4	1433.6	1355.4	1270.7	1181.9	1092.0	1004.1	920.5	842.7	771.5	707.0	648.9	596.8	550.0	508.1	470.4	436.6	406.1	378.6	353.6	331.0	310.5	291.7	274.6	258.9
360UB56.7	1947.0	1616.2	1540.0	1455.8	1364.6	1269.1	1172.4	1077.9	988.0	904.5	828.0	758.7	696.3	640.3	590.1	545.1	504.7	468.4	435.7	406.2	379.4	355.1	333.1	313.0	294.6	277.7
50.7	1682.3	1398.4	1333.1	1260.8	1182.6	1100.5	1017.3	935.7	858.1	785.8	719.6	659.6	605.5	556.9	513.3	474.2	439.1	407.6	379.1	353.4	330.2	309.1	289.9	272.4	256.4	241.7
44.7	1532.0	1255.5	1191.5	1120.9	1045.0	966.5	888.2	812.8	742.1	677.2	618.3	565.3	518.0	475.7	437.9	404.1	373.8	346.7	322.3	300.3	280.4	262.4	246.0	231.0	217.4	204.9
310UB46.2	1586.7	1318.3	1256.5	1188.2	1114.2	1036.6	958.0	881.1	807.8	739.7	677.3	620.7	569.8	524.0	483.0	446.2	413.1	383.4	356.7	332.5	310.6	290.8	272.7	256.2	241.2	227.4
40.4	1428.5	1173.7	1114.7	1049.7	979.7	907.1	834.5	764.3	698.4	637.6	582.5	532.8	488.3	448.5	413.0	381.2	352.7	327.2	304.2	283.4	264.7	247.7	232.2	218.1	205.3	193.5
32.0	1075.2	832.9	776.1	715.0	652.3	591.1	533.5	480.9	433.7	392.0	355.1	322.7	294.2	269.1	246.8	227.2	209.7	194.1	180.1	167.5	156.2	146.0	136.8	128.3	120.7	113.7
250UB37.3	1368.0	1061.4	989.5	912.1	832.7	754.9	681.6	614.6	554.5	501.2	454.1	412.7	376.3	344.2	315.8	290.6	268.3	248.3	230.4	214.4	199.9	186.9	175.0	164.2	154.4	145.5
31.4	1154.9	880.7	816.4	748.0	678.8	612.2	550.5	494.7	445.2	401.5	363.3	329.7	300.3	274.5	251.6	231.5	213.5	197.6	183.3	170.5	158.9	148.5	139.0	130.5	122.6	115.5
25.7	893.7	614.6	552.2	490.9	433.9	383.1	338.8	300.6	267.9	239.8	215.7	194.9	176.8	161.1	147.3	135.2	124.5	115.0	106.5	99.0	92.2	86.0	80.5	75.5	70.9	66.7
200UB29.8	1100.2	814.9	748.5	679.3	611.1	547.1	489.0	437.4	392.2	352.8	318.5	288.6	262.5	239.6	219.5	201.7	186.0	171.9	159.4	148.2	138.1	129.0	120.8	113.3	106.5	100.3
25.4	930.2	674.9	615.9	555.4	496.9	442.8	394.4	351.8	314.8	282.7	254.9	230.8	209.7	191.3	175.1	160.9	148.3	137.0	127.0	118.1	110.0	102.7	96.1	90.2	84.7	79.8
22.3	826.6	602.3	550.4	497.0	445.1	397.0	353.8	315.8	282.7	254.0	229.0	207.4	188.5	172.0	157.5	144.7	133.3	123.2	114.2	106.2	98.9	92.4	86.5	81.1	76.2	71.8
18.2	661.5	349.6	297.8	254.0	217.8	188.1	163.7	143.5	126.7	112.6	100.7	90.6	81.9	74.4	67.8	62.1	57.1	52.6	48.7	45.2	42.0	39.2	36.6	34.3	32.2	30.3
180UB22.2	812.2	393.5	331.6	280.7	239.5	206.1	178.9	156.5	138.0	122.5	109.5	98.4	88.9	80.7	73.5	67.3	61.8	57.0	52.7	48.9	45.5	42.4	39.6	37.1	34.8	32.7
18.1	662.4	316.7	266.5	225.4	192.1	165.2	143.4	125.4	110.6	98.2	87.7	78.8	71.2	64.6	58.9	53.9	49.5	45.6	42.2	39.1	36.4	33.9	31.7	29.7	27.9	26.2
16.1	587.5	277.1	232.8	196.7	167.6	144.1	125.0	109.3	96.3	85.5	76.4	68.6	62.0	56.3	51.3	46.9	43.1	39.7	36.7	34.1	31.7	29.5	27.6	25.8	24.3	22.8
150UB18.0	662.4	239.3	196.9	164.1	138.5	118.3	102.1	89.0	78.2	69.3	61.8	55.4	50.0	45.3	41.3	37.7	34.6	31.9	29.5	27.3	25.4	23.7	22.1	20.7	19.4	18.2
14.0	512.6	176.5	144.9	120.5	101.6	86.7	74.8	65.2	57.3	50.7	45.2	40.5	36.5	33.1	30.2	27.6	25.3	23.3	21.5	20.0	18.5	17.3	16.1	15.1	14.2	13.3

Table 4.5: Design load capacity table for UC members subject to axial compression buckling about strong axis (manually generated)

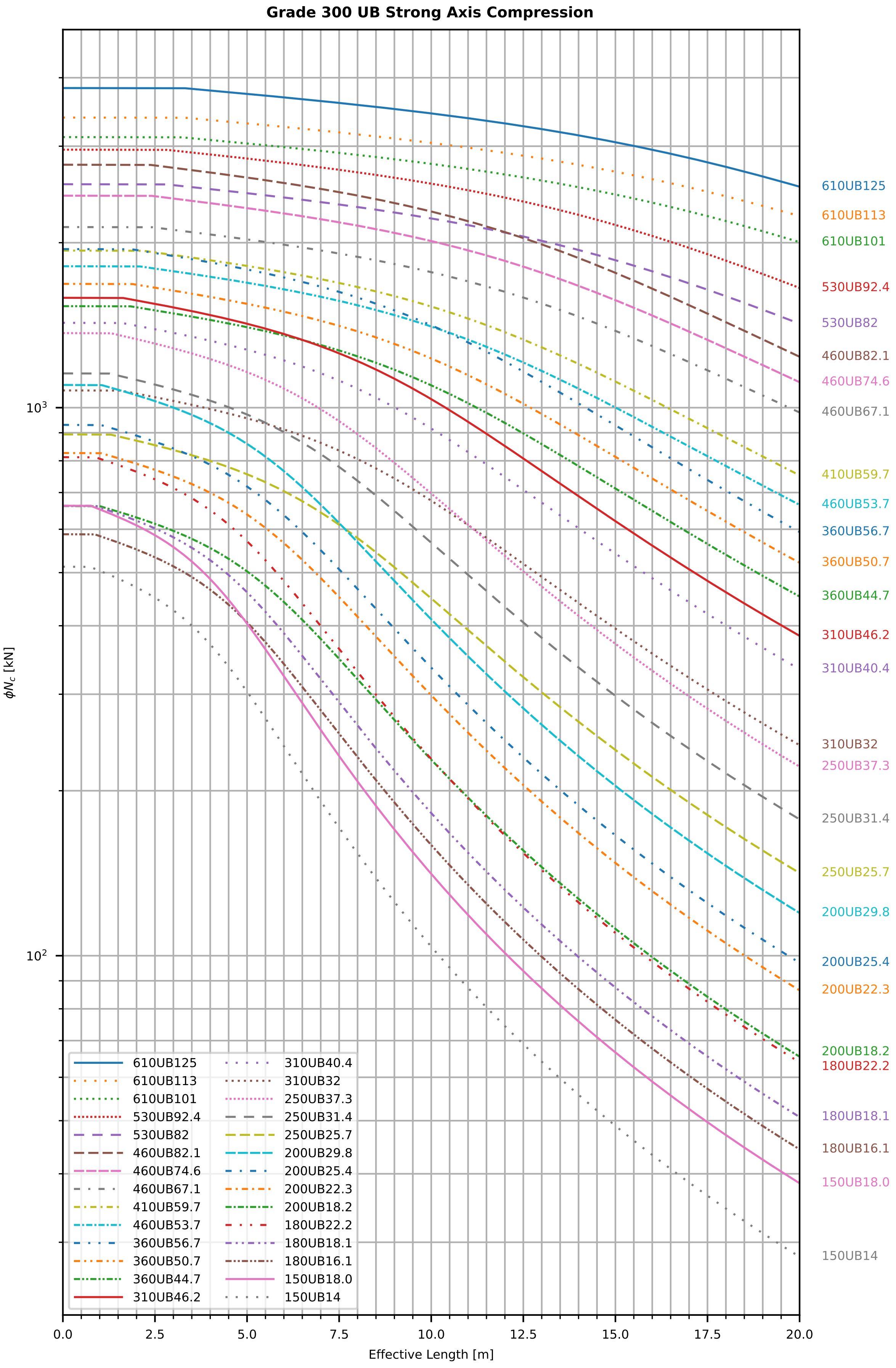
Grade 300 Steel UC Section Subject to Axial Compression **Strong** Axis Buckling

	ϕN_s (kN)	ϕN_c (kN)																							
L_e (m)	0	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9	9.5	10	11	12	13	14	15	16	17	18
310UC158	5065.2	5036.0	4971.4	4905.9	4838.8	4769.7	4697.9	4622.7	4543.6	4460.0	4371.3	4276.9	4176.6	4069.9	3956.9	3837.7	3712.9	3583.2	3314.1	3040.9	2774.5	2523.3	2292.1	2082.9	1895.5	1728.6
137	4410.0	4381.3	4324.2	4266.2	4206.9	4145.6	4081.8	4014.9	3944.5	3869.9	3790.6	3706.3	3616.4	3520.9	3419.8	3313.2	3201.7	3086.1	2847.2	2606.2	2372.9	2154.2	1954.1	1773.7	1612.7	1469.6
118	3780.0	3754.0	3704.6	3654.5	3603.2	3550.2	3494.9	3437.0	3375.9	3311.2	3242.4	3169.1	3091.0	3008.0	2920.1	2827.4	2730.7	2630.4	2423.7	2215.9	2015.4	1828.0	1657.0	1503.2	1366.1	1244.5
96.8	3348.0	3316.1	3270.1	3223.2	3175.0	3124.9	3072.5	3017.3	2958.7	2896.3	2829.6	2758.4	2682.4	2601.6	2516.1	2426.4	2333.4	2237.9	2044.0	1853.8	1674.5	1510.4	1363.0	1232.1	1116.7	1015.0
250UC89.5	2872.8	2820.9	2774.7	2727.1	2677.4	2624.9	2569.1	2509.2	2444.7	2375.1	2299.9	2219.2	2133.3	2042.9	1949.1	1853.3	1757.1	1661.9	1479.5	1313.4	1166.4	1038.3	927.5	831.8	749.2	677.6
72.9	2516.4	2463.8	2421.3	2377.4	2331.2	2282.3	2229.9	2173.4	2112.2	2045.9	1974.4	1897.6	1816.3	1731.3	1644.0	1556.0	1468.6	1383.3	1222.9	1079.7	954.9	847.4	755.2	676.1	608.1	549.4
200UC59.5	2057.4	1981.4	1936.8	1889.3	1838.0	1781.7	1719.8	1651.3	1576.3	1495.2	1409.4	1320.8	1231.9	1144.8	1061.4	982.9	910.0	842.8	725.4	628.1	547.6	480.8	425.1	378.2	338.5	304.6
52.2	1798.2	1730.8	1691.4	1649.6	1604.2	1554.5	1499.6	1439.1	1372.6	1300.9	1225.1	1147.1	1068.9	992.5	919.6	851.1	787.5	729.1	627.1	542.8	473.1	415.3	367.1	326.5	292.2	262.9
46.2	1593.0	1531.9	1496.6	1459.0	1418.1	1373.3	1323.8	1269.1	1209.1	1144.4	1076.2	1006.2	936.4	868.5	803.8	743.3	687.3	635.9	546.4	472.6	411.7	361.3	319.3	284.0	254.1	228.6
150UC37.2	1277.1	1195.2	1155.3	1110.8	1060.2	1002.7	938.3	868.6	796.3	724.6	656.4	593.4	536.5	485.7	440.7	400.9	365.8	334.8	283.0	241.9	209.0	182.2	160.2	142.0	126.6	113.6
30.0	1111.7	1034.2	997.1	955.2	907.2	852.6	791.8	726.9	661.0	597.1	537.6	483.6	435.5	393.1	355.8	323.1	294.4	269.1	227.0	193.8	167.3	145.7	128.1	113.4	101.1	90.7
23.4	858.2	794.4	764.1	729.6	690.0	644.8	594.9	542.4	490.0	440.2	394.5	353.7	317.6	286.1	258.5	234.4	213.3	194.8	164.2	140.0	120.8	105.1	92.4	81.7	72.9	65.3
100UC14.8	544.3	454.7	411.4	360.6	307.9	259.4	218.2	184.5	157.2	135.2	117.3	102.6	90.5	80.3	71.7	64.5	58.2	52.8	44.1	37.3	32.0	27.8	24.3	21.4	19.1	17.0

Table 4.6: Design load capacity table for UC members subject to axial compression buckling about weak axis (manually generated)

Grade 300 Steel UC Section Subject to Axial Compression **Weak** Axis Buckling

	ϕN_s (kN)	ϕN_c (kN)																							
L_e (m)	0	2	2.25	2.5	2.75	3	3.25	3.5	3.75	4	4.25	4.5	4.75	5	5.25	5.5	5.75	6	6.25	6.5	6.75	7	7.25	7.5	7.75	8
310UC158	5065.2	4835.6	4774.8	4711.9	4646.4	4578.1	4506.5	4431.1	4351.7	4267.8	4179.2	4085.7	3987.3	3884.0	3776.2	3664.3	3548.9	3430.9	3311.1	3190.6	3070.4	2951.2	2834.0	2719.5	2608.3	2500.7
137	4410.0	4206.4	4152.8	4097.4	4039.7	3979.4	3916.1	3849.5	3779.2	3704.9	3626.5	3543.7	3456.5	3365.1	3269.7	3170.7	3068.9	2964.8	2859.4	2753.5	2648.0	2543.7	2441.3	2341.4	2244.5	2151.0
118	3780.0	3602.2	3555.7	3507.6	3457.5	3405.1	3350.1	3292.1	3230.9	3166.2	3097.8	3025.5	2949.5	2869.8	2786.7	2700.6	2612.0	2521.7	2430.4	2338.9	2247.8	2157.9	2069.9	1984.2	1901.1	1821.1
96.8	3348.0	3175.6	3132.0	3086.6	3039.2	2989.4	2936.9	2881.4	2822.5	2760.2	2694.1	2624.4	2551.1	2474.4	2394.8	2312.8	2229.1	2144.4	2059.6	1975.4	1892.5	1811.5	1732.8	1656.8	1583.9	1514.0
250UC89.5	2872.8	2683.9	2639.4	2592.5	2542.8	2490.0	2433.6	2373.3	2309.0	2240.5	2168.2	2092.2	2013.3	1932.2	1849.8	1767.1	1685.0	1604.5	1526.3	1450.8	1378.5	1309.6	1244.3	1182.5	1124.3	1069.6
72.9	2516.4	2336.7	2295.1	2251.1	2204.1	2154.0	2100.3	2042.7	1981.2	1915.9	1847.0	1775.2	1701.1	1625.6	1549.8	1474.6	1400.9	1329.4	1260.6	1194.9	1132.5	1073.5	1018.0	965.8	916.8	871.0
200UC59.5	2057.4	1841.2	1793.0	1740.6	1683.5	1621.4	1554.4	1483.0	1408.3	1331.5	1254.2	1177.9	1104.0	1033.3	966.5	904.0	845.9	792.0	742.3	696.5	654.3	615.4	579.7	546.7	516.3	488.2
52.2	1798.2	1608.0	1565.6	1519.4	1469.1	1414.5	1355.5	1292.7	1227.0	1159.5	1091.8	1025.0	960.2	898.5	840.2	785.6	735.0	688.0	644.8	604.9	568.2	534.4	503.3	474.6	448.2	423.8
46.2	1593.0	1421.6	1383.4	1341.8	1296.5	1247.1	1193.9	1137.3	1078.2	1017.8	957.2	897.7	840.3	785.6	734.1	686.0	641.4	600.2	562.2	527.3	495.2	465.6	438.4	413.4	390.3	369.0
150UC37.2	1277.1	1054.4	1003.0	946.2	884.9	821.0	756.8	694.3	635.3	580.8	531.1	486.2	445.9	409.7	377.4	348.5	322.6	299.3	278.3	259.4	242.2	226.7	212.6	199.7	188.0	177.2
30.0	1111.7	902.7	854.2	800.7	743.8	685.3	627.6	572.6	521.5	474.9	432.9	395.3	361.8	332.0	305.4	281.6	260.4	241.4	224.3	209.0	195.1	182.5	171.0	160.6	151.1	142.4
23.4	858.2	685.1	644.6	600.4	553.9	507.0	461.6	419.0	380.1	345.0	313.6	285.8	261.1	239.3	219.9	202.6	187.2	173.4	161.0	149.9	139.9	130.8	122.6	115.1	108.2	102.0
100UC14.8	544.3	323.0	280.5	242.5	209.9	182.5	159.6	140.4	124.4	110.8	99.3	89.4	80.9	73.5	67.1	61.5	56.6	52.2	48.3	44.8	41.7	38.9	36.4	34.1	32.0	30.1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17.5
20.0
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610UB113
610UB101
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530UB82
460UB82.1
460UB74.6
460UB67.1
410UB59.7
460UB53.7
360UB56.7
360UB50.7
360UB44.7
310UB46.2
310UB40.4
310UB32
250UB37.3
250UB31.4
250UB25.7
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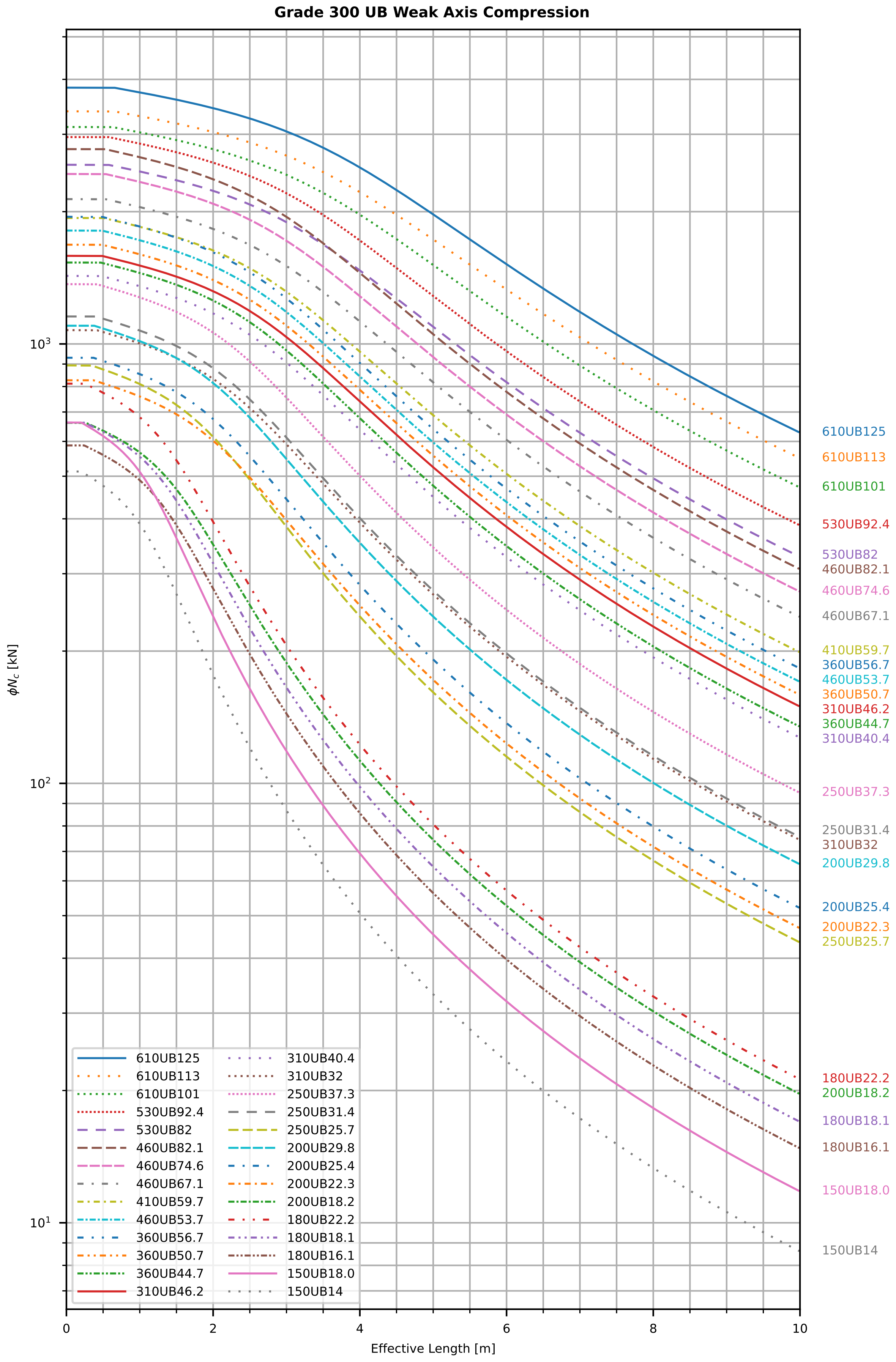
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2
4
6
8
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610UB113
610UB101
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460UB82.1
460UB74.6
460UB67.1
410UB59.7
460UB53.7
360UB56.7
360UB50.7
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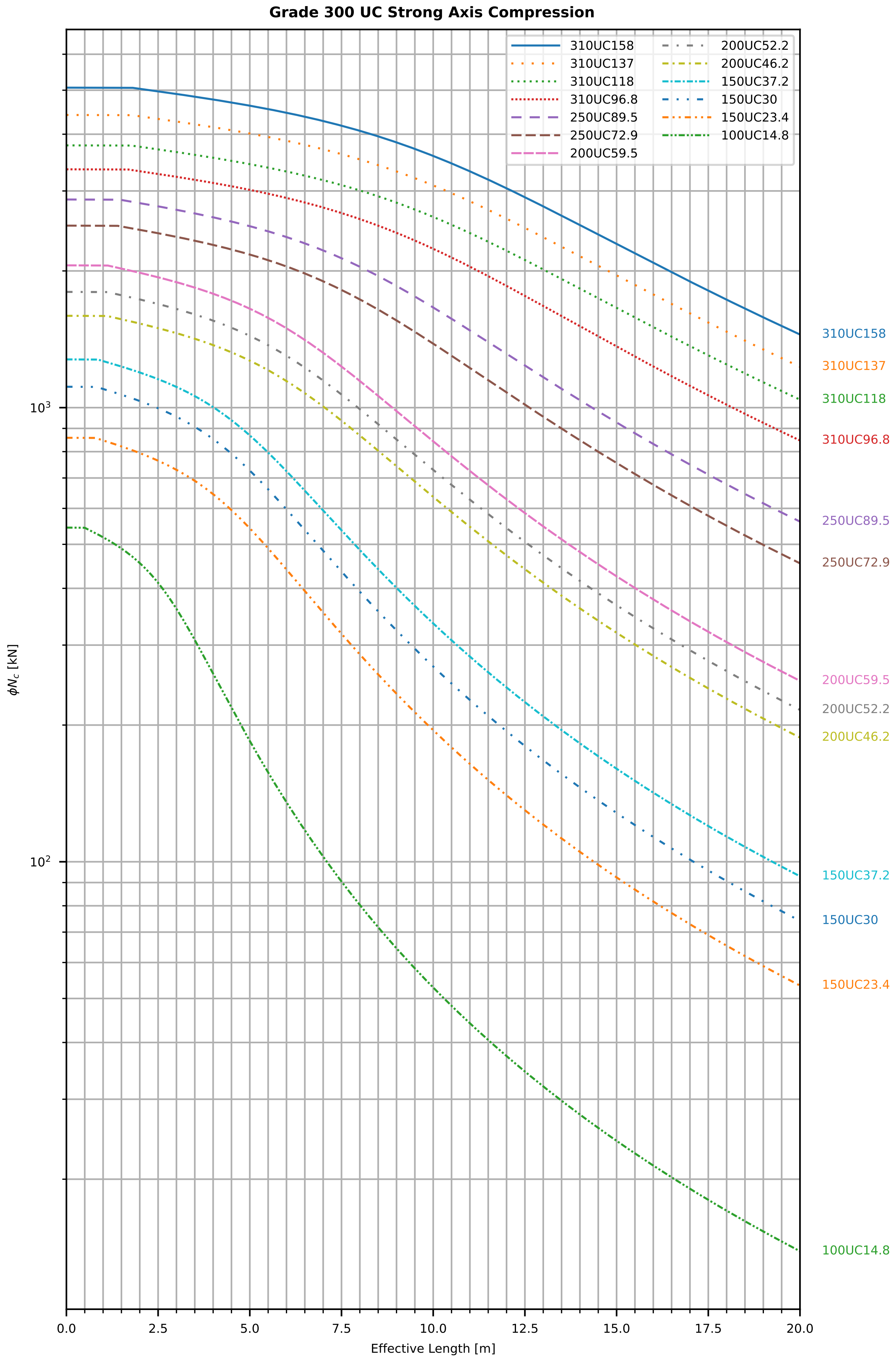
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250UC72.9
200UC59.5
200UC52.2
200UC46.2
150UC37.2
150UC30
150UC23.4
100UC14.8
10^3
10^2
ϕN_c [kN]
0.0
2.5
5.0
7.5
10.0
12.5
15.0
17.5
20.0
Effective Length [m]
310UC158
310UC137
310UC118
310UC96.8
250UC89.5
250UC72.9
200UC59.5
200UC52.2
200UC46.2
150UC37.2
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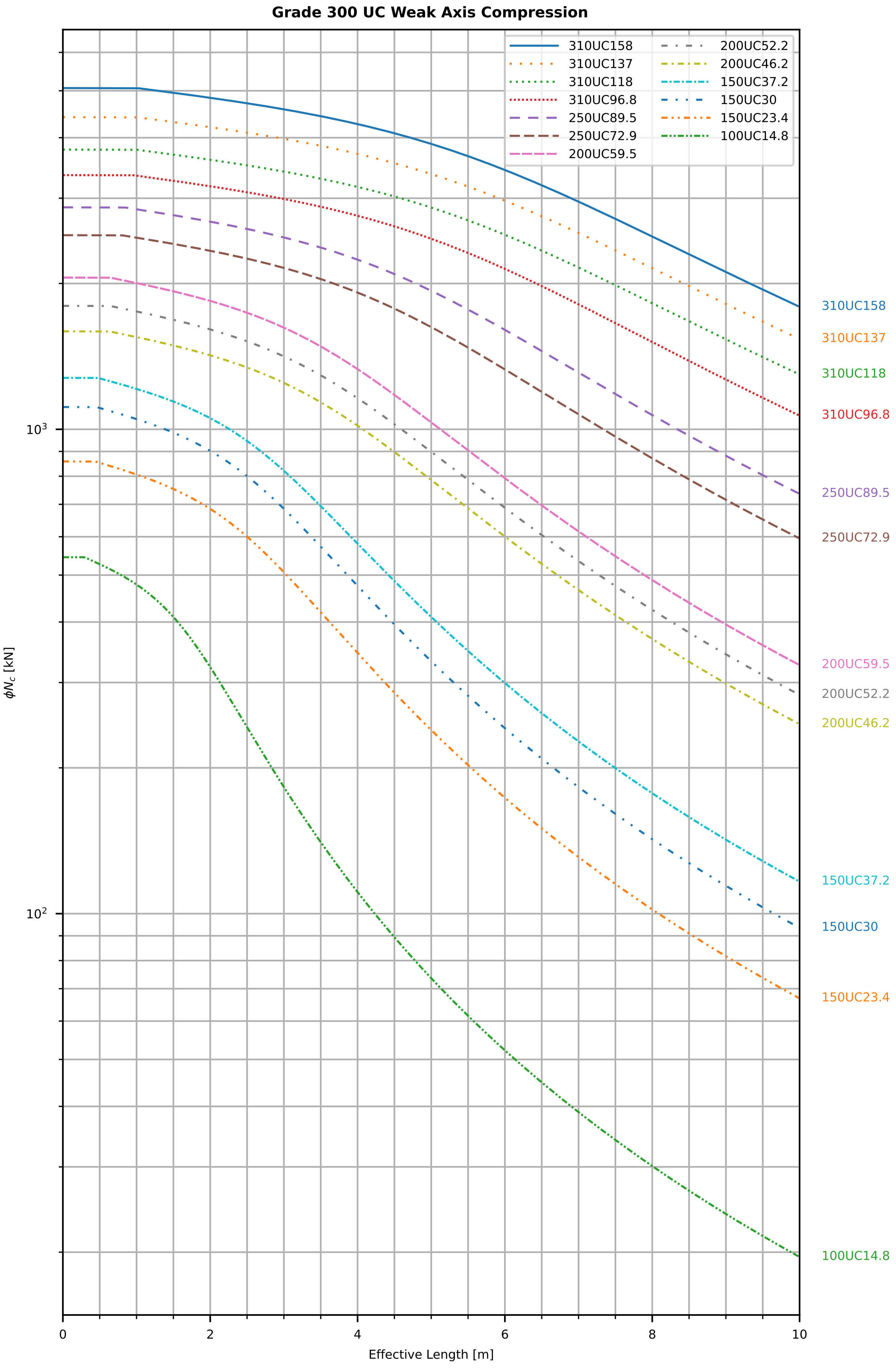
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150UC23.4
100UC14.8
10^3
10^2
ϕN_c [kN]
0
2
4
6
8
10
Effective Length [m]
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310UC137
310UC118
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250UC89.5
250UC72.9
200UC59.5
200UC52.2
200UC46.2
150UC37.2
150UC30
150UC23.4
100UC14.8

Example 4.5 *Axial Compression Example — UC*

Using Grade 300 steel, find the lightest UC without holes for $G = 400\,\text{kN}$ *and* $Q = 700\,\text{kN}$ *with* $L_e = 5\,\text{m}$.

Solution 4.5

For all hot-rolled UC sections, $t_f < 40\,\text{mm}$, thus $k_f = 1$ and $\alpha_b = 0$. Assume $f_y = 300\,\text{MPa}$.

Load combination gives

$$N^* = 1.2G + 1.5Q = 1.2 \times 400\,\text{kN} + 1.5 \times 700\,\text{kN} = 1530\,\text{kN}.$$

For the identical L_e along both axes, clearly the weak axis governs. Assume a moderate $r_y \approx 60\,\text{mm}$, this leads to

$$\lambda_n = \frac{L_e}{r_y}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{5\,\text{m}}{50\,\text{mm}} \times \sqrt{1} \times \sqrt{\frac{300}{250}} = 91.3.$$

Then $\alpha_c = 0.601$.

$$A_{n,min} = \frac{N^*}{\phi\alpha_c k_f f_y} = \frac{1530\,\text{kN}}{0.9 \times 0.601 \times 1 \times 300\,\text{MPa}} = 9430\,\text{mm}^2.$$

This gives us a reasonable reference value of A_n, we then try designations that have similar A_n.

Try 250UC89.5, update all quantities,

$$f_y = 280\,\text{MPa}, \quad r_y = 65.2\,\text{mm}, \quad \lambda_n = 81.2, \quad \alpha_c = 0.673, \quad A_{n,min} = 8420\,\text{mm}^2.$$

Since $A_n = 11\,400\,\text{mm}^2 > 8420\,\text{mm}^2$, 250UC89.5 works.

Try 250UC72.9, update all quantities,

$$f_y = 300\,\text{MPa}, \quad r_y = 64.5\,\text{mm}, \quad \lambda_n = 84.9, \quad \alpha_c = 0.646, \quad A_{n,min} = 8772\,\text{mm}^2.$$

Since $A_n = 9320\,\text{mm}^2 > 8772\,\text{mm}^2$, 250UC72.9 works.

Try 200UC59.5, update all quantities,

$$f_y = 300\,\text{MPa}, \quad r_y = 51.7\,\text{mm}, \quad \lambda_n = 105.9, \quad \alpha_c = 0.502, \quad A_{n,min} = 11\,280\,\text{mm}^2.$$

Since $A_n = 7620\,\text{mm}^2 < 11\,280\,\text{mm}^2$, 200UC59.5 does not work.

The lightest designation would be 250UC72.9. By using $N^* = 1530\,\text{kN}$, the optimal section is 250UC72.9 from Fig. 4.15.

Example 4.6 *A* $18\,\text{m}$ *long beam is braced laterally (pinned) at its ends for buckling about its strong axis, and braced (pinned) at the ends and at the* ***quarter*** *points for buckling about its weak axis. Find the lightest I section to carry* $N^* = 700\,\text{kN}$*. Use Grade 300 steel.*

Solution 4.6

The effective lengths are

$$L_{e,y} = 18\,\text{m}/4 = 4.5\,\text{m},$$
$$L_{e,x} = 18\,\text{m}.$$

An efficient member shall have similar L_e/r about both axes. This leads to $r_x/r_y \approx 4$. UC sections often have $r_x/r_y \approx 1.6$ to 1.8. UB sections often have $r_x/r_y \approx 3.5$ to 5.5. Thus use UB sections will be more efficient. From Table 4.3 and Table 4.4,

Section	$\phi N_{c,x}$ at 18 m	$\phi N_{c,y}$ at 4.5 m	
410UB59.7	881.5	811.4	Okay
53.7	782.2	707.0	Okay
360UB56.7	705.5	758.7	Okay
50.7	619.5	659.6	N.G.

Thus the lightest section is 410UB53.7.

4.3.1 Consideration of Effective Length

Individual Member

NZS 3404.1&2:1997 § 4.8.3.2 recommends the following values for k_e as shown in Fig. 4.17 for **individual** sway and braced members that are designed for load combinations that do **not** include earthquake loads. Those values are larger than the theoretical values because it is assumed that no connections are ideal, connections at member ends normally have some flexibility. It is worth noting that those values may be different in other codes.

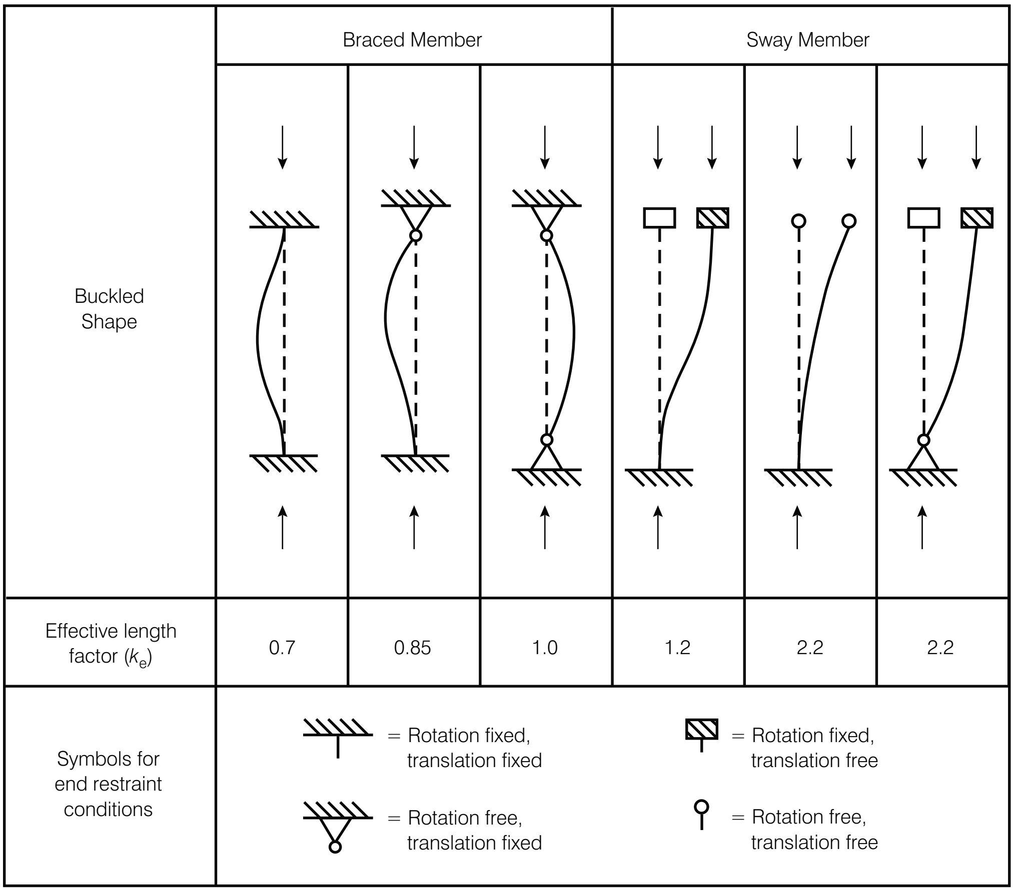


Figure 4.17: Effective length factor k_e

Frame Member

There are two types of frame members.

- **Braced Frame Member**
  In these frames, lateral stability is provided by structural walls, diagonal bracing or other similar means within the storey considered. For braced members, $1.0 \geqslant k_e \geqslant 0.5$.

- **Sway Frame Member**
  In these frames, lateral stability is provided by bending stiffness of rigidly connected beams or columns. The upper supports can also move down the same amount on each side. For sway members, $k_e \geqslant 1.0$.

Whether the beams are braced results in different rotation constraints on beam–column joints. Besides, the illustrations in Fig. 4.18 only show the cases when the far ends of beams are pinned. The fixity condition of beam far end also has an impact on the degree of rotation constraint offered. This will be discussed later.

In order to compute member capacities, two methods are available, namely the stability function and the γ-factor method.

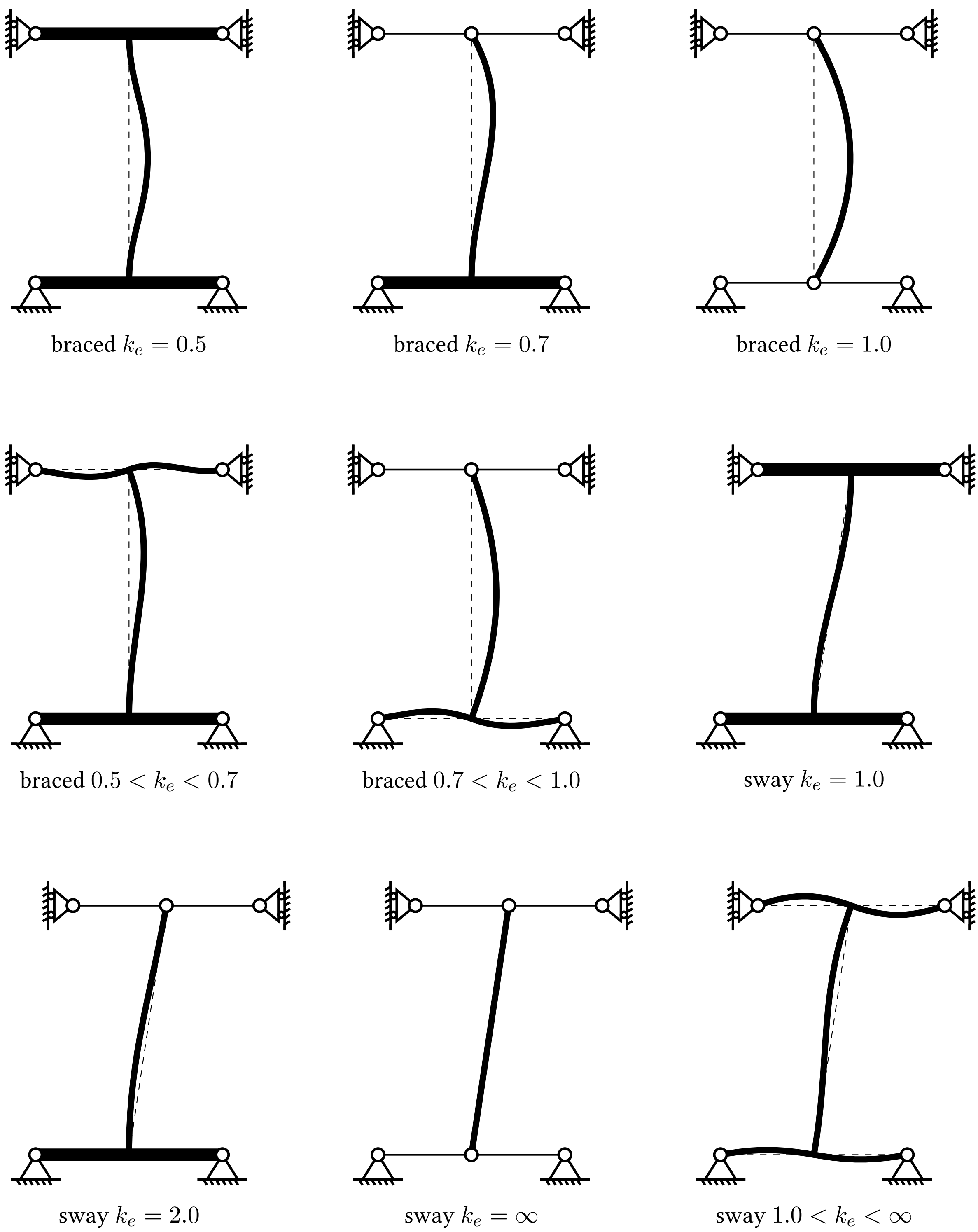


Figure 4.18: Illustration of different types of frame members with theoretical k_e shown

Stability Functions In the general case of different EI in the beams at the top and bottom of the frame, stability functions can be used to evaluate the buckling force in the member without needing to consider the effective lengths. These are included in computer programs but are not studied here.

The γ-Factor Method Alternatively, for frame members, the effective length factor k_e can be determined by a simplified method via stiffness ratios. The following assumptions of idealized conditions, which seldom exist in real structures, are adopted (ANSI/AISC 360-16 § 7.2):

- all members have constant cross section,
- all joints are rigid,
- joint restraint is distributed to the column above and below the joint in proportion to EI/L of the two columns,
- for braced frames, rotations at opposite ends of the beams are of equal magnitude, producing single curvature bending,
- for sway frames, rotations at opposite ends of the restraining beams are of equal magnitude, producing reverse curvature bending,
- the stiffness parameters $L\sqrt{\dfrac{P}{EI}}$ of all columns are equal,
- all columns buckle simultaneously,
- behaviour is purely elastic,
- no significant axial compression force exists in the girders,
- shear deformations are neglected.

For each compression member, it is possible to compute the stiffness ratios of two ends respectively via the following expression.

$$\gamma=\frac{\sum\limits_{\text{columns}}\dfrac{EI}{L}}{\sum\limits_{\text{beams}}\dfrac{\beta_eEI}{L}}=\frac{\sum\limits_{\text{columns}}\dfrac{I}{L}}{\sum\limits_{\text{beams}}\dfrac{\beta_eI}{L}},\tag{4.7}$$

in which

- NZS 3404.1&2:1997 § 4.8.3.4.2 $\sum\limits_{\text{columns}}\dfrac{I}{L}$ shall be calculated from the sum of the stiffnesses, in the plane of bending, of all the compression members rigidly connected at the end of the member under consideration, including the member itself.
- NZS 3404.1&2:1997 § 4.8.3.4.3 $\sum\limits_{\text{beams}}\dfrac{\beta_eI}{L}$ shall be calculated from the sum of the stiffnesses, in the plane of bending, of all the beams rigidly connected at the end of the member under consideration. The contributions of any beams pin-connected to the member shall be neglected.
- β_e is a modification factor to account for different beam far end conditions. This will be introduced later. For the moment, assume $\beta_e=1$.

With γ_1 and γ_2 (for two ends) at hand, k_e can be calculated by one of the following methods:

1. The exact solution for the assumptions above may be found using the following equations. These are transcendental functions that can be solved by numerical methods.

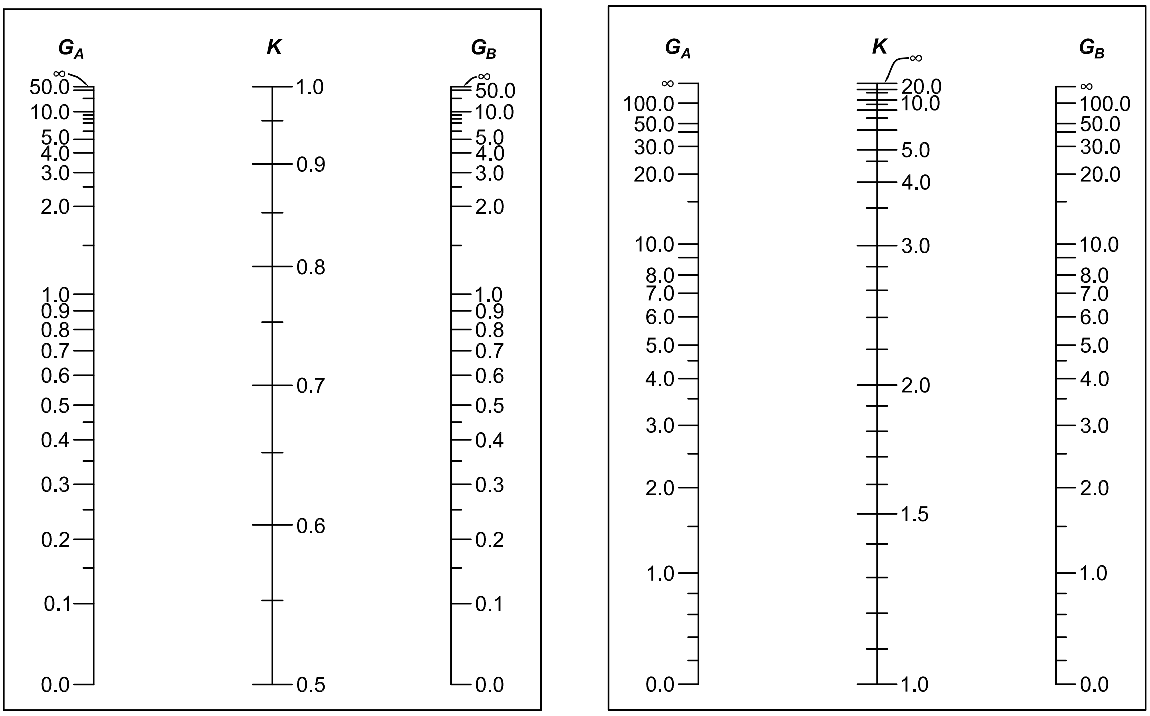
- braced member

$$\frac{\gamma_1\gamma_2}{4}\left(\frac{\pi}{k_e}\right)^2+\frac{\gamma_1+\gamma_2}{2}\left(1-\frac{\pi/k_e}{\tan\left(\pi/k_e\right)}\right)+\frac{2\tan\left(0.5\pi/k_e\right)}{\pi/k_e}=1.0$$

- sway member

$$\left(\gamma_1\gamma_2\left(\frac{\pi}{k_e}\right)^2-36\right)\tan\left(\frac{\pi}{k_e}\right)=6\left(\gamma_1+\gamma_2\right)\frac{\pi}{k_e}$$

2. US Alignment Charts. These charts used in ANSI/AISC 360-16 Specification for Structural Steel Buildings give the graphical solutions to the two equations above. They are shown in Fig. 4.19 in which G_A and G_B correspond to γ_1 and γ_2. Additional copies are provided at the end of this chapter.



(a) for braced members (b) for sway members

Figure 4.19: Alignment chart for k_e

3. AU/NZ Alignment Charts. These charts used in NZS 3404.1&2:1997 Fig. 4.8.3.3 are essentially identical to the ones used in the US code but presented in a different format. Fig. 4.20 shows those charts.

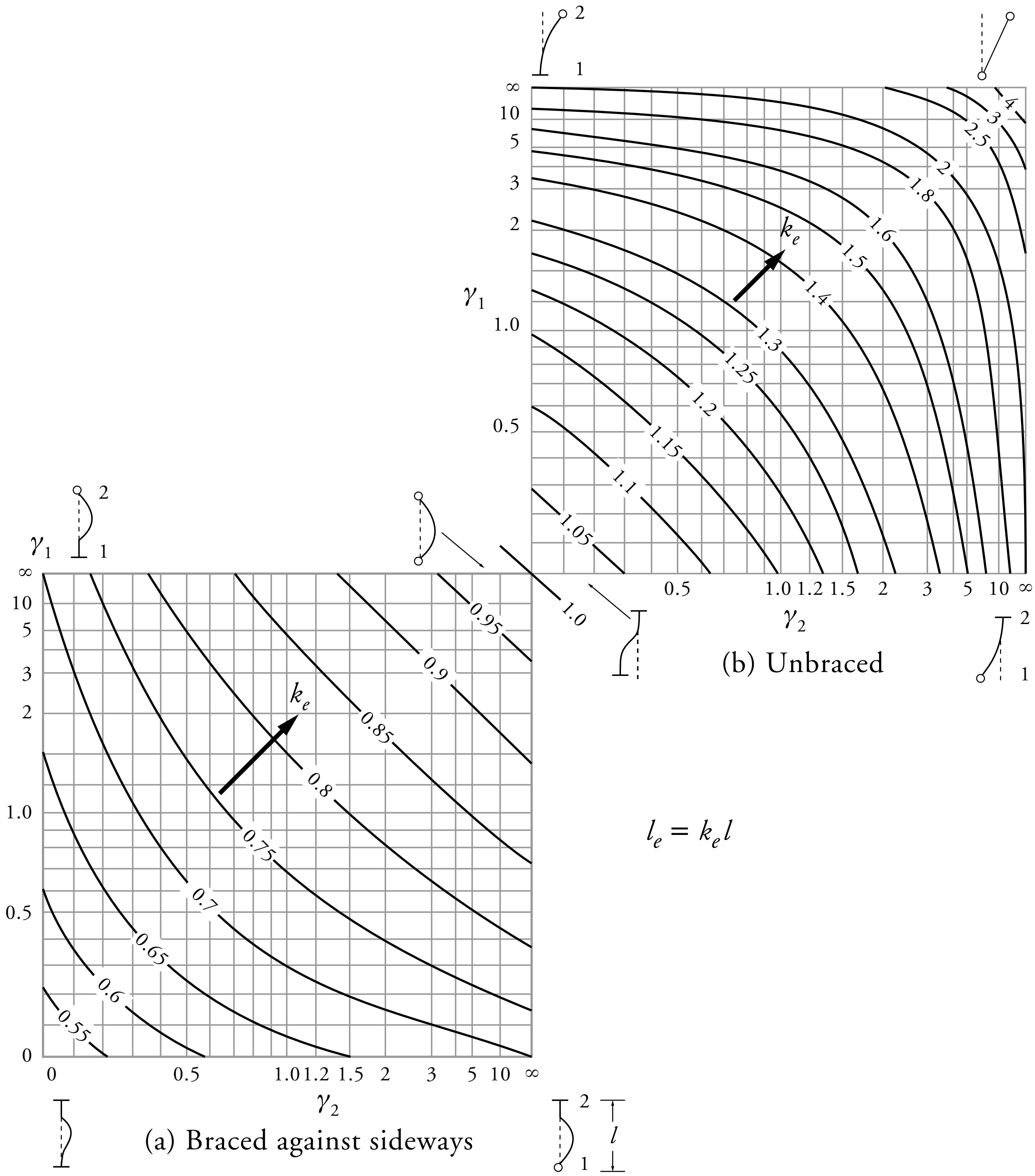


Figure 4.20: k_e for braced and sway members (Gorenc et al., 2015)

4. French Equations. These are used in the French code and they are an approximation to the two equations above. They give answers to an accuracy of better than 1 % to the true answer and any error results in a slightly conservative answer. This is better than the readability of the design charts.

   - braced member

$$k_e = \frac{3\gamma_1\gamma_2 + 1.4(\gamma_1 + \gamma_2) + 0.64}{3\gamma_1\gamma_2 + 2(\gamma_1 + \gamma_2) + 1.28} \tag{4.8}$$

- sway member

$$k_e = \sqrt{\frac{1.6\gamma_1\gamma_2 + 4(\gamma_1 + \gamma_2) + 7.5}{\gamma_1 + \gamma_2 + 7.5}} \tag{4.9}$$

The French method is probably the best for practical usage, although any of the methods above would be acceptable.

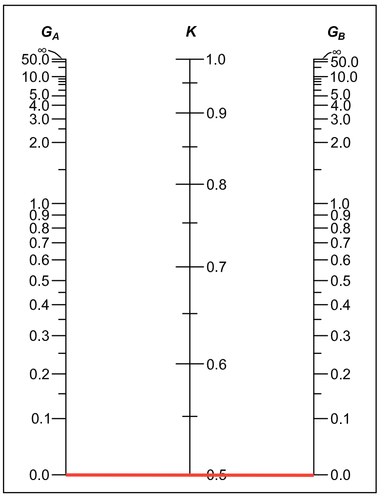
Examples of Alignment Charts

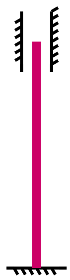
To use the alignment charts, one shall calculate γ_1 and γ_2 first. Draw a straight line defined by γ_1 and γ_2, the intersection gives the value of k_e.

Braced Individual Member Since both ends are fixed,

$$\gamma_{top} = \gamma_{bot} = 0.$$

From the chart, $k_e = 0.5$.

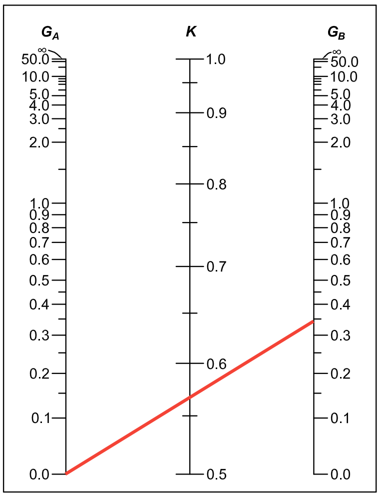


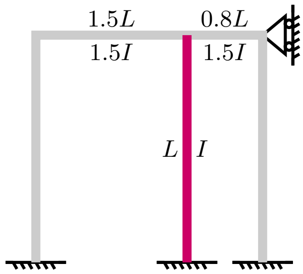


Braced Frame Member The top end is connected to two beams,

$$\gamma_{top} = \frac{1}{1.5/0.8 + 1.5/1.5} = 0.348, \qquad \gamma_{bot} = 0.$$

From the chart, $k_e \approx 0.57$. The French equation gives $k_e = 0.5704$.

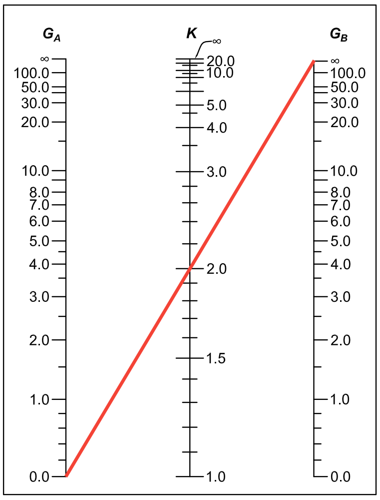


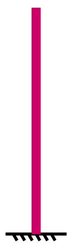


Sway Individual Member Since the top end is a free end and the bottom end is fixed,

$$\gamma_{top} = \infty, \qquad \gamma_{bot} = 0.$$

From the chart, $k_e = 2.0$.

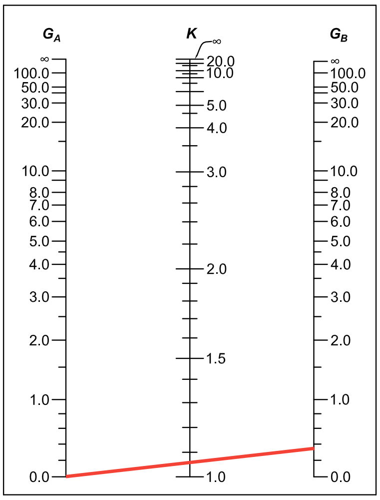


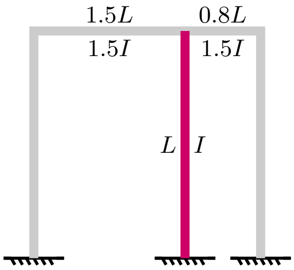


Sway Frame Member The top end is connected to two beams,

$$\gamma_{top} = \frac{1}{1.5/0.8 + 1.5/1.5} = 0.348, \qquad \gamma_{bot} = 0.$$

From the chart, $k_e \approx 1.05$. The French equation gives $k_e = 1.0644$.



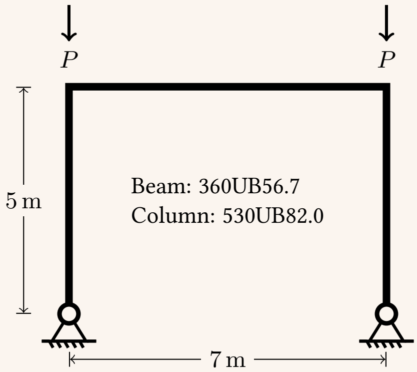


Remark

The stiffness ratio γ characterises the rotation ability of an end. A sufficiently large $\gamma \to \infty$ represents a pinned connection while a sufficiently small $\gamma \to 0$ represents a fixed connection. **However, those are idealised assumptions, in real world those perfect connections do not exist. Thus, γ is often taken as 1 for fully fixed connection and 10 for perfectly pinned connection.**

Example 4.7 *Worksheet Axial Compression Example — Frame*

For the frame shown, find axial compression capacity of column when the frame is a) braced and b) unbraced. Assume the out-of-plane buckling is fully prevented and in-plane deformations cause strong axis member bending. Use Grade 300 steel.



Solution 4.7

In this simple case, $\beta_e = 1$. The stiffness ratios can be computed as

$$\gamma_1 = \gamma_{top} = \frac{I_c/L_c}{\beta_e I_b/L_b} = 4.15, \qquad \gamma_2 = \gamma_{bot} = 10.$$

Note for the pinned connection, 10 is used.

- braced frame

By the French equation,

$$k_e = \frac{3\gamma_1\gamma_2 + 1.4\,(\gamma_1+\gamma_2) + 0.64}{3\gamma_1\gamma_2 + 2\,(\gamma_1+\gamma_2) + 1.28} = 0.941.$$

The modified slenderness ratio,

$$\lambda_n = \frac{k_e L}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{0.941 \times 5\,\text{m}}{213\,\text{mm}} \times \sqrt{0.902} \times \sqrt{\frac{300}{250}} = 22.97.$$

Thus $\alpha_c = 0.968$,

$$\phi N_c = \phi\alpha_c k_f A_n f_y = 0.9 \times 0.968 \times 0.902 \times 10\,500\,\text{mm}^2 \times 300\,\text{MPa} = 2475\,\text{kN}.$$

- sway frame

  By the French equation,

$$k_e = \sqrt{\frac{1.6\gamma_1\gamma_2 + 4\,(\gamma_1+\gamma_2) + 7.5}{\gamma_1+\gamma_2+7.5}} = 2.455.$$

The modified slenderness ratio,

$$\lambda_n = \frac{k_e L}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{2.455 \times 5\,\text{m}}{213\,\text{mm}} \times \sqrt{0.902} \times \sqrt{\frac{300}{250}} = 59.95.$$

Thus $\alpha_c = 0.809$,

$$\phi N_c = \phi\alpha_c k_f A_n f_y = 0.9 \times 0.809 \times 0.902 \times 10\,500\,\text{mm}^2 \times 300\,\text{MPa} = 2069\,\text{kN}.$$

4.3.2 Modifications to γ-Factor Method

Effect of Beam End Conditions

Recall Fig. 4.18 and Eq. (4.7),

$$\gamma = \frac{\displaystyle\sum_{\text{columns}} \frac{EI}{L}}{\displaystyle\sum_{\text{beams}} \frac{\beta_e EI}{L}} = \frac{\displaystyle\sum_{\text{columns}} \frac{I}{L}}{\displaystyle\sum_{\text{beams}} \frac{\beta_e I}{L}}.$$

NZS 3404.1&2:1997 § 4.8.3.4.3 uses a modifying factor β_e to account for various conditions at the far ends of the beams, which may affect the magnitude of moment on the joint. This corresponds to ANSI/AISC 360-16 Eq. C-A-7-4.

Table 4.7: Modifying factor β_e

fixity at far end of beam	beam restraining a braced member	beam restraining a sway member
pinned	1.5	0.5
rigidly connected to a column	1.0	1.0
fixed	2.0	0.67

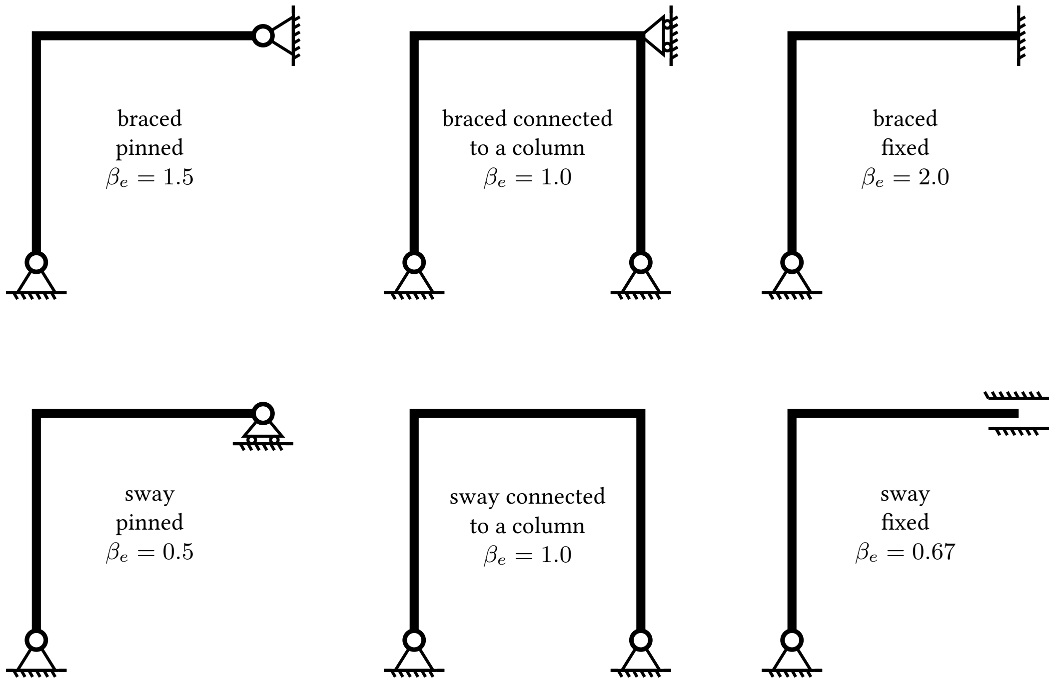


Figure 4.21: Far end of beam conditions

Member Yielding

Stocky columns which are subject to high axial forces may yield as a result of the residual stress. This reduces the elastic modulus, so the column has a 'close to' pinned connection at the yield end. This can lead to a saving of material.

We will not study this in this class, and it is not important for $f < 0.5f_y$.

Total Storey Sidesway

The design charts are based on the assumption that all columns buckle simultaneously and that the sidesway load for the storey can be obtained from the sidesway load of one column.

In gravity frames the resistance of one column to sidesway may be greater than that of other columns due to its loading or stiffness. It is assumed columns are axially rigid.

For design against sidesway, the gravity load which a frame can support may be split up in any proportion. An example is shown as follows.

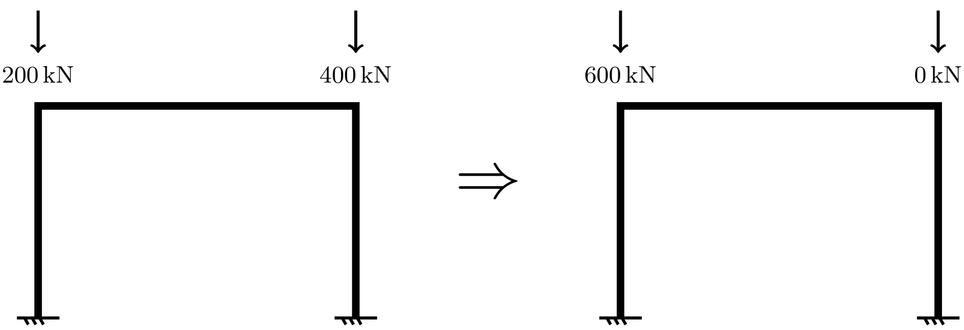
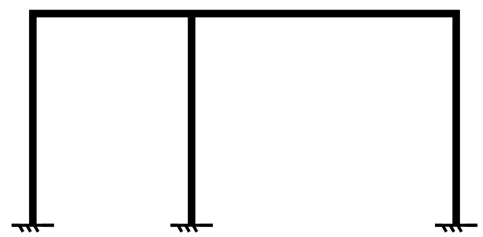


Figure 4.22: Transfer of vertical loads

However, the maximum load in any column must **not** exceed the maximum load that a column could support if it were braced against sidesway ($k_e = 1.0$).

For the following frame, assume the braced and sway capacities for each column are listed as follows.



braced capacity $\phi N_{c,b}$	500 kN	600 kN	400 kN
sway capacity $\phi N_{c,s}$	300 kN	500 kN	300 kN
	Column 1	Column 2	Column 3

For the sway mechanism, $\sum_{i=1}^{3} N_i^* = N^* \leqslant \sum_{i=1}^{3} \phi N_{c,s}^i = 1100\,\text{kN}$. For each column, the maximum load shall not exceed its braced capacity, that is

$$N_1^* \leqslant \phi N_{c,b}^1 = 500\,\text{kN}, \qquad N_2^* \leqslant \phi N_{c,b}^2 = 600\,\text{kN}, \qquad N_3^* \leqslant \phi N_{c,b}^3 = 400\,\text{kN}.$$

Now consider the following two cases.

- The loads for columns 1 and 2 are given as $N_1^* = 400\,\text{kN}$ and $N_2^* = 500\,\text{kN}$, what is the maximum load of column 3?
  Knowing that $N^* \leqslant 1100\,\text{kN}$ to prevent sway mechanism, $N_3^* \leqslant 1100\,\text{kN} - 400\,\text{kN} - 500\,\text{kN} = 200\,\text{kN}$. Thus the maximum load of column 3 should be $N_3^* = \min{(200\,\text{kN},\ 400\,\text{kN})} = 200\,\text{kN}$.
- The loads for columns 1 and 2 are given as $N_1^* = 300\,\text{kN}$ and $N_2^* = 300\,\text{kN}$, what is the maximum load of column 3?
  Knowing that $N^* \leqslant 1100\,\text{kN}$ to prevent sway mechanism, $N_3^* \leqslant 1100\,\text{kN} - 300\,\text{kN} - 300\,\text{kN} = 500\,\text{kN}$. Thus the maximum load of column 3 should be $N_3^* = \min{(500\,\text{kN},\ 400\,\text{kN})} = 400\,\text{kN}$.

Use of Effective Length Factor — Practical Considerations

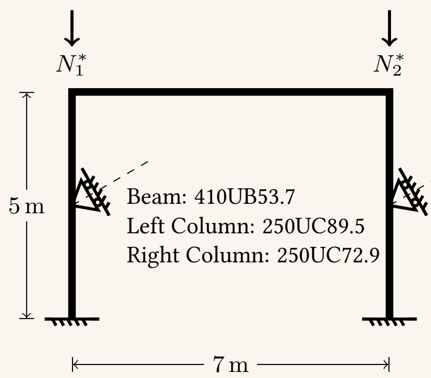
For columns in braced frames, it is always conservative to use $k_e = 1$. This has been recommended by Yura (1971). Based on this, and the uncertainty in assessing some of these connection stiffness, we will use $k_e = 1$ for the braced case consideration for columns in

1. sway frames, and
2. braced frames, where there is loading along the beam length causing column moment.

For columns in braced frames with no moment from the beams, k_e can be computed as being less than 1 using the techniques described above.

Example 4.8 *Worksheet Axial Compression Example — Sway Frame*

For the frame shown, the columns are supported out-of-plane at top, centre and bottom, find 1) the maximum value of axial load each column can carry and 2) the maximum value of $N_1^* + N_2^*$ the frame can carry. Note the out-of-plane (weak axis) buckling should be checked as well. Use Grade 300 steel.



	A mm^2	I_x 10^6 mm^6	r_x mm	r_y mm	f_y MPa
410UB53.7		188			
250UC89.5	11 400	143	112	65.2	280
250UC72.9	9320	114	111	64.5	300

Solution 4.8

- For sway capacity.
  - Left Column
    Find stiffness ratios. $\gamma_{bot} = \gamma_2 = 1$.

$$\gamma_{top} = \gamma_1 = \frac{I_c/L_c}{\beta_e I_b/L_b} = \frac{143 \times 10^6\,\mathrm{mm}^6/5\,\mathrm{m}}{1 \times 188 \times 10^6\,\mathrm{mm}^6/7\,\mathrm{m}} = 1.065.$$

From French equation, $k_e = 1.351$. Thus, the modified slenderness ratio,

$$\lambda_n = \frac{k_e L_x}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{1.351 \times 5\,\mathrm{m}}{112\,\mathrm{mm}} \times \sqrt{1} \times \sqrt{\frac{280}{250}} = 63.84.$$

Thus $\alpha_c = 0.786$,

$$\phi N_c = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.786 \times 1 \times 11\,400\,\mathrm{mm}^2 \times 280\,\mathrm{MPa} = 2259\,\mathrm{kN}.$$

Alternatively, $L_e = k_e L_x = 6.76\,\mathrm{m}$ could be used to get 2259 kN by linear interpolation from Table 4.5.

- Right Column
  Find stiffness ratios. $\gamma_{bot} = \gamma_2 = 1$.

$$\gamma_{top} = \gamma_1 = \frac{I_c/L_c}{\beta_e I_b/L_b} = \frac{114 \times 10^6\,\mathrm{mm}^6/5\,\mathrm{m}}{1 \times 188 \times 10^6\,\mathrm{mm}^6/7\,\mathrm{m}} = 0.849.$$

  From French equation, $k_e = 1.319$. Thus, the modified slenderness ratio,

$$\lambda_n = \frac{k_e L_x}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{1.319 \times 5\,\mathrm{m}}{111\,\mathrm{mm}} \times \sqrt{1} \times \sqrt{\frac{300}{250}} = 65.06.$$

  Thus $\alpha_c = 0.779$,

$$\phi N_c = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.779 \times 1 \times 9320\,\mathrm{mm}^2 \times 300\,\mathrm{MPa} = 1961\,\mathrm{kN}.$$

  Alternatively, $L_e = k_e L_x = 6.60\,\mathrm{m}$ could be used to get 1961 kN by linear interpolation from Table 4.5.

Thus the total $N^* = \sum N_i^* = 2259\,\mathrm{kN} + 1961\,\mathrm{kN} = 4220\,\mathrm{kN}$.

- For braced capacity.

  One can use French equation to find the corresponding k_e as shown before. For braced columns in a sway frame, use $k_e = 1$.

  - Left Column
    The modified slenderness ratio,

$$\lambda_n = \frac{k_e L_x}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{1 \times 5\,\mathrm{m}}{112\,\mathrm{mm}} \times \sqrt{1} \times \sqrt{\frac{280}{250}} = 47.25.$$

    Thus, $\alpha_c = 0.874$.

$$\phi N_c = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.874 \times 1 \times 11\,400\,\mathrm{mm}^2 \times 280\,\mathrm{MPa} = 2509\,\mathrm{kN}.$$

  - Right Column
    The modified slenderness ratio,

$$\lambda_n = \frac{k_e L_x}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{1 \times 5\,\mathrm{m}}{111\,\mathrm{mm}} \times \sqrt{1} \times \sqrt{\frac{300}{250}} = 49.34.$$

    Thus, $\alpha_c = 0.864$.

$$\phi N_c = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.864 \times 1 \times 9320\,\mathrm{mm}^2 \times 300\,\mathrm{MPa} = 2173\,\mathrm{kN}.$$

  From α_c, one can tell the braced capacity shall be greater than the sway capacity.

- For weak axis buckling.

  Again, we choose $k_e = 1$ for a conservative design. Since the columns are supported every half the length, $L_y = 2.5\,\mathrm{m}$.

- Left Column
  The modified slenderness ratio,

$$\lambda_n = \frac{k_e L_y}{r_y}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{1\times 2.5\,\text{m}}{65.2\,\text{mm}} \times \sqrt{1} \times \sqrt{\frac{280}{250}} = 40.58.$$

  Thus, $\alpha_c = 0.902$.

$$\phi N_c = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.902 \times 1 \times 11\,400\,\text{mm}^2 \times 280\,\text{MPa} = 2592\,\text{kN}.$$

- Right Column
  The modified slenderness ratio,

$$\lambda_n = \frac{k_e L_y}{r_y}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{1\times 2.5\,\text{m}}{64.5\,\text{mm}} \times \sqrt{1} \times \sqrt{\frac{300}{250}} = 42.46.$$

  Thus, $\alpha_c = 0.895$.

$$\phi N_c = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.895 \times 1 \times 9320\,\text{mm}^2 \times 300\,\text{MPa} = 2251\,\text{kN}.$$

The results match the capacity table well.

To sum up, the following design criteria shall be met.

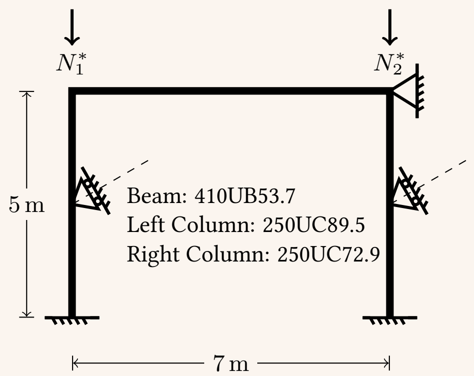
$$N_1^* \leqslant \min(2509\,\text{kN},\ 2592\,\text{kN}) = 2509\,\text{kN},$$
$$N_2^* \leqslant \min(2173\,\text{kN},\ 2251\,\text{kN}) = 2173\,\text{kN},$$
$$N_1^* + N_2^* \leqslant 4220\,\text{kN}.$$

If $N_1^* = 2500\,\text{kN}$, the frame fails with a sway mode when

$$N_2^* = \min(2173\,\text{kN}, 4220\,\text{kN} - N_1^*) = 1720\,\text{kN}.$$

Example 4.9 *Worksheet Axial Compression Example — Braced Frame*

Same as the previous example but the frame is now braced.



Solution 4.9

Since now the frame is braced, we use French equation to obtain k_e for strong axis buckling.

- Column 1.

  Given that $\gamma_1 = 1.065$ and $\gamma_2 = 1$, from French equation,

  $$k_e = \frac{3\gamma_1\gamma_2 + 1.4\,(\gamma_1 + \gamma_2) + 0.64}{3\gamma_1\gamma_2 + 2\,(\gamma_1 + \gamma_2) + 1.28} = 0.782.$$

  Then,

  $$\lambda_n = \frac{k_e L_x}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{0.782 \times 5\,\text{m}}{112\,\text{mm}} \times \sqrt{1} \times \sqrt{\frac{280}{250}} = 36.93.$$

  Thus $\alpha_c = 0.917$,

  $$\phi N_c = \phi\alpha_c k_f A_n f_y = 0.9 \times 0.917 \times 1 \times 11\,400\,\text{mm}^2 \times 280\,\text{MPa} = 2635\,\text{kN}.$$

- Column 2.

  Given that $\gamma_1 = 0.849$ and $\gamma_2 = 1$, from French equation,

  $$k_e = \frac{3\gamma_1\gamma_2 + 1.4\,(\gamma_1 + \gamma_2) + 0.64}{3\gamma_1\gamma_2 + 2\,(\gamma_1 + \gamma_2) + 1.28} = 0.768.$$

  Then,

  $$\lambda_n = \frac{k_e L_x}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{0.768 \times 5\,\text{m}}{111\,\text{mm}} \times \sqrt{1} \times \sqrt{\frac{300}{250}} = 37.87.$$

  Thus $\alpha_c = 0.913$,

  $$\phi N_c = \phi\alpha_c k_f A_n f_y = 0.9 \times 0.913 \times 1 \times 9320\,\text{mm}^2 \times 300\,\text{MPa} = 2298\,\text{kN}.$$

The weak-axis buckling capacities are the same as in the previous example. The design criteria considering both strong and weak axis buckling are

$$N_1^* \leqslant \min\,(2635\,\text{kN},\; 2592\,\text{kN}) = 2592\,\text{kN},$$
$$N_2^* \leqslant \min\,(2298\,\text{kN},\; 2251\,\text{kN}) = 2251\,\text{kN}.$$

If the frame is braced, in this particular case, the weak axis buckling governs.

4.3.3 Leaning Columns

Moment connections are expensive so frames often have two types of connections:

- moment connections that develop frame action (transfer moments) (Fully Restrained, FR)
- 'simple' connections (Partially Restrained, PR)

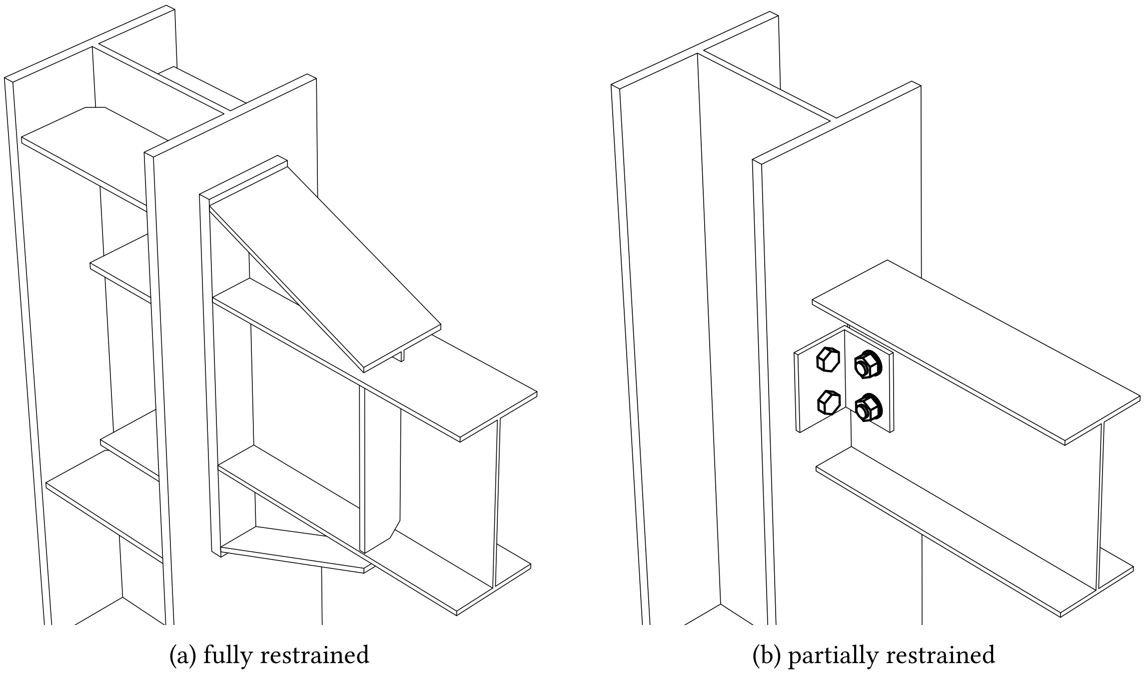


Figure 4.23: Connection types

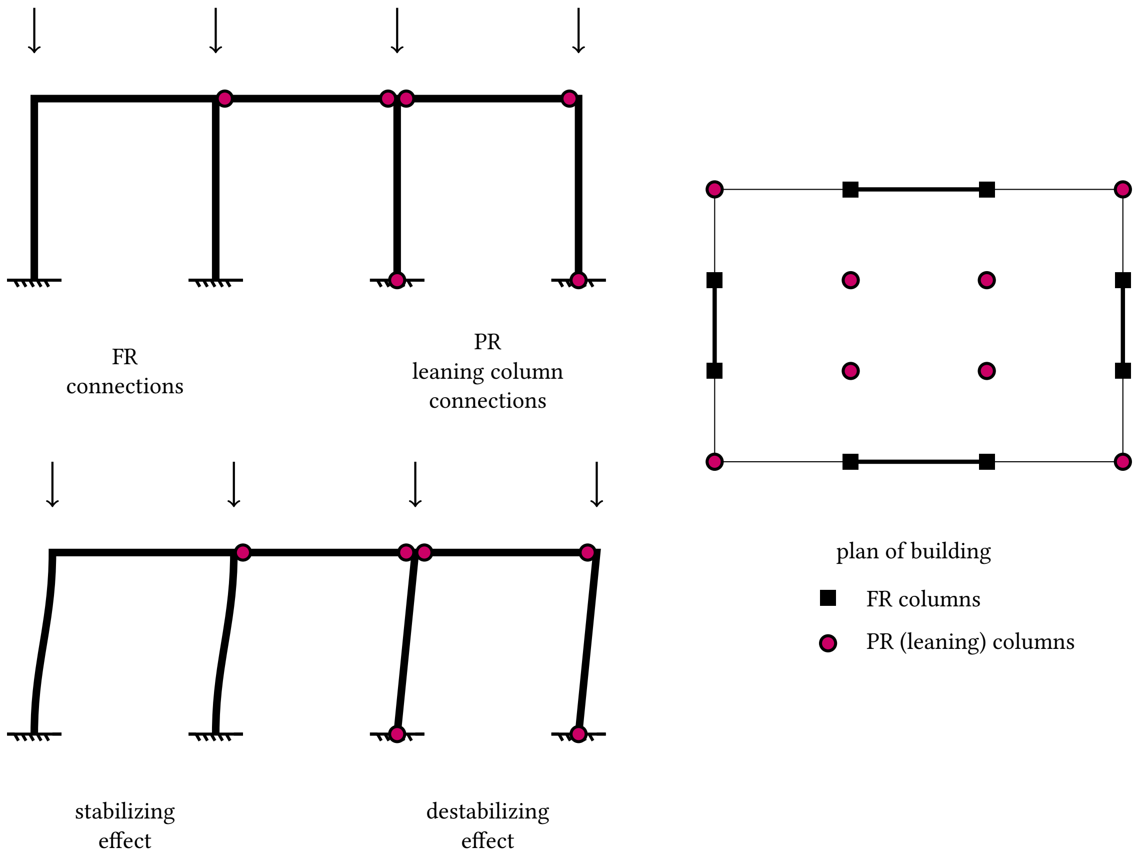
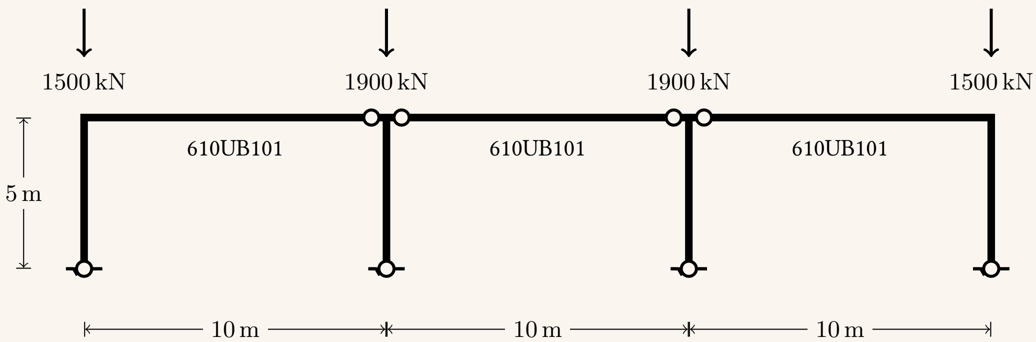


Figure 4.24: Destabilizaing effect of leaning columns

The FR frame provides a stabilizing effect for all of the columns, NZS 3404.1&2:1997 Steel structures standard requires that destabilizing effect of 'leaning columns' be accounted for. For 'leaning columns', use $k_e = 1$.

Example 4.10 Worksheet *Use Grade 300 steel. For the following sway frame, assume axial rigidity and out-of-plane buckling restrained. Design columns using UC sections.*



Solution 4.10

For exterior columns, the total force to be carried is

$$\sum N^* = 2 \times (1500\,\text{kN} + 1900\,\text{kN}) = 6800\,\text{kN}.$$

Try 310UC158, $f_y = 280\,\text{MPa}$ and $r_x = 139\,\text{mm}$,

$$\gamma_{top} = \gamma_1 = \frac{I_c/L_c}{\beta_e I_b/L_b} = \frac{388 \times 10^6\,\text{mm}^6/5\,\text{m}}{0.5 \times 761 \times 10^6\,\text{mm}^6/10\,\text{m}} = 2.039,$$
$$\gamma_{bot} = \gamma_2 = 10.$$

Note the far end of the beam is pinned and sway is allowed, from Table 4.7, $\beta_e = 0.5$. From French equation, $k_e = 2.126$. Thus,

$$\lambda_n = \frac{k_e L}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{2.126 \times 5\,\text{m}}{139\,\text{mm}} \times \sqrt{1} \times \sqrt{\frac{280}{250}} = 80.92.$$

This leads to $\alpha_c = 0.674$,

$$\phi N_c = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.674 \times 1 \times 20\,100\,\text{mm}^2 \times 280\,\text{MPa} = 3415\,\text{kN}.$$

Thus, the total capacity of two columns is

$$\sum \phi N_c = 2 \times 3415\,\text{kN} = 6830\,\text{kN} > 6800\,\text{kN}.$$

For interior leaning columns, assume $f_y = 280\,\text{MPa}$, try $r_x = 100\,\text{mm}$,

$$\lambda_n = \frac{k_e L}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = \frac{1 \times 5\,\text{m}}{100\,\text{mm}} \times \sqrt{1} \times \sqrt{\frac{280}{250}} = 52.92.$$

Thus $\alpha_c = 0.846$,

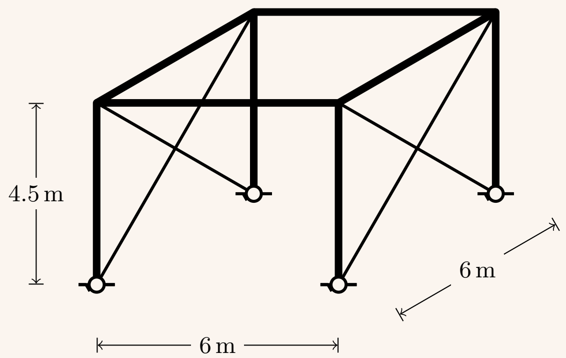
$$A_n \geqslant \frac{N^*}{\phi \alpha_c k_f f_y} = \frac{1900\,\text{kN}}{0.9 \times 0.846 \times 1 \times 280\,\text{MPa}} = 8909\,\text{mm}^2.$$

Thus choose 250UC72.9 for interior columns. It can be checked 250UC72.9 satisfies the design criterion by updating all quantities while the next designation 200UC59.5 does not. The braced capacity is greater than the sway capacity.

In conclusion, choose 250UC72.9 for interior columns and 310UC158 for exterior columns.

Example 4.11 *Loading Dock*

A loading dock is supported by four columns. It is braced in one direction and unbraced in the other. Find the column strength for different combinations of beam (310UB46.2 and 610UB125) and column (250UC89.5 and 310UC158) sections. The columns are oriented so that the weak axis direction is braced.



Solution 4.11

For all cases, $\beta_e = 1$. Using the French equations, the effective lengths for each combination can be found as follows.

beam	column		γ_{bot}	γ_{top}	k_e	L_e
310UB46.2	250UC89.5	braced (weak axis)	10	0.65	0.83	3.75
		unbraced (strong axis)	10	1.91	2.10	9.45
310UB46.2	310UC158	braced (weak axis)	10	1.67	0.90	4.04
		unbraced (strong axis)	10	5.17	2.58	11.61
610UB125	250UC89.5	braced (weak axis)	10	0.07	0.71	3.21
		unbraced (strong axis)	10	0.19	1.70	7.67
610UB125	310UC158	braced (weak axis)	10	0.17	0.75	3.36
		unbraced (strong axis)	10	0.52	1.79	8.07

By looking up the design tables, the critical loads can be found.

beam	column	total weight kg	braced capacity kN	unbraced capacity kN	critical kN	load/weight $\mathrm{kN\,kg^{-1}}$
310UB46.2	250UC89.5	2719.80	2310.07	1766.14	1766.14	2.60
310UB46.2	310UC158	3952.80	4254.20	3146.73	3146.73	3.18
610UB125	250UC89.5	4611.00	2441.89	2103.52	2103.52	1.82
610UB125	310UC158	5844.00	4472.80	4054.05	4054.05	2.77

Frame stiffness and therefore buckling capacity depend on stiffness of beams and columns. A proper combination shall be chosen to optimise the design (maximise load carried per weight).

4.3.4 Design of Columns in Frames

In the previous discussion it is assumed that the columns are axially stiff.

- When columns are not axially stiff and loading is asymmetric, axial shortening occurs which leads to sidesway and moments.
- When frame is not symmetric and columns are not axially stiff, sidesway and moments would be induced.
- When member loads and lateral forces are present, moments develop in members.

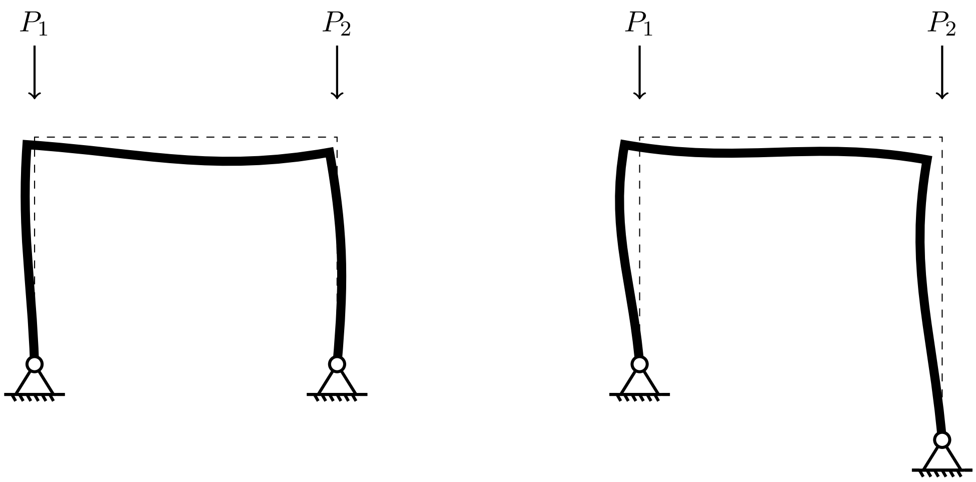


Figure 4.25: Sidesway due to non-rigid columns

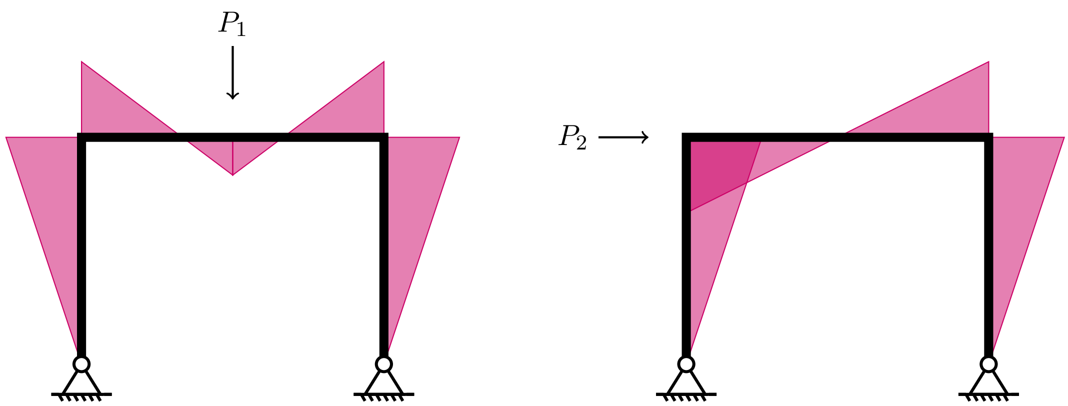
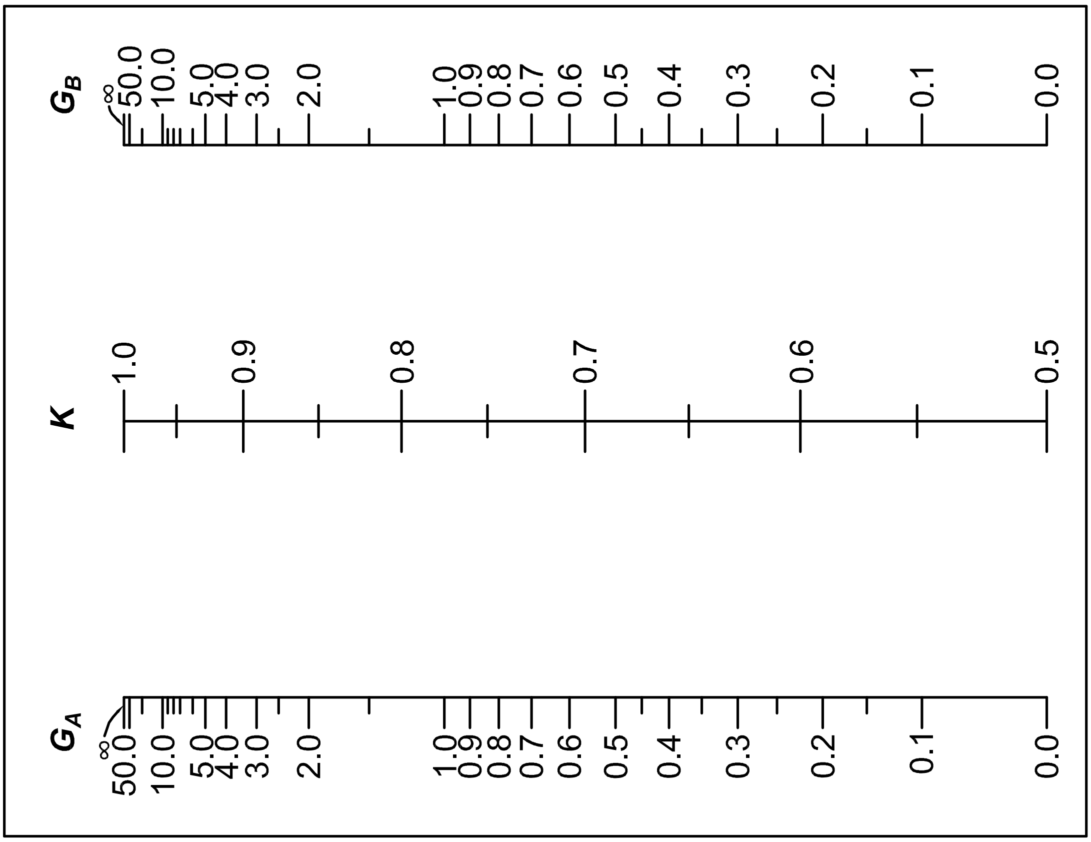
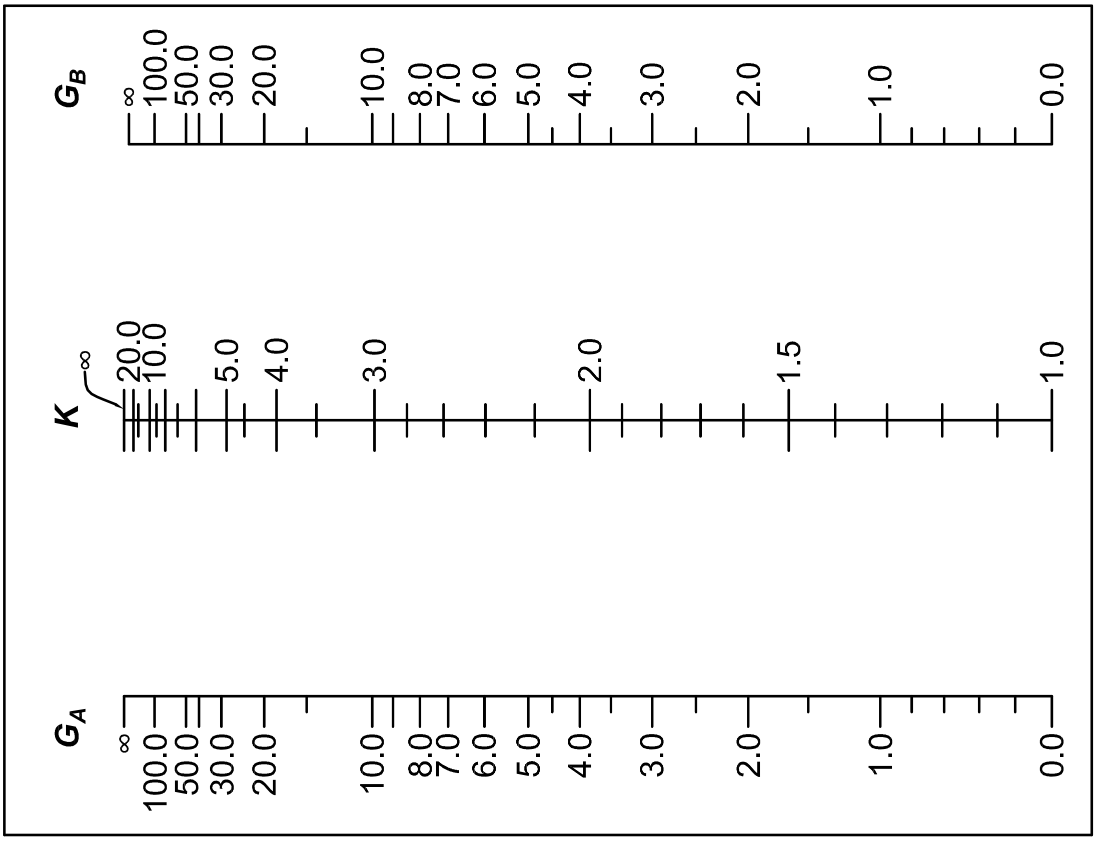


Figure 4.26: Moment developed in columns due to horizontal loads

If moments occur in conjunction with axial force, then he member shall be designed for both moment and axial force. Furthermore, axial force will enhance moment (P-Δ and P-δ effects). We will discuss this in design of bending members.



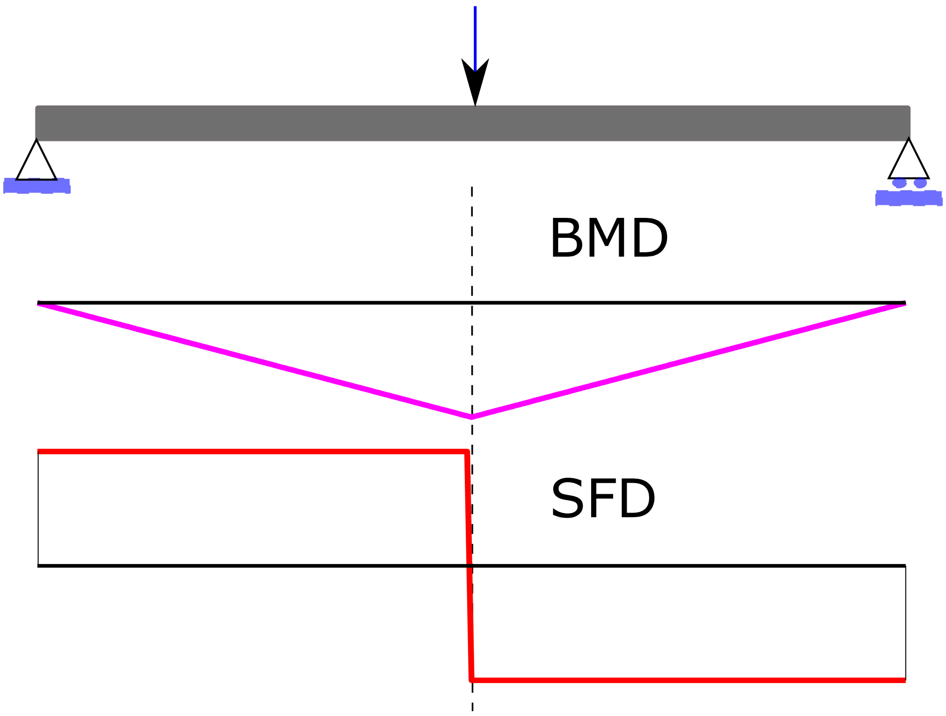
(a) alignment chart for k_e for braced members



(b) alignment chart for k_e for sway members

Bending Members

Beams are generally horizontal members supporting vertical loads and sometimes also frame lateral effects (e.g., from wind and earthquakes).



5.1 Limit States

The limit states of a bending member may consist of the following aspects.

- strength
  - flexural

    * section
        · yielding
        · local buckling
    * member lateral buckling
        · FLT buckling with residual stress
        · FLT buckling with warping
  - shear
    * yielding
    * buckling
  - concentrated load
    * yielding
    * buckling

- serviceability
  - excessive deflection
  - excessive vibration

Interaction may occur between limit states (e.g., beam yielding and local buckling). Such interactions should be considered in design. The only requirement for a well designed beam is that: *The capacity must be greater than the demands for each limit state.*

The most efficient beams to carry flexure are those with the areas of steel in tension and compression as far apart as possible. The limit states above, and constructability/installation/cost issues give limit the practical sizes that can be used.

We will look at the limit states in turn.

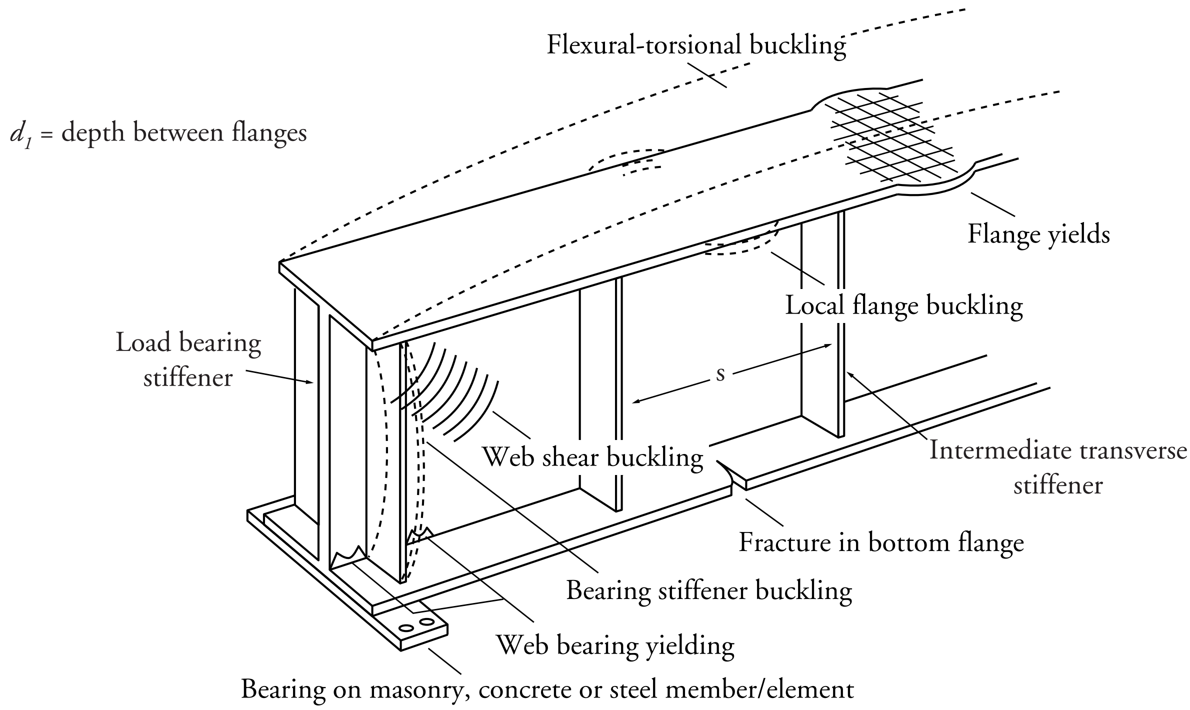


Figure 5.1: Critical areas for consideration of web stiffeners (Gorenc et al., 2015)

5.2 Section Flexural Yielding Strength

We often adopt the **elastic perfectly plastic** idealisation for steel material response.

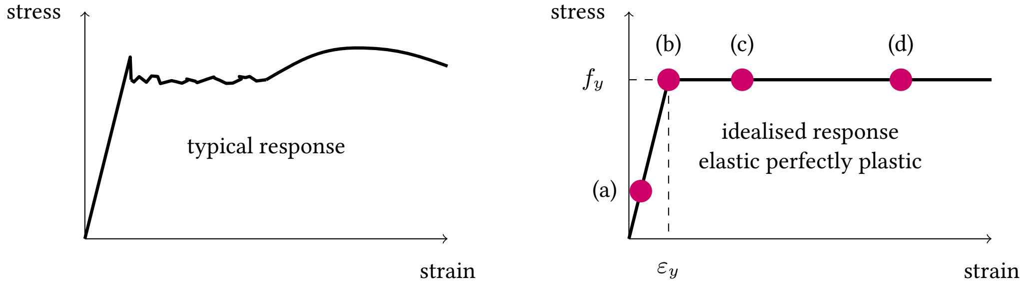


Figure 5.2: Idealisation of steel response

The development of plasticity of a rectangular section subject to increasing bending can thus be illustrated as follows. The material responses of extreme fibres, that correspond to four states, are labelled in the above figure as well.

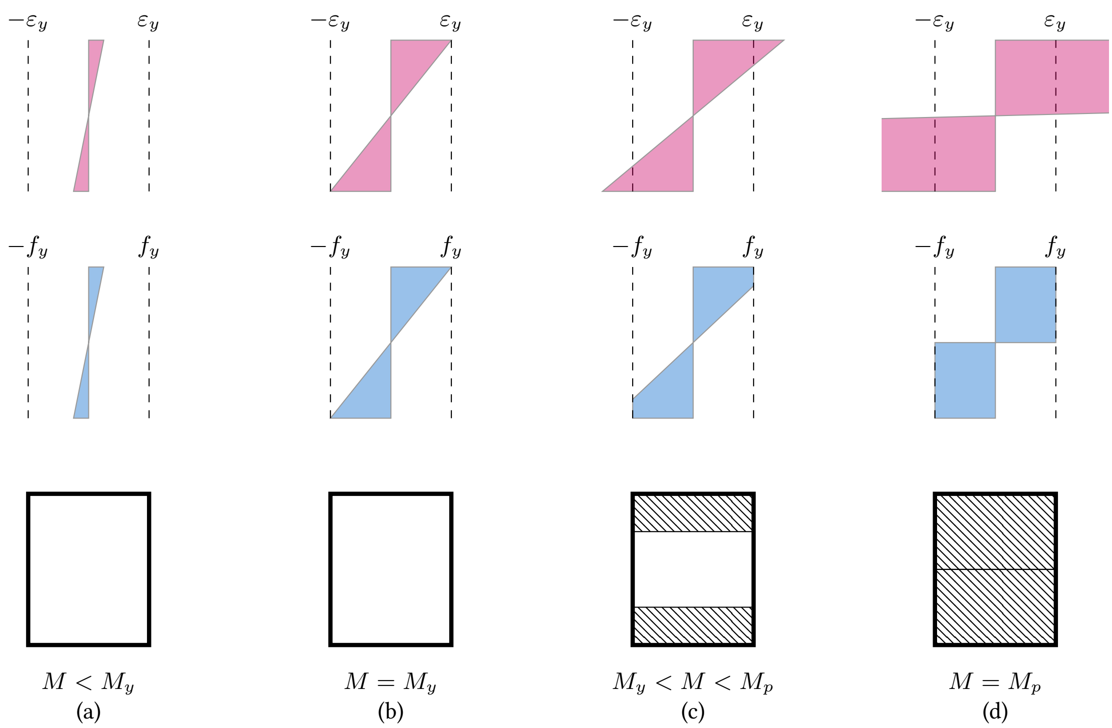


Figure 5.3: Development of plasticity of a rectangular section

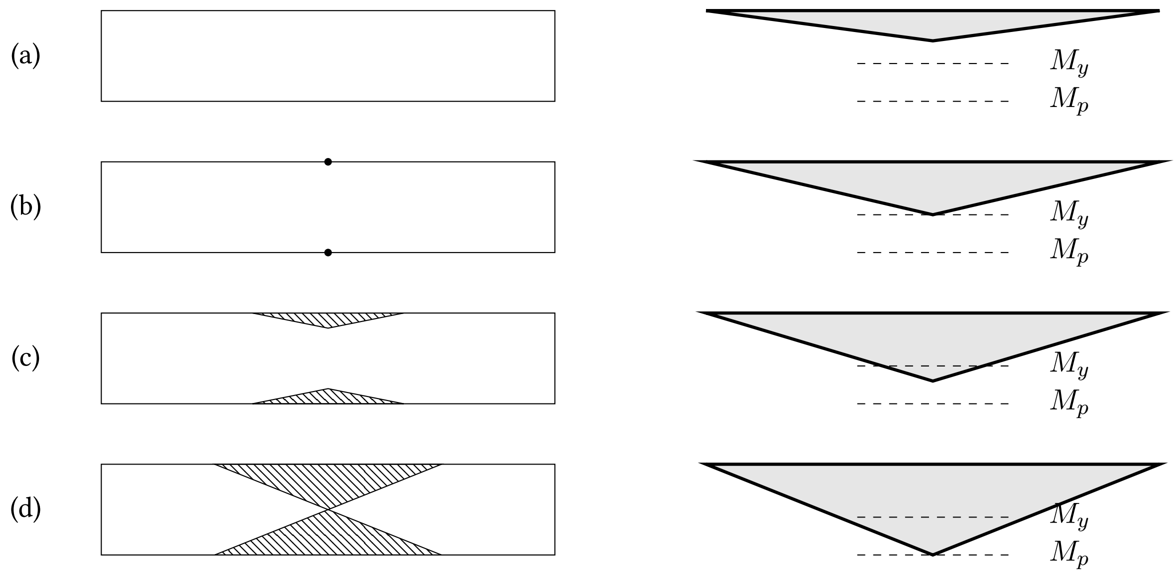
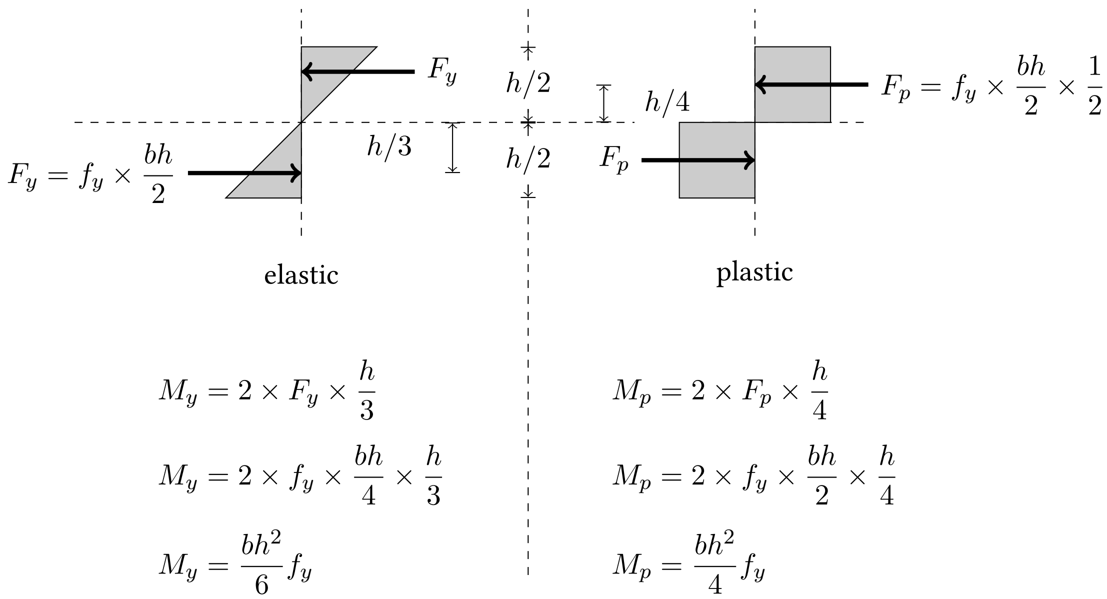


Figure 5.4: Development of plasticity of a rectangular section in a beam

Denote the height and width of the section to be h and b, the yield moment M_y and plastic moment M_p can be computed as

$$M_y = \frac{bh^2}{6} f_y = Z f_y, \qquad M_p = \frac{bh^2}{4} f_y = S f_y,$$

where Z and S are elastic and plastic section moduli. Note in ANSI/AISC 360-16 Specification for Structural Steel Buildings, S is used for elastic section modulus and Z is used for plastic section modulus, in Eurocode 3, W_{el} and W_{pl} are used. No matter which convention is used, the plastic section modulus is always greater than the elastic section modulus.



The moduli Z and S for each section are given in the specification manual. Like form factor k_f, there is no need to calculate them in practice. The ratio between S and Z is defined as the shape factor.

$$\text{SF} = \frac{S}{Z}.$$

For rectangular sections, SF $= 1.5$. The value of shape factor would vary between 1 and 1.5 depending on different section shapes. As the shape factor increases, more of the length of the beam yields resulting in less concentration of the inelastic rotation, but a higher likelihood of element buckling instability.

If one plots section curvature versus moment, the following response can be obtained.

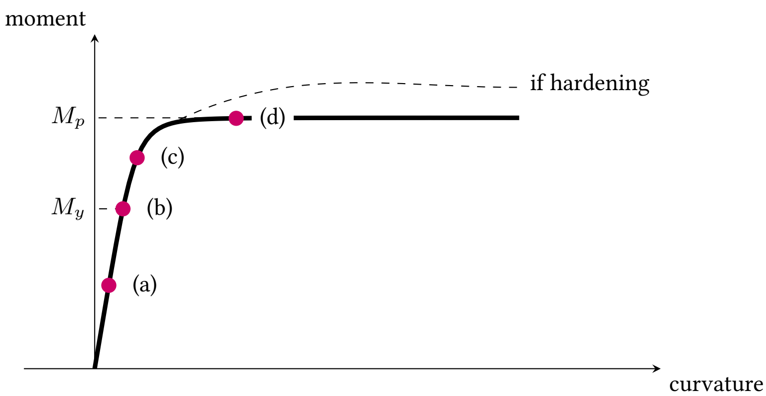


Figure 5.5: Section response

If the section **does not buckle** and is **ductile**, then the section strength is closer to M_p than to M_y. The fact that we have strain hardening in the steel means that M_p can easily be obtained in actual members, although M_p implies an infinite curvature with the steel model chosen.

5.2.1 Calculation of Plastic Modulus

The plastic modulus S for random sections with uniform yield stress is essentially its **first moment of area**.

$$S_x = \int_A |y| \, \mathrm{d}A, \qquad S_y = \int_A |x| \, \mathrm{d}A,$$

where x and y are the perpendicular distances (lever arms) to the centroid of element $\mathrm{d}A$ from the corresponding plastic neutral axes.

The plastic neutral axis coincides with the centroid of section which splits the area into two equal halves.

It should be emphasised that the above statements are **only** valid for sections with uniform yield stress.

» 5.3 Strength Considering Local Buckling

Sections with high element slendernesses are likely to buckle before the plastic moment capacity is reached.

The slenderness of any **flat element** i of the section for flexure, $\lambda_{e,i}$, is obtained in a similar way as it is for compression members (NZS 3404.1&2:1997 § 5.2.2.1):

$$\lambda_{e,i} = \frac{b_i}{t_i}\sqrt{\frac{f_{y,i}}{250\,\mathrm{MPa}}},$$

where b_i is again the clear width of the element outstand from the face of the supporting plate element or the clear width of the element between the faces of supporting plate elements and t_i is the element thickness.

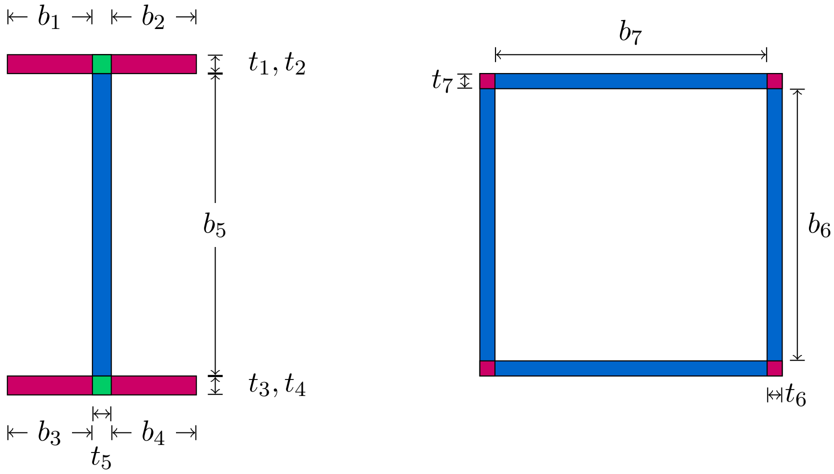


Figure 5.6: Flat elements in different sections

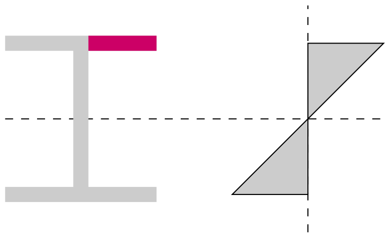
The slenderness for the **whole section** λ_s is set to λ_e for the element with the **greatest** ratio of λ_e/λ_{ey}. Similarly, the slenderness limits for the whole section λ_{sp} and λ_{sy} are taken as λ_{ep} and λ_{ey} for the element with the greatest ratio of λ_e/λ_{ey}. The element slenderness limits λ_{ep} and λ_{ey} are given in Table 5.1 (NZS 3404.1&2:1997 Table 5.2).

Table 5.1: Values of slenderness limits for flat elements

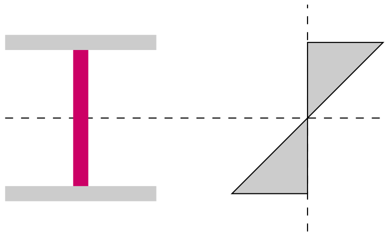
longitudinal edges supported	compression distribution	residual stress	λ_{ep}	λ_{ey}	λ_{ed}
one	uniform	SR	10	16	35
		HR	9	16	35
		LW, CF	8	15	35
		HW	8	14	35
	gradient	SR	10	25	
		HR	9	25	
		LW, CF	8	22	
		HW	8	22	
both	uniform	SR	30	45	90
		HR	30	45	90
		LW, CF	30	40	90
		HW	30	35	90
	gradient	Web of RHS and SHS	45	60	
		Other	85	130	

The following are some examples on different cases of supported edges and compression distributions.

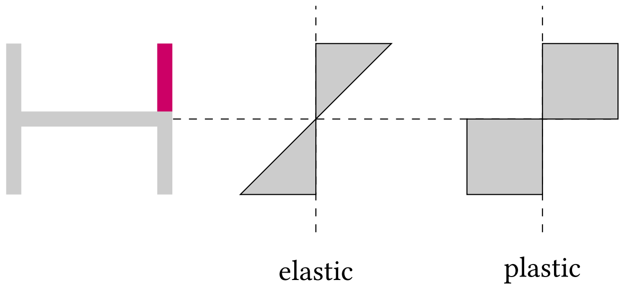
- For flanges of I sections subject to strong axis bending, only **one** edge is supported, both ends experience the same magnitude of normal stress, thus the compression distribution pattern is **uniform**. For HR sections, $\lambda_{ep} = 9$ and $\lambda_{ey} = 16$.



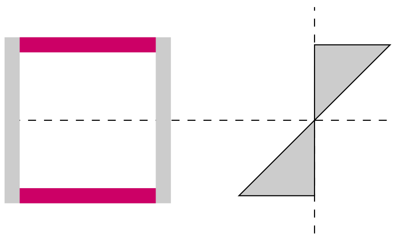
- For web of I sections subject to strong axis bending, **both** edges are supported, one end experiences compression while the other experiences tension, thus the compression distribution pattern is **gradient**. For HR sections, $\lambda_{ep} = 85$ and $\lambda_{ey} = 130$.



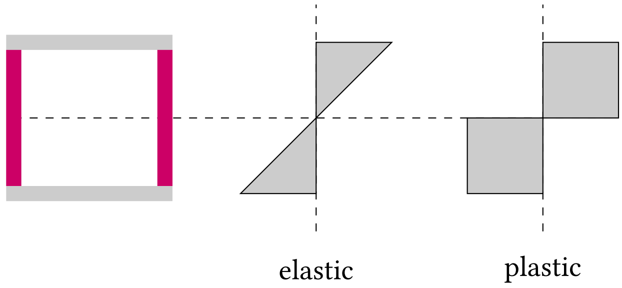
- For flanges of I sections subject to weak axis bending, only **one** edge is supported, but one end experiences compression/tension while the other has zero normal stress, thus the compression distribution pattern is **gradient**. For HR sections, $\lambda_{ep} = 9$ and $\lambda_{ey} = 25$.



- For flanges of RHS/SHS sections subject to strong axis bending, **both** edges are supported, both ends experience the same magnitude of normal stress, thus the compression distribution pattern is **uniform**. For HW sections, $\lambda_{ep} = 30$ and $\lambda_{ey} = 35$.



- For webs of RHS/SHS sections subject to strong axis bending, **both** edges are supported, one end experiences compression while the other experiences tension, thus the compression distribution pattern is **gradient**. Thus, $\lambda_{ep} = 45$ and $\lambda_{ey} = 60$.



Sections of different slendernesses are defined in the following way:

- $\lambda_s \leqslant \lambda_{sp}$ — compact
- $\lambda_{sp} < \lambda_s \leqslant \lambda_{sy}$ — non-compact
- $\lambda_{sy} < \lambda_s$ — slender

The real behaviour of different types of sections, accounting for plastic material response, can be described in the following figure.

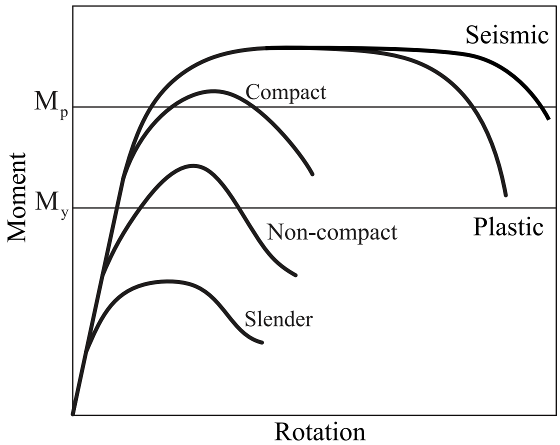


Figure 5.7: Flexural behaviour of different types of sections

The nominal section moment capacity M_s shall be calculated as

$$M_s = f_y Z_e, \tag{5.1}$$

where Z_e is the effective section modulus which shall be determined by section type and section slenderness ratio.

Compact Section ($M = M_c$)

When section slenderness ratio is smaller than the section plasticity limit, $\lambda_s \leqslant \lambda_{sp}$, the section is **compact**, reaching a strength of $M_c = \min(M_p,\ 1.5M_y)$. NZS 3404.1&2:1997 § 5.2.3 defines Z_e to be the compact modulus Z_c for compact sections, which shall be the smaller of S and $1.5Z$. The compactness is given along with other section specifications.

$$Z_e = Z_c = \min(S,\ 1.5Z)\,. \tag{5.2}$$

When holes exist, S and Z shall be recomputed considering holes (NZS 3404.1&2:1997 § 5.2.7).

Non-compact Section ($M_y \leqslant M \leqslant M_c$)

When section slenderness ratio is between the section plasticity and yield limits, $\lambda_{sp} < \lambda_s \leqslant \lambda_{sy}$, the section is **non-compact**. Then effective section modulus Z_e shall be calculated via linear interpolation (NZS 3404.1&2:1997 § 5.2.4).

$$Z_e = Z + \frac{\lambda_{sy} - \lambda_s}{\lambda_{sy} - \lambda_{sp}} \left(Z_c - Z \right), \tag{5.3}$$

where Z_c is the compact modulus defined in Eq. (5.2).

Slender Section ($M < M_y$)

When section slenderness ratio is greater than the section yield limit, $\lambda_s > \lambda_{sy}$, the section is **slender**. For slender sections, many cases shall be considered. For simplicity, one can use the following expression (NZS 3404.1&2:1997 § 5.2.5.2) to calculate Z_e as a conservative design.

$$Z_e = Z \left(\frac{\lambda_{sy}}{\lambda s} \right)^2 . \tag{5.4}$$

It shall be noted that slender sections are often not economical thus generally avoided.

If one plots M_s against λ_s, the curve has a similar shape as seen in compression members in which elastic buckling is expected for slender sections, and residual stress effects control the behaviour. The effective section moduli Z_e for both axes are also given in the specification.

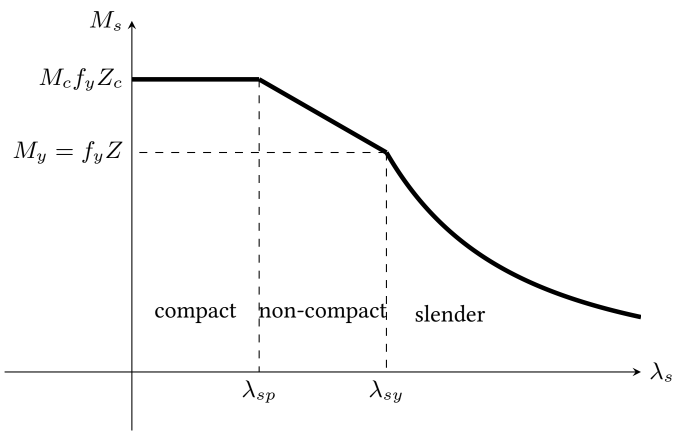


Figure 5.8: Section capacity as a function of slenderness

5.4 Strength Considering Member Buckling

A bending member, like a compression member, may undergo lateral buckling. However, only the part of the section in compression has the tendency to buckle.

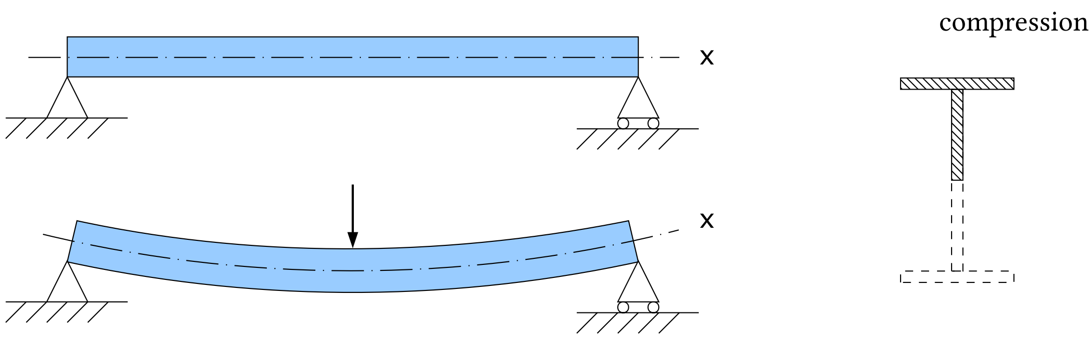


Figure 5.9: Compression develops in the upper part of a beam

Beam buckling over its length is generally called Flexural Lateral Torsional (FLT) buckling. Because part of the section is in tension, and doesn't tend to buckle, it restrains the total connection.

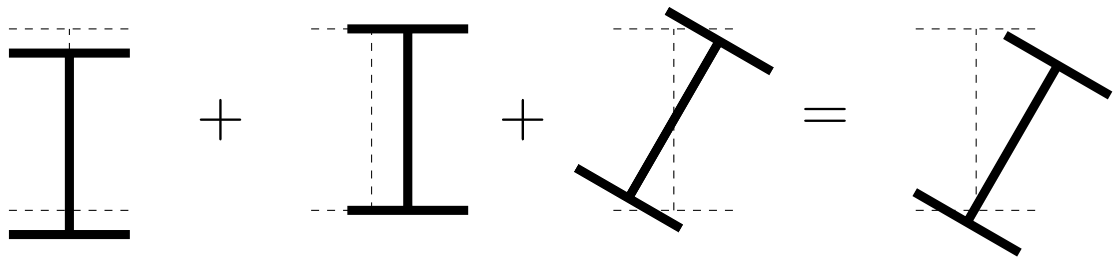


Figure 5.10: Components of FLT buckling

The amount of FLT buckling depends on

- slenderness of compression part of section
- orientation of section
- member length
- member bracing type and positions
- type and location of loading

Interested readers can check this[1] video.

5.4.1 Orientation Effect

I sections bending about their strong axis with an unbraced compression flange are susceptible to buckling. However, strengths of other sections are not always affected by buckling. Lateral buckling is not generally significant in the following members.

[1] https://www.youtube.com/watch?v=XSl7ZntK94E

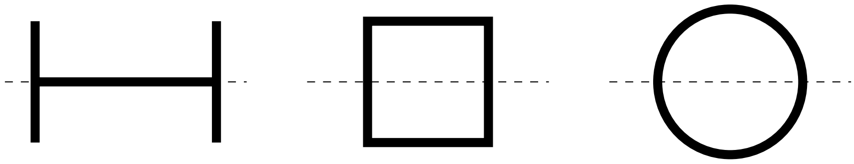


Figure 5.11: Sections less sensitive to lateral buckling

5.4.2 Uniform Moment

Reference Moment

From the mechanics of materials, it can be shown that the elastic buckling moment of a beam bending about its strong axis subject to a constant moment over its length, sometimes called the reference buckling moment, M_o, is given by (NZS 3404.1&2:1997 Eq. 5.6.1.1(4))

$$M_o = \underbrace{\sqrt{\frac{\pi^2 EI_y}{L_e^2}}}_{\text{lateral}} \underbrace{\sqrt{GJ + \frac{\pi^2 EI_w}{L_e^2}}}_{\text{torsional}}, \tag{5.5}$$

where

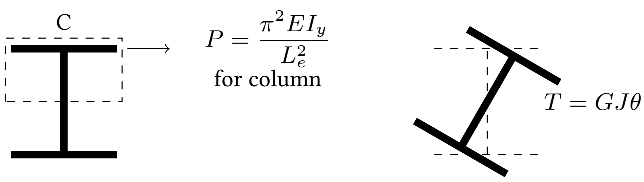
E = elastic modulus, 200 GPa
G = shear modulus, 80 GPa
I_y = the second moment of area about weak axis
J = torsion constant
I_w = warping constant
L_e = effective length of the beam **segment** considered

The lateral buckling term, EI_y, indicates that the compressive part of the section wants to move sideways.

There are two sources of twist,

- pure torsion (Saint Venant), related to term GJ;
- warping torsion, related to term EI_w.

The GJ term (i.e., St. Venant term) is for twist assuming plane sections remain plane.



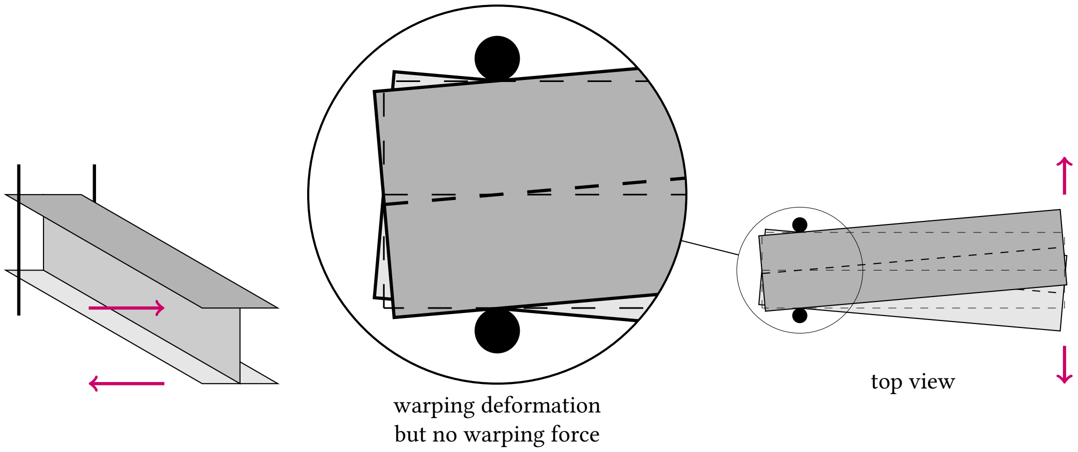
Warping refers to the deformation that plane sections do not remain plane. If warping deformation is restrained, warping stress would be developed. When there is no warping effect, $I_w = 0$,

$$M_o = \frac{\pi}{L_e}\sqrt{EI_yGJ}. \tag{5.6}$$

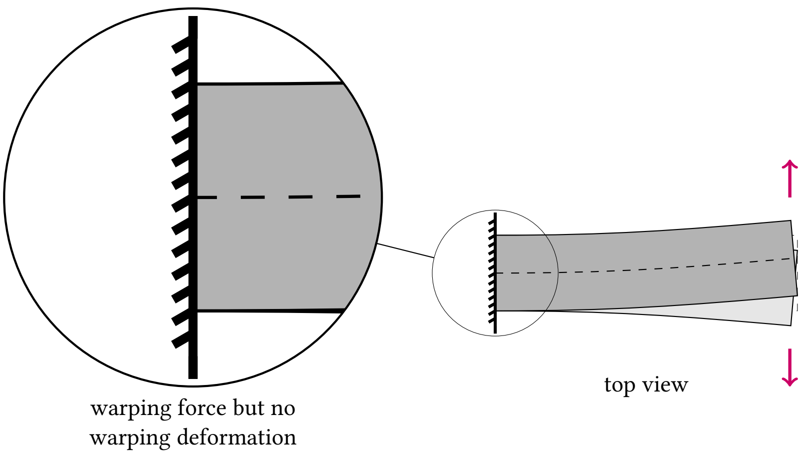
Warping is important for shorter members.

What is this thing called warping? Warping is the deformation such that plane sections do not remain plane.

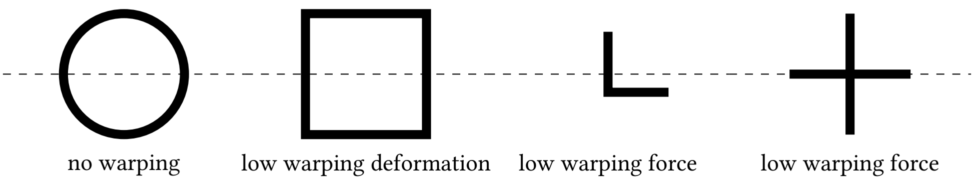
For a simply supported member subjected to end torsion, plane sections do not remain plane at ends. There are warping deformations but no warping stresses. In such a configuration, torsion is resisted by St. Venant effects only. The M_0 is a function of GJ.



If one end is restrained, plane sections remain plane at end, which means warping deformations are restrained **but** warping stresses/forces exist. In such a configuration, end rotation is less and member is stronger. The M_0 is a function of **both** EI_w and GJ.



Circular sections are free from warping. All other sections warp. Hollow sections have low warping deformation and angles have low warping stress but high warping deformation.



Based on this fact, for angles, rectangular/square/circular hollow sections, narrow rectangular sections and T sections, one shall use Eq. (5.6) to calculate M_o (NZS 3404.1&2:1997 Cl. 5.6.1.3 to Cl. 5.6.1.6). For all other sections, one shall use Eq. (5.5) to compute reference moment.

The most significant warping stress is often developed in open sections with several legs with all elements not framing into one point. For example, I and C sections. If readers want to learn more about warping, check this[2] video.

To calculate M_o, the effective length of a segment of the member L_e shall be defined. It is found as follows (NZS 3404.1&2:1997 § 5.6.3.1).

$$L_e = k_t k_l k_r L, \tag{5.7}$$

where

L = the beam **segment** or **subsegment** length between (partial (P), full (F), or lateral (L)) restraints
k_t = the twist restraint factor
k_l = the load height factor
k_r = the rotation restraint factor

A **segment** or **subsegment** may also be between the unrestrained (U) end of a cantilever and an adjacent section that is fully or partially restrained.

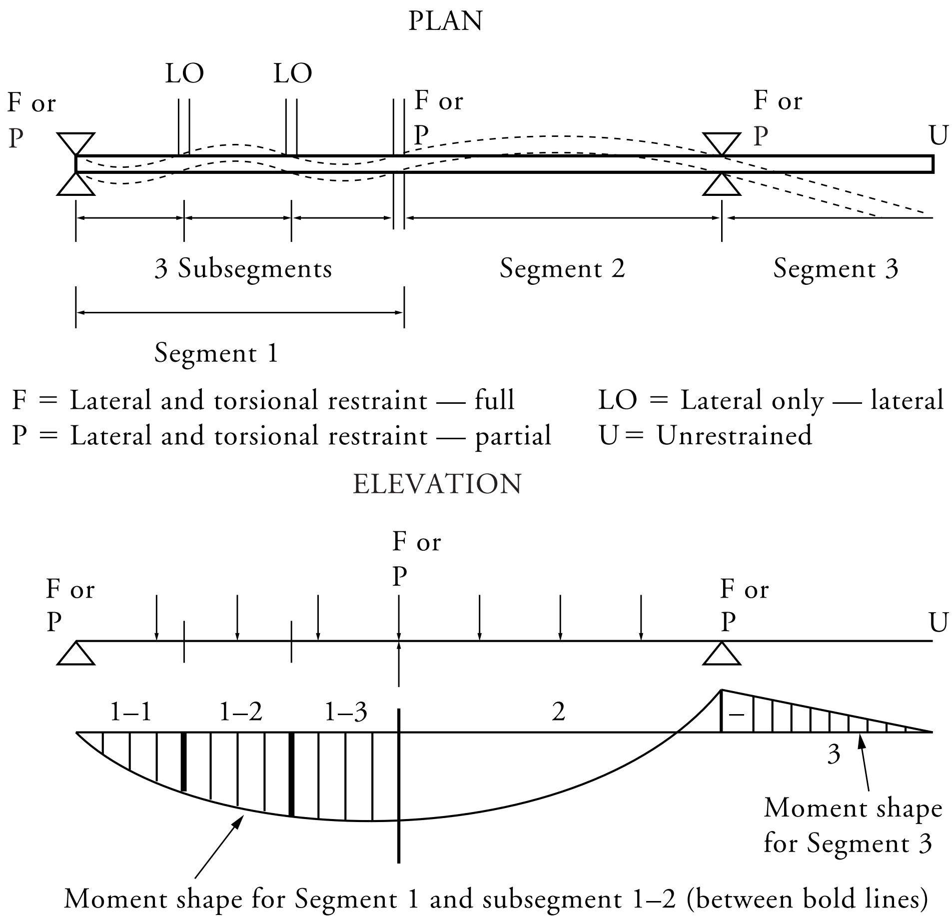


Figure 5.12: The division of a beam into segments and subsegments (Gorenc et al., 2015)

[2]https://www.youtube.com/watch?v=QtyGWZDPtCI

The factor k_t is for twist about z-axis (beam chord) at segment ends. It depends on the segment end restraint which may be categorised as F, L, P and U.

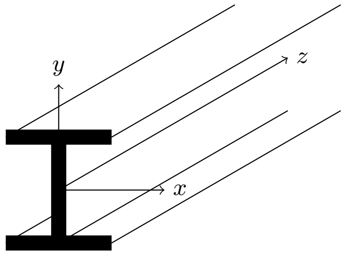


Figure 5.13: Definition of local axes

The factor k_l is the height of load abobe the neutral axis. Depending on where the load is applied, some cases are more likely to buckle.

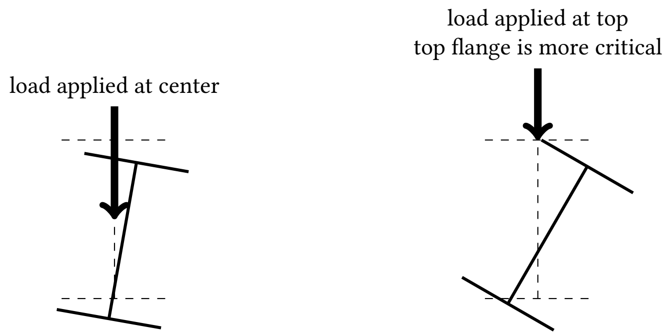


Figure 5.14: Loads applied to different heights

The k_r factor considers warping. If warping restraint exists, $k_r < 1$, segments become more rigid, then warping deformation is limited so that beam can carry more load.

The determination of k_t, k_l and k_r is summarised in Table 5.2.

Table 5.2: Parameters k_t, k_l and k_r for different end restraints

end restraint	k_t	k_l load height position			k_r ends with minor axis rotation restraints		
		shear centre	top flange				
			load within segment	load at segment end	none	one	both
FF	1.0	1.0	1.4	1.0	1.0	0.9	0.7
FL	1.0	1.0	1.4	1.0	1.0	1.0	1.0
LL	1.0	1.0	1.4	1.0	1.0	1.0	1.0
FU	1.0	1.0	2.0	2.0	1.0	1.0	1.0
FP	Eq. (5.8)	1.0	1.4	1.0	1.0	0.9	0.7
PL	Eq. (5.8)	1.0	1.4	1.0	1.0	1.0	1.0
PU	Eq. (5.8)	1.0	2.0	2.0	1.0	1.0	1.0
PP	Eq. (5.9)	1.0	1.4	1.0	1.0	0.9	0.7

Details and other considerations can be found in NZS 3404.1&2:1997 Table 5.6.3. The following two expressions are used in Table 5.2.

$$k_t = 1 + \frac{d}{8n_w L}\left(\frac{t_f}{t_w}\right)^3, \tag{5.8}$$

$$k_t = 1 + \frac{d}{4n_w L}\left(\frac{t_f}{t_w}\right)^3, \tag{5.9}$$

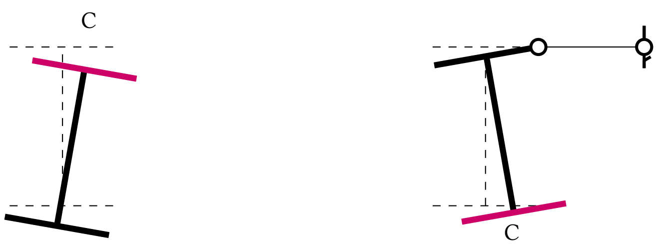
in which,

$$\begin{aligned} d &= \text{depth of section} \\ L &= \text{segment length} \\ n_w &= \text{number of webs} \\ t_f &= \text{thickness of critical flange (the flange in compression)} \\ t_w &= \text{thickness of web} \end{aligned}$$

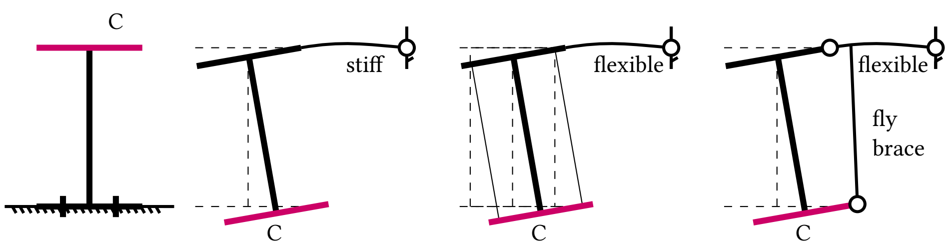
Segment End Restraint Type

For the definitions and classifications of restraints, readers can refer to NZS 3404.1&2:1997 § 5.4.2. Here some illustrations of each type are presented. Coloured flange denotes the critical flange — the flange in compression.

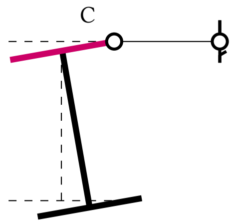
Unrestrained (U) There is no critical flange lateral restraint nor twist restraint in the following cases.



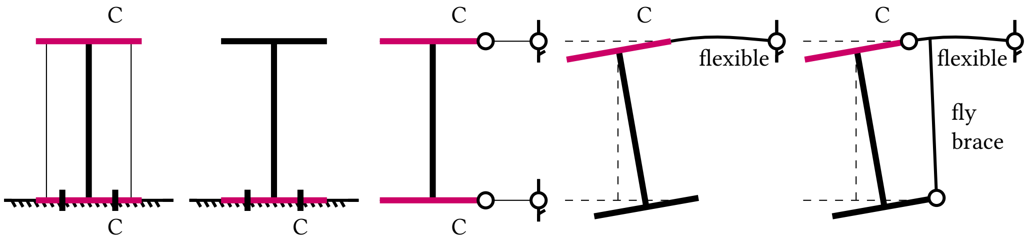
Partially Restrained (P) There is either non-critical flange lateral restraint or partial twist restraint in the following cases.



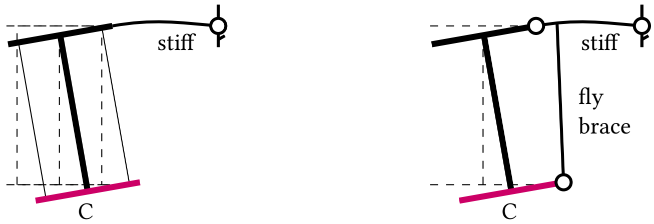
Laterally Restrained (L) There is critical flange lateral restraint but no twist restraint in the following case.



Fully Restrained (F) The following cases have critical flange lateral restraint and effective twist restraint.

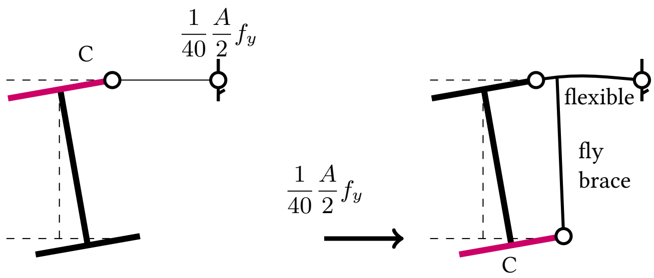


The following cases have non-critical flange lateral restraint and effective twist restraint.



Design of Restraints

The lateral restraint at any cross section considered to be fully, partially or laterally restrained, is designed to resist a transverse force acting on the critical flange of $2.5\,\%$ of the maximum force in the critical flanges.



Effect of Section Yielding on Beam Strength

With M_o, now we could define the **critical** bending strength M_{cr} under uniform flexure.

$$M_{cr} = \alpha_s M_s, \tag{5.10}$$

where α_s is the slenderness reduction factor to account for residual stresses and section yielding. It shall be computed computed as (NZS 3404.1&2:1997 Eq. 5.6.1.1(3))

$$\alpha_s = 0.6\left(\sqrt{\left(\frac{M_s}{M_o}\right)^2+3}-\frac{M_s}{M_o}\right). \tag{5.11}$$

When $M_s \ll M_o$, $\alpha_s = 0.6\times\sqrt{3} = 1.03 \approx 1$. As $M_s/M_o \to \infty$, M_{cr} is less than M_o.

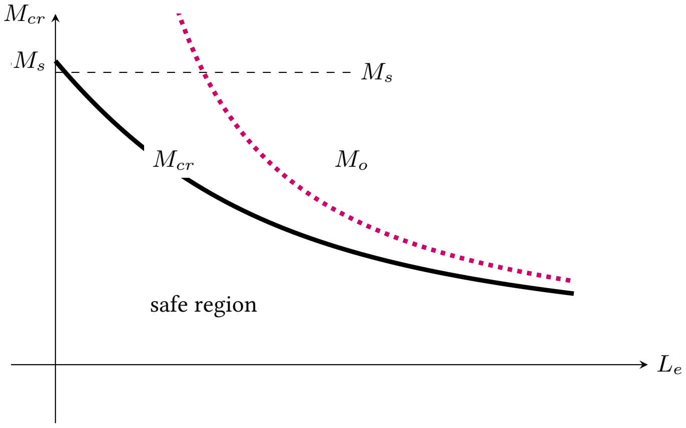


Figure 5.15: Critical moment as a function of effective length

5.4.3 Effect of Loading Type

The critical strength M_{cr} is obtained under uniform moment. However, not all members are subjected to uniform moment. To account for non-uniform moment, an additional factor α_m is used so that the nominal member moment capacity M_b can be calculated as

$$M_b = \min\left(\alpha_m M_{cr},\ M_s\right) = \min\left(\alpha_m\alpha_s M_s,\ M_s\right) = \min\left(\alpha_m\alpha_s,\ 1\right)\cdot M_s, \tag{5.12}$$

where α_m can be taken as 1, but this is in general too conservative in many situations.

Alternatively, it can be calculated from the maximum member moment M_m^*, which is taken as positive, and the moments at the **quarter** points of the target segments, M_2^*, M_3^* and M_4^* as

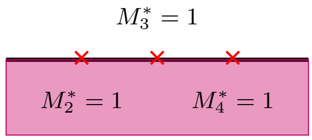
$$\alpha_m = \min\left(2.5,\ \frac{1.7M_m^*}{\sqrt{(M_2^*)^2+(M_3^*)^2+(M_4^*)^2}}\right). \tag{5.13}$$

NZS 3404.1&2:1997 § 5.6.1.1.1 also gives other methods to compute α_m.

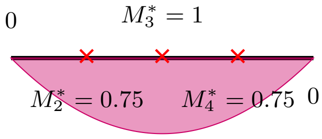
For a segment that does not contain a plastic hinge and is fully laterally braced, then $\alpha_m = 1$ and $M_s/M_o \to 0$, this leads to $\alpha_s \approx 1$, thus $M_b \approx M_s$.

For exmaple, for a **segment** of a beam:

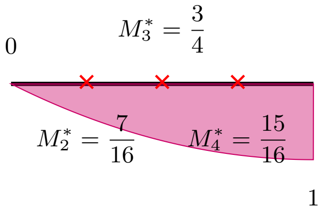
- For uniform bending, $M_m^* = M_2^* = M_3^* = M_4^*$, this leads to $\alpha_m = 0.98 \approx 1$. The uniform moment is indeed the critical case as all sections are subjected to the same magnitude of load which is easier to buckle.



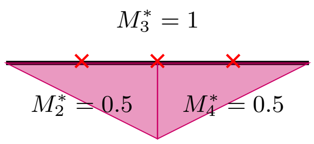
- For members under UDL, $\alpha_m = \dfrac{1.7 \times 1}{\sqrt{0.75^2 + 1^2 + 0.75^2}} = 1.17.$



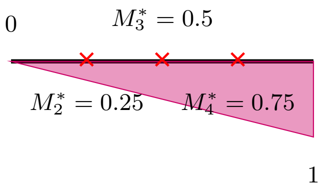
- For **half span** of members under UDL, $\alpha_m = \dfrac{1.7 \times 1}{\sqrt{\left(\frac{7}{16}\right)^2 + \left(\frac{3}{4}\right)^2 + \left(\frac{15}{16}\right)^2}} = 1.33.$



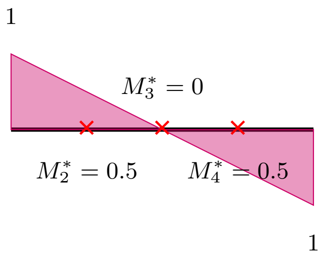
- For members under mid-span point load, $\alpha_m = \dfrac{1.7 \times 1}{\sqrt{0.5^2 + 1^2 + 0.5^2}} = 1.39.$



- For **half span** of members under mid-span point load, $\alpha_m = \dfrac{1.7 \times 1}{\sqrt{0.25^2 + 0.5^2 + 0.75^2}} = 1.82.$



- For members under reverse curvature, $\alpha_m = \dfrac{1.7 \times 1}{\sqrt{0.5^2 + 0^2 + 0.5^2}} = 2.40.$

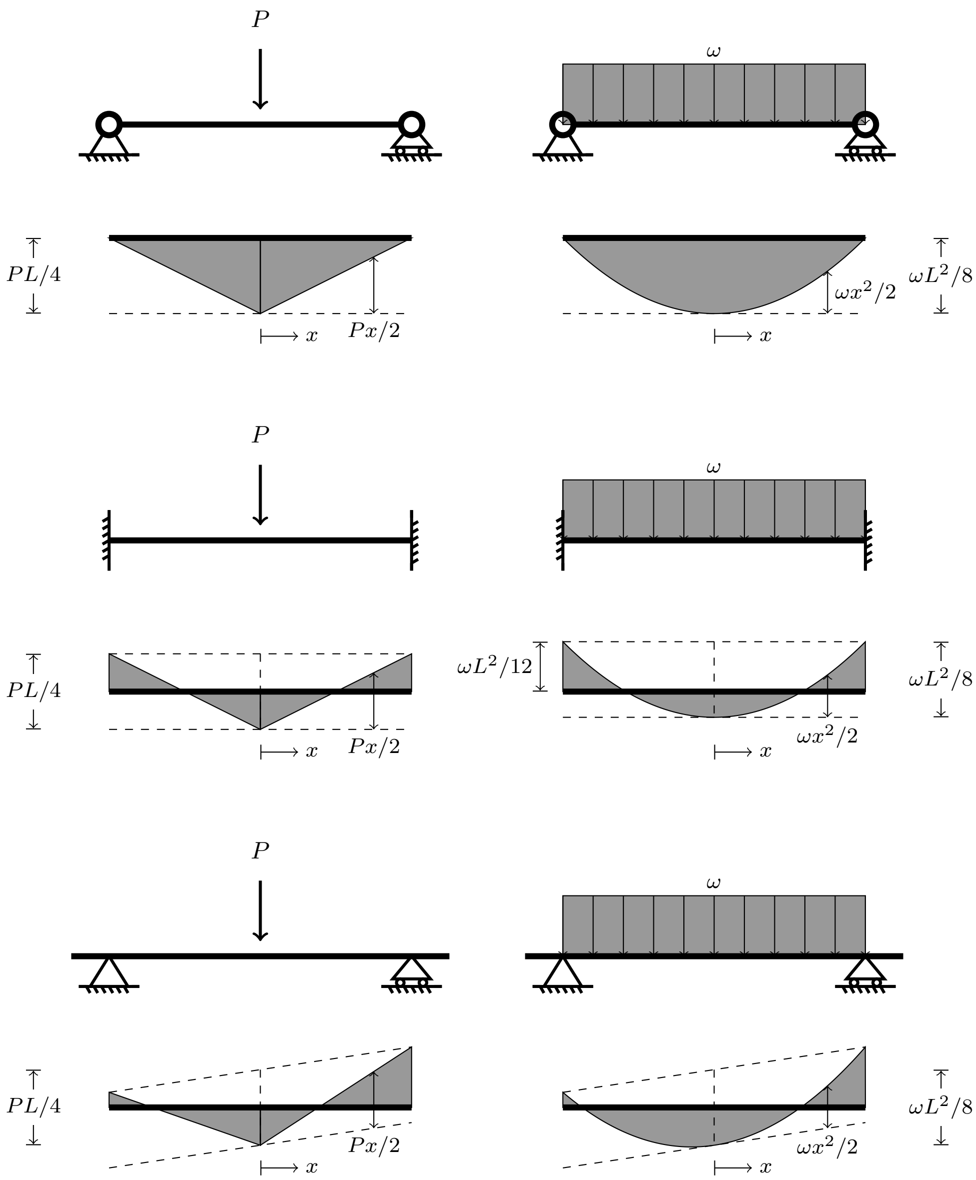


A member, which does not contain a plastic hinge, is fully laterally braced if $M_b = M_s$.

Design capacity tables and charts may be used for analysis and design. These are generally provided with $\alpha_m = 1$. To find the strength of a member with a certain L_e, it is necessary to:

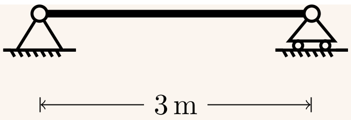
1. look up ϕM_s and $\phi \alpha_s M_s$ from tables and charts,
2. compute α_m according to bending moment diagram,
3. find member strength $\phi M_b = \min(\phi \alpha_m \alpha_s M_s,\ \phi M_s)$.

The following is a review of hand calculation of bending moment diagrams.



Example 5.1 *Worksheet Strong Axis Bending Example — Simply Supported Beam*

Determine the maximum factored moment M^* that a $3\,\mathrm{m}$ long simply supported 310UB32.0 Grade 300 steel beam can carry when fully laterally restrained.



Solution 5.1

- Member Slenderness
  From property table, $t_f = 8\,\mathrm{mm}$, $t_w = 5.5\,\mathrm{mm}$, $b_f = 149\,\mathrm{mm}$ and $d = 298\,\mathrm{mm}$.

  – flange

  $$\lambda_{e,f} = \frac{b_f - t_w}{2t_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{149\,\mathrm{mm} - 5.5\,\mathrm{mm}}{2 \times 8\,\mathrm{mm}} \times \sqrt{\frac{320}{250}} = 10.15,$$
  $$\lambda_{e,f}/\lambda_{ey,f} = 0.634.$$

  – web

  $$\lambda_{e,w} = \frac{d_1}{t_w}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} = \frac{282\,\mathrm{mm}}{5.5\,\mathrm{mm}} \times \sqrt{\frac{320}{250}} = 58.01,$$
  $$\lambda_{e,w}/\lambda_{ey,w} = 0.446.$$

  Thus flange governs, $\lambda_s = \lambda_{e,f} = 10.15$, $\lambda_{sy} = \lambda_{ey,f} = 16$ and $\lambda_{sp} = \lambda_{ep,f} = 9$.

- Section Capacity
  Since $\lambda_{sp} < \lambda_s < \lambda_{sy}$, this is a non-compact section. Compute compact section modulus.

  $$\begin{aligned} Z_c &= \min\left(S,\ 1.5Z\right) \\ &= \min\left(475 \times 10^3\,\mathrm{mm}^3,\ 1.5 \times 424 \times 10^3\,\mathrm{mm}^3\right) \\ &= 475 \times 10^3\,\mathrm{mm}^3. \end{aligned}$$

  Then,

  $$\begin{aligned} Z_e &= Z + \frac{\lambda_{sy} - \lambda_s}{\lambda_{sy} - \lambda_{sp}}\left(Z_c - Z\right) \\ &= 424 \times 10^3\,\mathrm{mm}^3 + \frac{16 - 10.15}{16 - 9}\left(475 \times 10^3\,\mathrm{mm}^3 - 424 \times 10^3\,\mathrm{mm}^3\right) \\ &= 466.6\,\mathrm{cm}^3. \end{aligned}$$

  Thus,

  $$M_s = f_y Z_e = 320\,\mathrm{MPa} \times 466.6\,\mathrm{cm}^3 = 149.3\,\mathrm{kN\,m}, \qquad \phi M_s = 134.4\,\mathrm{kN\,m}.$$

- Member Capacity
  Since the beam is fully laterally restrained, $\alpha_s = 1$, $\alpha_m = 1$, then

  $$M^* \leqslant \phi M_b = \phi M_s = 134.4\,\mathrm{kN\,m}.$$

  This agrees with the design capacity table.

Example 5.2 Worksheet *Same as in the previous example. Now the beam is only fully braced at its ends, find the maximum M^*. Assume a uniform moment over the beam length and loads are applied at beam ends.*

Solution 5.2

Since the moment is uniform, $\alpha_m = 1$. There is one segment of length 3 m, the factors can be found to be

$$k_t = 1.0, \qquad k_l = 1.0, \qquad k_r = 1.0.$$

Thus, $L_e = 3\,\mathrm{m}$. The reference moment is

$$\begin{aligned} M_o &= \sqrt{\frac{\pi^2 EI_y}{L_e^2}}\sqrt{\left(GJ + \frac{\pi^2 EI_w}{L_e^2}\right)} \\ &= \sqrt{\frac{\pi^2 \cdot 200\,\mathrm{GPa} \times 442\,\mathrm{cm}^4}{3\,\mathrm{m} \times 3\,\mathrm{m}}\left(80\,\mathrm{GPa} \times 8.65\,\mathrm{cm}^4 + \frac{\pi^2 \cdot 200\,\mathrm{GPa} \times 92\,900\,\mathrm{cm}^6}{3\,\mathrm{m} \times 3\,\mathrm{m}}\right)} \\ &= 162.7\,\mathrm{kN\,m}. \end{aligned}$$

Then,

$$\begin{aligned} \alpha_s &= 0.6\left(\sqrt{\left(\frac{M_s}{M_o}\right)^2 + 3} - \frac{M_s}{M_o}\right) \\ &= 0.6\left(\sqrt{\left(\frac{149.3\,\mathrm{kN\,m}}{162.7\,\mathrm{kN\,m}}\right)^2 + 3} - \frac{149.3\,\mathrm{kN\,m}}{162.7\,\mathrm{kN\,m}}\right) = 0.6254. \end{aligned}$$

The member capacity is

$$\begin{aligned} M_b &= \min\left(\alpha_m \alpha_s M_s,\ M_s\right) \\ &= \min\left(1 \times 0.6254 \times 149.3\,\mathrm{kN\,m},\ 149.3\,\mathrm{kN\,m}\right) = 93.38\,\mathrm{kN\,m}, \\ M^* &\leqslant \phi M_b = 84.05\,\mathrm{kN\,m}. \end{aligned}$$

The design table gives the same value.

Example 5.3 Worksheet *Same as the previous example. But now the beam is subjected to reverse curvature, find the maximum M^*.*

Solution 5.3

The α_m has been calculated previously, which is

$$\alpha_m = \frac{1.7 \times 1}{\sqrt{0.5^2 + 0^2 + 0.5^2}} = 2.4.$$

Factor α_s remains the same. Thus,

$$\begin{aligned} M_b &= \min\left(\alpha_m\alpha_s M_s,\ M_s\right) \\ &= \min\left(2.4 \times 0.6254 \times 149.3\,\mathrm{kN\,m},\ 149.3\,\mathrm{kN\,m}\right) = 149.3\,\mathrm{kN\,m}, \\ M^* &\leqslant \phi M_b = 134.4\,\mathrm{kN\,m}. \end{aligned}$$

5.5 Strength Design Concept

5.5.1 Strong Axis Bending

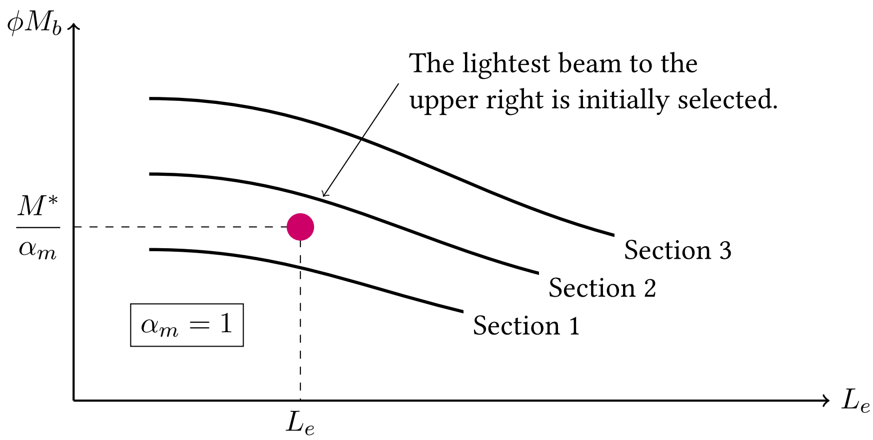
A bending member bent about the strong axis which is analysed by the elastic method with/without redistribution shall satisfy (NZS 3404.1&2:1997 § 5.1.1)

$$M_x^* \leqslant \phi M_{bx}. \tag{5.14}$$

Unlike the compression members, in which N_b can never be greater than N_s, for bending members, M_{bx} can be either smaller than or equal to M_{sx}. In design, it is necessary to first compute M_{sx}.

To design a member, the following approaches can be used.

- Use iterative method. In this procedure involves choosing sizes, and evaluating whether it meets the code criteria. Many sizes may have to be tried to find the most economical safe size.
- Use tables and charts via the following steps:
  1. find α_m from BMD and compute M^*/α_m,
  2. look up a proper section in design capacity table/chart ($\alpha_m = 1$) so that $\phi\alpha_s M_s > M^*/\alpha_m$,



  3. check if the selected section satisfies $\phi M_s > M^*$, if not, choose another section.

5.5.2 Weak Axis Bending

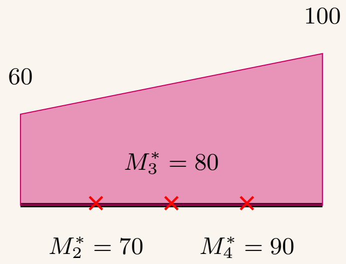
A bending member bent about the weak axis which is analysed by the elastic method with/without redistribution shall satisfy (NZS 3404.1&2:1997 § 5.1.2)

$$M_y^* \leqslant \phi M_{sy}. \tag{5.15}$$

Since FLT buckling does not occurs about weak axis, the member capacity is simply its section capacity.

Example 5.4 *Strong Axis Bending Example*

A beam is braced at 3 m *centers. In the critical section, the bending moment pattern increases linearly from* 60 kN m *to* 100 kN m *over that range. Based on flexural strength alone, what Grade 300 beam is satisfactory?*



Solution 5.4

The α_m can be computed as

$$\alpha_m = \frac{1.7 \times 100\,\mathrm{kN\,m}}{\sqrt{(70\,\mathrm{kN\,m})^2 + (80\,\mathrm{kN\,m})^2 + (90\,\mathrm{kN\,m})^2}} = 1.2205.$$

This lead to

$$\frac{M^*}{\alpha_m} = \frac{100\,\mathrm{kN\,m}}{1.2205} = 81.9\,\mathrm{kN\,m}.$$

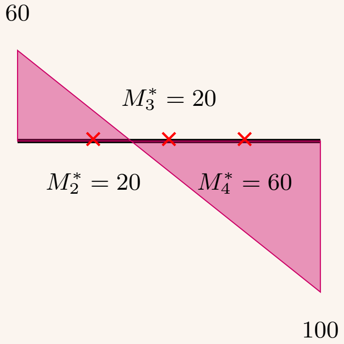
From the design capacity table, 310UB32.0 gives $\phi M_b = 84.1\,\mathrm{kN\,m}$ for $L_e = 3\,\mathrm{m}$. Check if section capacity is satisfied.

$$\phi M_s = 134.5\,\mathrm{kN\,m} > M^* = 100\,\mathrm{kN\,m}.$$

Thus 310UB32.0 satisfies the demand.

Example 5.5 *Strong Axis Bending Example*

A beam is braced at 3 m *centers. In the critical section, the bending moment pattern increases linearly from* −60 kN m *to* 100 kN m *over that range. Based on flexural strength alone, what Grade 300 beam is satisfactory?*



Solution 5.5

The α_m can be computed as

$$\alpha_m = \frac{1.7 \times 100\,\mathrm{kN\,m}}{\sqrt{(20\,\mathrm{kN\,m})^2 + (20\,\mathrm{kN\,m})^2 + (60\,\mathrm{kN\,m})^2}} = 2.5628 > 2.5,$$

use $\alpha_m = 2.5$. This leads to

$$\frac{M^*}{\alpha_m} = \frac{100\,\mathrm{kN\,m}}{2.5} = 40\,\mathrm{kN\,m}.$$

From the design capacity table, 200UB25.4 gives $\phi M_b = 46.0\,\mathrm{kN\,m}$ for $L_e = 3\,\mathrm{m}$. Check if section capacity is satisfied.

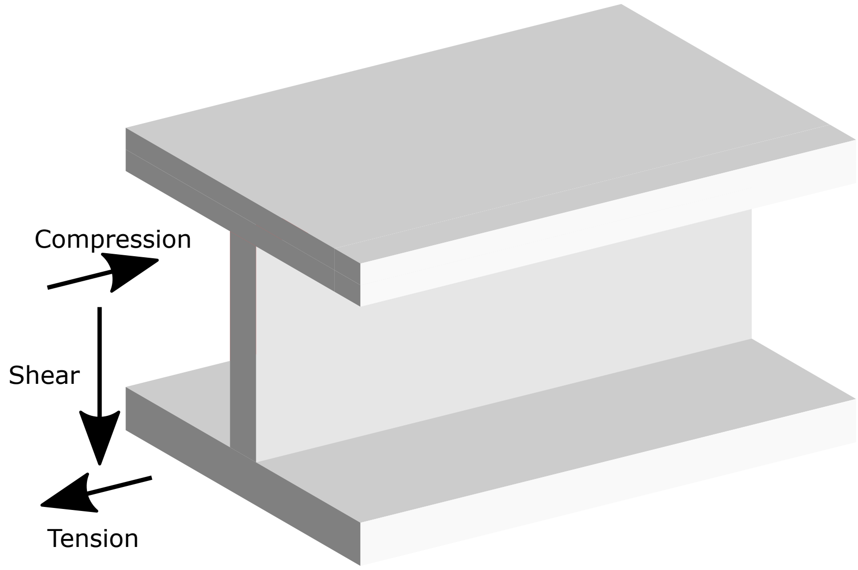
$$\phi M_s = 74.6\,\mathrm{kN\,m} < M^* = 100\,\mathrm{kN\,m}.$$

Thus need to pick a larger section. Eventually, 250UB31.4 is chosen.

5.6 Design for Shear

5.6.1 Shear Flow On Thin–Walled Sections

The shear stress is **not** uniform. There are two types of shear stresses on thin–walled sections: vertical shear and horizontal shear. Consider a I section under strong axis bending, the section is subjected to compression/tension and shear forces.



Horizontal Shear

The shear stress on the longitudinal cut (light grey) needs to equilibrate the normal stress on the free body. According to stress equilibrium of a 2D plane, the shear stress on the section (dark grey) is assumed to be equal to that on the longitudinal cut (light grey).

The horizontal shear stress is mostly in flanges. Due to symmetry, it is self-equilibrating. The horizontal shear stress on web is often negligible for thin–walled sections.

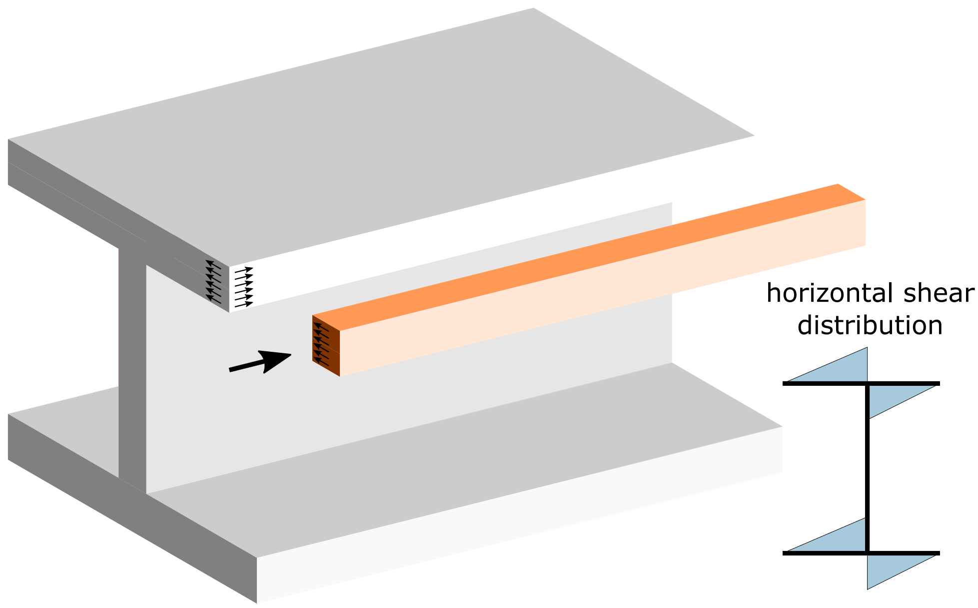


Figure 5.16: Horizontal shear of free body cut on flange

Vertical Shear

For vertical shear stress, similar analysis can be performed.

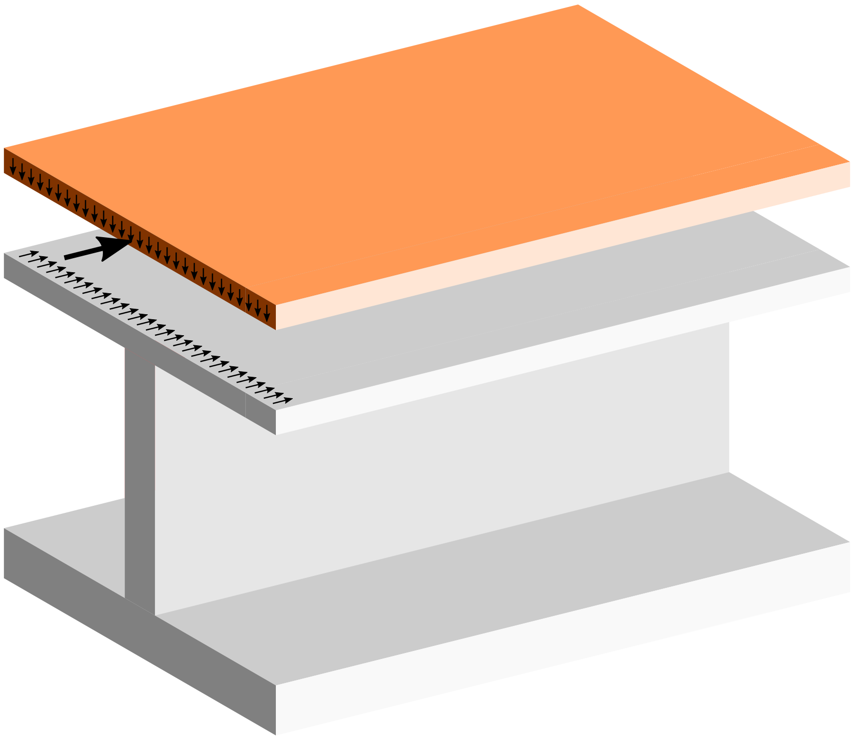


Figure 5.17: Vertical shear of free body cut on flange

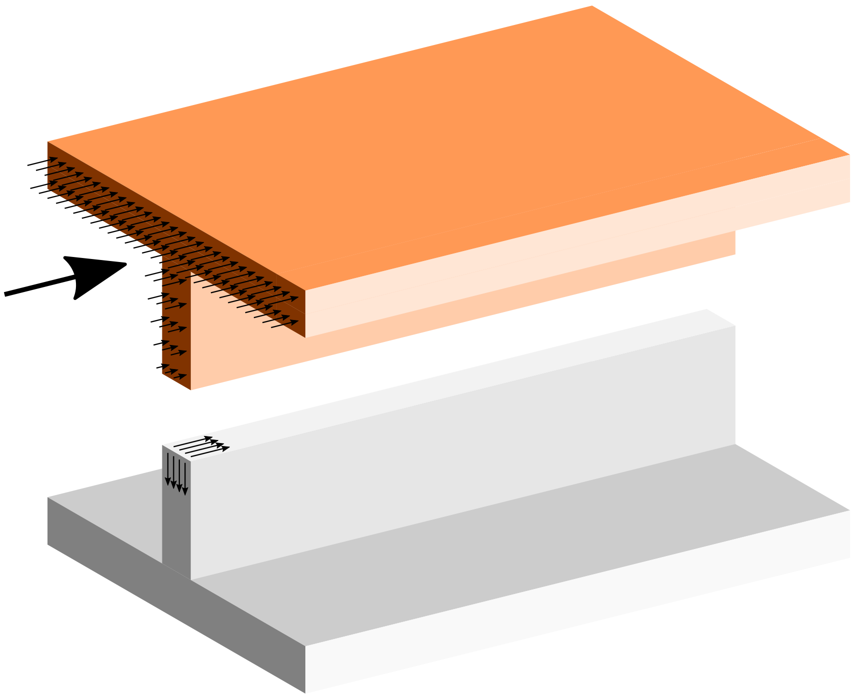


Figure 5.18: Vertical shear of free body cut on web

The vertical shear in the web carries almost all of the applied shear force to a section. The flanges carry the majority of the bending (normal stress). The vertical shear stress distribution is depicted in Fig. 5.19. Interested readers can check this[3] video for more elaborations.

[3] https://www.youtube.com/watch?v=f08Y39UiC-o

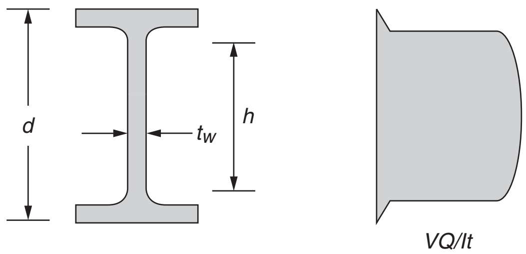


Figure 5.19: Vertical shear distribution in an I section

5.6.2 Shear Capacity

In practical design, we mainly consider the **vertical** shear acting on web. It is further assumed the vertical shear is uniform/constant over the depth. The shear stress can be computed using the following expression.

$$q = \frac{VQ}{Ib}.$$

A flat plate unstiffened web in an I section shall satisfy

$$V^* \leqslant \phi V_v. \tag{5.16}$$

The nominal shear capacity V_v shall be determined as

$$V_v = \min\left(V_w,\ V_b\right). \tag{5.17}$$

The shear capacity associated with web yielding V_w (NZS 3404.1&2:1997 § 5.11.4.1) is,

$$V_w = 0.6 f_{yw} A_w = 0.6 f_{yw} d t_w \tag{5.18}$$

where f_{yw} is the yield shear stress. For an unstiffened web, $A_w = dt_w$ is the area of web.

The shear capacity associated with web buckling V_b (NZS 3404.1&2:1997 § 5.11.5.1) is,

$$V_b = V_w \left(\frac{82}{\dfrac{d_p}{t_w}\sqrt{\dfrac{f_y}{250\,\mathrm{MPa}}}}\right)^2 = 0.6 f_y A_w \left(\frac{82}{\dfrac{d_p}{t_w}\sqrt{\dfrac{f_y}{250\,\mathrm{MPa}}}}\right)^2 \tag{5.19}$$

where d_p is the depth of the deepest web panel, this is equal to the distance between the insides of the flanges, d_1, for a web without horizontal stiffening plates.

Also, NZS 3404.1&2:1997 Cl. 5.10.1.1 imposes the maximum web slenderness ratio as

$$\frac{d_p}{t_w}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}} \leqslant 180. \tag{5.20}$$

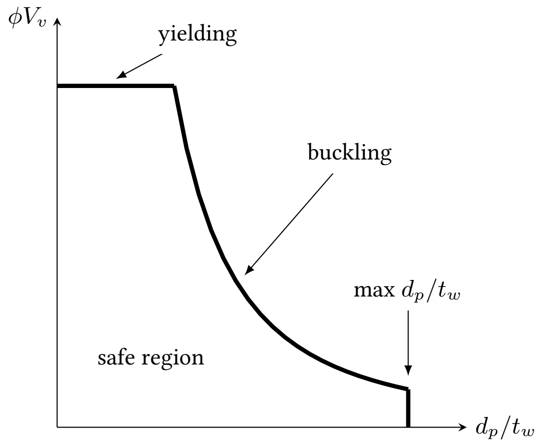


Figure 5.20: Design region of shear capacity

5.6.3 Interaction Between Bending and Shear

For the majority of practical situations, we can ignore a reduction in bending strength due to shear, or a reduction in shear strength due to bending also being carried by the section. This is because the bending is carried mainly by the flanges of an I beam bending about its strong axis, the shear is resisted mainly by the web. However, for cases in which both V^* is close to ϕV_v and M^* is close to ϕM_s, a reduction in strength should be considered. Interested readers can refer to Joint Committee of the Welding Research Council and ASCE (1971) for elaborations.

To account for such an interaction, to design a web under shear in the presence of bending moment, it shall satisfy (NZS 3404.1&2:1997 § 5.12.2)

$$V^* \leqslant \phi V_{vm}, \tag{5.21}$$

where

$$V_{vm} = \begin{cases} V_v, & \text{for } \dfrac{M^*}{\phi M_s} \leqslant 0.75, \\ V_v\left(2.2 - 1.6\dfrac{M^*}{\phi M_s}\right), & \text{for } 0.75 < \dfrac{M^*}{\phi M_s} \leqslant 1. \end{cases} \tag{5.22}$$

To illustrate, the following graph can be used.

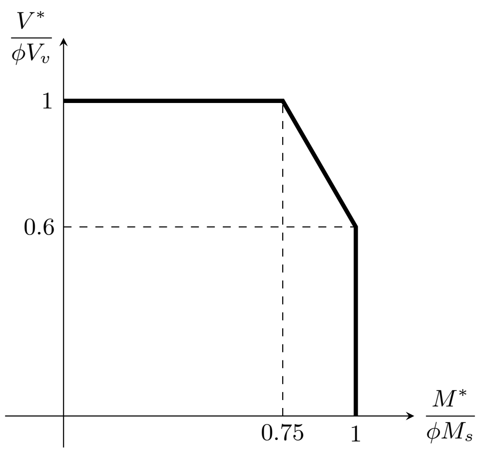


Figure 5.21: Interaction between shear and moment

Example 5.6 *Determine the maximum factored UDL, ω^*, that can be applied to the previous fully restrained 310UB32.0 beam using Grade 300 steel.*

Solution 5.6

- **flexure**
  Given that $M^* \leqslant \phi M_b$ and the maximum moment at midspan

$$M^* = \frac{\omega^* l^2}{8},$$

  the maximum ω^* can be computed as

$$\frac{\omega^* l^2}{8} \leqslant \phi M_b,$$
$$\omega^* \leqslant \frac{8}{l^2}\phi M_b = \frac{8}{3\,\mathrm{m} \times 3\,\mathrm{m}} \times 134.4\,\mathrm{kN\,m} = 119.5\,\mathrm{kN\,m^{-1}}.$$

- **shear**
  Check the factor,

$$\frac{82}{\dfrac{d_p}{t_w}\sqrt{\dfrac{f_y}{250\,\mathrm{MPa}}}} = \frac{82}{\dfrac{282\,\mathrm{mm}}{5.5\,\mathrm{mm}}\sqrt{\dfrac{320\,\mathrm{MPa}}{250\,\mathrm{MPa}}}} = 1.41 > 1.$$

  Thus yield capacity governs, that is

$$V_v = 0.6 f_y A_w = 0.6 \times 320\,\mathrm{MPa} \times 298\,\mathrm{mm} \times 5.5\,\mathrm{mm} = 314.7\,\mathrm{kN}.$$

  Then the maximum shear at ends shall satisfy

$$\frac{\omega^* l}{2} = V^* \leqslant \phi V_v,$$
$$\omega^* \leqslant \frac{2}{l}\phi V_v = \frac{2}{3\,\mathrm{m}} \times 0.9 \times 314.7\,\mathrm{kN} = 188.8\,\mathrm{kN\,m^{-1}}.$$

$$\omega^* \leqslant 188.8\,\mathrm{kN\,m^{-1}}$$

Thus flexure governs, the maximum ω^* is $119.5\,\mathrm{kN\,m^{-1}}$. It is noted that shear tends to govern in very short members.

5.7 Design for Bearing

Modes of failure near a concentrated load are web buckling and web yielding. To compute either we need to know how the force is transferred into the web of the beam.

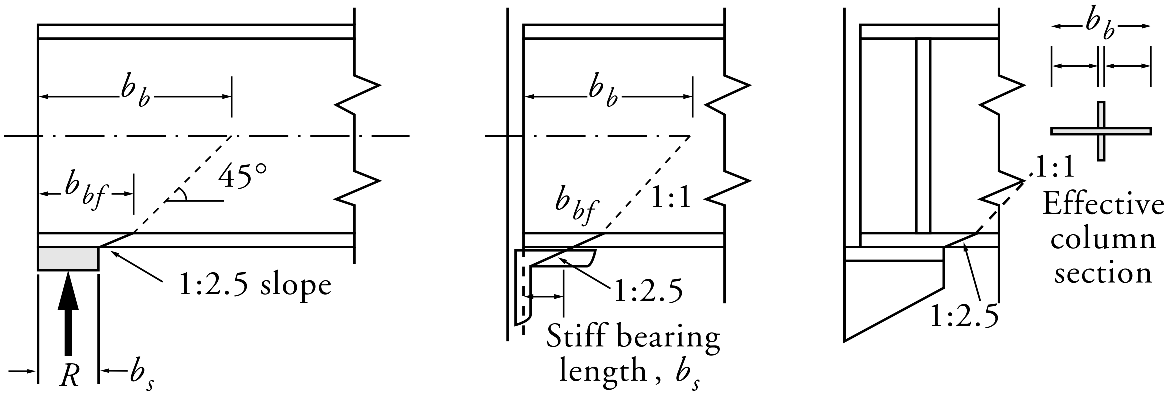
The design froce R^* on a web shall then satisfy (NZS 3404.1&2:1997 § 5.13.2)

$$R^* \leqslant \phi R_b = \min\left(\phi R_{by},\ \phi R_{bb}\right), \tag{5.23}$$

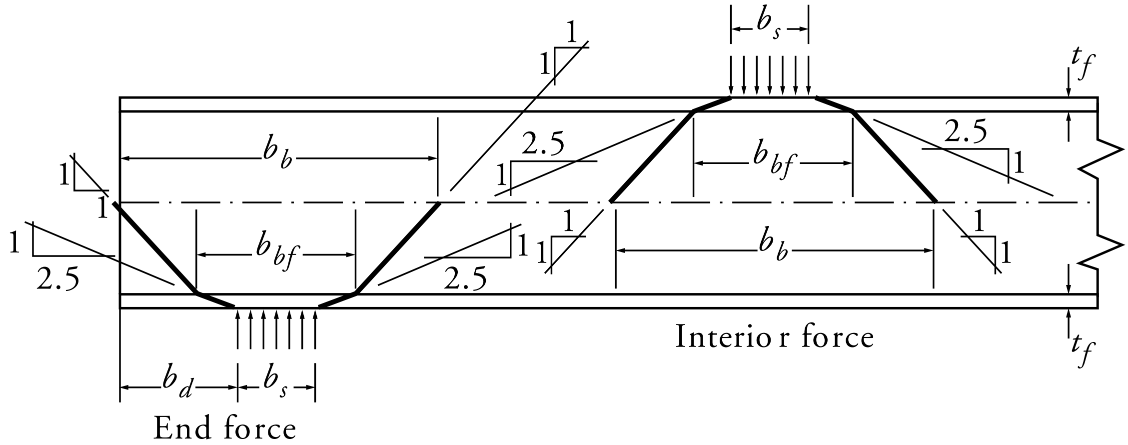
where R_b shall be the smaller of R_{by} and R_{bb} as defined below.

5.7.1 Force Dispersion

Point load is an idealised concept that does not exist in practice. All loads are applied to regions of finite areas. NZS 3404.1&2:1997 § 5.13.1 requires the dispersion of load through the flange shall be taken at a slope of 1:2.5 to the surface of the flange while the dispersion of load to the flange shall be taken at a slope of 1:1 through solid material.



(a) Force dispersion at end bearing points.



(b) General force dispersion in I-section flange and web

Figure 5.22: Web bearing and the load dispersion method (Gorenc et al., 2015)

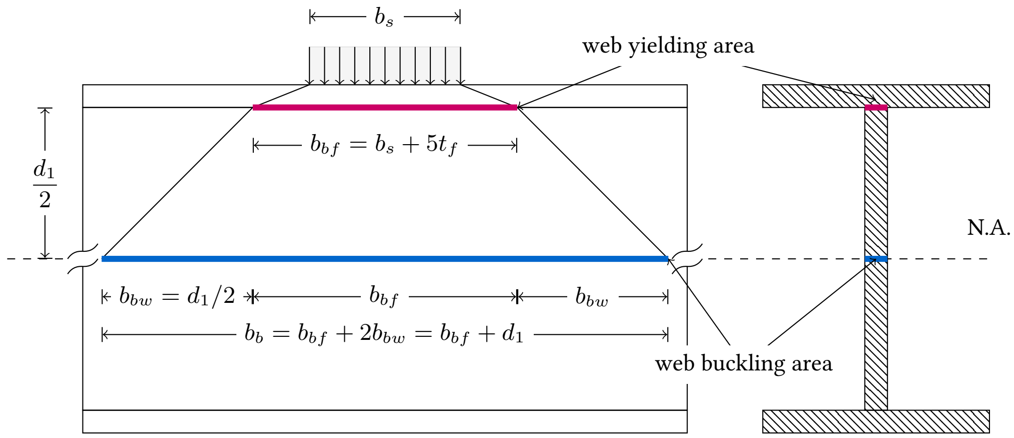


Figure 5.23: Interior force

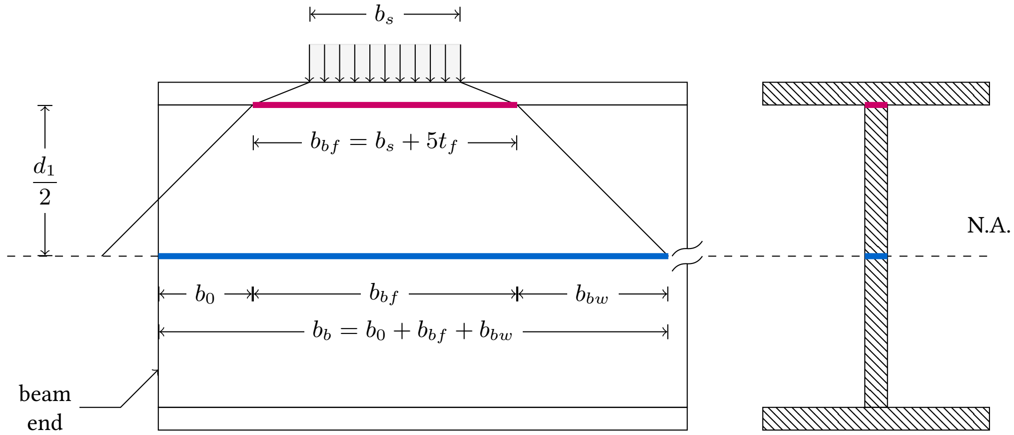


Figure 5.24: End force

If loading is applied from another section, then the way the force should be computed on the beam being designed in shown below.

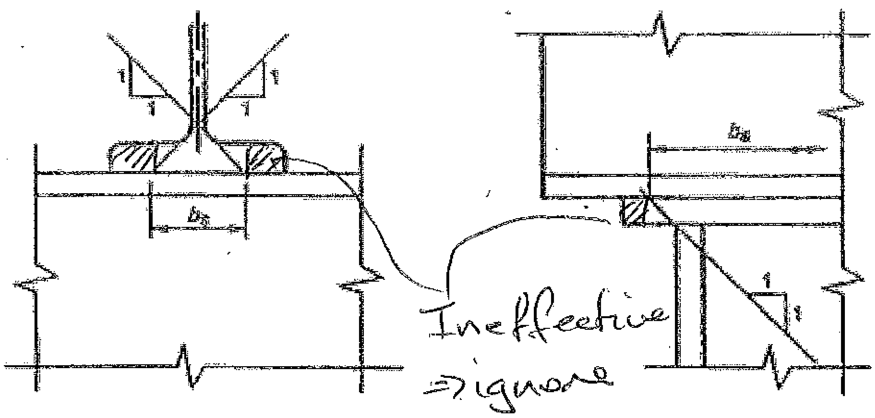


Figure 5.25: Ineffective regions are ignored

5.7.2 Yielding

Web yielding will occur over the area $b_{bf}t_w$ when there is either high load or small area. The nominal bearing yield capacity of a web shall be calculated as (NZS 3404.1&2:1997 § 5.13.3.1)

$$R_{by} = 1.25 b_{bf} t_w f_y. \tag{5.24}$$

The factor 1.25 considers the following aspects:

- potential higher strength under compression,
- Poisson's ratio,
- force distribution may be conservative, and
- triaxial stresses from flange.

5.7.3 Buckling

Web buckling is assumed to occur following the pattern as shown in absence of web stiffeners.

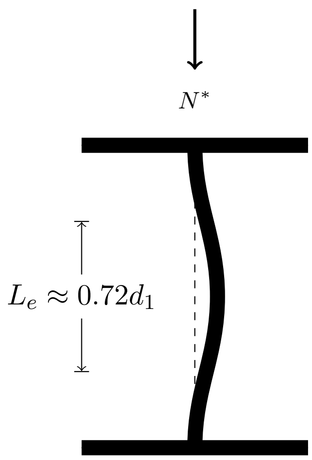


Figure 5.26: Web buckling due to bearing

The critical stress is at centre (mid height) of beam web over the area $b_b t_w$. The nominal bearing buckling capacity R_{bb} is determined as the axial compression capacity using $\alpha_b = 0.5$ and $k_f = 1$ with slenderness ratio $L_e/r = 2.5d_1/t_w$ (NZS 3404.1&2:1997 § 5.13.4).

$$R_{bb} = \alpha_c N_s = \alpha_c b_b t_w f_y. \tag{5.25}$$

It shall be noted $f_y = f_{y,web}$ shall be taken as the yield strength of web. The slenderness ratio $L_e/r = 2.5d_1/t_w$ is equivalent to considering $L_e \approx 0.72d_1$. For web with size $t_w \times d_1$, the radius of gyration is

$$r = r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{t_w^3 d_1}{12 t_w d_1}} = \frac{t_w}{\sqrt{12}}, \tag{5.26}$$

this leads to

$$\frac{L_e}{r} = \frac{0.72d_1}{t_w/\sqrt{12}} \approx 2.5\frac{d_1}{t_w}. \tag{5.27}$$

The above is valid for I sections. Different equations are needed for other sections. Interested readers can refer to NZS 3404.1&2:1997 § 5.14.

5.7.4 Web Stiffener

If the requirements for yielding or buckling above are not satisfied then load bearing stiffeners must be provided. They can be provided in one or both sides, should fit tightly, and may or may not extend over the whole depth.

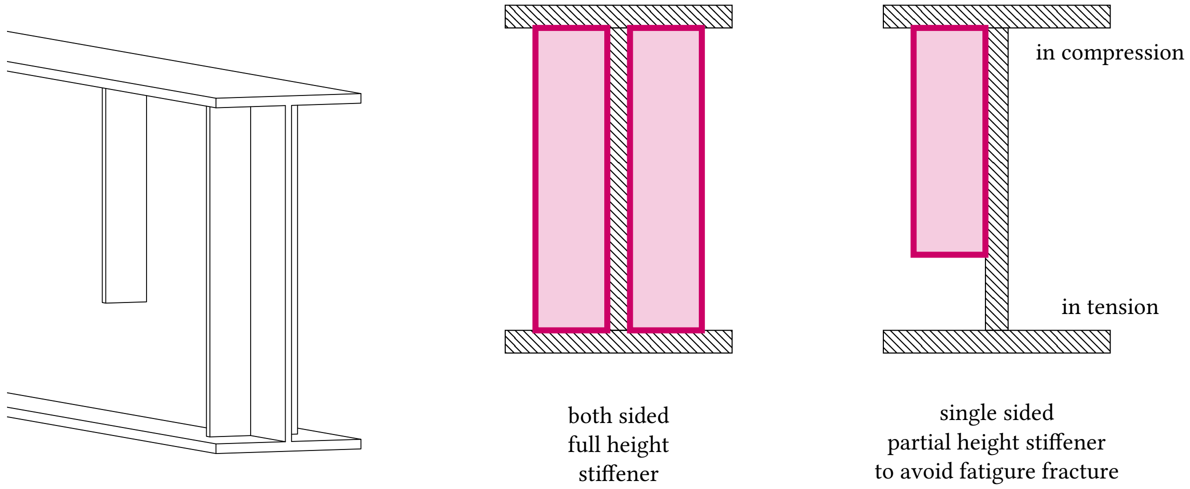


Figure 5.27: Web stiffeners

Web Yielding

The design force R^* shall satisfy (NZS 3404.1&2:1997 5.14.1)

$$R^* \leqslant \phi R_{sy}, \tag{5.28}$$

where

R_{sy} = nominal yield capacity of the stiffened web

The nominal yield capacity of the stiffened web shall be computed as

$$R_{sy} = R_{by} + A_s f_{ys}, \tag{5.29}$$

where

R_{by} = nominal bearing yield capacity
A_s = area of the stiffener in contact with the flange
f_{ys} = yield stress of the stiffener

Web Buckling

The design force R^* shall satisfy (NZS 3404.1&2:1997 5.14.2)

$$R^* \leqslant \phi R_{sb}, \tag{5.30}$$

where R_{sb} is the nominal buckling capacity of the stiffened web which shall be determined in a way similar to compression members as follows.

$$R_{sb} = \alpha_c N_s = \alpha_c k_f A_n f_y. \tag{5.31}$$

In which, α_c shall be computed according to § 4.2.2 with modified slenderness ratio λ_n be

$$\lambda_n = \frac{L_e}{r}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\mathrm{MPa}}}, \tag{5.32}$$

using $k_f = 1$, $\alpha_b = 0.5$, r be the radius of gyration about the neutral axis of A_n parallel to web, L_e be

- $0.7d_1$ where flanges are restrained against rotation in plane of stiffener, or
- d_1 where flanges are **not** restrained against rotation in plane of stiffener.

The net area A_n can be taken as the summation of the gross area of the stiffener(s) plus the effective area of the web as shown.

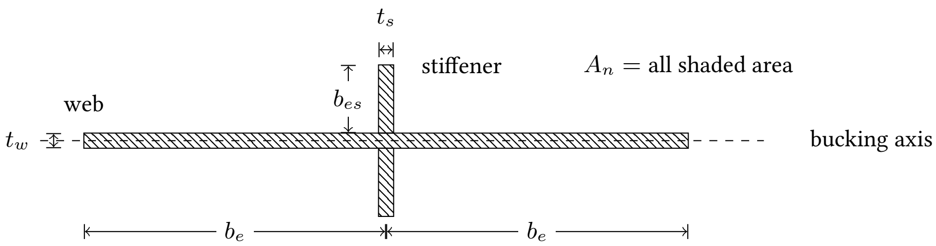


Figure 5.28: Plan view at web buckling surface

The effective width b_e of each side of the stiffener(s) centreline shall be the smaller of $17.5t_w\sqrt{\frac{250\,\mathrm{MPa}}{f_y}}$ and $s/2$ where s is the stiffener spacing.

The width to thickness ratio of any stiffener element should also satisfy

$$\frac{b_{es}}{t_s}\sqrt{\frac{f_{ys}}{250\,\mathrm{MPa}}} \leqslant 15, \tag{5.33}$$

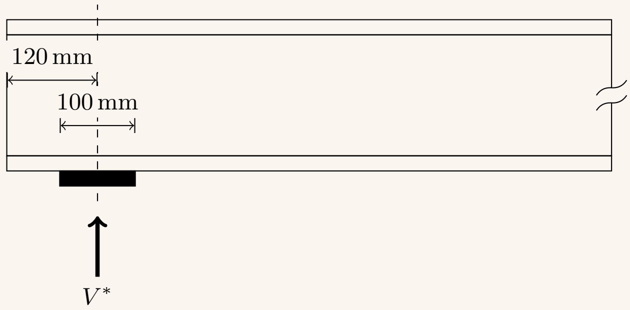
where b_{es} is the width of stiffener, t_s is the thickness of stiffener and f_{ys} is the yield strength of stiffener.

It is easiest to prevent buckling problems by providing a sufficiently large support.

Example 5.7 *Worksheet Bearing Design*

If a $3\,\mathrm{m}$ *long Grade 300 310UB32.0 beam subject to UDL of* $\omega = 119\,\mathrm{kN\,m^{-1}}$ *is supported on a* $100\,\mathrm{mm}$ *wide plate at either end, and if the beam continues* $120\,\mathrm{mm}$ *beyond the centre of the support, is it satisfactory?*

Solution 5.7



For force dispersion,

$$\begin{aligned}
b_{bf} &= b_s + 5t_f = 100\,\text{mm} + 5 \times 8\,\text{mm} = 140\,\text{mm},\\
b_{bw} &= \frac{d_1}{2} = 141\,\text{mm},\\
b_0 &= 120\,\text{mm} - 0.5 \times 140\,\text{mm} = 50\,\text{mm},\\
b_b &= \min\left(b_{bf} + 2b_{bw}, b_{bf} + b_{bw} + b_0\right)\\
&= \min\left(140\,\text{mm} + 2 \times 141\,\text{mm}, 140\,\text{mm} + 141\,\text{mm} + 50\,\text{mm}\right)\\
&= 331\,\text{mm}.
\end{aligned}$$

The shear demand is

$$V^* = \frac{\omega L}{2} = \frac{119\,\text{kN}\,\text{m}^{-1} \times 3\,\text{m}}{2} = 178.5\,\text{kN}.$$

For web yielding,

$$\begin{aligned}
\phi R_{by} &= \phi 1.25 b_{bf} t_w f_y\\
&= 0.9 \times 1.25 \times 140\,\text{mm} \times 5.5\,\text{mm} \times 320\,\text{MPa}\\
&= 277.2\,\text{kN} > V^* = 178.5\,\text{kN}.
\end{aligned}$$

For web buckling,

$$\begin{aligned}
\frac{L_e}{r} &= 2.5\frac{d_1}{t_w} = 2.5 \times \frac{282}{5.5} = 128.2,\\
\lambda_n &= \frac{L_e}{r}\sqrt{k_f}\sqrt{\frac{f_y}{250\,\text{MPa}}} = 128.2 \times 1 \times \sqrt{\frac{320\,\text{MPa}}{250\,\text{MPa}}} = 145.
\end{aligned}$$

Using $\alpha_b = 0.5$, one can obtain $\alpha_c = 0.288$. Thus,

$$\begin{aligned}
\phi R_{bb} &= \phi \alpha_c b_b t_w f_y\\
&= 0.9 \times 0.288 \times 331\,\text{mm} \times 5.5\,\text{mm} \times 320\,\text{MPa}\\
&= 151.0\,\text{kN} < V^* = 178.5\,\text{kN}.
\end{aligned}$$

NOT GOOD

It can be seen than web buckling governs and $V^* > \phi R_{bb}$. One can increase b_s, extend beam, or add stiffener to avoid buckling failure.

5.8 Biaxial Loading

The biaxial bending can be decomposed into strong axis and weak axis components.

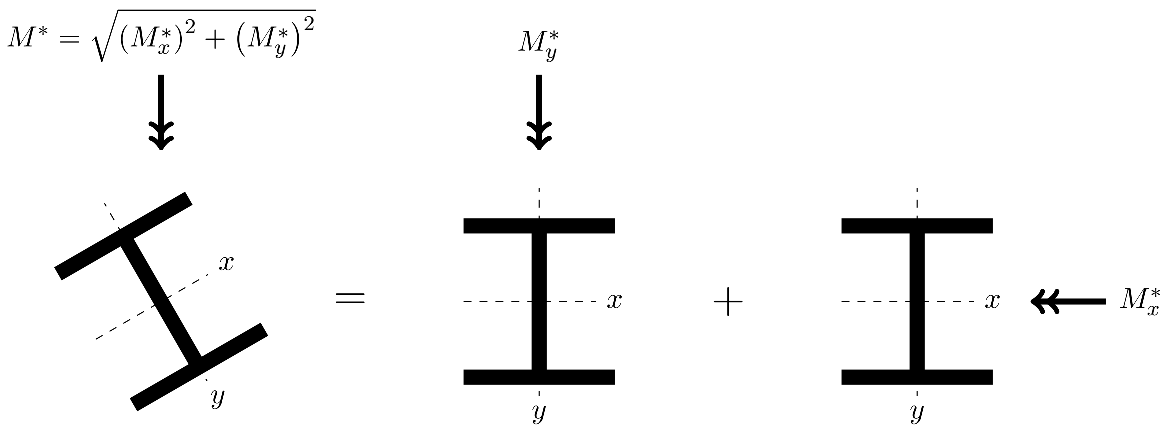


Figure 5.29: Decomposition of biaxial moment into x and y components

In absence of axial force, NZS 3404.1&2:1997 Cl. 8.4.5.1 requires

$$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{cy}}\right)^{1.4} \leqslant 1.0, \tag{5.34}$$

where major axis bending occurs about the x-axis and minor axis bending ocurs about the y-axis. Furthermore, $M_{cx} = \min(M_{sx}, M_{bx})$ for the major (strong) axis and $M_{cy} = M_{sy}$ for the minor (weak) axis. FLT buckling does **not** occur for bending about weak axis.

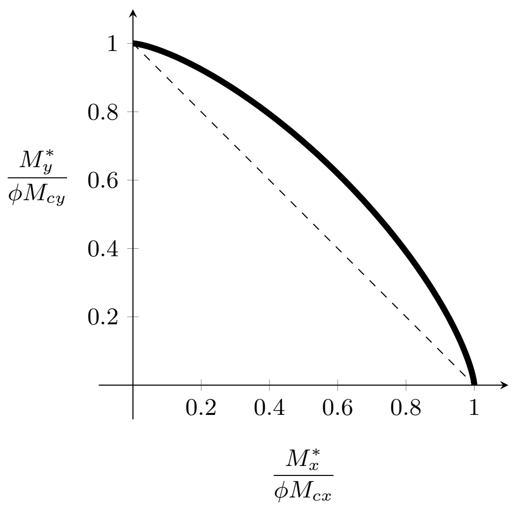


Figure 5.30: Envelop of biaxial moments

Example 5.8 *Biaxial Design*

The factored moments on a beam, including self–weight, are $M_x^* = 200\,\mathrm{kN\,m}$ *and* $M_y^* = 50\,\mathrm{kN\,m}$. *Select a Grade 300 UB section to resist these moments assuming full lateral support of the compression flange.*

Solution 5.8

Try 460UB74.6,

$$
\begin{aligned}
&\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{cy}}\right)^{1.4} \\
=&\left(\frac{M_x^*}{\phi M_{sx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{sy}}\right)^{1.4} \\
=&\left(\frac{200\,\mathrm{kN\,m}}{448.2\,\mathrm{kN\,m}}\right)^{1.4} + \left(\frac{50\,\mathrm{kN\,m}}{70.7\,\mathrm{kN\,m}}\right)^{1.4} \\
=&0.939 < 1.0
\end{aligned}
$$

Since the member is fully supported, $M_{cx} = M_{sx}$ and $M_{cy} = M_{sy}$.

For unbraced members, $M_{cx} = M_{bx}$ which depends on α_m, unbraced length, etc.

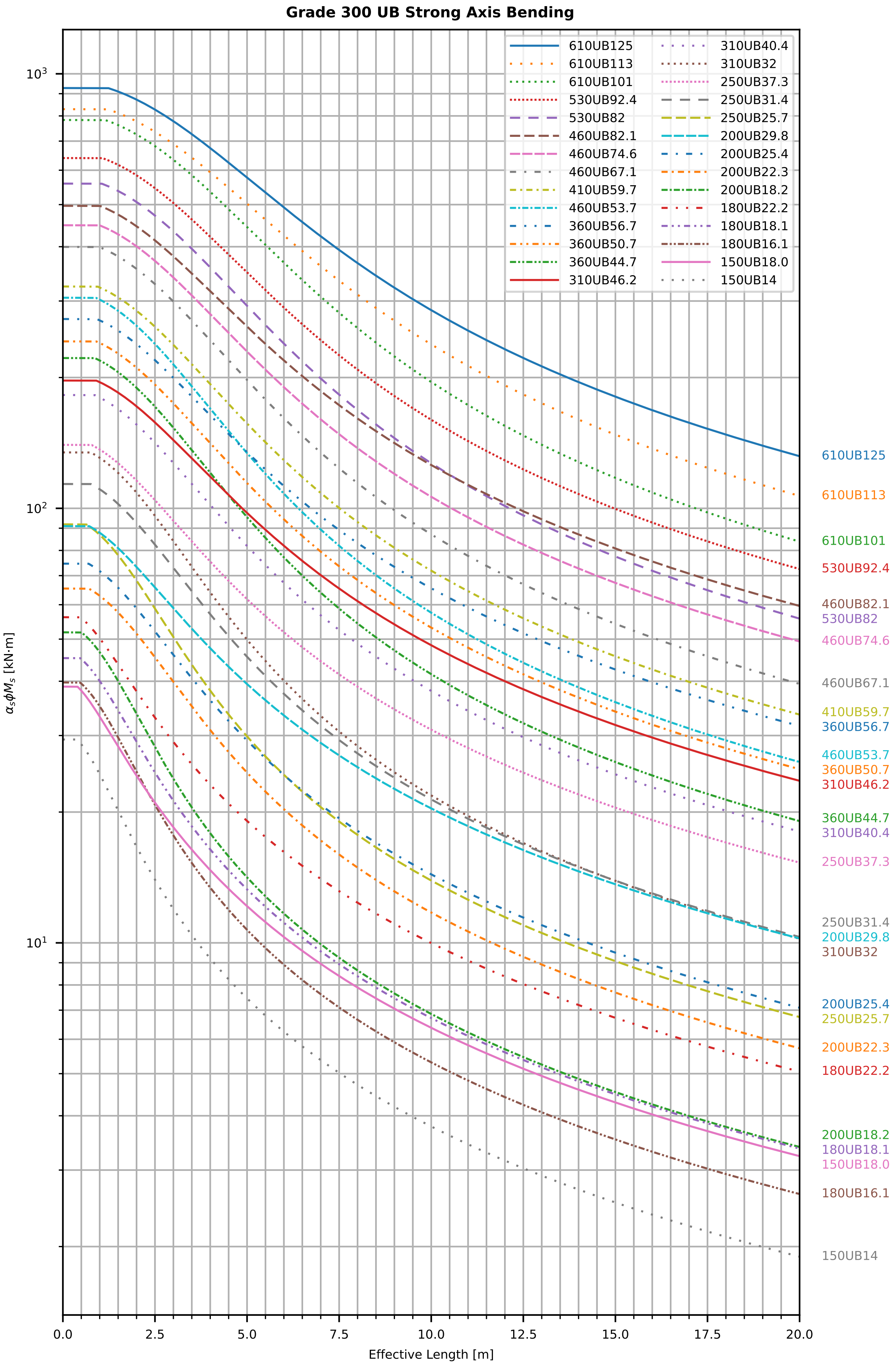
Table 5.3: Design load capacity table for members subject to strong axis bending ($\alpha_m = 1.0$)

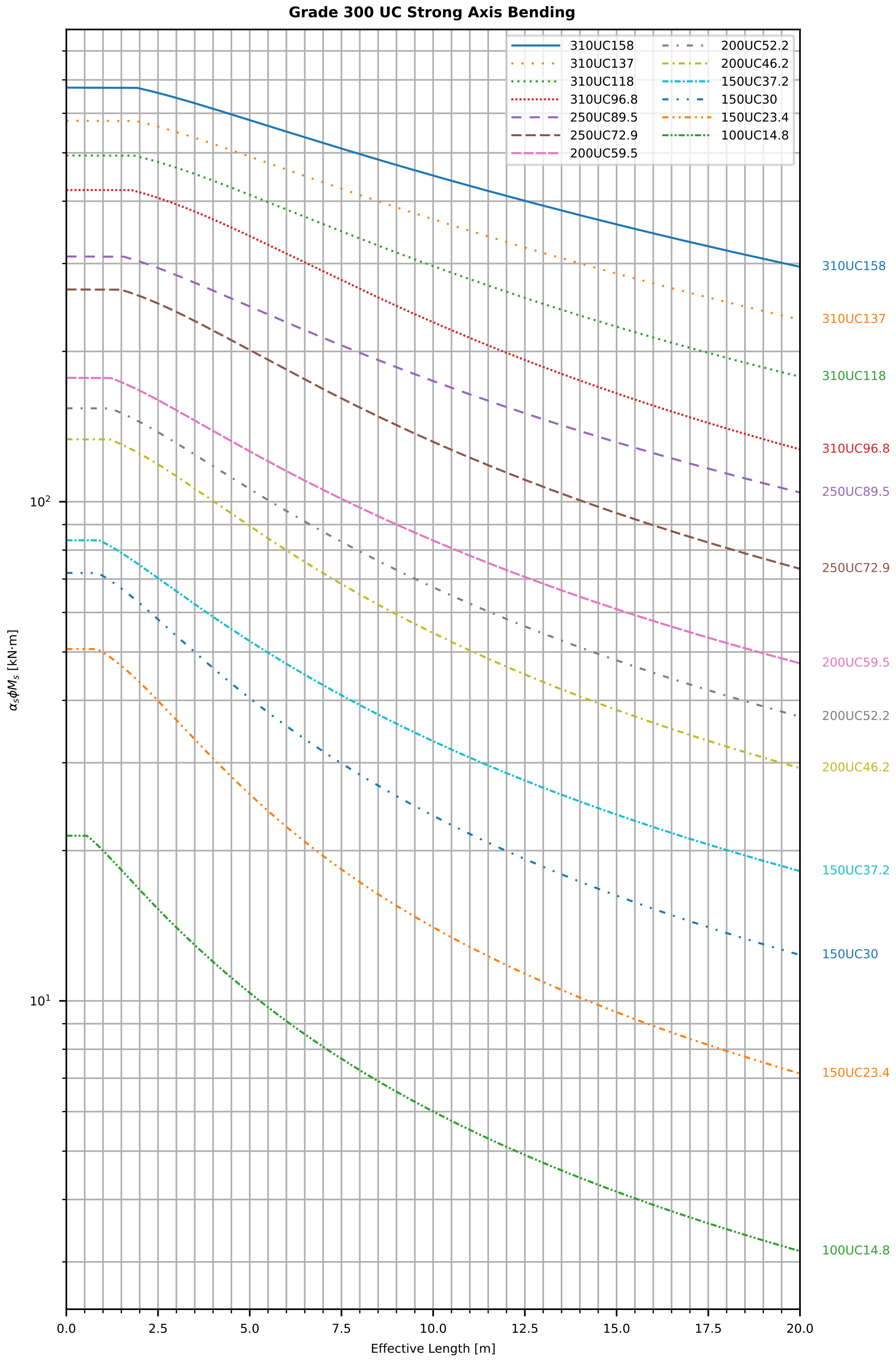
Grade 300 UB **Strong** Axis Bending ($\alpha_m = 1.0$)

	ϕM_{sx} (kN)	ϕM_{sy} (kN)	$\alpha_s \phi M_s$ (kN)																										
L_e (m)	0.00	0.00	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00
610UB125	927.4	129.8	927.4	926.1	910.3	892.3	872.2	850.5	827.3	803.0	777.9	752.3	726.4	700.4	674.7	649.3	624.5	600.3	577.0	532.8	492.4	455.7	422.7	393.2	366.7	343.1	321.9	302.9	285.8
610UB113	829.1	113.7	829.1	826.9	812.3	795.6	777.1	756.9	735.4	712.8	689.5	665.6	641.5	617.3	593.4	569.8	546.7	524.2	502.5	461.7	424.5	390.9	360.9	334.1	310.3	289.1	270.2	253.4	238.4
610UB101	783.0	104.2	783.0	777.4	762.1	744.7	725.3	704.2	681.8	658.3	634.1	609.4	584.5	559.7	535.3	511.3	488.0	465.4	443.8	403.5	367.3	335.2	306.8	281.8	259.9	240.6	223.6	208.6	195.3
530UB92.4	639.9	92.3	639.9	631.3	617.3	601.5	584.0	565.3	545.5	525.1	504.2	483.2	462.3	441.7	421.7	402.2	383.5	365.7	348.7	317.4	289.6	265.2	243.8	225.0	208.5	194.0	181.3	169.9	159.9
530UB82.0	558.9	78.0	558.9	550.2	537.5	523.0	507.1	489.9	471.8	453.1	434.0	414.8	395.6	376.8	358.5	340.8	323.8	307.6	292.3	264.1	239.4	217.8	199.0	182.6	168.4	156.0	145.1	135.5	127.0
460UB82.1	496.8	78.8	496.8	486.4	474.3	460.6	445.8	430.0	413.7	397.0	380.2	363.5	347.2	331.3	315.9	301.3	287.3	274.1	261.6	238.9	218.9	201.4	186.1	172.7	160.9	150.5	141.3	133.1	125.8
460UB74.6	448.2	70.7	447.9	438.4	427.2	414.6	400.9	386.3	371.1	355.5	339.9	324.3	309.1	294.2	279.9	266.3	253.3	241.0	229.5	208.4	190.1	174.1	160.2	148.1	137.5	128.2	120.0	112.8	106.3
460UB67.1	399.6	62.1	399.0	390.2	379.9	368.3	355.7	342.2	328.1	313.7	299.2	284.8	270.6	256.9	243.6	231.0	219.0	207.7	197.1	177.9	161.2	146.8	134.4	123.6	114.3	106.1	99.0	92.7	87.1
410UB59.7	324.0	54.8	322.3	314.7	305.8	295.8	285.0	273.5	261.7	249.8	237.8	226.1	214.6	203.6	193.1	183.2	173.8	164.9	156.7	141.9	129.0	118.0	108.4	100.2	93.0	86.7	81.1	76.2	71.9
410UB53.7	305.3	49.8	302.2	294.2	284.9	274.4	263.1	251.2	239.0	226.7	214.4	202.4	190.9	179.8	169.4	159.6	150.4	141.9	134.0	120.0	108.1	97.9	89.3	82.0	75.7	70.2	65.4	61.2	57.5
360UB56.7	272.7	52.1	270.8	264.1	256.4	247.9	238.7	229.2	219.3	209.5	199.7	190.2	180.9	172.1	163.7	155.7	148.2	141.2	134.7	122.8	112.6	103.6	95.9	89.1	83.2	77.9	73.3	69.2	65.5
360UB50.7	242.2	45.4	240.2	234.1	227.0	219.2	210.7	201.8	192.7	183.5	174.4	165.6	157.0	148.8	141.0	133.6	126.7	120.3	114.3	103.6	94.3	86.3	79.5	73.5	68.4	63.8	59.8	56.3	53.1
360UB44.7	221.8	40.3	218.9	212.8	205.8	197.9	189.4	180.5	171.4	162.3	153.3	144.5	136.1	128.2	120.6	113.6	107.1	101.0	95.4	85.5	77.1	70.0	64.0	58.8	54.3	50.5	47.1	44.1	41.5
310UB46.2	196.8	44.0	195.2	190.3	184.6	178.4	171.7	164.8	157.7	150.6	143.6	136.8	130.2	124.0	118.0	112.4	107.2	102.2	97.6	89.2	82.0	75.7	70.2	65.4	61.1	57.4	54.1	51.1	48.4
310UB40.4	182.3	40.0	179.9	174.9	169.1	162.7	155.9	148.8	141.5	134.3	127.2	120.3	113.8	107.5	101.7	96.2	91.0	86.3	81.9	74.1	67.4	61.6	56.7	52.5	48.8	45.6	42.7	40.2	38.0
310UB32.0	134.5	25.0	130.7	126.0	120.7	114.8	108.7	102.4	96.1	90.0	84.1	78.5	73.3	68.5	64.1	60.0	56.3	52.9	49.9	44.5	40.1	36.4	33.3	30.6	28.4	26.4	24.7	23.2	21.8
250UB37.3	140.0	33.4	136.6	132.1	127.0	121.5	115.9	110.1	104.4	98.9	93.6	88.6	83.9	79.5	75.4	71.6	68.1	64.8	61.8	56.4	51.9	47.9	44.5	41.5	38.8	36.5	34.5	32.6	30.9
250UB31.4	113.8	26.3	110.6	106.8	102.4	97.6	92.6	87.5	82.5	77.6	73.0	68.6	64.4	60.6	57.0	53.8	50.8	48.0	45.5	41.1	37.4	34.3	31.6	29.3	27.3	25.5	24.0	22.6	21.4
250UB25.7	91.9	17.8	86.9	82.7	78.0	73.1	68.2	63.4	58.8	54.5	50.5	46.9	43.7	40.7	38.1	35.7	33.5	31.6	29.9	26.9	24.4	22.3	20.5	19.0	17.7	16.6	15.6	14.7	13.9
200UB29.8	91.0	24.9	87.9	84.6	81.0	77.2	73.3	69.5	65.8	62.2	58.9	55.8	52.8	50.1	47.6	45.3	43.2	41.2	39.4	36.2	33.4	31.0	28.8	27.0	25.4	23.9	22.6	21.4	20.4
200UB25.4	74.6	19.8	71.7	68.8	65.7	62.3	58.8	55.4	52.1	48.9	46.0	43.2	40.7	38.3	36.2	34.2	32.4	30.7	29.2	26.6	24.3	22.4	20.8	19.3	18.1	17.0	16.0	15.2	14.4
200UB22.3	65.4	17.4	62.9	60.4	57.6	54.6	51.5	48.4	45.4	42.5	39.8	37.3	35.0	32.8	30.9	29.1	27.5	26.0	24.6	22.3	20.3	18.6	17.2	16.0	14.9	14.0	13.1	12.4	11.7
200UB18.2	51.8	9.9	46.7	43.4	40.0	36.7	33.6	30.8	28.2	25.9	23.9	22.1	20.5	19.1	17.9	16.8	15.8	14.9	14.1	12.8	11.7	10.7	9.9	9.2	8.6	8.1	7.6	7.2	6.9
180UB22.2	56.2	11.7	50.2	47.0	43.7	40.7	37.9	35.3	33.0	30.9	29.0	27.3	25.8	24.4	23.1	22.0	20.9	20.0	19.1	17.5	16.2	15.0	14.0	13.2	12.4	11.7	11.1	10.5	10.0
180UB18.1	45.2	9.4	40.1	37.2	34.3	31.5	29.0	26.7	24.7	22.8	21.2	19.8	18.5	17.4	16.4	15.5	14.7	13.9	13.3	12.1	11.1	10.3	9.6	8.9	8.4	7.9	7.5	7.1	6.7
180UB16.1	39.7	8.2	35.0	32.4	29.7	27.1	24.8	22.7	20.8	19.1	17.7	16.4	15.3	14.3	13.4	12.6	11.9	11.3	10.7	9.7	8.9	8.2	7.6	7.1	6.7	6.3	5.9	5.6	5.3
150UB18.0	38.9	7.7	33.2	30.6	28.3	26.2	24.2	22.5	21.0	19.7	18.5	17.4	16.4	15.5	14.7	14.0	13.3	12.7	12.1	11.2	10.3	9.6	8.9	8.4	7.9	7.4	7.1	6.7	6.4
150UB14.0	29.4	5.7	24.4	22.2	20.1	18.3	16.6	15.2	14.0	12.9	12.0	11.1	10.4	9.8	9.2	8.7	8.2	7.8	7.5	6.8	6.2	5.8	5.4	5.0	4.7	4.4	4.2	4.0	3.8

Table 5.4: Design load capacity table for members subject to strong axis bending ($\alpha_m = 1.0$)

Grade 300 UC **Strong** Axis Bending ($\alpha_m = 1.0$)																										
	ϕM_{sx} (kN)	ϕM_{sy} (kN)	$\alpha_s \phi M_s$ (kN)																							
L_e (m)	0.00	0.00	1.00	1.50	2.00	2.50	3.00	3.50	4.00	4.50	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00	13.00	14.00	15.00	16.00	17.00	18.00	19.00	20.00
310UC158	675.4	304.9	675.4	675.4	672.9	659.2	644.3	628.7	612.8	597.0	581.4	551.4	523.3	497.0	472.7	450.2	429.2	409.8	391.8	375.0	359.4	344.9	331.3	318.6	306.7	295.6
137	579.6	262.1	579.6	579.6	576.7	564.2	550.5	536.0	521.2	506.3	491.5	463.0	436.3	411.6	388.9	368.0	348.8	331.3	315.1	300.2	286.5	273.8	262.1	251.2	241.1	231.7
118	493.9	222.3	493.9	493.9	490.7	479.4	466.9	453.6	439.8	425.8	412.0	385.1	359.9	336.7	315.6	296.5	279.1	263.4	249.1	236.0	224.2	213.3	203.3	194.2	185.8	178.0
96.8	421.2	187.4	421.2	421.2	416.8	406.2	394.3	381.4	368.0	354.3	340.6	314.1	289.4	266.9	246.8	228.8	212.9	198.8	186.2	174.9	164.8	155.8	147.6	140.2	133.5	127.3
250UC89.5	310.0	142.9	310.0	310.0	302.9	294.0	284.5	274.7	264.9	255.4	246.1	228.7	213.0	198.8	186.0	174.5	164.1	154.7	146.3	138.6	131.5	125.1	119.3	113.9	108.9	104.3
72.9	266.2	122.6	266.2	265.8	258.3	249.5	240.1	230.2	220.3	210.6	201.2	183.7	168.1	154.4	142.4	131.8	122.5	114.4	107.1	100.7	94.9	89.7	85.0	80.8	76.9	73.4
200UC59.5	177.1	80.7	177.1	173.6	167.0	159.8	152.6	145.5	138.7	132.2	126.2	115.2	105.6	97.3	90.0	83.6	78.0	73.0	68.5	64.5	60.9	57.7	54.8	52.1	49.7	47.5
52.2	153.9	70.2	153.9	150.6	144.5	137.9	131.0	124.3	117.9	111.8	106.1	95.8	87.0	79.5	73.0	67.4	62.5	58.2	54.4	51.1	48.1	45.4	43.0	40.9	38.9	37.1
46.2	133.4	60.2	133.4	130.3	124.8	118.8	112.5	106.3	100.3	94.6	89.3	79.9	71.9	65.2	59.5	54.6	50.4	46.7	43.5	40.7	38.3	36.1	34.1	32.3	30.7	29.3
150UC37.2	83.7	37.0	83.0	79.0	74.6	70.3	66.2	62.3	58.8	55.5	52.6	47.3	42.9	39.1	35.9	33.1	30.7	28.6	26.7	25.1	23.6	22.3	21.1	20.0	19.1	18.2
30.0	72.0	31.7	71.0	67.0	62.6	58.1	53.9	50.0	46.4	43.2	40.4	35.5	31.6	28.4	25.7	23.5	21.6	20.0	18.6	17.3	16.2	15.3	14.4	13.7	13.0	12.4
23.4	50.7	21.2	49.9	46.9	43.5	39.9	36.5	33.4	30.6	28.1	25.9	22.3	19.5	17.3	15.5	14.0	12.8	11.8	10.9	10.2	9.5	8.9	8.4	7.9	7.5	7.2
100UC14.8	21.4	9.9	20.0	18.3	16.7	15.3	14.0	12.9	12.0	11.1	10.4	9.1	8.1	7.3	6.6	6.0	5.5	5.1	4.7	4.4	4.1	3.9	3.7	3.5	3.3	3.2


Grade 300 UB Strong Axis Bending
610UB125
610UB113
610UB101
530UB92.4
530UB82
460UB82.1
460UB74.6
460UB67.1
410UB59.7
460UB53.7
360UB56.7
360UB50.7
360UB44.7
310UB46.2
310UB40.4
310UB32
250UB37.3
250UB31.4
250UB25.7
200UB29.8
200UB25.4
200UB22.3
200UB18.2
180UB22.2
180UB18.1
180UB16.1
150UB18.0
150UB14
10^3
10^2
10^1
$\alpha_s \phi M_s$ [kN·m]
0.0
2.5
5.0
7.5
10.0
12.5
15.0
17.5
20.0
Effective Length [m]


Grade 300 UC Strong Axis Bending
310UC158
310UC137
310UC118
310UC96.8
250UC89.5
250UC72.9
200UC59.5
200UC52.2
200UC46.2
150UC37.2
150UC30
150UC23.4
100UC14.8
10^2
10^1
$\alpha_s \phi M_s$ [kN·m]
0.0
2.5
5.0
7.5
10.0
12.5
15.0
17.5
20.0
Effective Length [m]

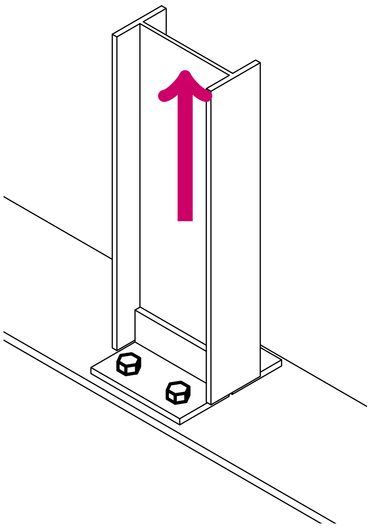
Bolted Connections

6.1 Connection Types

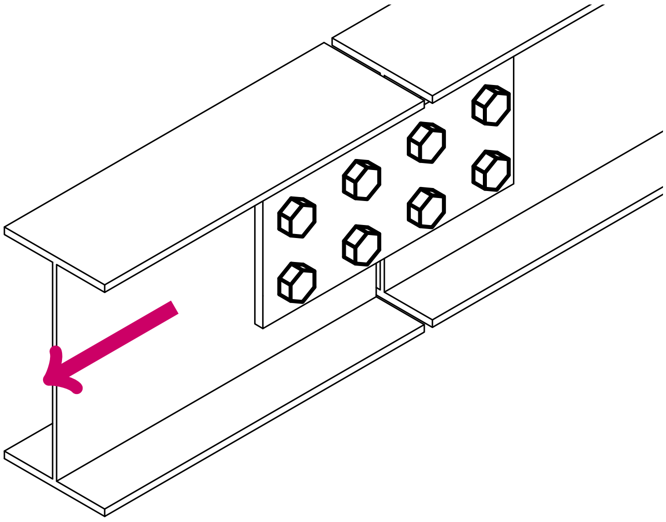
Because of experiences in the NZ construction industry, it is common practice not to conduct major structural welding at the construction site. Instead, the preferred method used is 'shop welding and site bolting'. This method has advantages in terms of construction speed, simplicity and quality control that are not enjoyed in other countries. However, proper planning is necessary to ensure that the building will fit together properly at the construction site.

Different connections can put different demands on the bolts.

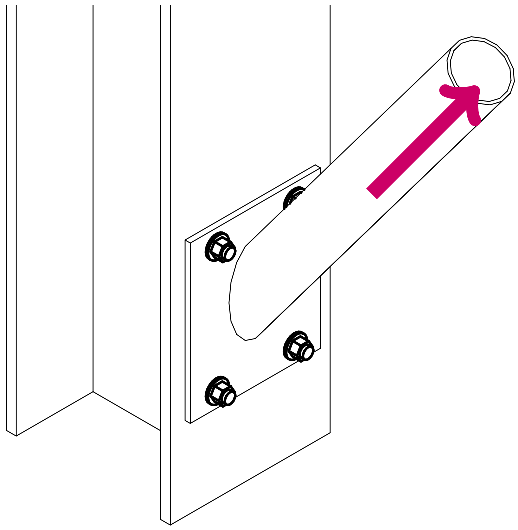
- Axial Tension — all bolts have the same tension



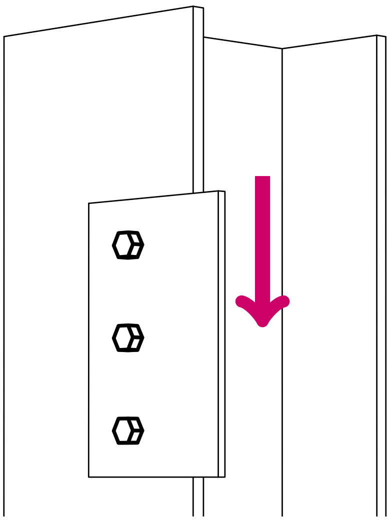
- Shear — all bolts have the same shear



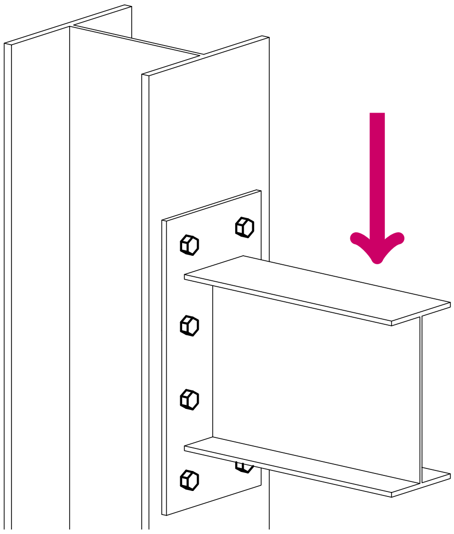
- Uniform Tension and Shear — all bolts have the same force



- Eccentric Shear



- Combined Tension and Shear



6.2 Connector Types

Standard connection types are now available in the Steel Connect[1]. This means that connections can be specified rapidly using standard notation, and every element of the connection does not need to be designed independently. It is the purpose of this chapter to explain how connections should be designed.

- **Pins**
  Used in 'zero moment' connections in truss joints.
- **Rivets**
  - Hot steel is deformed into shape
  - Shrinks as it cools giving high pretensioning
  - Labour intensive

[1] https://www.scnz.org/techincal-resources/connections-guide/

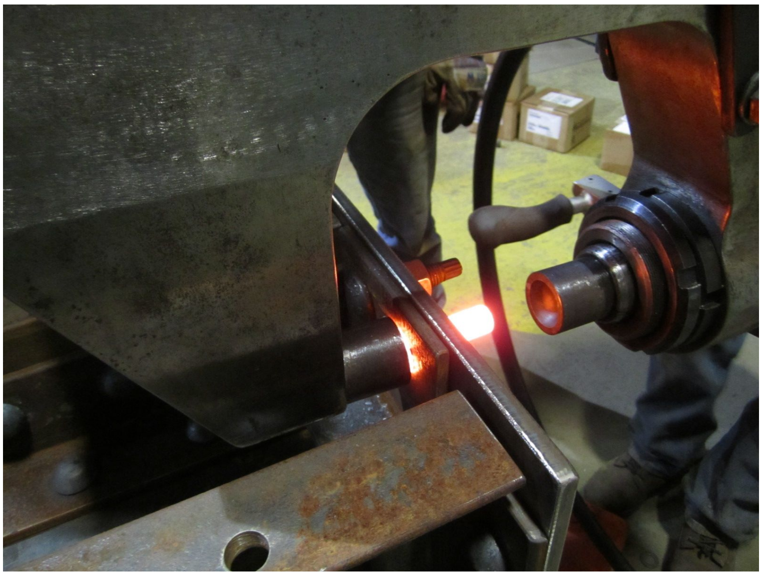


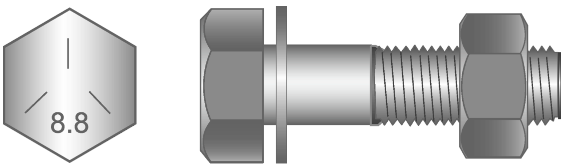
Figure 6.1: Installing a rivet

- **Bolts**
  Different head markings also exist depending on the mill, whether it is metric or imperial, and the preference of the mill. It is important to purchase bolts through an SCNZ approved distributor to ensure quality control.

  Bolts should be specified according to the grade and standard. (E.g, other grade 8.8 bolts are used for machinery, and these have different properties.) Most structural bolts are galvanised (rather than Zinc coated, or black).

  - Ordinary

  - High Strength (HS)



6.3 Bolt Strength

The types of bolts used in NZ for general structural use are of two types.

Grade	Tensile Strength, f_{uf}	Yield Strength, f_{yf}	Type	Standard
Grade 4.6	400 MPa	240 MPa	Ordinary or Mild Steel	AS 1111.1
Grade 8.8	830 MPa	660 MPa	High Strength or HSFG	AS/NZS 1252

HSFG stands for High Strength Friction Grip.

For Grade X.Y bolts, the ultimate strength f_{uf} is about $100 \times X$ MPa, the yield strength f_{yf} is about $10 \times X \times Y$ MPa.

Grade 4.6 bolts are mode of low carbon steel (similar to Grade 250 steel) and they are used mostly in secondary members.

Grade 8.8 bolts are make of medium carbon steel using quenching and tempering to enhance the properties. They are used in main framing.

Almost all bolts currently used in Australasia are now made in China.

6.4 Washer Size

Washers are useful to spread load from the bolt head or nut onto the material being clamped. Also, as washers have similar characteristics, they provide a relatively consistent coefficient of friction during rotation of the nut relative to the bolt when tightening during installation.

The acceptable dimension ranges for flat round washers for high strength structural bolting are given in AS/NZS 1252.1:2016 Fig. 4.1. In this class, a washer thickness of 3.5 mm for computations for all bolt sizes may be used as this value satisfies the dimensional criteria. In practice, the possible range of dimensions should be considered.

6.5 Bolt Dimension

The bolt dimensions are shown in Table 6.1.

Table 6.1: Dimensions of structural bolts

Type	shank diameter E	head height C	core area A_c	tensile area A_s	shank area A_o	Grade 4.6 width A	Grade 4.6 width B	Grade 4.6 nut height	Grade 8.8 width A	Grade 8.8 width B	Grade 8.8 nut height	thread pitch
M12	12	8	76.2	84.3	113	18	20	11				1.75
M16	16	11	144	157	201	24	26	15	27	31	17	2.0
M20	20	13	225	245	314	30	33	18	34	39	21	2.5
M24	24	16	324	353	452	36	40	22	41	47	24	3.0
M30	30	20	519	561	706	46	51	26	50	58	31	3.5
M36	36	24	759	817	1016	55	61	31	60	69	37	4.0

All numbers are in mm or mm^2, values are taken from AS 1111.1 and AS/NZS 1252.1:2016 .

The shank area is computed as $A_o = \pi \dfrac{E^2}{4}$. The core area is computed as $A_c = \pi \dfrac{D^2}{4}$. The tensile area is computed by using pitch diameter which includes threads thus is smaller than E (major diameter) but greater than D (minor diameter).

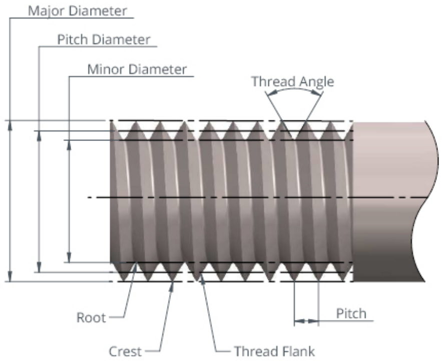
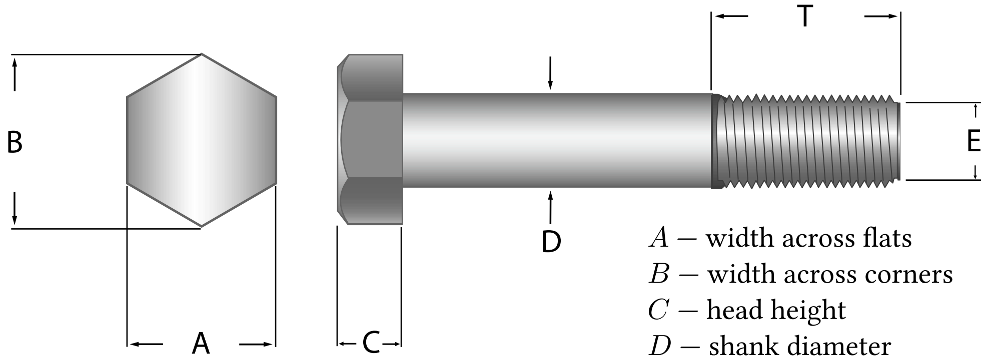


Figure 6.2: Bolt diameters (https://www.kelstonactuation.com/imagelibrary/screw-thread-principle.jpg)

If not given, the width across corners can be computed according to the width across flats.

$$B \approx \frac{2}{\sqrt{3}} A \approx 1.1547A. \tag{6.1}$$



A — width across flats
B — width across corners
C — head height
D — shank diameter
E — core diameter
T — thread length

Figure 6.3: Bolt dimensions

Bolt length is generally measured from the inside of the head of the bolt to the end of the bolt. Some bolts are threaded along their whole length. The minimum length of threaded bolt is shown as follows.

Table 6.2: Minimum length of thread

Nominal Length of Bolt, L	Minimum Length of Thread, T
$L \leqslant 125\,\text{mm}$	$2D + 6$
$125\,\text{mm} \leqslant L \leqslant 200\,\text{mm}$	$2D + 12$
$200\,\text{mm} \leqslant L$	$2D + 25$

According to NZS 3404.1&2:1997 14.3.6.1.2, a bolt should have

- At least one clear thread above the nut after tightening, and

- The following minimum number of threads beneath the nut after tightening
  - for snug tightened bolts: one clear thread
  - for tensioned bolts (i.e., proof loaded bolts) with diameter, d_b:

bolt length between bolt head and nut, l_g	No. threads
$l_g \leqslant 4d_b$	5
$4d_b < l_g \leqslant 8d_b$	7
$4d_b < l_g$	10

Note: The term "grip length" or "grip" of a bolt is defined differently in NZS 3404.1&2:1997 Append. K1.2.4, AS/NZS 1252.1:2016 § 2.3.5 and international standards. Sometimes it includes the distance between the bolt head and nut, and other times it is this minus the thickness of washer(s). Full definition is required when using this term.

Figure 6.4: Bolt, washer and nut

6.6 Installation of Bolts

Bolts may be installed in two ways:

- Grade 4.6 Bolts
  - Snug Tightened (4.6/S)
- Grade 8.8 Bolts
  - Snug Tightened (8.8/S)
  - Proof Loaded (8.8/T)

Snug tightening ensures that the bolt is in full contact with the material. It is defined as ‘the full effort of one man on a hand wrench tightening the bolt’. It is assumed that the surfaces are clean and flat.

Proof loading may be carried out by any of the following methods.

1. **Torque wrench**
   The torque to turn the bolt is measured giving an indication of the bolt tensile force. It must be calibrated. Clean and flat surfaces are assumed.

2. **Specified nut rotation method, a.k.a. turn-of-nut method**
   Snug tighten is initially performed. Then the nut is turned a specified amount relative to the bolt as shown.

"Snug" tighten — Mark nut and bolt — Desired position achieved (1/3 turn illustrated)

Figure 6.5: Steps of turn-of-nut method

The amount of turn required depends on the bolt type. This method is

- cheap,
- less dependent on surface conditions, and
- easy to inspect.

To fully tension (i.e., proof load) a bolt, the number of turns of the nut relative to the bolt after snug tightening to achieve the desired bolt tension, where surfaces beneath the bolt head and nut are parallel, should be the following for different bolt diameters, d_b (NZS 3404.1&2:1997 Table 14.3.6.1.2).

bolt length between underside of bolt head and end of bolt, l	rotations
$l \leqslant 4d_b$	1/3 turn
$4d_b < l \leqslant 8d_b$	1/2 turn
$8d_b < l$	2/3 turn

3. **Direct tension indicator (DTI)**
   Tightening on washers with protrusions.

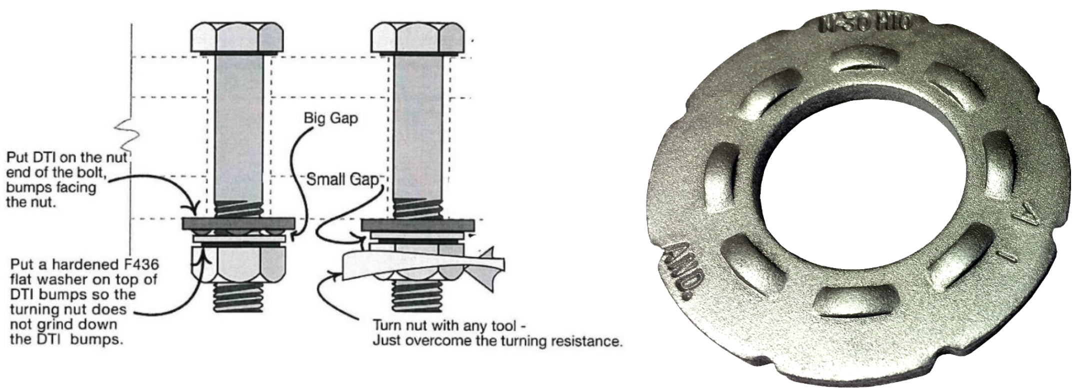


Figure 6.6: Direct tension indicator and protrusion

This method is

- less dependent on surface conditions,

- easy to inspect, and
- hard to cheat with.

4. **Tension Control Bolts**
   Fracture of bolt end occurs at the notch when proof load is reached.

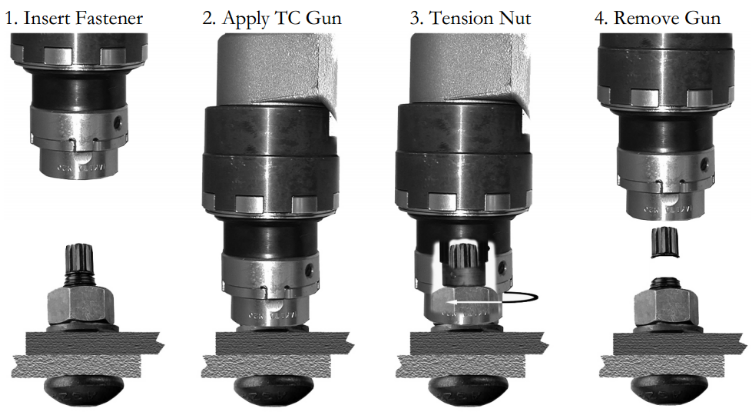


Figure 6.7: Steps of installing TC bolts

This method is

- fast and easy to inspect, and
- dependent on surface conditions.

All of the proof loading methods above require the bolt to sustain some permanent deformation. Therefore, after proof loading they should **not** be reused.

6.7 Erection Tolerances

In design of bolts, one shall consider the size of tools used.

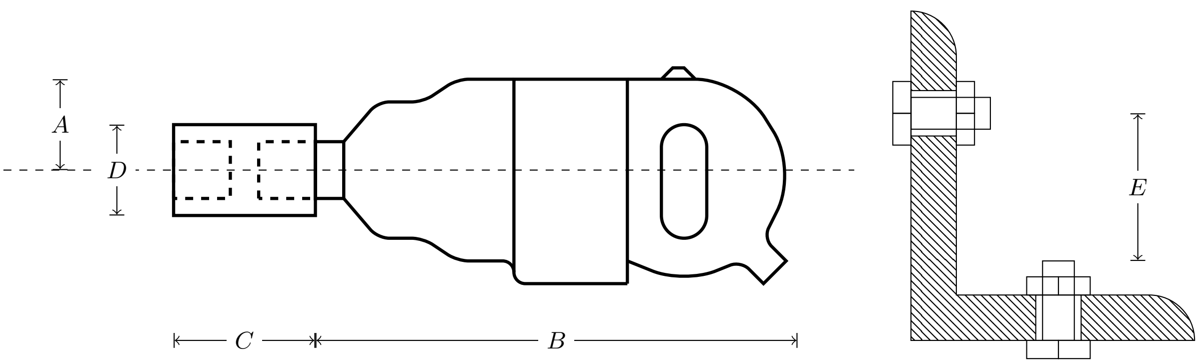


Figure 6.8: Definition of impact wrench sizes (Australian Steel Institute, 2016)

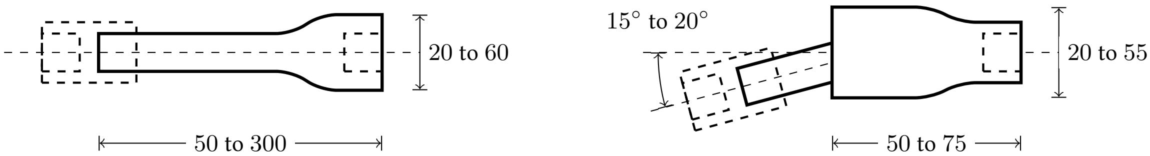
Table 6.3: Impact wrench sizes (Australian Steel Institute, 2016)

Impact wrench type	B mm	A mm
Normal	to 370	55
Heavy	some to 600	65

Table 6.4: Impact wrench sizes (Australian Steel Institute, 2016)

Nominal Bolt Diameter	Sockets 20 mm drive			Sockets 25 mm drive		
			Clearance			Clearance
	C	D	E	C	D	E
mm	mm	mm	mm	mm	mm	mm
16	54	48	30	60	58	35
20	57	58	35	63	58	35
24	58	61	35	70	68	40

If extension bar and/or universal joint are used, their dimensions shall also be considered.



(a) Extension bar

(b) Universal joint

Figure 6.9: Dimension of extension bar and universal joint (Australian Steel Institute, 2016)

6.8 Modes of Carrying Shear Force

6.8.1 Shear Planes

Bolts can be in single (one shear plane) or double (two shear planes) shear as shown below. Threads may be iNcluded (N) or eXcluded (X) from the shear plance (e.g., M20X bolt).

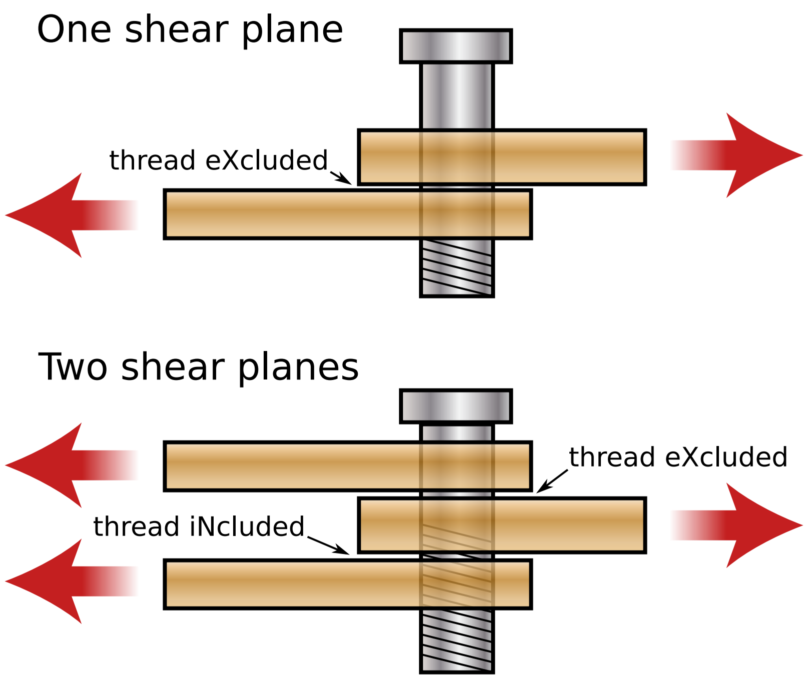


Figure 6.10: Illustration of number of shear planes (https://commons.wikimedia.org/wiki/File:Bolt-in-shear.svg)

6.8.2 Snug Tightening

Shear forces are transferred through **shear in the bolt**. The following illustration shows two shear planes.

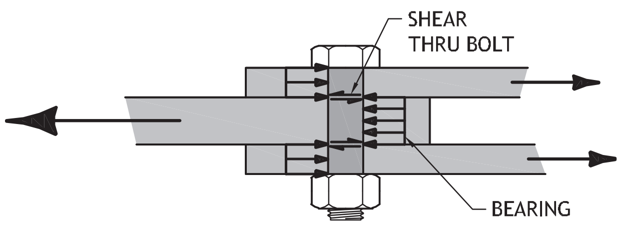


Figure 6.11: Snug tighten bolts (McMullin et al., 2018)

6.8.3 Proof Loading

This involves tightening the bolt in such a way that it provides a large compressive force on the elements it connects. Force is transferred through **friction in the plates**. Friction resistance is dependent on both axial (bolt) force P and surface condition (friction coefficient μ). No slip occurs until friction is overcome. When the friction force is overcome, shear force is transferred through both **bolt shear** and **friction**.

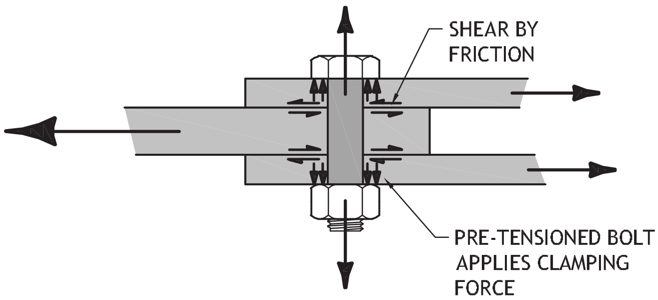


Figure 6.12: Proof loading bolts (McMullin et al., 2018)

There are two types of 8.8/T bolts.

- 8.8/TB — Tension Bearing
- 8.8/TF — Tension Friction

These are identical expect that the /TF bolting has treatment of the mating surfaces such that the friction is increased.

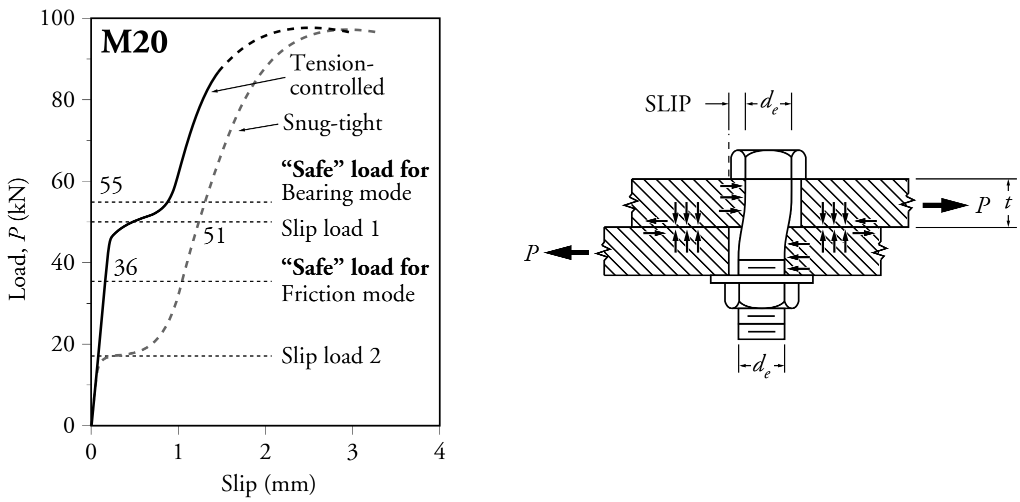


Figure 6.13: Behaviour of high-strength structural bolts. Slip load 1 applies to tension-controlled HS structural bolts (i.e., proof loaded bolts) using a slip factor of 0.35. Slip load 2 applies to snug-tight bolts. (Gorenc et al., 2015)

» 6.9 Minimum Bolt Proof Loads

It is necessary to tighten the bolt to obtain its 'proof load' in order to get the maximum benefit from it.

Proof loading the bolt is tightening it such as it stretches until it applies at least the 'proof load', P_L. The bolt is then in tension applying a compressive force of P_L to the plates. The compression causes friction between the plates and minimizes deformation.

The minimum proof load for specific bolts is specified in the following table (NZS 3404.1&2:1997 Table 15.2.5.1).

Table 6.5: Minimum Bolt tension for property class 8.8 bolts

Nominal diameter of bolt	Minimum bolt tension (proof load, kN)
M16	95
M20	145
M22	180
M24	210
M30	335
M36	490

The axial force of a proof loaded bolt should be as high as possible without causing a risk of major bolt deformation or fracture. The minimum bolt proof load in old standards was determined by a mixed consideration of the following:

- 0.2 % offset strain,
- 0.5 % extension under load,
- 70 % of f_u.

It is now being changed to simply be $0.7f_u$.

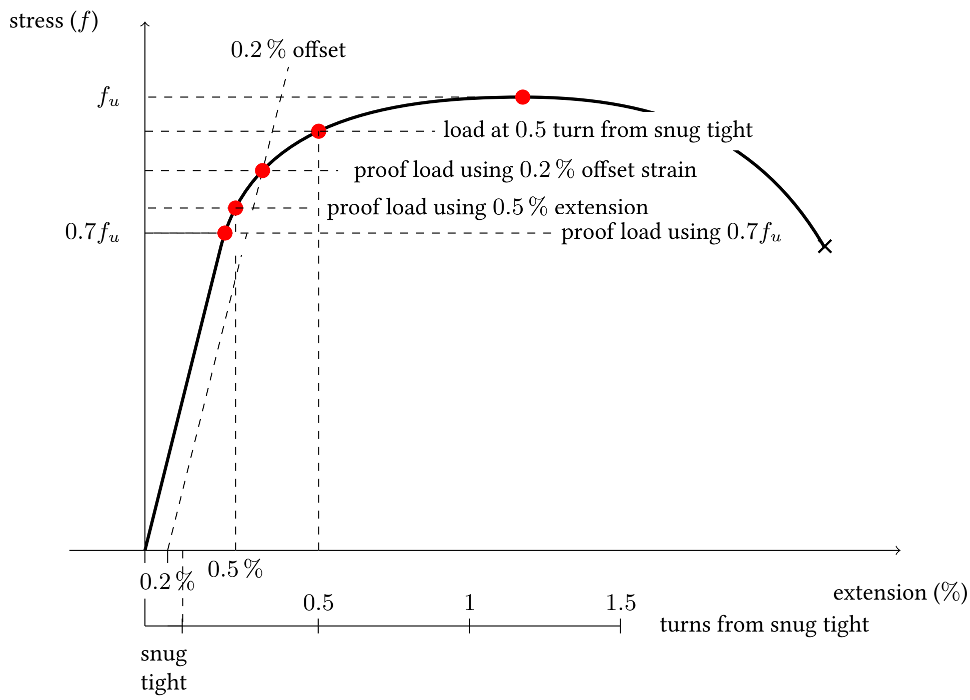


Figure 6.14: Proof load

6.10 Strengths of Different Bolt Types

Table 6.6: Grade 4.6/S bolted connection strengths for each surface per shear plane

Size	Shear (eXcluded) ϕV_{fx} (kN)	Shear (iNcluded) ϕV_{fn} (kN)	Axial Tension ϕN_f (kN)
M12	22.4	15.1	27
M16	39.9	28.6	50.2
M20	62.3	44.6	78.4
M24	89.7	64.3	113
M30	140	103	180
M36	202	151	261

Table 6.7: Grade 8.8 bolted connection strengths per shear plane

Size	Strength /S or /T			Serviceability /T				
						Shear ϕV_{sf} (kN) for $\mu = 0.35$		
	Shear (eXcluded) ϕV_{fx} (kN)	Shear (iNcluded) ϕV_{fn} (kN)	Axial Tension ϕN_f (kN)	Proof Load N_{tf} (kN)	Axial Tension ϕN_{tf} (kN)	Standard $k_h = 1.0$	Short $k_h = 0.85$	Long $k_h = 0.7$
M16	82.7	59.3	104	95	66.5	23.3	19.8	16.3
M20	129	92.6	163	145	101.5	35.5	30.2	24.9
M24	186	133	234	210	147	51.5	43.7	36.0
M30	291	214	373	335	234.5	82.1	69.8	57.5
M36	419	312	542	490	343	120.1	102.0	84.0

6.11 Strength Design

The strength reduction factor ϕ shall be taken as follows for bolted connections.

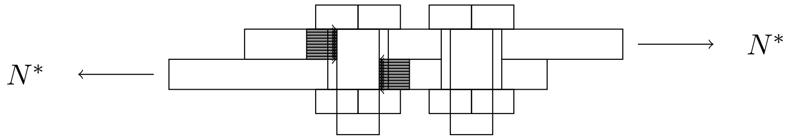
Table 6.8: Strength reduction factor for bolted connections

Design capacity	Section	ϕ	Location
(ULS) bolt in shear	NZS 3404.1&2:1997 § 9.3.2.1	0.8	in bolt
(ULS) bolt in tension	NZS 3404.1&2:1997 § 9.3.2.2	0.8	in bolt
(ULS) bolt in combined shear and tension	NZS 3404.1&2:1997 § 9.3.2.3	0.8	in bolt
(ULS) ply in bearing	NZS 3404.1&2:1997 § 9.3.2.4	0.9	on steel
(ULS) bolt group	NZS 3404.1&2:1997 § 9.4	0.8	in bolt
(SLS) friction type	NZS 3404.1&2:1997 § 9.3.3	0.7	between steel

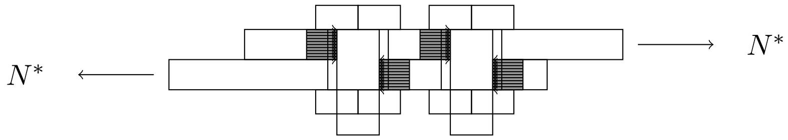
6.11.1 Connection Behaviour

Load is initially carried by only one bolt due to inaccurate hole placement. After it yields, other bolts can carry loads. The concept is illustrated in the following figure. **The shear force in a connection is often assumed to be carried equally by each of the bolts.** Bearing connections are used when slip is not important.

Initially left bolt is loaded.



After left bolt yields, right bolt can also carry load.



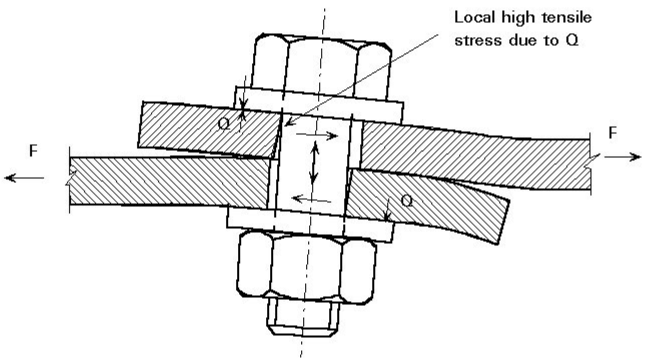


Figure 6.15: Actual deformation of bolted single shear tension connection (http://fgg-web.fgg.uni-lj.si/~/pmoze/esdep/master/wg11/l0310.htm)

6.11.2 Yield on Plate Gross Area

This was discussed in § 3.1, see Eq. (3.2).

6.11.3 Fracture on Plate Net Area

This was discussed in § 3.1, see Eq. (3.3).

6.11.4 Bolt Shear Failure

The design strength of bolts in shear is affected by the ultimate shear strength of the steel, the bolt net area, the mode by which shear force is carried, the distribution of force among bolts in a connection.

The core area of the bolt, A_c, is used when the shear plane passes through the threaded area. A_c is given in Table 6.1 and it is often $0.75A_o$ to $0.8A_o$ where A_o is the shank area.

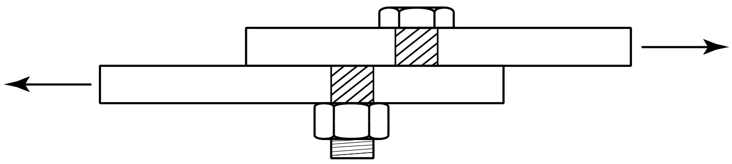


Figure 6.16: Bolt shear failure

Experiments show that

- the ultimate shear stress in bolts is $\tau_u \approx 0.62 f_u$. The value of 0.62 is close to $\tau_y/f_y = 1/\sqrt{3} = 0.577$ given by von Mises yielding criterion of steel;
- in connections which are more than $1300\,\mathrm{mm}$ long, the forces are not shared evenly over all bolts, so bolts should be designed to resist a greater shear force;
- a bolt in double shear can carry twice as much shear force as one in single shear (as long as they both have the same shear area).

The design shear force V_f^* for a bolt shall satisfy (NZS 3404.1&2:1997 § 9.3.2.1)

$$V_f^* \leqslant \phi V_f = \phi 0.62 k_r f_{uf} \left(n_n A_c + n_x A_o\right), \tag{6.2}$$

where

ϕ = strength reduction factor, 0.8
k_r = reduction factor
f_{uf} = minimum tensile strength of the bolt
n_n = number of shear planes **iNcluding** threads intercepting the shear plane
A_c = minor diameter area of the bolt
n_x = number of shear planes **eXcluding** threads intercepting the shear plane
A_o = nominal plain shank area of the bolt

The factor k_r shall be taken as 1.0 for lap connections with lengths up to $300\,\mathrm{mm}$ and 0.75 for lap connections with lengths over $1300\,\mathrm{mm}$. Linear interpolation shall be used for lengths between $300\,\mathrm{mm}$ and $1300\,\mathrm{mm}$. For all other connections, $k_r = 1.0$.

It can be alternatively expressed as

$$V_f = n_n V_{fn} + n_x V_{fx}, \tag{6.3}$$

with

$$V_{fn} = 0.62 k_r f_{uf} A_c, \tag{6.4}$$
$$V_{fx} = 0.62 k_r f_{uf} A_o. \tag{6.5}$$

The nominal capacity V_f is the summation of capacities of all shear planes of two types.

6.11.5 Plate Bearing and Tearing Failure Beside Bolts

For bolts to develop their strength, the material around bolts must be strong enough to resist bolt forces. That is, the plate material should not fail in **bearing**, and if bolts are near the side or edge of the plate or if bolt holes near other holes, then the possibility of **plate yielding or facture** should be considered in the assessment of the maximum force that the bolt can carry. Different values of edge distance are given for holes of different sizes and shapes.

Bearing causes deformation and elongation of the plate beside the hole. Strength loss will occur due to plate fracture as discussed later.

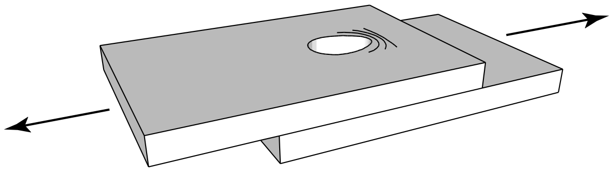


Figure 6.17: Bearing failure

Codes generally have an allowable resistance, R_n, greater than $A_b f_u$. This is due to the following reasons.

- The part of the plate in bearing is subject to compressive force.
  - The ultimate strength f_u is found from a tension test.
  - In compression, the strength is larger (see Fig. 2.3).
- Swelling of the plate (Poisson's effect) causes an increase in bolt compressive forces. This cause a triaxial loaded state which leads to higher strength.
- Collapse does not occur as a result of bearing failure.

If the strength of bolt is less than the strength of plate, then design for bearing of the bolt should be carried out.

Bolt tear out failure occurs when the distance between bolt hole and the edge of plate, or the distance between adjacent bolt holes in the line of force is small.

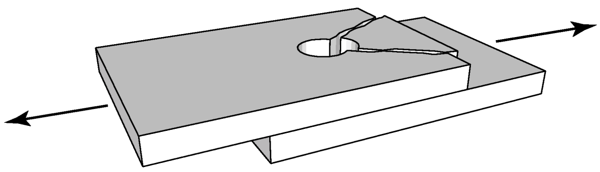


Figure 6.18: Tear out failure

Edge failure due to bolt loading in a direction which is not in the line of force should also be considered.

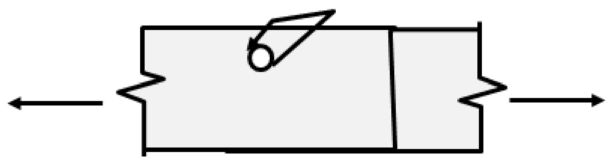


Figure 6.19: Edge failure of plate

Oversize holes may require special edge distance and bearing design because loading may be more concentrated.

According to NZS 3404.1&2:1997 § 9.3.2.4.1, a ply subject to a design bearing force V_b^* due to bolt in shear shall satisfy

$$V_b^* \leqslant C_1 \phi V_b. \tag{6.6}$$

For **non-seismic** design, $C_1 = 1.0$. For seismic design, $C_1 < 1.0$. It could be as low as 0.6 for seismic design according to NZS 3404.1&2:1997 § 12.9.4.3. Also, using $C_1 = 0.6$ is a good default for all NZ connections as it will reduce the bolt hole deformation, and the possibility of significantly pinched hysteretic behaviour causing large impacts during earthquake shaking. The nominal bearing capacity V_b of a ply shall be the **smaller** of the following two.

1. **Ply bearing failure beside bolt (Eq. 9.3.2.4(1))**

$$V_b = 3.2 d_f t_p f_{up}, \tag{6.7}$$

where

d_f = bolt diameter
t_p = ply thickness
f_{up} = ply ultimate strength

2. **Bolt tearing out failure (Eq. 9.3.2.4(2))**

$$V_b = a_e t_p f_{up}, \tag{6.8}$$

where

a_e = minimum distance from hole edge to edge of ply in direction of force plus half of d_f

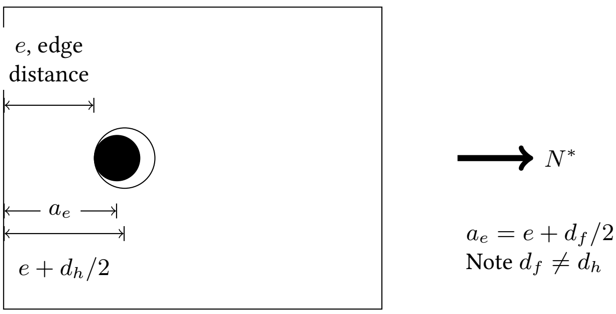


Figure 6.20: Illustration of a_e

The following requirements shall be met to avoid potential failure modes.

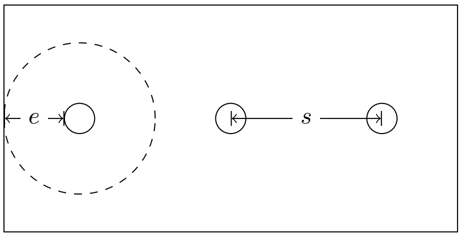


Figure 6.21: Illustration of minimum pitch s and edge distance e

1. **Bolt spacing failure**
   The minimum pitch s (the distance between centres of bolt holes)

   - in any direction shall be at least $2.5d_f$. (NZS 3404.1&2:1997 § 9.6.1)

   $$s \geqslant 2.5d_f. \tag{6.9}$$

   - in the direction of force, the computation of s should be such that Eq. (6.8) is satisfied. Here, to compute a_e, the 'minimum distance from hole edge to edge of ply in direction of force plus half of d_f' is taken as 'the clear distance between bolt holes in the direction of the component of force plus half of d_f'.

2. **Bolt side failure**
   NZS 3404.1&2:1997 § 9.6.2.1 defines the edge distance to be the distance from the nearer edge of the hole to the physical edge of the plate or rolled section **plus** half of d_f. The edge distance $a_e = e + d_f/2$ shall meet the following minimum values for different cases:

   - $1.75d_f$ for sheared or hand flame cut edge
   - $1.5d_f$ for rolled plate, flat bar or section: machine flame cut, sawn or planed edge
   - $1.25d_f$ for rolled edge of a rolled flat bar or section

3. **Corrosion between plates**
   NZS 3404.1&2:1997 § 9.6.4 requires the maximum distance from the centre of any bolt hole to the edge of the plate to be the smaller of $12t_p$ and 150 mm.

   $$s \leqslant \min\left(12t_p,\ 150\,\text{mm}\right). \tag{6.10}$$

6.11.6 Block Tearing Failure

Block shear/tearing failure is not explicitly considered in NZS 3404.1&2:1997 Steel structures standard. However, it has caused connection failures in the past.

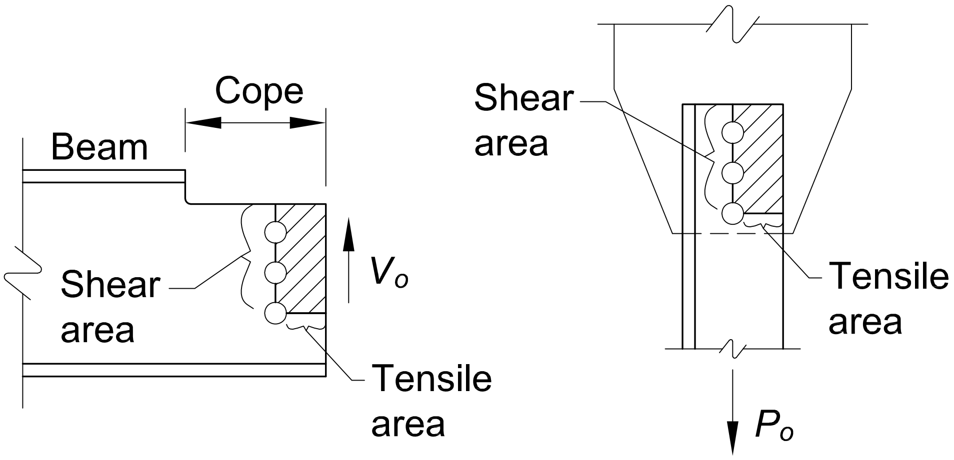


Figure 6.22: Illustration of block tearing failure

Similar to tension members, in this course, we the following expression to determine capacity when tension stress is uniform.

$$N^* \leqslant \phi 0.95 \left(0.6 A_{ev} f_u + A_{nt} f_u\right), \tag{6.11}$$

where

$$\begin{aligned} \phi &= \text{strength reduction factor, } 0.9 \\ A_{ev} &= \text{effective area subject to shear} \\ A_{nt} &= \text{net area subject to tension} \end{aligned}$$

The effective shear area is taken as the average of net and gross shear areas.

$$A_{ev} = \frac{1}{2}\left(A_{gv} + A_{nv}\right), \tag{6.12}$$

where A_{nv} and A_{gv} have been introduced previously and are also illustrated in the following figure.

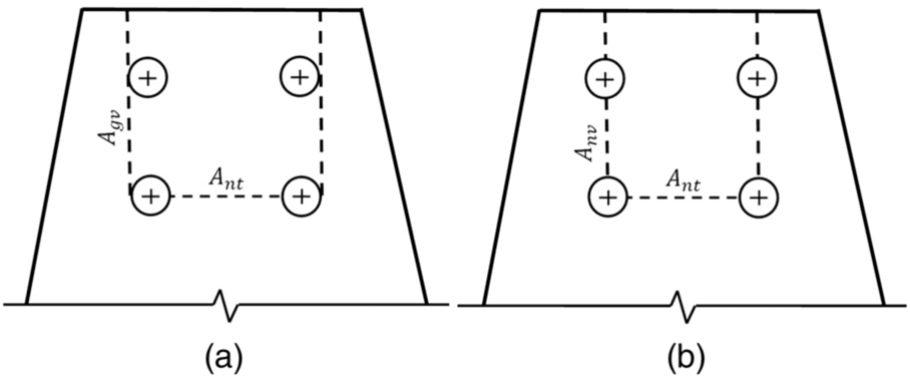


Figure 6.23: Definition of net and gross areas

Figure 6.24: Failure surface of block shear/tearing (https://m2ukblog.wordpress.com/2016/05/28/block-shear-failure-in-tension-members/)

6.11.7 Bolt Tension Failure

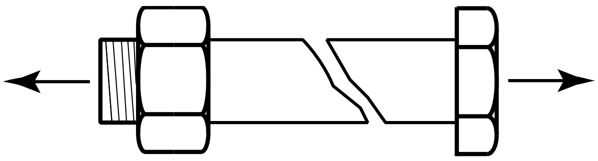


Figure 6.25: Bolt tension failure

NZS 3404.1&2:1997 9.3.2.2 requires

$$N_{tf}^* \leqslant \phi N_{tf} = \phi A_s f_{uf}, \tag{6.13}$$

where

ϕ = strength reduction factor, 0.8
N_{tf} = nominal tension capacity of a bolt
A_s = tensile stress area of a bolt
f_{uf} = minimum tensile strength of a bolt

6.11.8 Bolt Combined Failure

NZS 3404.1&2:1997 9.3.2.3 requires

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leqslant 1.0. \tag{6.14}$$

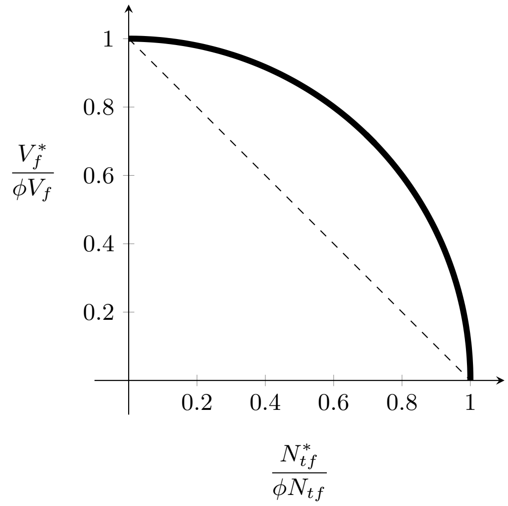


Figure 6.26: Envelop of axial force versus shear

6.11.9 Bending Failure of Bolts

Bending failure is not usually aa problem expect for very long bolts. Standard methods can be used.

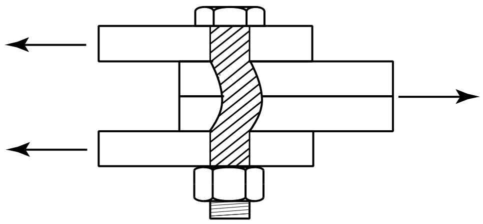


Figure 6.27: Bending failure of bolts

6.11.10 Fatigue Failure of Bolts

The strength of bolts is reduced by fatigue and methods are available for design.

6.11.11 Prying

Prying forces, Q, act on the flange of an I or T section when the web is in tension and the flange stiffness is moderate. The prying forces cause a) larger bolt forces, and b) greater flange moments. The actual size of Q is difficult to assess. However, it can be significant. A design method is described by Salmon et al. (2009).

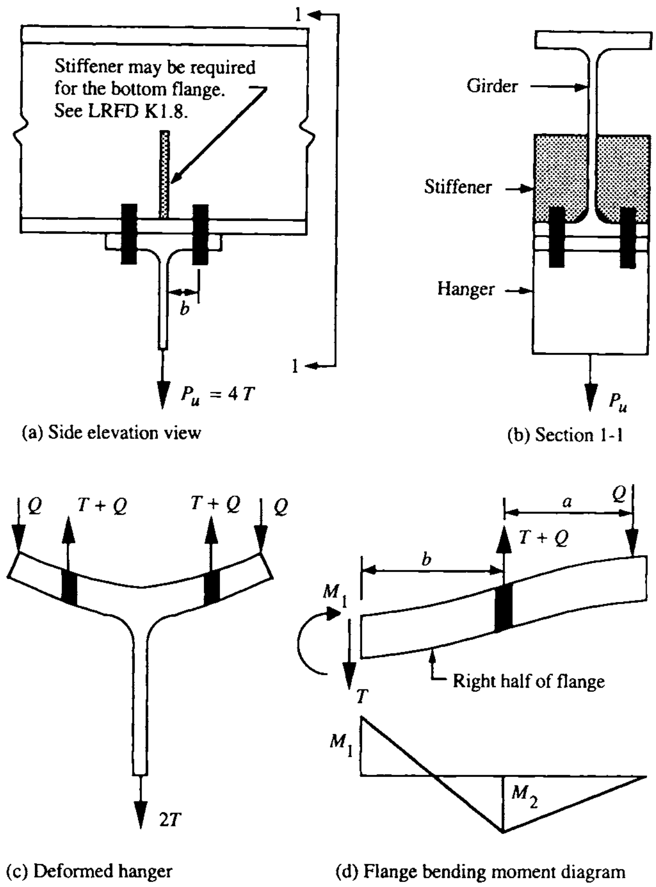


Figure 6.28: T section hanger connection (Smith and Smith, 1996)

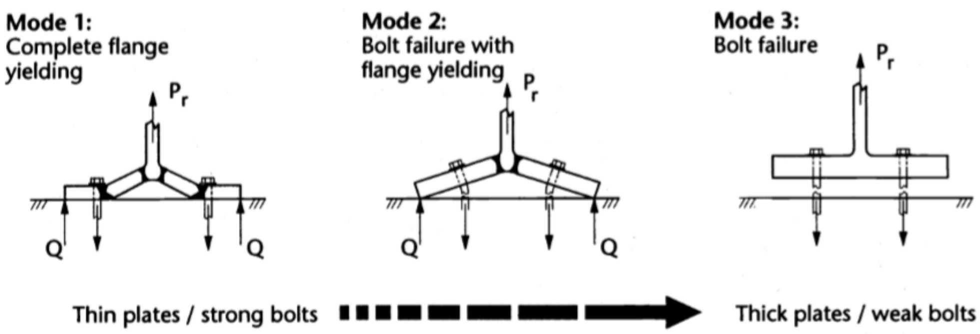
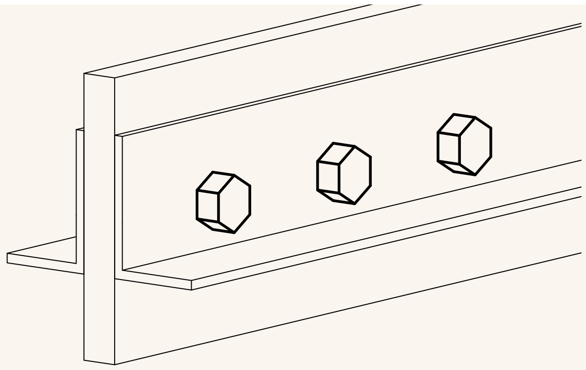


Figure 6.29: Prying failure modes (https://www.structures-simplified.com/2020/08/why-prying-force-is-important.html)

Example 6.1 *Worksheet Double Angle Bolted Connection*

Assume connection length is smaller than 300 mm, *find the lightest pair of Grade 300 angles with long legs back-to-back to carry* $N^* = 320\,\text{kN}$. *Using bearing type M16 Grade 8.8N bolts, a bolt spacing* $s = 40\,\text{mm}$ *and an end distance of* 30 mm.



Solution 6.1

1. Determine the number of bolts required.
   The connection length is smaller than 300 mm, $k_r = 1.0$. The shear capacity per bolt shall be calculated as

$$\begin{aligned}\phi V_f &= \phi 0.62 k_r f_{uf} \left(n_n A_c + n_x A_o\right)\\ &= \phi 0.62 k_r f_{uf} n_n A_c\\ &= 0.8 \times 0.62 \times 1 \times 830\,\mathrm{MPa} \times 2 \times 144\,\mathrm{mm}^2\\ &= 118.6\,\mathrm{kN}.\end{aligned}$$

   The above value can also be computed as $2 \times 59.3\,\mathrm{kN} = 118.6\,\mathrm{kN}$, see Table 6.7.

   The number of bolts is then

$$n \geqslant \frac{N^*}{\phi V_f} = 2.70.$$

   Thus use 3 bolts.

2. Check bearing on angle.
   The bearing force on each bolt is

$$V_b^* = \frac{320\,\mathrm{kN}}{3 \times 2} = 53.3\,\mathrm{kN}.$$

   The bearing capacity of each bolt shall be greater than force,

$$\phi 3.2 d_f t_p f_{up} = \phi V_b \geqslant V_b^*$$

   This leads to

$$t_p \geqslant \frac{53.3\,\mathrm{kN}}{0.9 \times 3.2 \times 16\,\mathrm{mm} \times 440\,\mathrm{MPa}} = 2.63\,\mathrm{mm}.$$

3. Check end distance.

$$\phi a_e t_p f_{up} = \phi V_b \geqslant V_b^*.$$

   This gives

$$t_p \geqslant \frac{53.3\,\mathrm{kN}}{0.9 \times 29\,\mathrm{mm} \times 440\,\mathrm{MPa}} = 4.64\,\mathrm{mm}.$$

4. Check yielding on gross area.
   Assume $f_y = 320\,\mathrm{MPa}$ and $f_u = 440\,\mathrm{MPa}$,

$$A_g \geqslant \frac{1}{2}\frac{N^*}{\phi f_y} = \frac{1}{2}\frac{320\,\mathrm{kN}}{0.9 \times 320\,\mathrm{MPa}} = 555.6\,\mathrm{mm}^2.$$

5. Check fracture on net area.
   It can be looked up $k_{te} = 1.0$.

$$A_e \geqslant \frac{1}{2}\frac{N^*}{\phi 0.85 k_{te} f_u} = \frac{1}{2}\frac{320\,\mathrm{kN}}{0.9 \times 0.85 \times 1 \times 440\,\mathrm{MPa}} = 475.3\,\mathrm{mm}^2.$$

   The gross area should satisfy

$$A_g \geqslant 475.3\,\mathrm{mm}^2 + 18t_p.$$

6. Check edge distance in any direction.
   (a) The minimum distance:

$$L_{e,min} = 1.25d_f = 1.25 \times 16\,\mathrm{mm} = 20\,\mathrm{mm}.$$

   This is equivalent to 21 mm to bolt centre.
   (b) The maximum distance:

$$L_{e,max} = \min\left(150\,\mathrm{mm},\ 12t_p\right) = 60\,\mathrm{mm},$$

   assuming $t_p = 5\,\mathrm{mm}$.

A quick summary, the desired section shall meet the following requirements:

1. $t_p \geqslant 4.64\,\mathrm{mm}$
2. $A_g \geqslant 555.6\,\mathrm{mm}^2$
3. $A_g \geqslant 475.3\,\mathrm{mm}^2 + 18t_p$
4. $21\,\mathrm{mm} \leqslant$ Edge Distance $\leqslant 60\,\mathrm{mm}$

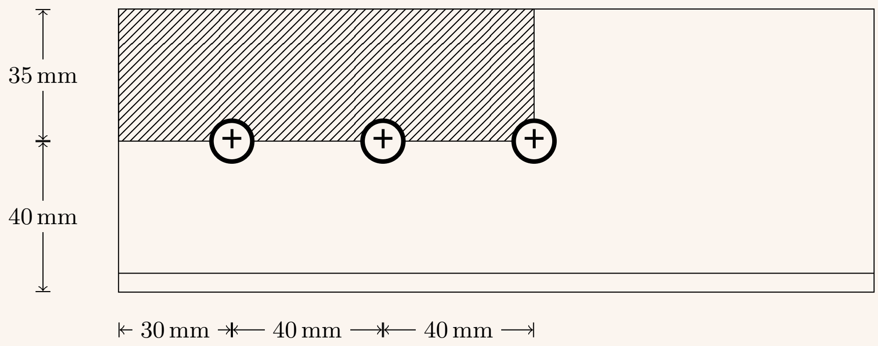
Try 65×50×5UA, $A_g = 512\,\mathrm{mm}^2$. This does not satisfy the requirement.

Try 75×50×5UA, $A_g = 560\,\mathrm{mm}^2 > 555.6\,\mathrm{mm}^2$, okay. Check net area, $A_{g,min} = 475.3\,\mathrm{mm}^2 + 18\,\mathrm{mm} \times 5\,\mathrm{mm} = 565.3\,\mathrm{mm}$. The difference is $0.9\,\%$, okay.

Check bolt spacing, $s = 40\,\mathrm{mm} \geqslant 2.5d_f = 2.5 \times 16\,\mathrm{mm} = 40\,\mathrm{mm}$.

$$\begin{aligned}\phi V_b &= \phi a_e t_p f_{up} \\ &= 0.9 \times \left(40\,\mathrm{mm} - 18\,\mathrm{mm} + \frac{16\,\mathrm{mm}}{2}\right) \times 5\,\mathrm{mm} \times 440\,\mathrm{MPa} \\ &= 59.4\,\mathrm{kN} > 53.3\,\mathrm{kN}.\end{aligned}$$

According to Table 2.8, choose a gauge length of 40 mm. Thus try the following layout.



Check block tearing.

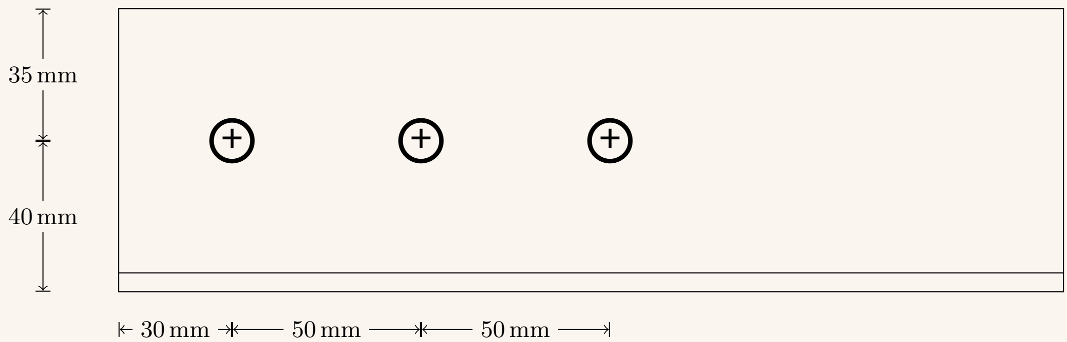
$$
\begin{aligned}
A_{gv} &= 2 \times 110\,\mathrm{mm} \times 5\,\mathrm{mm} = 1100\,\mathrm{mm}^2, \\
A_{gt} &= 2 \times 35\,\mathrm{mm} \times 5\,\mathrm{mm} = 350\,\mathrm{mm}^2, \\
A_{nt} &= 2 \times (35\,\mathrm{mm} - 0.5 \times 18\,\mathrm{mm}) \times 5\,\mathrm{mm} = 260\,\mathrm{mm}^2, \\
A_{nv} &= 2 \times (110\,\mathrm{mm} - 2.5 \times 18\,\mathrm{mm}) \times 5\,\mathrm{mm} = 650\,\mathrm{mm}^2.
\end{aligned}
$$

Use Eq. (6.11),

$$
\begin{aligned}
&\phi 0.95\,(0.6 A_{ev} f_u + A_{nt} f_u) \\
=&0.9 \times 0.95 \times \left(0.6 \times \frac{1100\,\mathrm{mm}^2 + 650\,\mathrm{mm}^2}{2} \times 440\,\mathrm{MPa} + 260\,\mathrm{mm}^2 \times 440\,\mathrm{MPa}\right) \\
=&295\,\mathrm{kN} < N^* = 320\,\mathrm{kN}.
\end{aligned}
$$

NOT GOOD

Block tearing is the controlling mode and must be considered. Therefore, to keep lightest angles, the spacing of bolts must be increased. The following spacing is satisfactory.



6.12 Serviceability Design

For the serviceability conditions, serviceability loads must be used.

6.12.1 Shear Slip

NZS 3404.1&2:1997 § 9.3.3.1 requires a bolt subjected only to a design shear force V_{sf}^* in the plane of the interfaces to satisfy

$$V_{sf}^* \leqslant \phi V_{sf} = \phi \mu_s n_{ei} N_{ti} k_h, \tag{6.15}$$

where

$$\begin{aligned}
\phi &= \text{strength reduction factor, } 0.7\\
V_{sf} &= \text{nominal shear capacity of a bolt}\\
\mu_s &= \text{slip factor}\\
&= 0.35 \text{ for clean as-rolled surfaces}\\
&= 0.48 \text{ for flame cleaned surfaces}\\
&= 0.53 \text{ for grit-blasted surfaces}\\
&= 0.18 \text{ for galvanized surfaces as received}\\
&= 0.30 \text{ for galvanized surfaces as lightly sandblasted}\\
&= \text{test values for other conditions}\\
n_{ei} &= \text{number of effective interfaces}\\
N_{ti} &= \text{minimum bolt tension at installation (NZS 3404.1\&2:1997 Table 15.2.5.1)}\\
k_h &= \text{factor for different hole types}\\
&= 1.0 \text{ for standard holes}\\
&= 0.85 \text{ for short slotted and oversize holes}\\
&= 0.70 \text{ for long slotted holes}
\end{aligned}$$

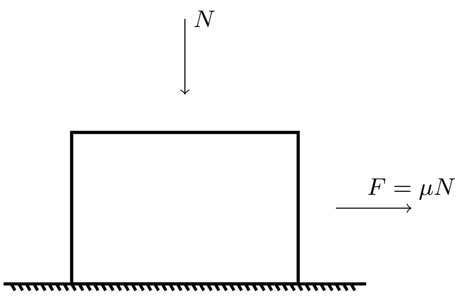


Figure 6.30: Friction force

6.12.2 Gap Opening (Tension Behaviour)

$$N_{tf}^* \leqslant \phi N_{tf} = \phi N_{ti}, \tag{6.16}$$

where

$$\begin{aligned}
\phi &= \text{strength reduction factor, } 0.7\\
N_{tf} &= \text{nominal tension capacity}\\
N_{ti} &= \text{minimum bolt tension at installation}
\end{aligned}$$

6.12.3 Shear Slip Under Tension

In combined shear and tension, a bolt shall satisfy

$$\frac{V^*_{sf}}{\phi V_{sf}} + \frac{N^*_{tf}}{\phi N_{tf}} \leqslant 1.0. \tag{6.17}$$

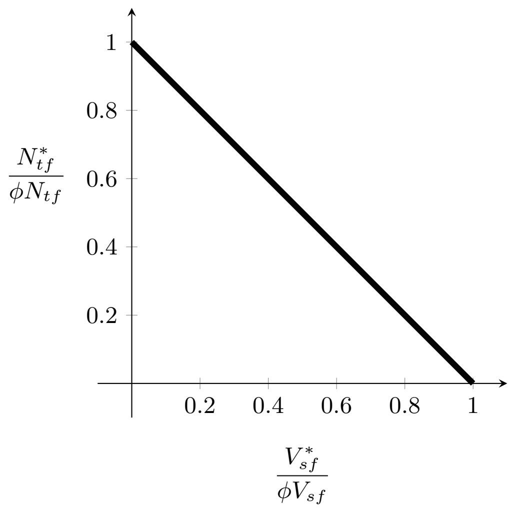


Figure 6.31: Envelop of tension versus shear

Example 6.2 *Redesign the connection in the previous example as a slip–critical (or friction type) connection without special treatment of the steel given that the dead load force is one quarter of the live load force.*

Solution 6.2

Knowing $V_L = 4V_D$,

$$\begin{aligned} V^* &= 1.2V_D + 1.5V_L \\ &= 1.8V_L = 320\,\text{kN}, \\ V_L &= 177.8\,\text{kN}. \end{aligned}$$

Then the serviceability combination is

$$V^*_{sf} = 1.2V_D + 0.4V_L = 124.4\,\text{kN}.$$

Determine the number of bolts. The capacity per bolt is

$$\begin{aligned} \phi V_{sf} &= \phi \mu_s n_{ei} N_{ti} k_h \\ &= 0.7 \times 2 \times 0.35 \times 1 \times 95\,\text{kN} \\ &= 46.6\,\text{kN}. \end{aligned}$$

This leads to

$$n \geqslant \frac{V^*_{sf}}{\phi V_{sf}} = \frac{124.4\,\text{kN}}{46.6\,\text{kN}} = 2.67.$$

Thus use 3 M16/N 8.8/TB bolts as before.

All other checks for strength are the same as before. The maximum resistance of the 3 bolt connection is $3 \times 46.6\,\mathrm{kN} = 139.8\,\mathrm{kN}$.

Welded Connections

7.1 Basic Types of Welded Joint

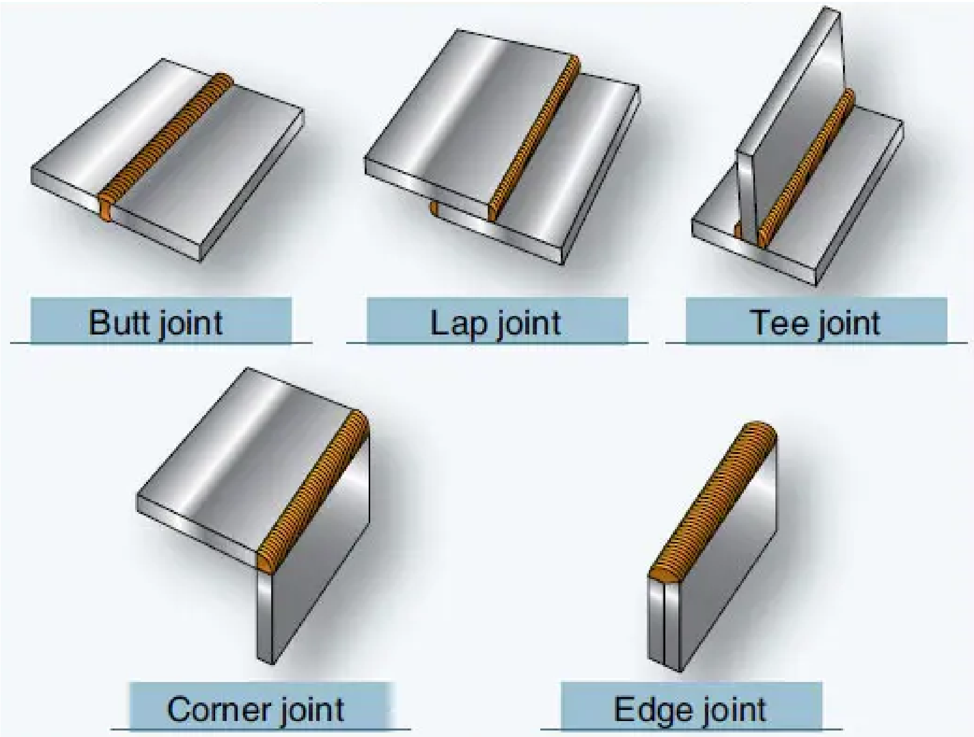


Figure 7.1: Basic types of welded joints (https://www.flight-mechanic.com/welded-joints-using-oxy-acetylene-torch/)

Lapped joints are the most common. Here are some examples.

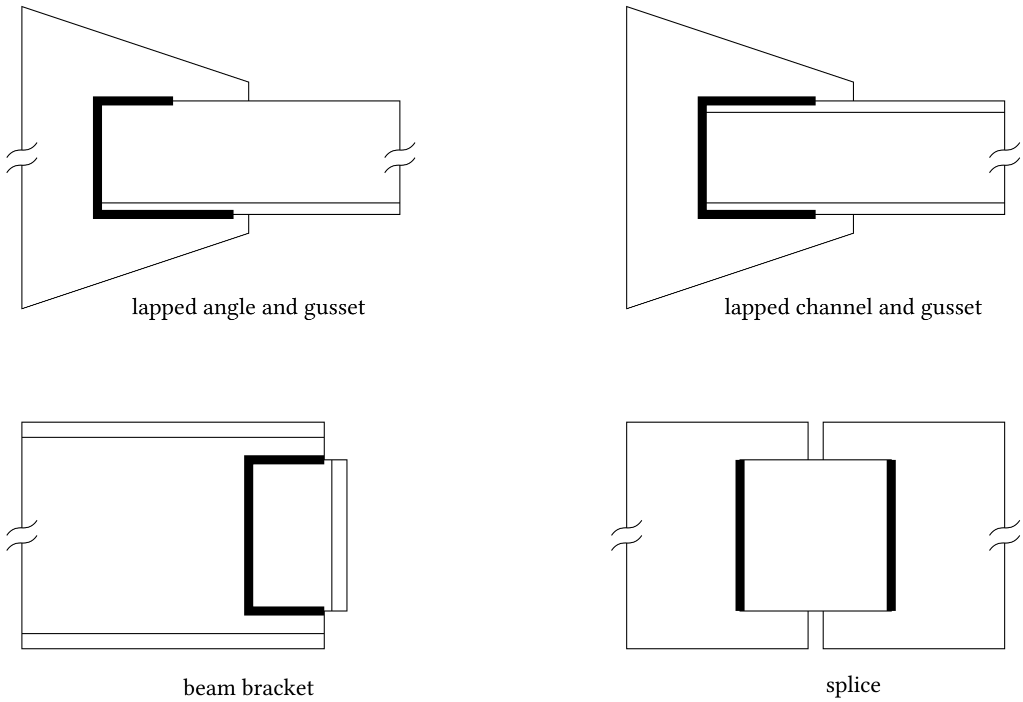


Figure 7.2: Examples of lapped joints

7.2 Weld Categories and Types

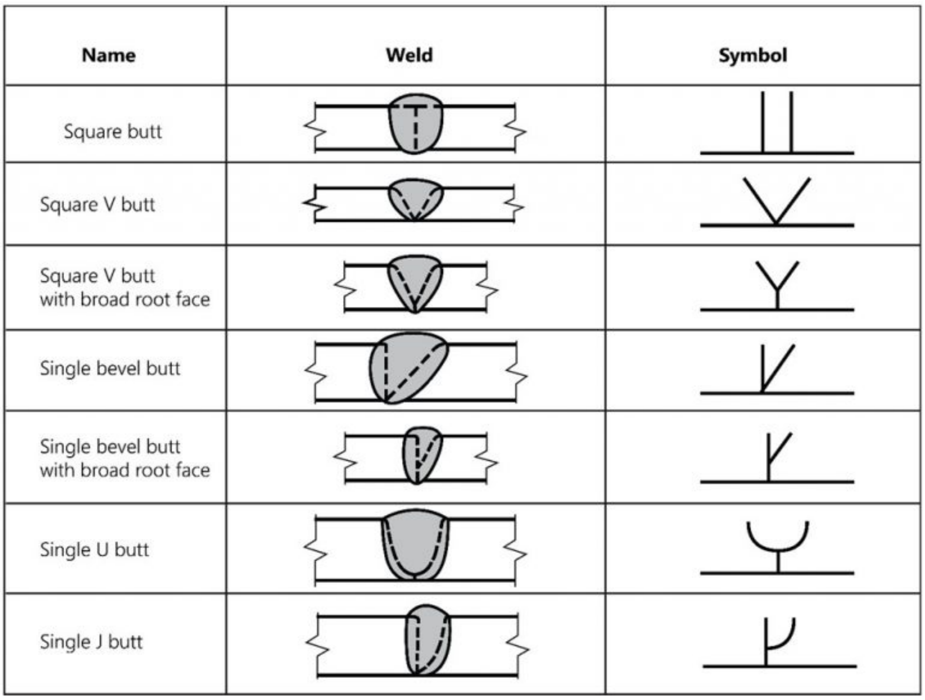
NZS 3404.1&2:1997 Steel structures standard permits the use of two weld categories.

- GP — General Purpose (for design $\phi = 0.6$)
  For general welds where demand is less than the weld capacity.
- SP — Structural Purpose (for design $\phi = 0.8$ but $\phi = 0.9$ for butt welds)
  For welds:
  - where demand is greater than GP weld capacity;
  - subject to high cycle fatigue loading; or
  - in main framing subjecting to earthquake loading.

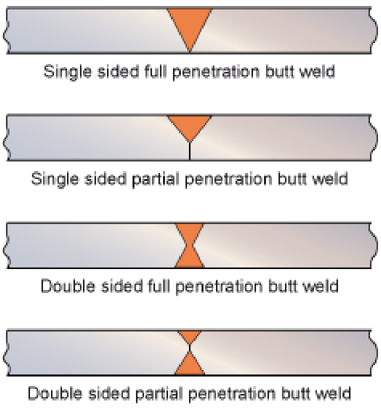
GP welds allow larger imperfections than SP welds do.

NZS 3404.1&2:1997 Steel structures standard deals with six types of welds as shown below.

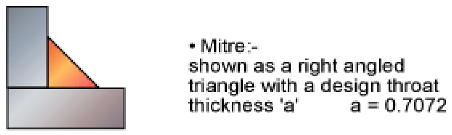
- Complete penetration butt/groove weld

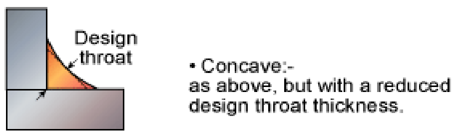


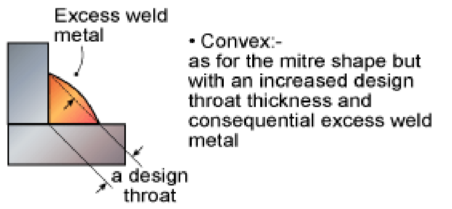
- Incomplete penetration butt/groove weld (not at a corner or T-joint)



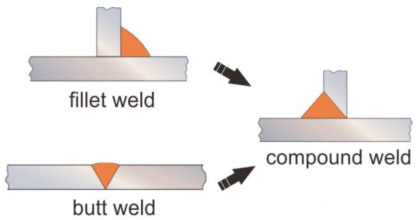
- Fillet weld



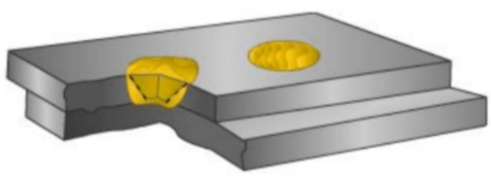




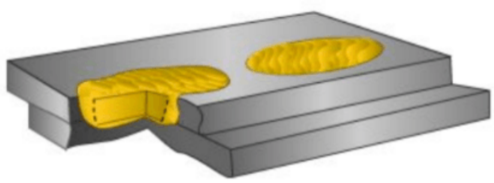
- Compound weld



- Plug weld



- Slot weld



7.3 Weld Process

Welding is **a materials joining process which produces coalescence of materials by heating the to suitable temperatures**.

Heat is used to melt the base material and a filler material in order that flow of material will occur and that fusion will take place.

Several welding processes are available. The most common are

- Shield Metal Arc Welding (SMAW)

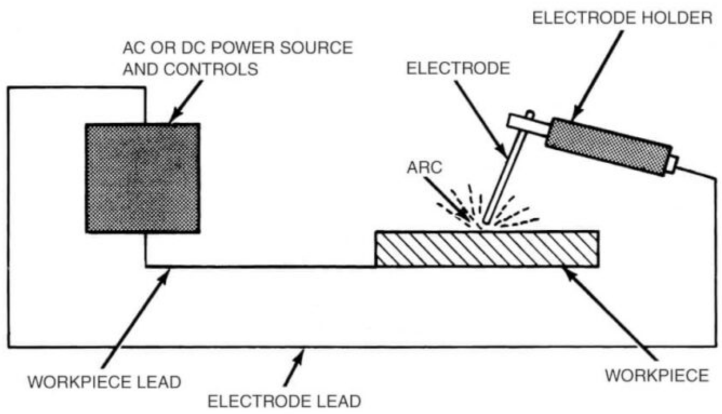


Figure 7.3: Shield metal arc welding (https://www.fabtechexpo.com/blog/2018/01/04/shielded-metal-arc-welding-basics)

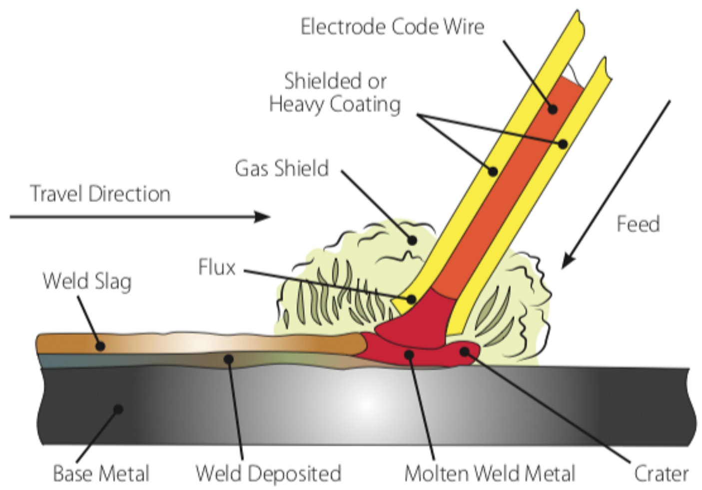


Figure 7.4: Shield metal arc welding (https://www.jasic.co.uk/guide-to-mma-welding)

This includes

- Manual Metal Arc Welding (MMAW)
  Welding with a stick electrode with flux coating. The shielding may perform the following functions.
  - Produce a gaseous shield to exclude air and stabilize the arc.
  - Introduce other materials, e.g., deoxidizers to refine grain structure of weld metal.
  - Produce a slag blanket to protect it from air and retard cooling.

  It is slow (expensive) but versatile.

– Gas Metal Arc Welding (GMAW)
  Welding with a steel wire fed through a gun with gas (e.g., CO_2, Ar, O_2) shielding.
  It is faster but wind dependent — not for field welding.
– Flux Cored Arc Welding (FCAW)
  Welding with a hollow steel wire filled with flux fed through a gun — sometimes inert shielding as is used too.
  It is fast but needs good access.

- Submerged Arc Welding (SAW)

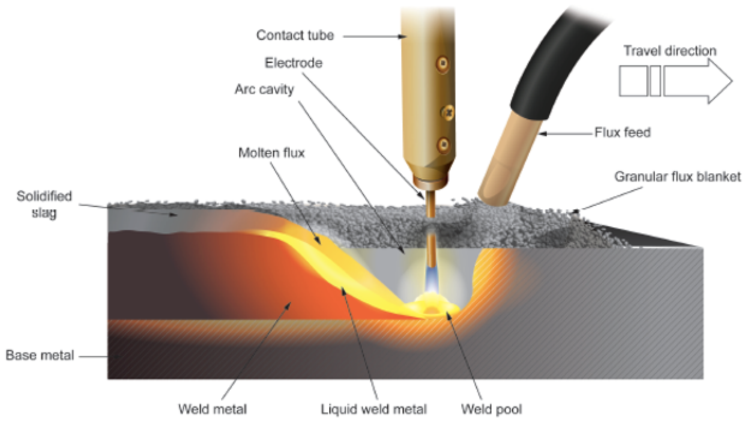


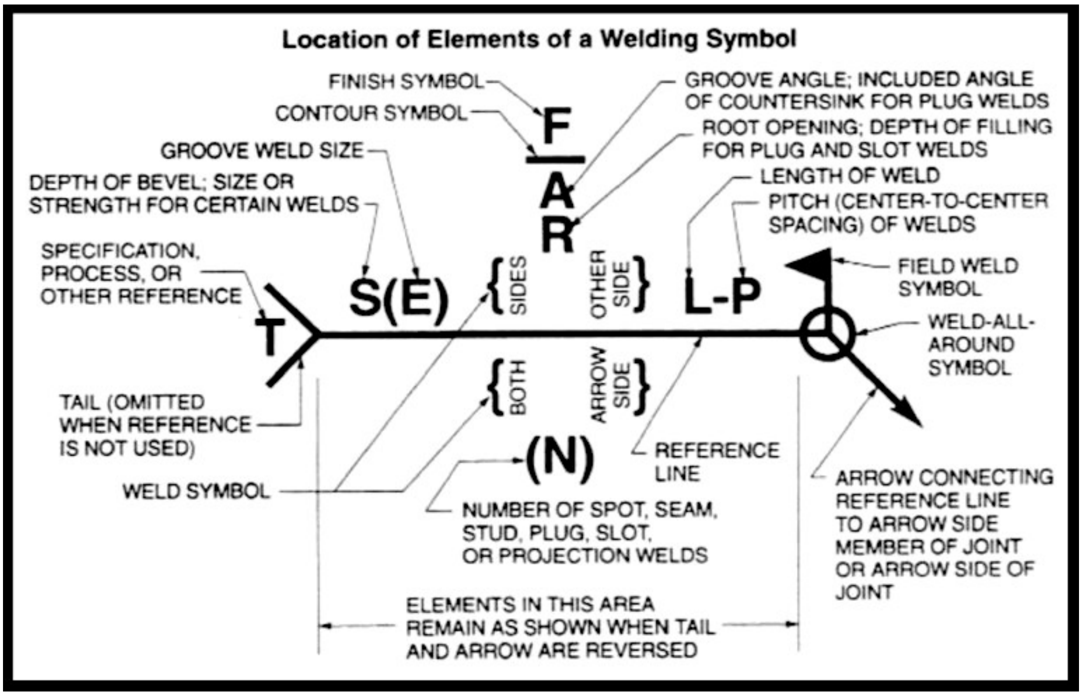
Figure 7.5: Submerged arc welding (https://www.cwbgroup.org/sites/default/files/imgs/saw-fig1.png)

The flux is the special feature of this method. The granular flux is usually laid out automatically along the seam ahead of the advancing weld. It

– provides a cover which allows the weld to be made without splatter, sparks or smoke;
– protects the weld from the atmosphere; and
– produces better welds of higher consistent quality.

SAW is usually used in welding shops (not at the job site) with automatic equipment and long runs of weld.

7.4 Standard Weld Symbols



Some additional notes shall be explained.

- The letters CP in the tail of the arrow indicate a complete penetration butt weld.
- The tail should be omitted if no reference T is required.
- The size of a fillet weld shall be to the left of the symbol. The vertical line on the fillet weld symbol is **always** on the left.
- For an incomplete penetration butt weld, the design throat thickness hsall be to the left of the symbol. Where no design throat thickness is shown, a complete penetration butt weld is assumed required.
- Arrow side and other side welds are made the same size unless otherwise dimensioned.
- Symbols only apply between abrupt changes in direction of welding unless governed by the ‘weld all round’ symbol or otherwise dimensioned.

Weld symbols are summarized in the following table.

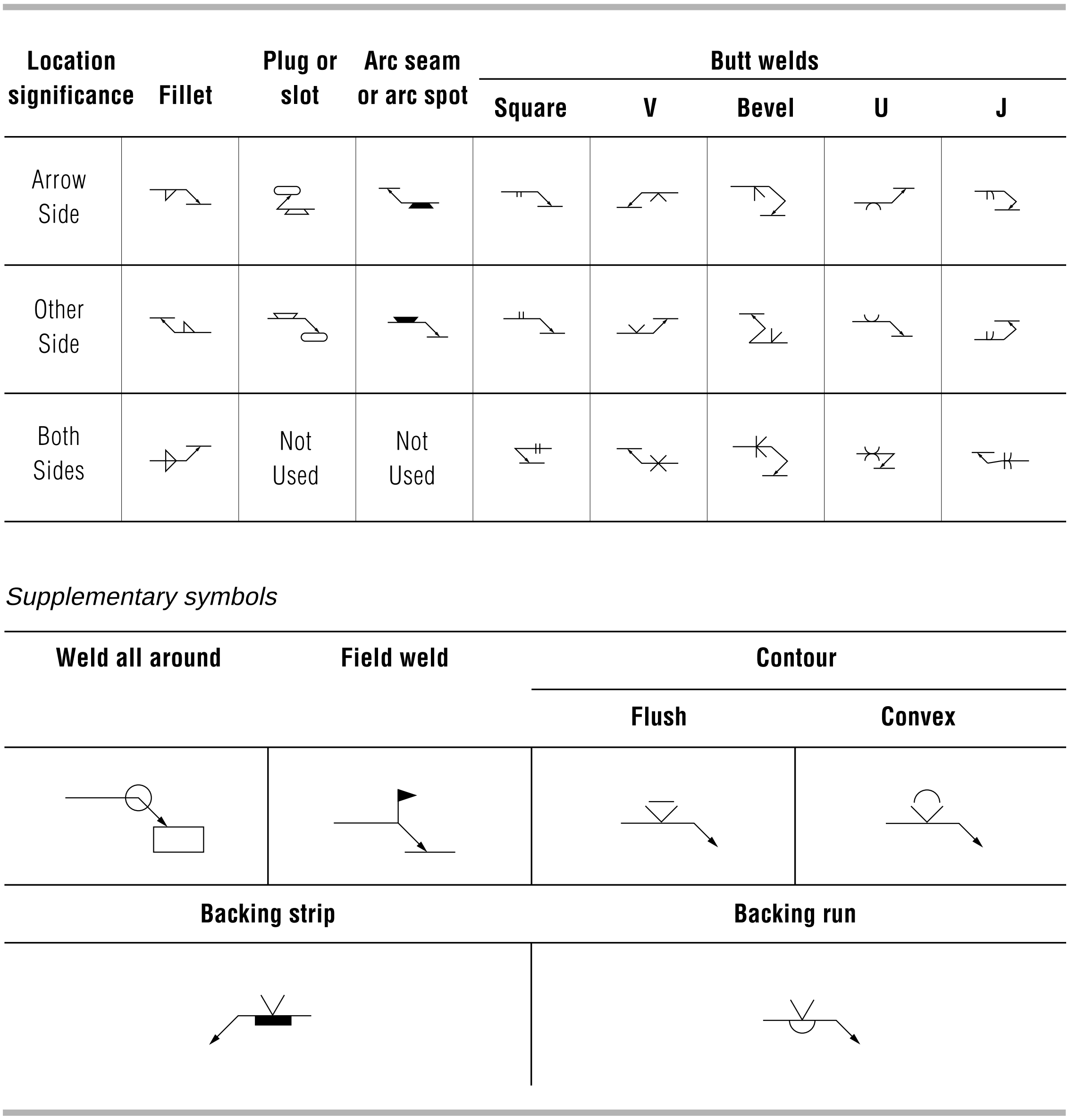


Figure 7.6: Welding symbols (Gorenc et al., 2015)

The following are some examples of welding symbols taken from (Corgan, 2017) on drawings and their physical meanings. All units are inches.

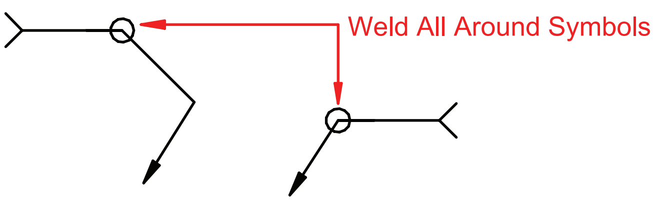


Figure 7.8: Weld all around welds (Corgan, 2017)

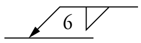
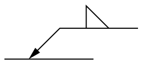
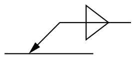
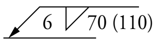
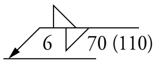
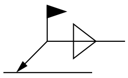
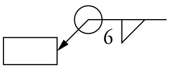
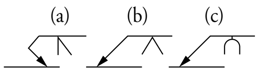
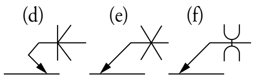
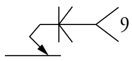
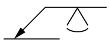
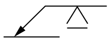
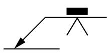
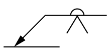
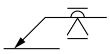
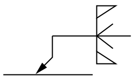
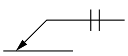
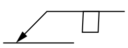
No.	Symbols	Description of weld
1		Continuous, one-sided fillet weld of 6 mm leg size along the length of the line indicated by arrow. Fillet weld is on the arrow side of the joint.
2		Same as for 1, but the weld is on the side opposite to where the arrow points.
3		Continuous, double-sided fillet weld.
4		Intermittent 6 mm fillet weld having incremental lengths of 70 mm spaced at 180 mm. Arrow side only.
5		Staggered intermittent weld as for 4 (both sides).
6		As for 3, but the flag indicates that this weld is to be done in the field.
7		As for 1, but instead of being applied along a line, this weld is to be carried out all around, and this is indicated by a small circle.
8		Butt welds: (a) single bevel; (b) single vee; (c) single U.
9		Butt welds: (d) double bevel; (e) double vee; (f) double U.
10		Same as 9(d) but a special procedure is to be used as specified under item 9 of the procedure sheet.
11		Same as 8(b) but weld is to have a convex contour.
12		Same as 8(b) but weld is to have a flush contour obtained by grinding.
13		Same as 8(b) but a backing strip is to be used.
14		Same as 8(b) but the root of the weld is to be gouged and a backing weld run applied.
15		Same as 14, but both faces are to be ground flush.
16		Double-bevel butt weld reinforced with fillet welds for a better stress dispersion.
17		Square butt weld. No grooves are prepared for this weld (suitable only for thin plates).
18		Plug weld. Weld is on arrow side of the joint.

Figure 7.7: Examples of use of welding symbols (Gorenc et al., 2015)

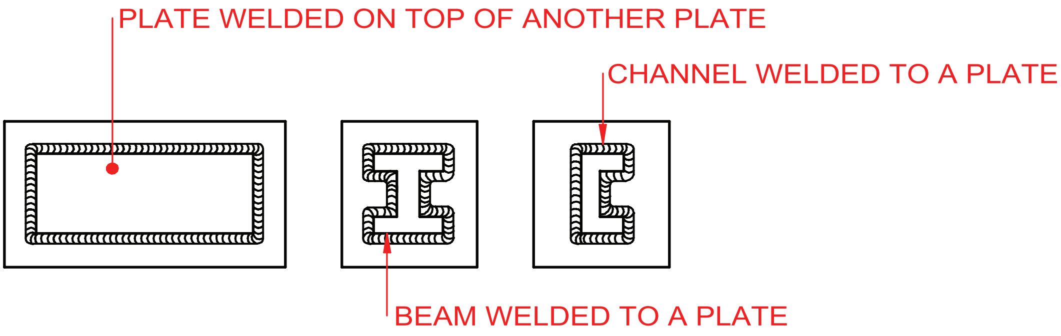


Figure 7.9: Examples of weld all around welds (Corgan, 2017)

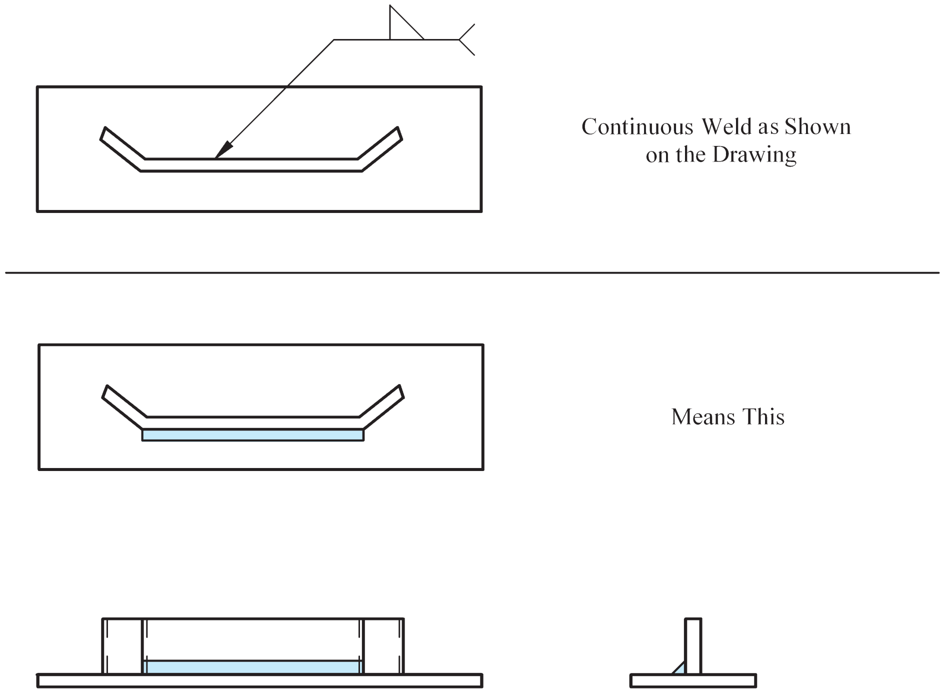


Figure 7.10: Examples of continuous welds (Corgan, 2017)

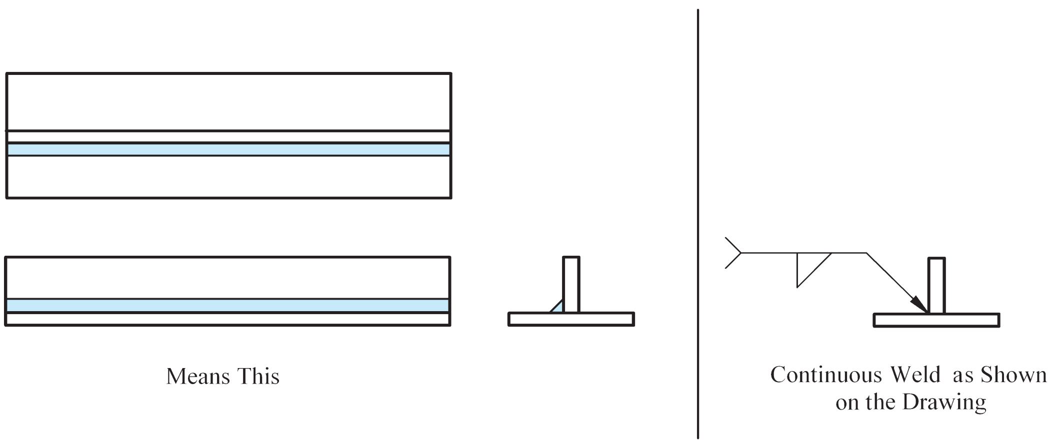


Figure 7.11: Examples of continuous welds (Corgan, 2017)

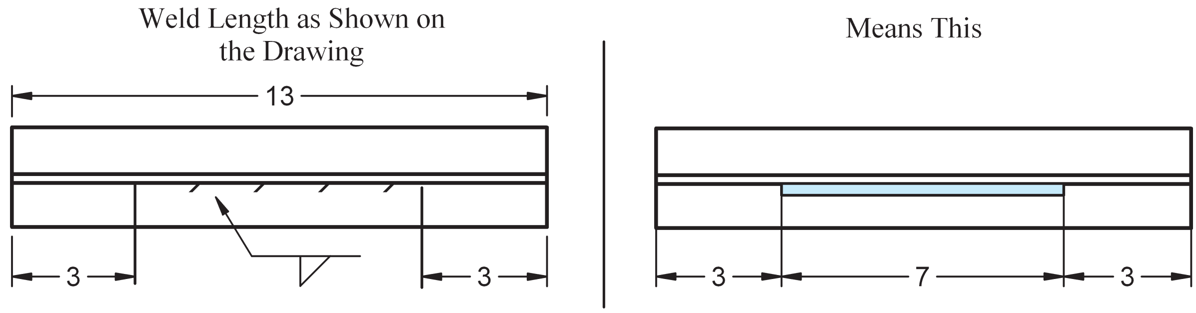


Figure 7.12: Weld length specified on welding symbol between extension lines (unit: inch) (Corgan, 2017)

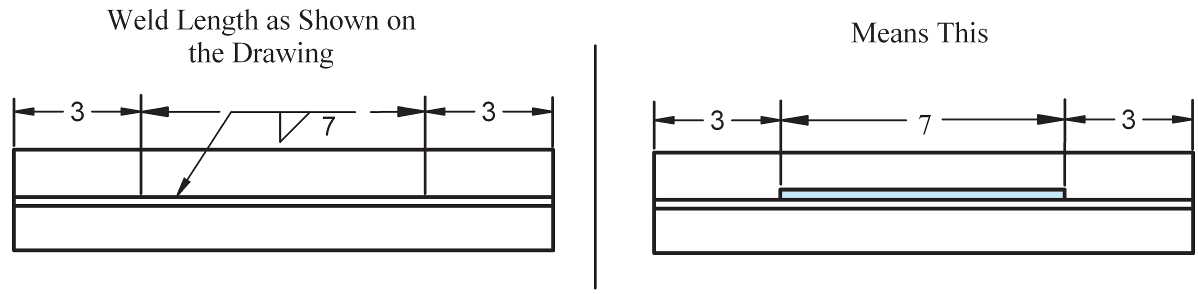


Figure 7.13: Weld length specified on welding symbol between extension lines with section lines representing the weld area (unit: inch) (Corgan, 2017)

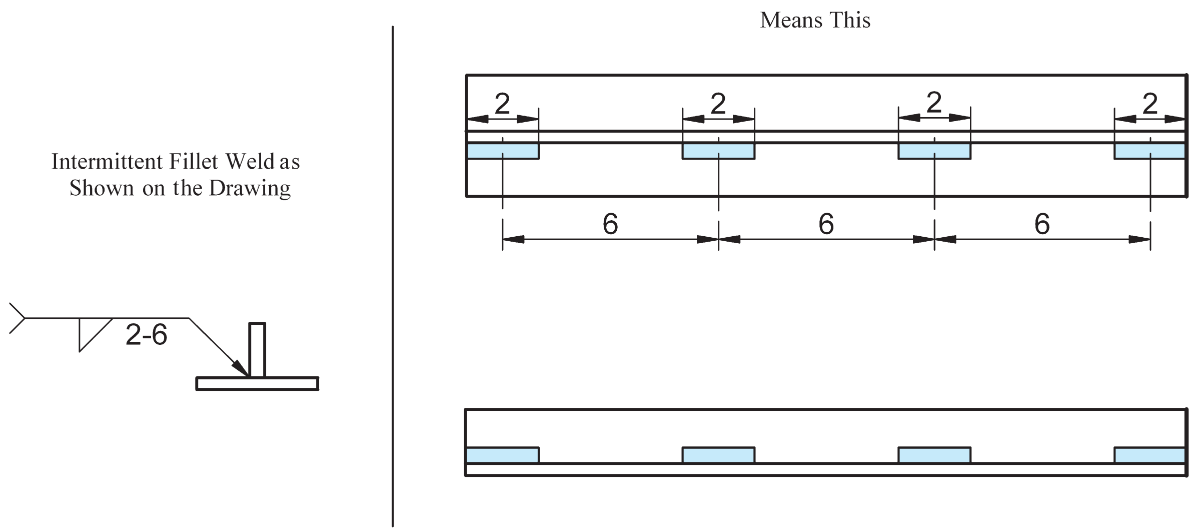


Figure 7.14: Example of intermittent welds (unit: inch) (Corgan, 2017)

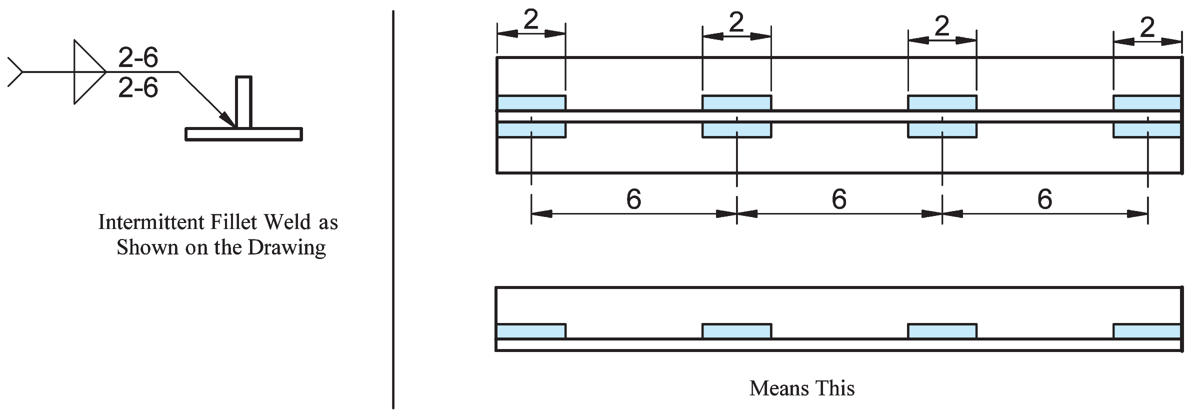


Figure 7.15: Example of chain intermittent welds (unit: inch) (Corgan, 2017)

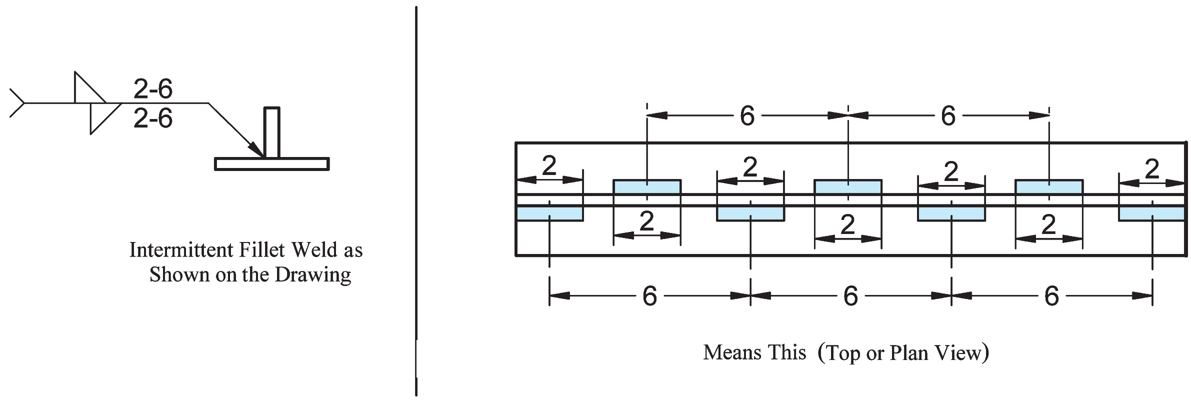


Figure 7.16: Example of staggered intermittent welds (unit: inch) (Corgan, 2017)

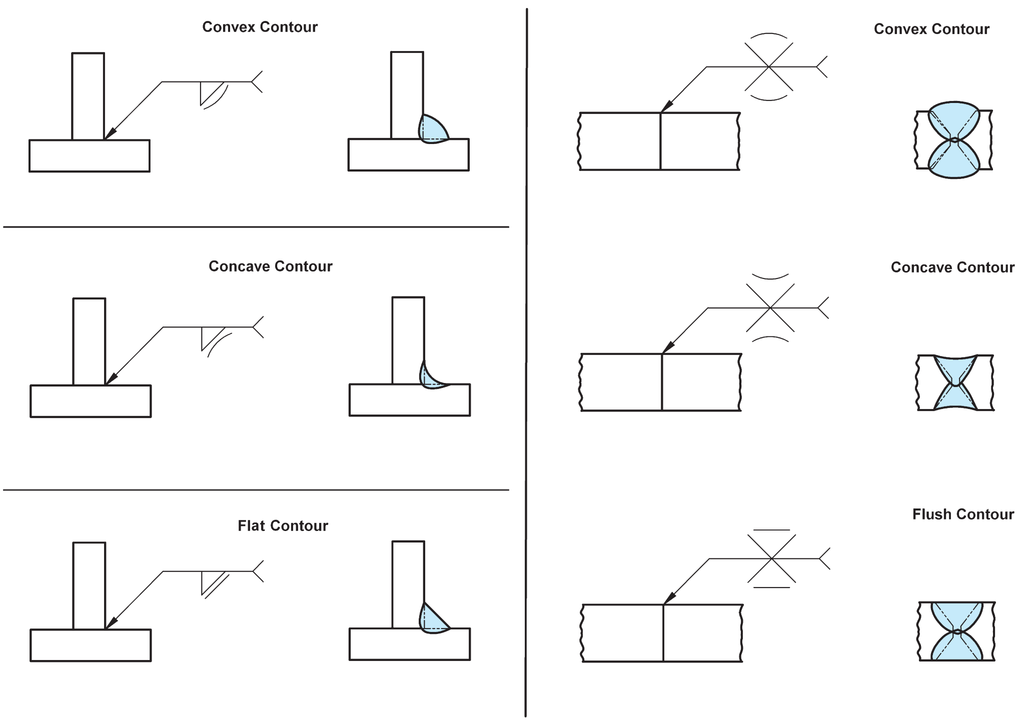


Figure 7.17: Contour symbols (Corgan, 2017)

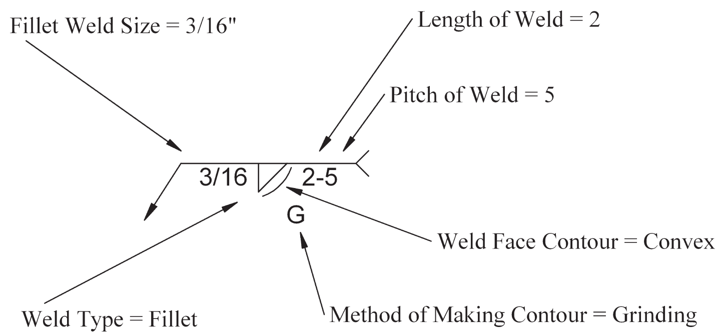


Figure 7.18: Example of a welding symbol for a fillet weld (unit: inch) (Corgan, 2017)

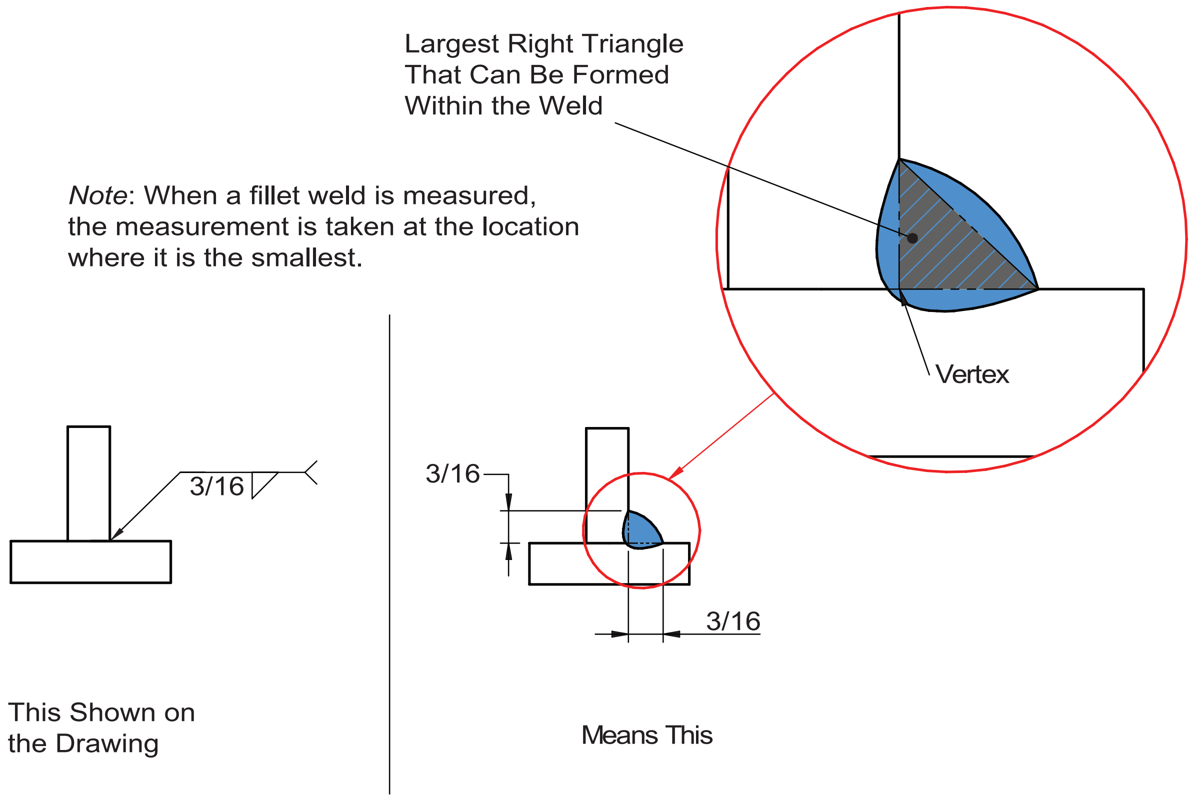


Figure 7.19: Fillet weld size (unit: inch) (Corgan, 2017)

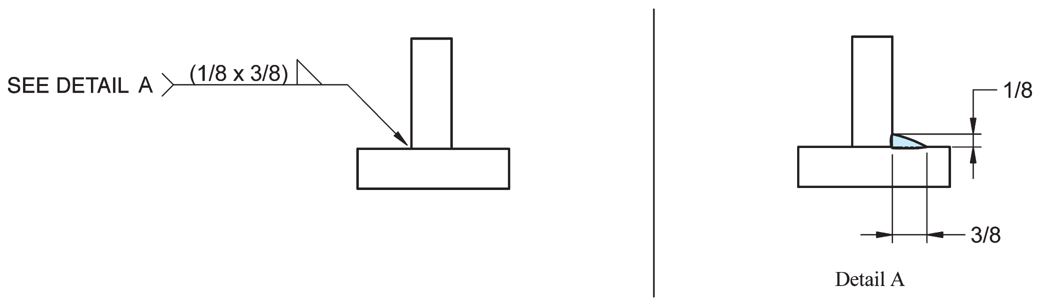


Figure 7.20: Unequal leg fillet with detail drawing (unit: inch) (Corgan, 2017)

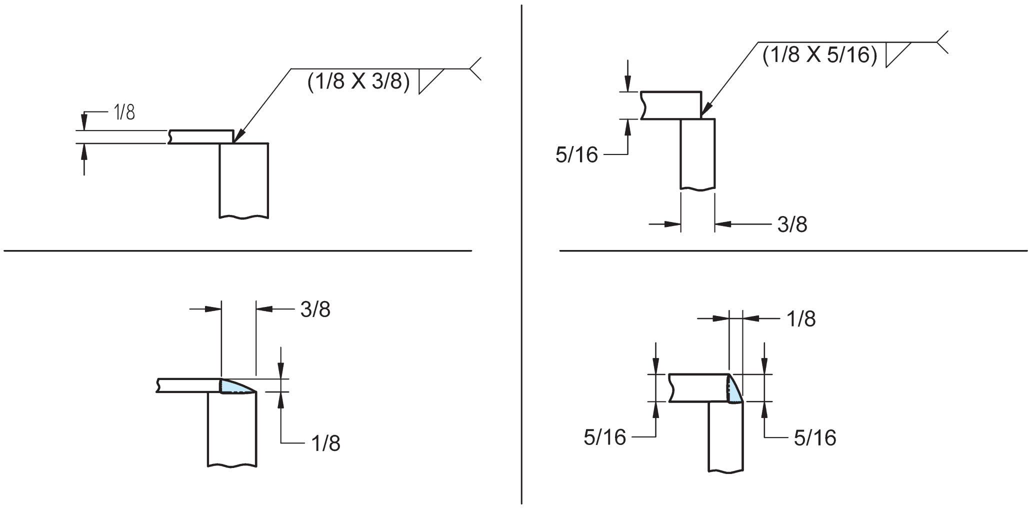


Figure 7.21: Unequal leg fillets that could be shown without detail (unit: inch) (Corgan, 2017)

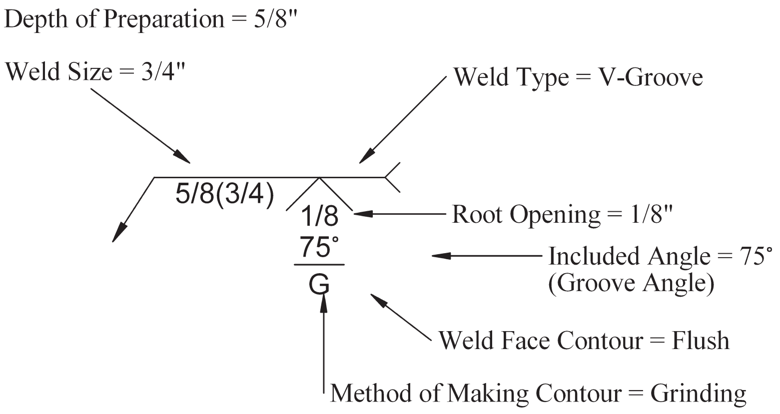


Figure 7.22: Welding symbol example for a groove weld (unit: inch) (Corgan, 2017)

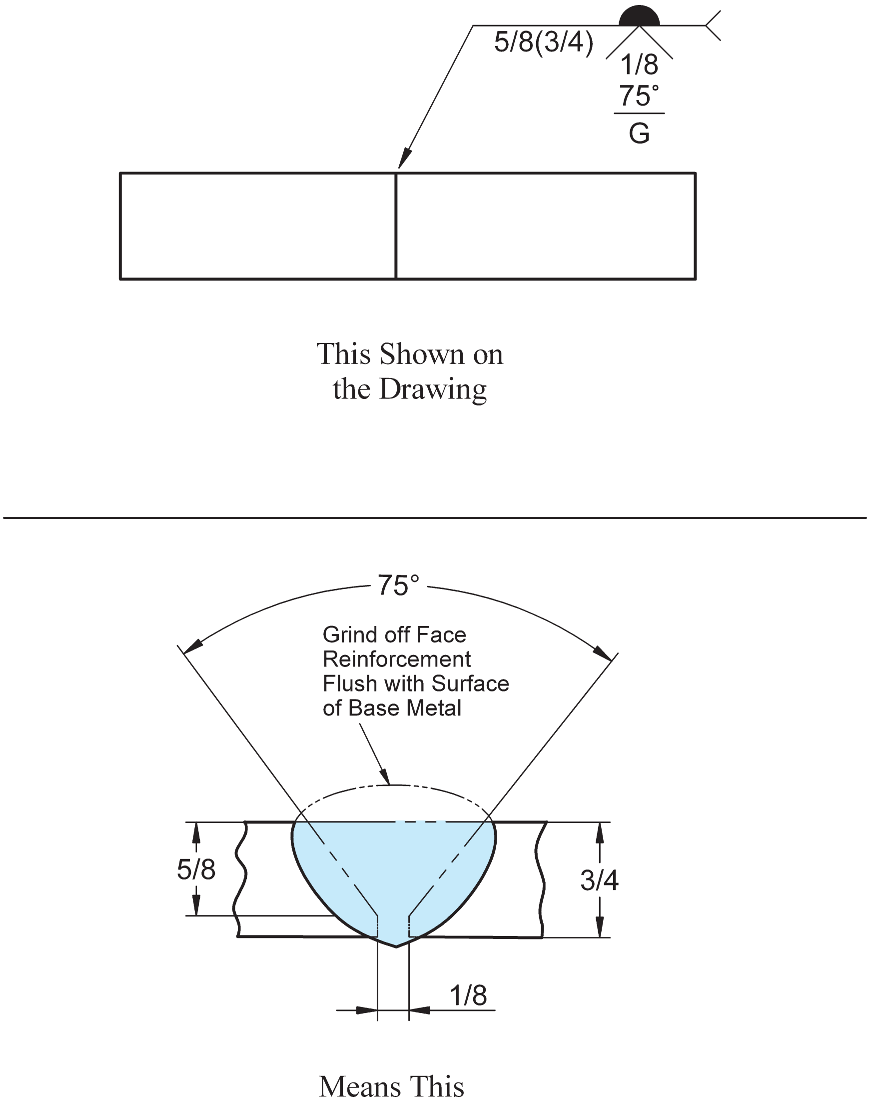


Figure 7.23: Example of a groove/butt weld (unit: inch) (Corgan, 2017)

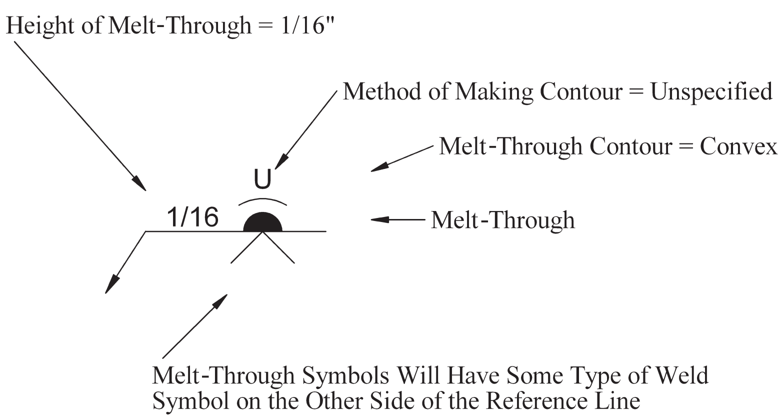


Figure 7.24: Welding symbol with melt–through (unit: inch) (Corgan, 2017)

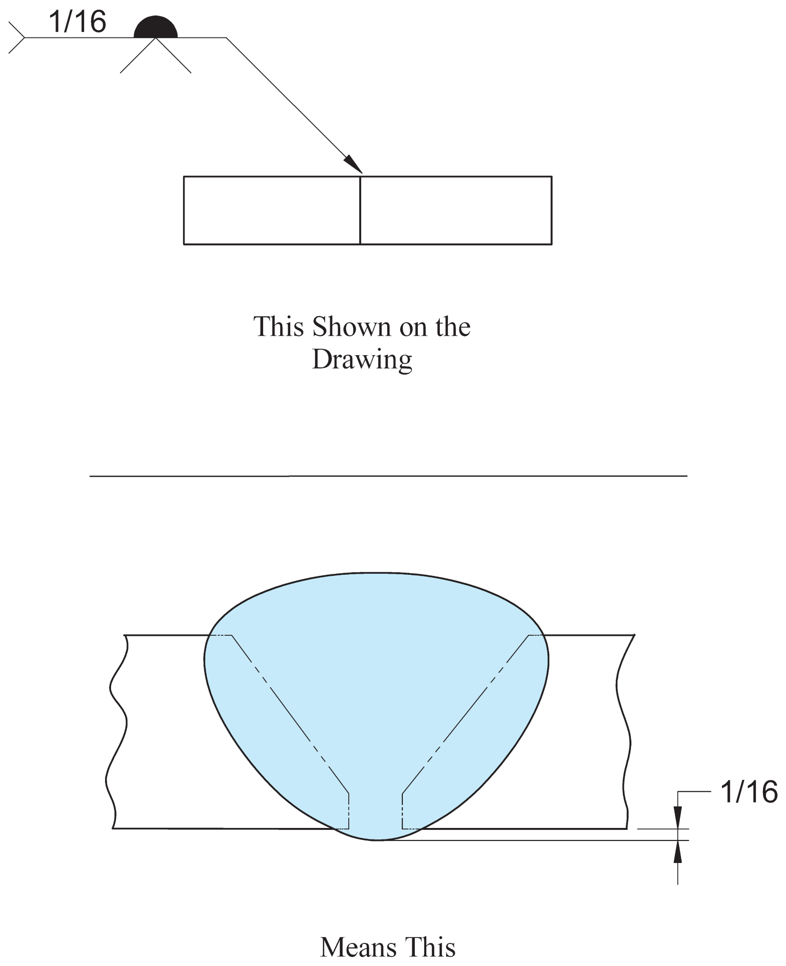


Figure 7.25: Melt–through example (unit: inch) (Corgan, 2017)

7.5 Electrodes Used for Welding

Table 7.1: Nominal tensile strength of weld metal (f_{uw})

MMAW	SAW, FCAW, GMAW	f_{uw}
E41XX	W40X	410 MPa
E48XX	W50X	480 MPa

The values of 'X' represent the usage, or they are related to the Charpy impact test. All of these welds may be used with Grade 250 to Grade 350 steel. This system is used in this book.

A new system of weld specification is available according to AS/NZS 4855:2007 Welding consumables — Covered electrodes for manual metal arc welding of non-alloy and fine grain steels — Classification.

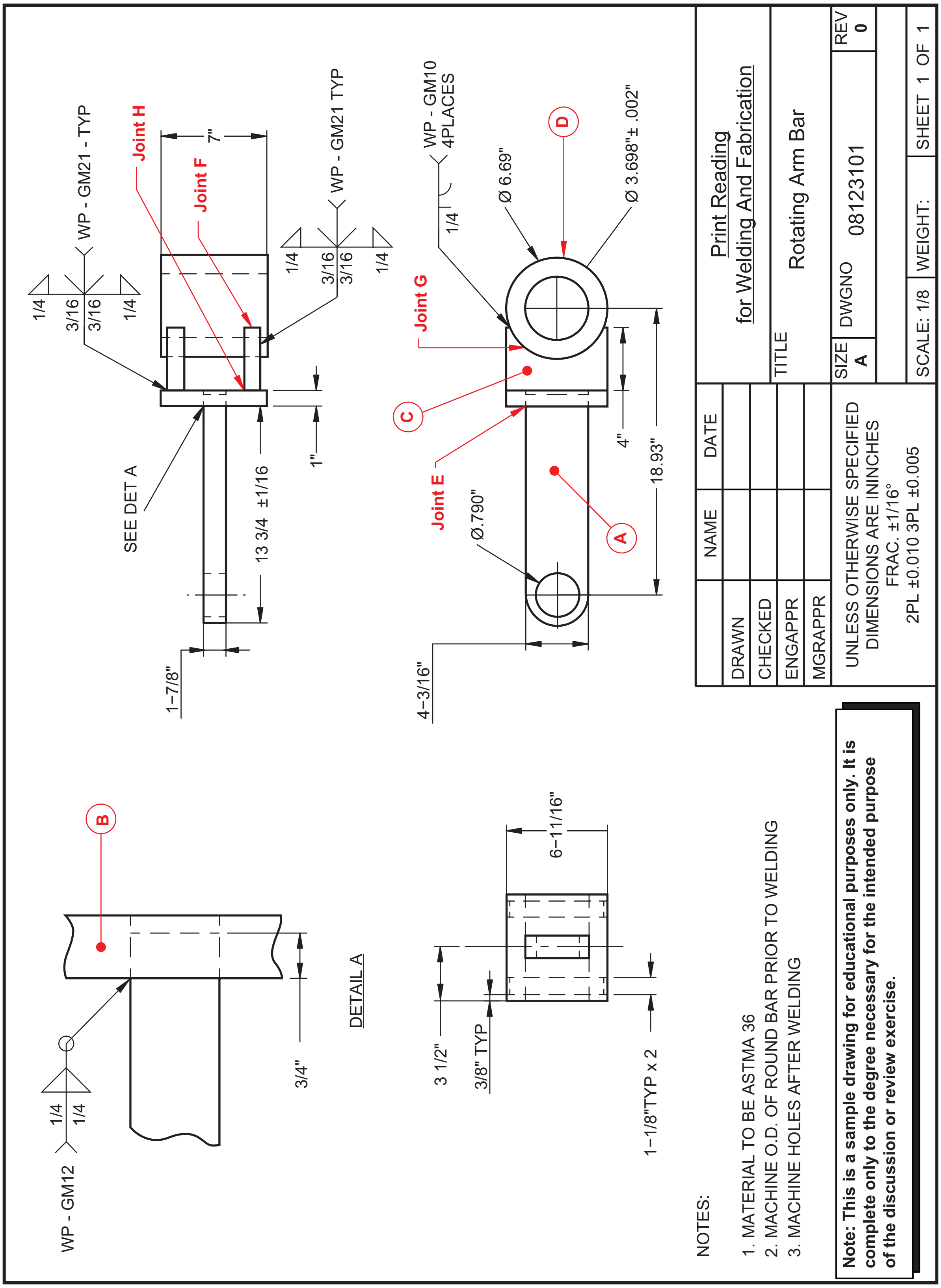

WP - GM21 - TYP
1/4
3/16
3/16
1/4
Joint H
Joint F
7"
WP - GM21 TYP
1/4
3/16
3/16
1/4
WP - GM10
4PLACES
1/4
Ø 6.69"
D
Ø 3.698"± .002"
SEE DET A
13 3/4 ±1/16
1"
Joint G
C
Joint E
Ø.790"
A
4"
18.93"
1–7/8"
4–3/16"
B
WP - GM12
1/4
1/4
3/4"
DETAIL A
6–11/16"
3 1/2"
3/8" TYP
1–1/8"TYP x 2
NOTES:
1. MATERIAL TO BE ASTMA 36
2. MACHINE O.D. OF ROUND BAR PRIOR TO WELDING
3. MACHINE HOLES AFTER WELDING
Note: This is a sample drawing for educational purposes only. It is complete only to the degree necessary for the intended purpose of the discussion or review exercise.
Print Reading for Welding And Fabrication
TITLE
Rotating Arm Bar
NAME
DATE
DRAWN
CHECKED
ENGAPPR
MGRAPPR
UNLESS OTHERWISE SPECIFIED DIMENSIONS ARE ININCHES
FRAC. ±1/16°
2PL ±0.010 3PL ±0.005
SIZE
A
DWGNO
08123101
REV
0
SCALE: 1/8
WEIGHT:
SHEET 1 OF 1

Table 7.2: Nominal tensile strength of weld metal (f_{uw})

	Symbol	f_{uw}
System A	A-E35	440 MPa
	A-E38	470 MPa
	A-E42	500 MPa
	A-E46	530 MPa
	A-E50	560 MPa
System B	B-E43	430 MPa
	B-E49	490 MPa
	B-E55	550 MPa
	B-E57	570 MPa

System A is used in Europe (based on yield stress), while System B is based on ultimate stress and is more common in Australasia.

The industry standard welding strength is $f_{uw} = 490\,\text{MPa}$, so this should be specified in practice (where possible).

7.6 Weld Size and Strength

- **Butt Weld**
  Complete penetration butt weld should be the same thickness as the material they are connecting. By using an electrode of sufficient strength ($f_{uw} \geqslant f_u$) and an SP weld, the effect of the weld can be ignored since the weld strength does not limit the member strength.
- **Fillet Weld**
  - **Preferred Sizes**
    Preferred sizes of a fillet weld with t_w less than 15 mm are 2 mm, 4 mm, 5 mm, 6 mm, 8 mm, 10 mm and 12 mm (NZS 3404.1&2:1997 § 9.7.3.2).
  - **Minimum Length**
    The minimum length l_w is $4t_w$ to ensure good fusion.
  - **Minimum Size**

Table 7.3: Minimum size of fillet weld t_w

Thickness of thickest part joined t (mm)	t_w (mm)
$t \leqslant 7$	3
$7 < t \leqslant 10$	4
$10 < t \leqslant 15$	5
$15 < t$	6

  - **Maximum Size**

Table 7.4: Maximum size of fillet weld t_w

Thickness of material alongside which fillet weld is to be made t (mm)	t_w (mm)
$t < 6$	t
$6 \leqslant t$	$t - 1$

This is necessary to a) prevent yielding of base material and b) indicate actual throat thickness.

The **weld strength per unit length** shall satisfy (NZS 3404.1&2:1997 § 9.7.3.10)

$$v_w^* \leqslant \phi v_w = \phi 0.6 f_{uw} t_t k_r, \tag{7.1}$$

where

$$\begin{aligned}
\phi &= \text{strength reduction factor} \\
&= 0.8 \text{ for SP} \\
&= 0.6 \text{ for GP} \\
v_w^* &= \text{vectorial summation of design force per unit length on weld effective area} \\
v_w &= \text{nominal capacity of a fillet weld per unit length} \\
f_{uw} &= \text{nominal tensile strength of weld material} \\
0.6 f_{uw} &= \text{nominal shear strength of weld material} \\
t_t &= \text{design throat thickness, usually equal to } t_w/\sqrt{2} \text{ where } t_w \text{ is the leg length, or fillet weld size} \\
k_r &= \text{reduction factor for welded lap connection (otherwise } k_r = 1.0) \\
&= 1.00 \text{ if } l_w \leqslant 1.7\,\mathrm{m} \\
&= 1.10 - 0.06 L_w \text{ if } 1.7\,\mathrm{m} < l_w \leqslant 8.0\,\mathrm{m} \\
&= 0.62 \text{ if } l_w > 8.0\,\mathrm{m}
\end{aligned}$$

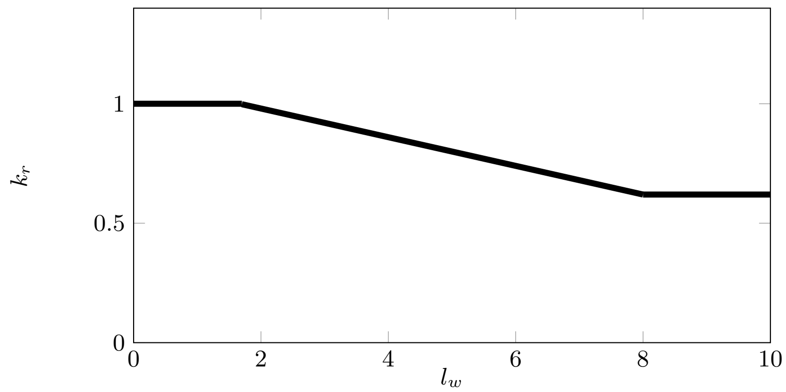


Figure 7.26: k_r as a function of l_w

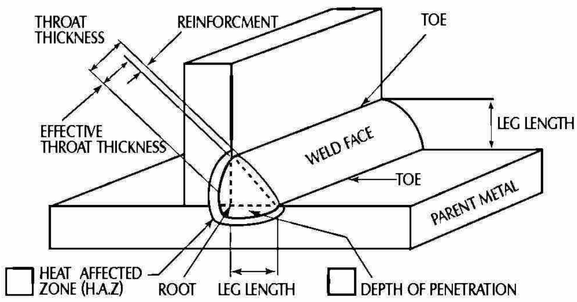


Figure 7.27: Weld terminology (`http://mdme.atspace.com/modules/7759G_Mechanical_Design/welds/Welded_Joints.html`)

Fillet welds can be loaded with components of force in shear ($\tau_y \approx \sigma_y/\sqrt{3}$) and tension/compression (σ_y). Design conservatively considers that the strength is based on $\tau_y \approx 0.6\sigma_y$ for all components of loading.

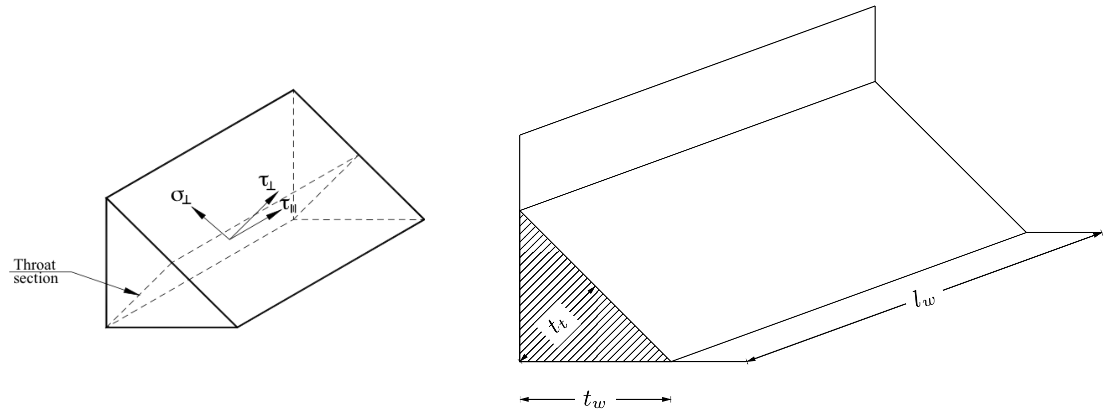


Figure 7.28: Weld stress components (https://offshorestructures.wordpress.com/2014/11/02/welded-lap-joints/)

The vectorial summation of design force per unit length can be associated with the stress components shown as follows. This is considered in some other standards with the strengths in each component direction (e.g., shear or tension).

$$v_w^* = t_t\sqrt{\sigma_\perp^2 + \tau_\perp^2 + \tau_\parallel^2} \tag{7.2}$$

Table 7.5: Dependable capacities of equal leg fillet welds ϕv_w (kN mm^{-1}) with $k_r = 1.0$

t_w	GP Welds			SP Welds		
(mm)	E41XX/W40X	E48XX/W50X	E49XX	E41XX/W40X	E48XX/W50X	E49XX
3	0.313	0.367	0.374	0.417	0.489	0.499
4	0.417	0.489	0.499	0.557	0.652	0.665
5	0.522	0.611	0.624	0.696	0.815	0.832
6	0.626	0.733	0.748	0.835	0.978	0.998
8	0.835	0.978	0.998	1.113	1.303	1.330
10	1.044	1.222	1.247	1.392	1.629	1.663
12	1.252	1.466	1.497	1.670	1.955	1.996

For example, for $t_w = 3\,\text{mm}$, SP with E41XX electrodes, the dependable strength per unit length of weld is

$$\begin{aligned}\phi v_w &= 0.8 \times 0.6 \times 410\,\text{MPa} \times \frac{3\,\text{mm}}{\sqrt{2}} \times 1.0 \\ &= 0.417\,\text{kN}\,\text{mm}^{-1}.\end{aligned}$$

When using SP longitudinal fillet welds to RHS with $t < 3\,\text{mm}$, SP capacities should be multiplied by $0.7/0.8$ to account for the different ϕ factors.

Welds greater than $8\,\text{mm}$ require multiple passes and tend to be less economical.

7.7 Fillet Weld Root Gaps

Where there is a separation/gap between plates (i.e., root gaps), the fillet weld size, t_w, is given by the inscribed triangle (which does not include the root gap) as shown (NZS 3404.1&2:1997 § 9.7.3.1 and AS/NZS 5131 § 7.5.8).

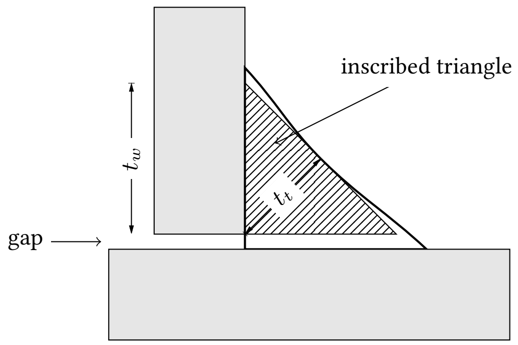


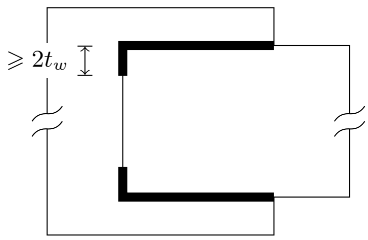
Figure 7.29: Fillet weld with root gap

Fabrication tolerances are given in AS/NZS 5131 § 7.5.8 Appendix F and NZS 3404.1&2:1997 § 14.4. Design/construction documentation should be clear as to how root gaps are considered (e.g., by the designer accounting for the tolerances with an increased t_w, or by the fabricator).

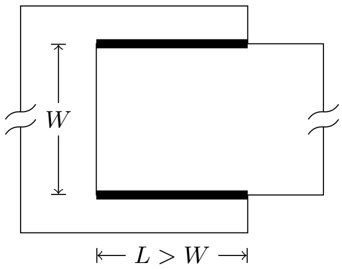
7.8 Good Practice for Welded Members

Although not shown in NZS 3404.1&2:1997 Steel structures standard, the following are some good practice for welded members.

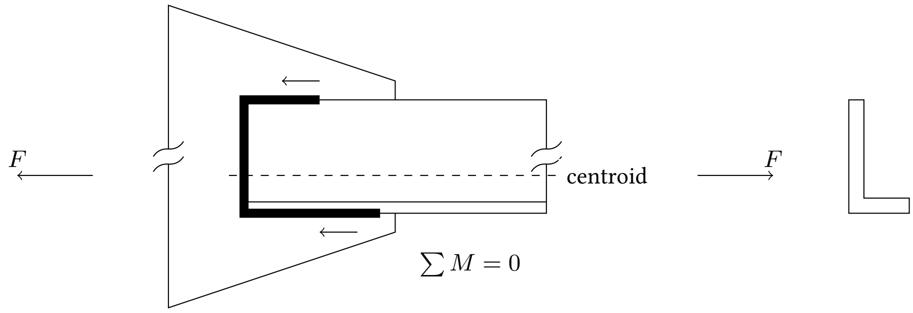
- It is good practice to use end returns. The effective length L_e includes returns.



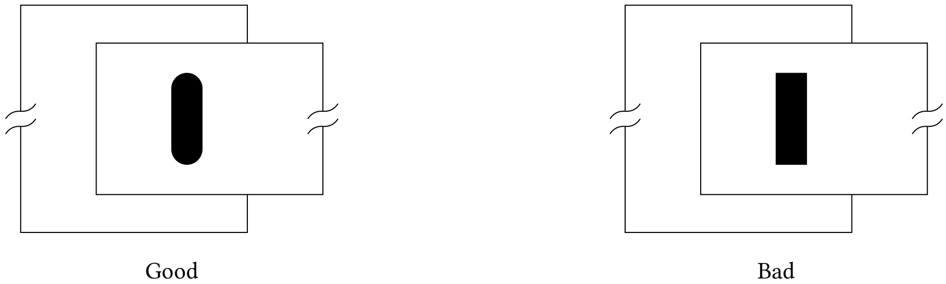
- For flat bars with only longitudinal fillet welds, make $L > W$, to minimise stress concentration.



- For weld placement in connections subject to repeated stress, center of weld resistance should coincide with member centroid unless special allowance is made for the eccentricity.



- For plug and slotted holes, make ends rounded to avoid stress concentration.



7.9 Member Design Considerations

7.9.1 Yielding on Gross Area

This has been previously studied, see Eq. (3.2).

$$N^* \leqslant \phi A_g f_y. \tag{7.3}$$

7.9.2 Fracture on Effective Net Area

This has been previously studied, see Eq. (3.3).

$$N^* \leqslant \phi 0.85 k_{te} A_n f_u. \tag{7.4}$$

7.9.3 Block Tearing Failure

Connections connected by welds should be checked for block tearing failure as for the bolted connections, see § 6.11.6.

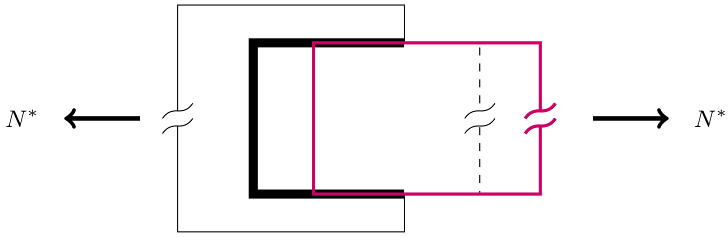


Figure 7.30: Block tear out of welded connection

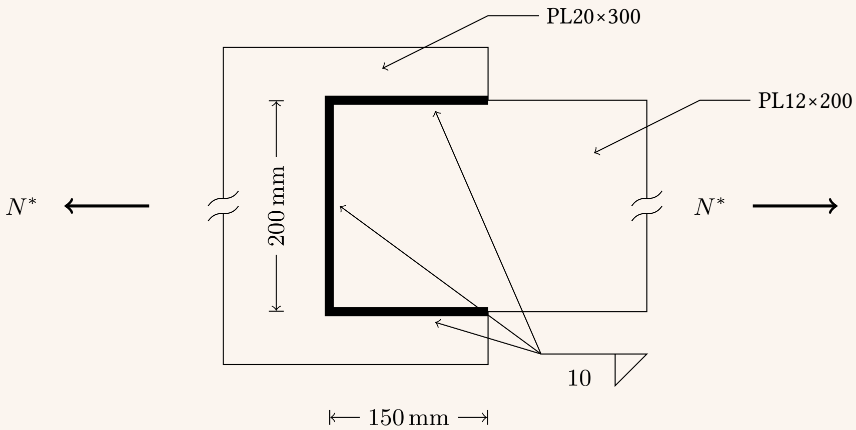
Since there is no need to distinguish between gross and net areas for welded connections, assuming the plate has a uniform thickness t_p, it is possible to denote $A_v = A_{gv} = A_{nv} = l_v t_p$ and $A_t = A_{gt} = A_{nt} = l_t t_p$, then Eq. (6.11) becomes

$$N^* \leqslant \phi 0.95\,(0.6 l_v + l_t)\, t_p f_u \tag{7.5}$$

ϕ = strength reduction factor, 0.9
l_v = weld length subject to shear
l_t = weld length subject to tension
t_p = plate thickness

Example 7.1 *Fillet Weld Analysis*

Find the strength of the connection shown if Grade 300 steel and E41XX electrodes are used for GP welds.



Solution 7.1

1. The effective throat thickness is

$$t_t = \frac{\sqrt{2}}{2} \times t_w = 7.07\,\text{mm}.$$

Weld capacity per unit length

$$\begin{aligned}\phi V_w &= \phi 0.6 f_{uw} t_t k_r \\ &= 0.6 \times 0.6 \times 410\,\text{MPa} \times 7.07\,\text{mm} \times 1 \\ &= 1.044\,\text{kN}\,\text{mm}^{-1}.\end{aligned}$$

This value can be seen in Table 7.5. The total capacity is then

$$1.044\,\mathrm{kN\,mm^{-1}} \times 500\,\mathrm{mm} = 521.8\,\mathrm{kN}.$$

2. Check yielding of plate,

$$\phi A_g f_y = 0.9 \times 12\,\mathrm{mm} \times 200\,\mathrm{mm} \times 310\,\mathrm{MPa} = 669.6\,\mathrm{kN}.$$

3. Check fracture of plate,

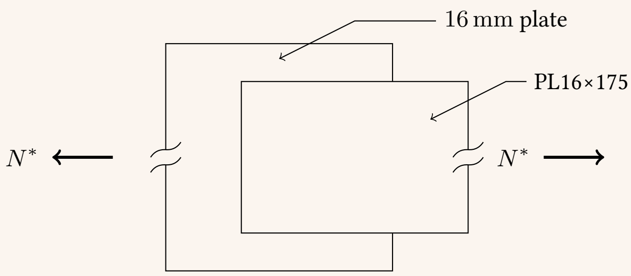
$$\phi 0.85 k_{te} A_n f_u = 0.9 \times 0.85 \times 1 \times 12\,\mathrm{mm} \times 200\,\mathrm{mm} \times 430\,\mathrm{MPa} = 789.5\,\mathrm{kN}.$$

4. Check block tearing, note block tearing can only happen for PL20×300 plate,

$$\begin{aligned} &\phi 0.95\,(0.6 l_v + l_t)\, t_p f_u \\ =&0.9 \times 0.95 \times (0.6 \times 300\,\mathrm{mm} + 200\,\mathrm{mm}) \times 20\,\mathrm{mm} \times 430\,\mathrm{MPa} \\ =&2794\,\mathrm{kN}. \end{aligned}$$

Thus weld strength governs, $N^* \leqslant 521.8\,\mathrm{kN}$.

Example 7.2 *Determine the size and length of fillet weld for the lap joint of two Grade 300 steel plates with $N^* = 640\,\mathrm{kN}$ as shown. Assume plate yield and fracture requirements are satisfied, check block tearing.*



Solution 7.2

The maximum size of fillet weld is $t_w = t - 1\,\mathrm{mm} = 15\,\mathrm{mm}$.

The minimum size of fillet weld is $t_w = 6\,\mathrm{mm}$.

Try 6 mm W40X SP weld, $L_{min} = 4 \times t_w = 24\,\mathrm{mm}$. Assume $k_r = 1.0$, the capacity per unit length is

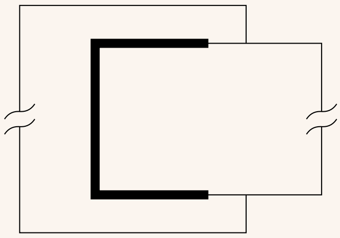
$$\begin{aligned} \phi v_w &= \phi 0.6 f_{uw} t_t k_r \\ &= 0.8 \times 0.6 \times 410\,\mathrm{MPa} \times \frac{6\,\mathrm{mm}}{\sqrt{2}} \times 1 \\ &= 0.835\,\mathrm{kN\,mm^{-1}}. \end{aligned}$$

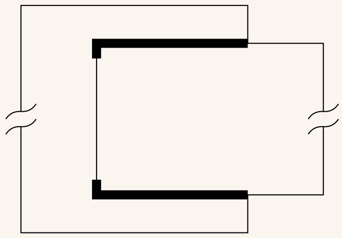
This value can be seen in Table 7.5.

The weld length required is

$$L_{min} = \frac{640\,\text{kN}}{0.835\,\text{kN}\,\text{mm}^{-1}} = 766.5\,\text{mm}.$$

Possible weld placements are the follows.





Which one is better?

Check block tearing using the first placement,

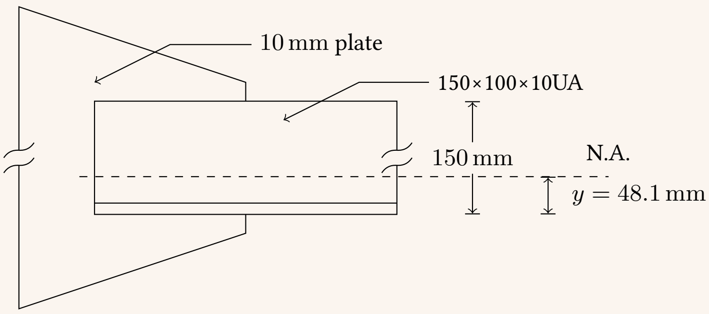
$$\begin{aligned}
&\phi 0.95\left(0.6 l_v + l_t\right) t_p f_u \\
=&0.9 \times 0.95 \times (0.6 \times 591.5\,\text{mm} + 175\,\text{mm}) \times 16\,\text{mm} \times 430\,\text{MPa} \\
=&3111\,\text{kN} > N^* = 640\,\text{kN}.
\end{aligned}$$

Check block tearing using the second placement,

$$\begin{aligned}
&\phi 0.95\left(0.6 l_v + l_t\right) t_p f_u \\
=&0.9 \times 0.95 \times 0.6 \times 766.5\,\text{mm} \times 16\,\text{mm} \times 430\,\text{MPa} \\
=&2705\,\text{kN} > N^* = 640\,\text{kN}.
\end{aligned}$$

Example 7.3 *Design fillet welds to develop the full strength of the angle below considering that it is subject to repeated loading. Use FCAW and Grade 350 steel.*

Note that if no load is specified, then design connection to resist angle capacity.



Solution 7.3

1. Angle Capacity

$$\phi A_g f_y = 0.9 \times 2300\,\mathrm{mm}^2 \times 360\,\mathrm{MPa} = 745\,\mathrm{kN},$$
$$\phi 0.85 k_{te} A_n f_u = 0.9 \times 0.85 \times 0.85 \times 2300\,\mathrm{mm}^2 \times 480\,\mathrm{MPa} = 718\,\mathrm{kN}.$$

Thus, $\phi N_t = 718\,\mathrm{kN}$.

2. Weld Length
The maximum size of fillet weld is $t_w = t - 1\,\mathrm{mm} = 9\,\mathrm{mm}$.

The minimum size of fillet weld is $t_w = 4\,\mathrm{mm}$.

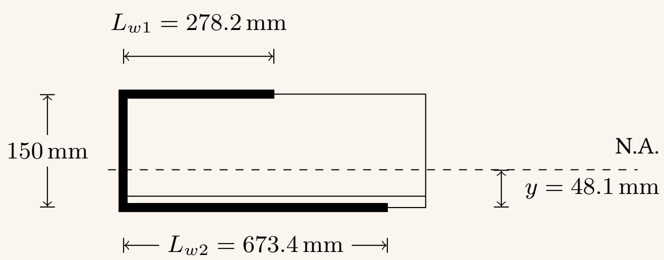
Try 4 mm W50X FCAW SP weld, $L_{min} = 4 \times t_w = 16\,\mathrm{mm}$. Assume $k_r = 1.0$, the capacity per unit length is

$$\begin{aligned}\phi v_w &= \phi 0.6 f_{uw} t_t k_r \\ &= 0.8 \times 0.6 \times 480\,\mathrm{MPa} \times \frac{4\,\mathrm{mm}}{\sqrt{2}} \times 1 \\ &= 0.652\,\mathrm{kN\,mm^{-1}}.\end{aligned}$$

The weld length required is

$$L_{min} = \frac{718\,\mathrm{kN}}{0.652\,\mathrm{kN\,mm^{-1}}} = 1101.6\,\mathrm{mm}.$$

3. Position of Welds
Balance welds so that there is no eccentricity.



Assume the top length is L_{w1}, take moment about N.A. of angle section,

$$\underbrace{L_{w1} \times (150\,\mathrm{mm} - 48.1\,\mathrm{mm})}_{\text{top segment}} + \underbrace{150\,\mathrm{mm} \times (150\,\mathrm{mm}/2 - 48.1\,\mathrm{mm})}_{\text{vertical segment}} = \underbrace{(1101.6\,\mathrm{mm} - L_{w1} - 150\,\mathrm{mm}) \times 48.1\,\mathrm{mm}}_{\text{bottom segment}},$$

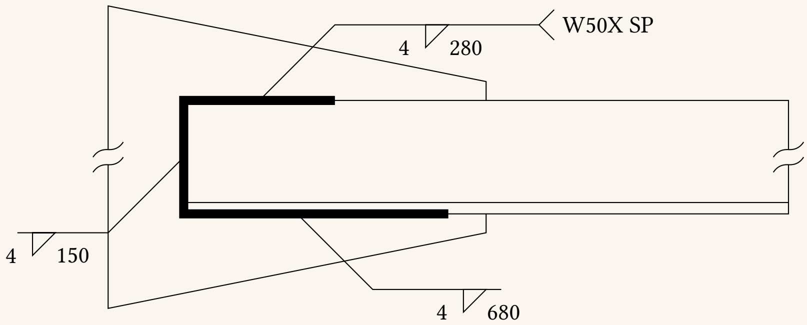
this gives

$$\begin{aligned} L_{w1} &= 278.2\,\mathrm{mm}, \qquad \text{use} \quad L_{w1} = 280\,\mathrm{mm}, \\ L_{w2} &= 673.4\,\mathrm{mm}, \qquad \text{use} \quad L_{w2} = 680\,\mathrm{mm}. \end{aligned}$$

Check block tearing, $l_v = L_{w1} + L_{w2}$,

$$
\begin{aligned}
&\phi 0.95\,(0.6 l_v + l_t)\, t_p f_u \\
=&0.9 \times 0.95 \times (0.6 \times 960\,\text{mm} + 150\,\text{mm}) \times 10\,\text{mm} \times 480\,\text{MPa} \\
=&2980\,\text{kN} > \phi N_t = 717.9\,\text{kN}.
\end{aligned}
$$

All weld lengths are greater than L_{min}. Final layout.



7.10 Quality of Welded Connection

The quality of welded connections depends on

- weldability of steel
  This is a measure of the ability to produce a crack–free and sound structural joint.
- proper preparation of the welded connections
  This involves cleanliness and alignment.
- proper procedures
  - good welding positions

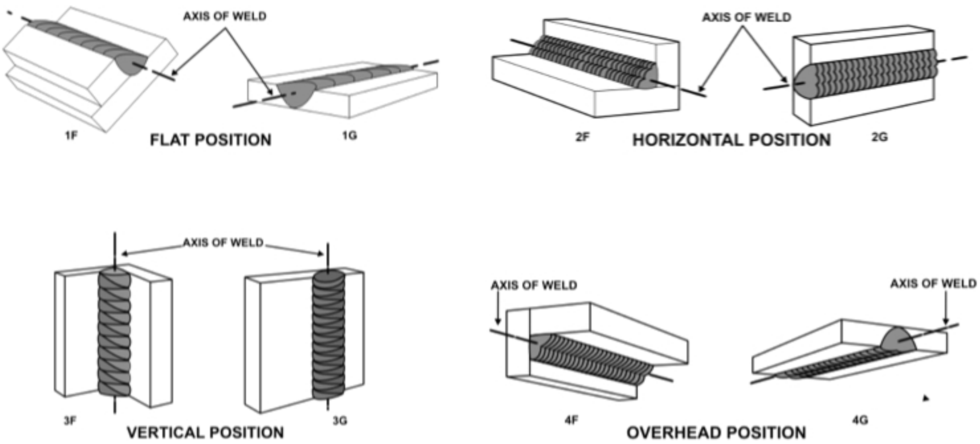


Figure 7.31: Weld positions (https://weldguru.com/welding-positions/)

– control of distortion of the welded member

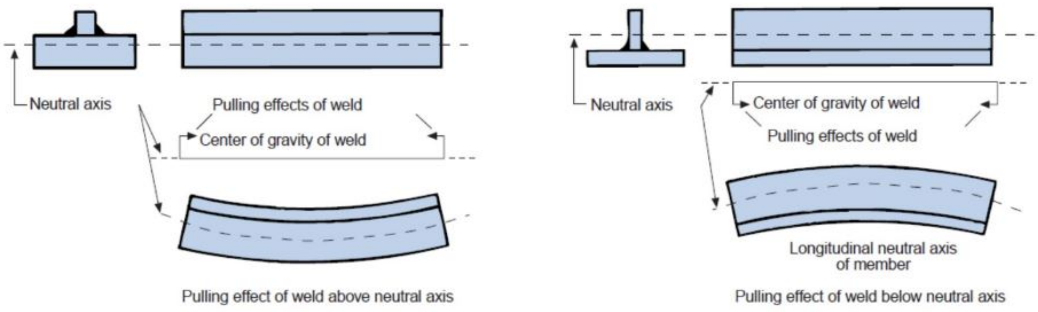


Figure 7.32: Weld distortion (https://weldinganswers.com/7-ways-to-control-distortion-in-welding/)

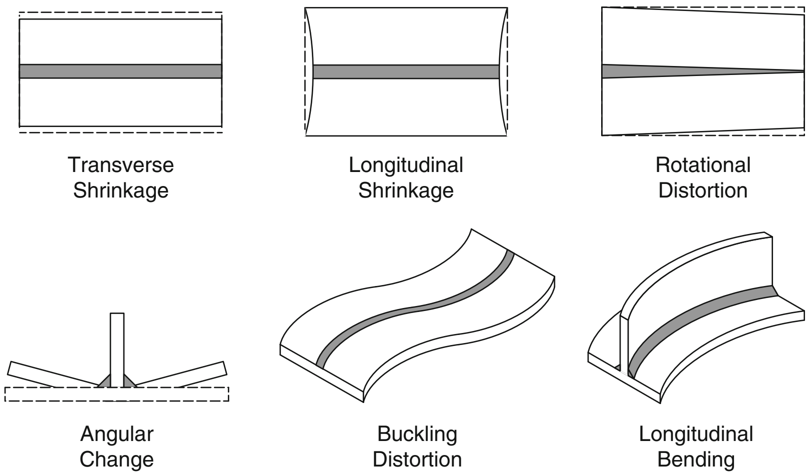


Figure 7.33: Weld distortion (Hetnarski, 2014)

– welding sequences to limit distortion

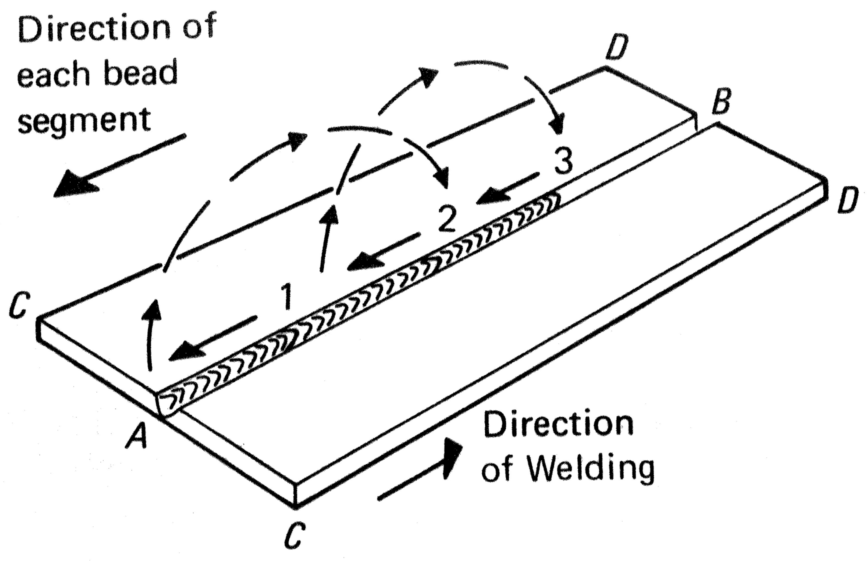


Figure 7.34: Back step weld (https://www.fabricatingandmetalworking.com/2013/02/how-to-control-the-warping-of-parts-in-thin-sheet-metal/)

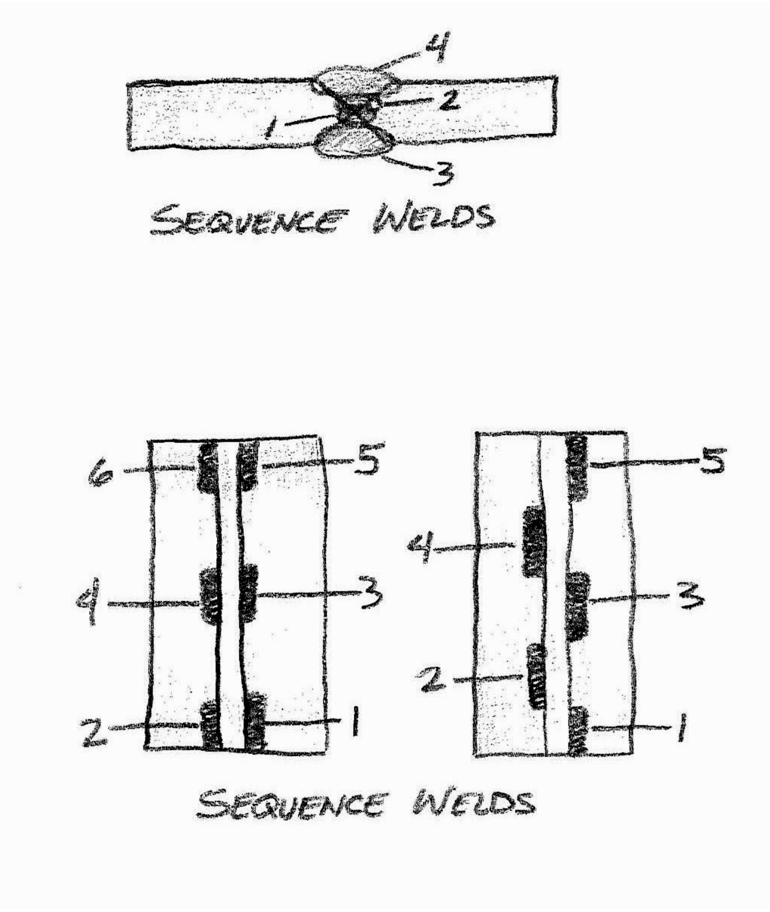


Figure 7.35: Weld sequence (https://axisfab.com/weld-shrinkage/)

- minimum weld thickness
- few passes

Figure 7.36: Multiple pass fillet weld (https://www.mig-welding.co.uk/arc-fillet-joints.htm)

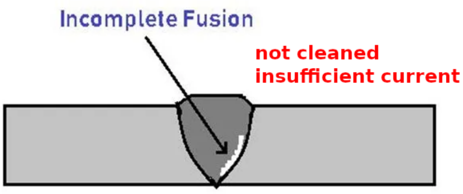
  - correct current and voltage for weld material, correct rate of welding, etc.
  - correct electrode for type of steel chosen
    If weld material is much stronger than plate material then plate failure may occur. An electrode matching plate material is required.

7.11 Possible Weld Defects

Welds need to be inspected by ultrasound or other techniques to ensure that the performance of the connection will be adequate.

Interested readers can refer to this page[1] for full version.

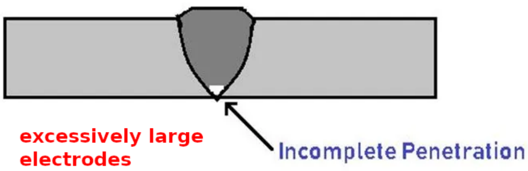
- **Incomplete Fusion**
  These types of welding defects occur when there is a shortage of suitable fusion between the metal and weld. It may also be visible between adjacent weld beads. This produces a gap inside the joint that is not filled with molten metal.



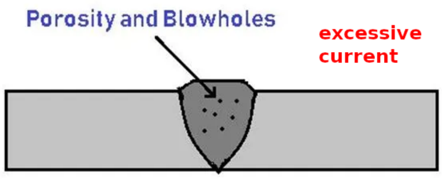
- **Incomplete Penetration**
  In these types of welding defects, penetration is defined as the distance from the uppermost surface of the base plate to the maximum extent of the weld nugget.

[1] https://www.theengineerspost.com/welding-defects/

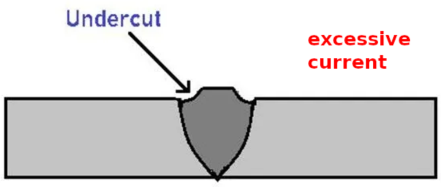
Incomplete penetration happens when the metal groove is not entirely filled, which means that the weld metal does not fully spread through the joint thickness.



- **Porosity and Blowhole**
  Porosity is a group of small bubbles and blowholes are relatively large hidden holes or pores. They are mainly caused by trapped gases. Porosity is a result of weld metal contamination.

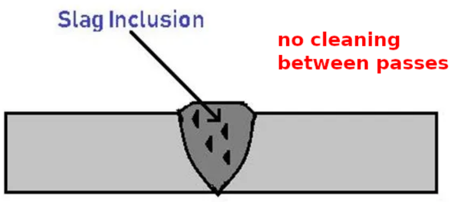


- **Undercut**
  Undercut in welding makes imperfection, it is the formation of grooves in the weld toe, which decreases the cross-sectional thickness of the base metal. As a result of this weld and workpiece get weakened.



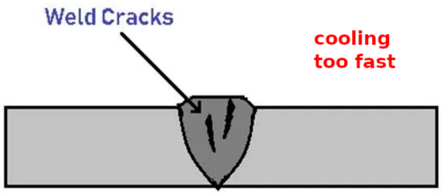
- **Slag Inclusion**
  Slag inclusion is welding defects that are usually visible in welds. The slag is a dangerous substance that appears as a product of stick welding, flux-core arc welding, and submerged arc welding.

  It is can occur when the flux, which is a solid shielding material applied when welding, melts in the weld or on the surface of the weld region. Slag inclusion decreases the strength of the joint and hence makes it weaker.



- **Weld Crack**
  These are the most dangerous types of welding defects. It is almost not allowed by all standards

in the production. It can appear on the surface, in the weld metal, or in an area affected by strong heat.



» 7.12 Possible Plate Defects

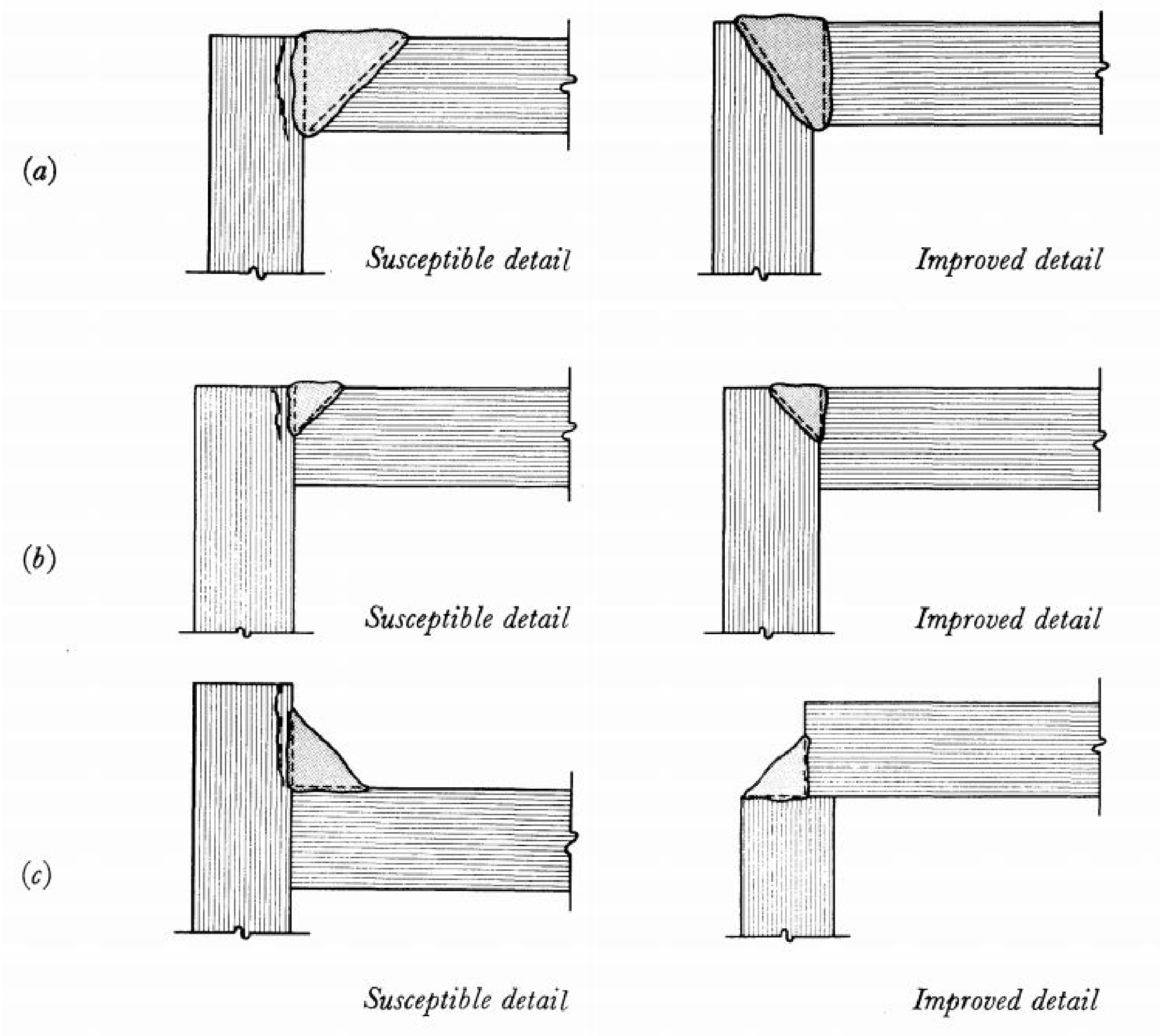


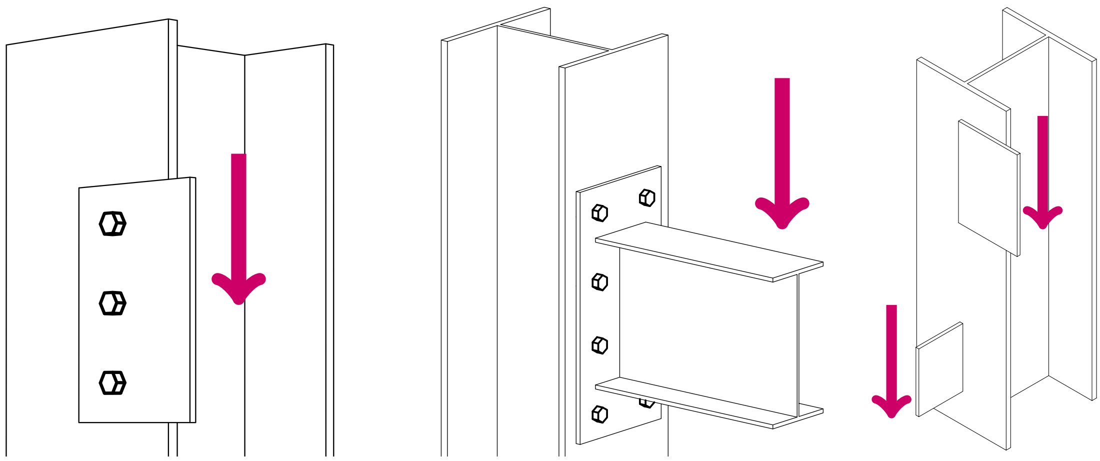
Figure 7.37: Susceptible and improved details (https://buildingfailures.com/2014/11/28/overview-of-lamellar-tearing-and-representative-case-studies/)

Part II

Eccentric Connections

» 8.1 Types of Connections

Some of the principles of bolted steel connections are similar. We will treat them together in this section.



» 8.2 Eccentric Bolted Connections

Bolts are subject to different shear forces, two design methods exist, namely,

- Elastic Design Method — simple, slightly conservative
- Ultimate Strength Method — manuals

8.2.1 Elastic Design Method

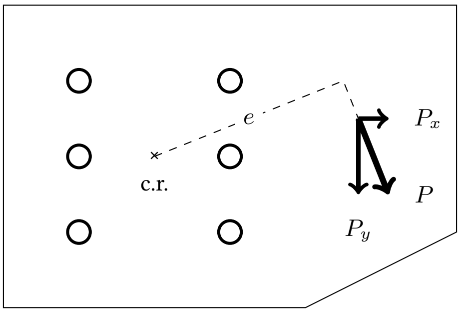


Figure 8.1: Decomposition of arbitrary force into x and y components

Basis for Elastic Computation

Assumption Basic assumptions for elastic computations are made as follows.

- Connector plate is rigid.
- Fasteners deform elastically.
- Fastener forces can be split into direct shear and torsional shear forces with appropriate vectorial components.

Notation The following notations will be used in the analysis.

$$
\begin{aligned}
B_i &= \text{fastener direct shear force} \\
B_{i,x} &= \text{fastener direct shear force component in } x \text{ direction} \\
B_{i,y} &= \text{fastener direct shear force component in } y \text{ direction} \\
P &= \text{applied force} \\
P_x &= \text{applied force component in } x \text{ direction} \\
P_y &= \text{applied force component in } y \text{ direction} \\
r_i &= \text{(positive) distance from c.r. (centre of rigidity) to fastener } i \\
k_i &= \text{fastener stiffness} \\
e &= \text{eccentricity} \\
\theta &= \text{plate rotation} \\
\delta_i &= \text{fastener deformation due to plate rotation } \theta \\
x_{cr} &= x \text{ coordinate of c.r. in the appropriate coordinate system} \\
y_{cr} &= y \text{ coordinate of c.r. in the appropriate coordinate system} \\
T_i &= \text{fastener force due to applied torque}
\end{aligned}
$$

Equilibrium In-plane forces shall satisfy statics equilibrium equations.

$$\sum F_x = 0, \quad \longrightarrow \quad \sum_{i=1}^{n} B_{i,x} = P_x, \tag{8.1}$$

$$\sum F_y = 0, \quad \longrightarrow \quad \sum_{i=1}^{n} B_{i,y} = P_y, \tag{8.2}$$

$$\sum M = 0, \quad \longrightarrow \quad \sum_{i=1}^{n} T_i r_i = Pe. \tag{8.3}$$

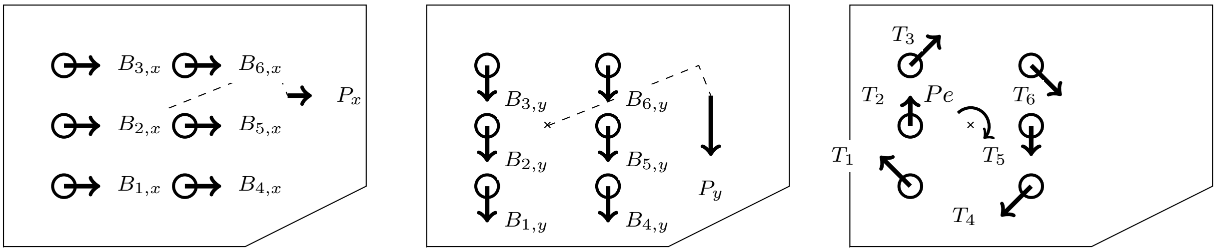


Figure 8.2: Equilibria of bolt forces

Kinematics/Compatibility

$$\theta = \frac{\delta_i}{r_i}. \tag{8.4}$$

Constitutive Relationship

$$T_i = k_i \delta_i, \qquad \delta_i = \frac{T_i}{k_i}. \tag{8.5}$$

Direct Shear Force Assume $k = k_i$,

$$B_{i,x} = P_x \frac{k_i}{\sum_{i=1}^{n} k_i} = \frac{P_x}{n}, \qquad B_{i,y} = P_y \frac{k_i}{\sum_{i=1}^{n} k_i} = \frac{P_y}{n}. \tag{8.6}$$

Torque Due to equilibrium,

$$Pe = \sum_{i=1}^{n} T_i r_i, \qquad \text{about c.r.} \tag{8.7}$$

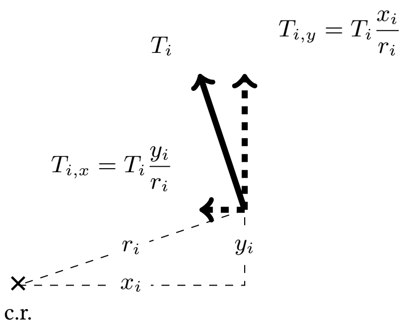
Knowing that $T_i = k_i \delta_i = k_i r_i \theta$, assume $k = k_i$,

$$Pe = \sum_{i=1}^{n} k_i \theta r_i^2 = k\theta \sum_{i=1}^{n} r_i^2, \qquad \longrightarrow \qquad \theta = \frac{Pe}{k \sum_{i=1}^{n} r_i^2}. \tag{8.8}$$

Then,

$$T_i = k r_i \frac{Pe}{k \sum_{i=1}^{n} r_i^2} = Pe \frac{r_i}{\sum_{i=1}^{n} r_i^2}. \tag{8.9}$$

The bolt force due to torsion, T_i, needs to be resolved into components in x and y directions ($T_{i,x}$ and $T_{i,y}$) and added to the direction x and y forces ($B_{i,x}$ and $B_{i,y}$).



*Only magnitudes are shown.
Force components can be signed.
Need to consider the specific geometry in each problem.

The total bolt force may be found from the forces in each direction,

$$V_{f,i}^* = \sqrt{(B_{i,x} + T_{i,x})^2 + (B_{i,y} + T_{i,y})^2}. \tag{8.10}$$

This is discussed further in the procedure below.

Elastic Analysis Procedure

Assume all n fasteners are of the same size ($k = k_i$, $A = A_i$) and load is vertical, viz., $P_x = 0$.

1. Determine centre of rigidity of fastener group

$$x_{cr} = \frac{\sum_{i=1}^n x_i k_i}{\sum_{i=1}^n k_i} = \frac{\sum_{i=1}^n x_i A_i}{\sum_{i=1}^n A_i} = \frac{\sum_{i=1}^n x_i}{n},$$
$$y_{cr} = \frac{\sum_{i=1}^n y_i k_i}{\sum_{i=1}^n k_i} = \frac{\sum_{i=1}^n y_i A_i}{\sum_{i=1}^n A_i} = \frac{\sum_{i=1}^n y_i}{n},$$

   where
   x_i = signed x distance from c.r. of fastener i
   y_i = signed y distance from c.r. of fastener i

2. Calculate distance to c.r. for each fastener

$$r_i = \sqrt{x_i^2 + y_i^2}.$$

3. Determine polar moment of distance

$$J = \sum_{i=1}^n r_i^2.$$

4. Determine fastener direct shear force

$$B_{i,y} = P_y \frac{A_i}{\sum_{i=1}^n A_i} = \frac{P_y}{n}.$$

5. Determine torsional shear force on critical fastener

$$T_i = P_y e \frac{r_i}{J}.$$

6. Resolve T_i into $T_{i,x}$ and $T_{i,y}$

$$T_{i,x} = T_i \frac{y_i}{r_i},$$
$$T_{i,y} = T_i \frac{x_i}{r_i}.$$

7. Find the critical shear on fastener

$$V^*_{f,i} = \sqrt{\left(B^2_{i,y} + T^2_{i,y}\right) + T^2_{i,x}}.$$

8. Select bolt size

$$V^*_{f,max} \leqslant \phi V_f.$$

Bracket Capacity Check

The effective area should be used to obtain Z_x and A_v for strength calculation.

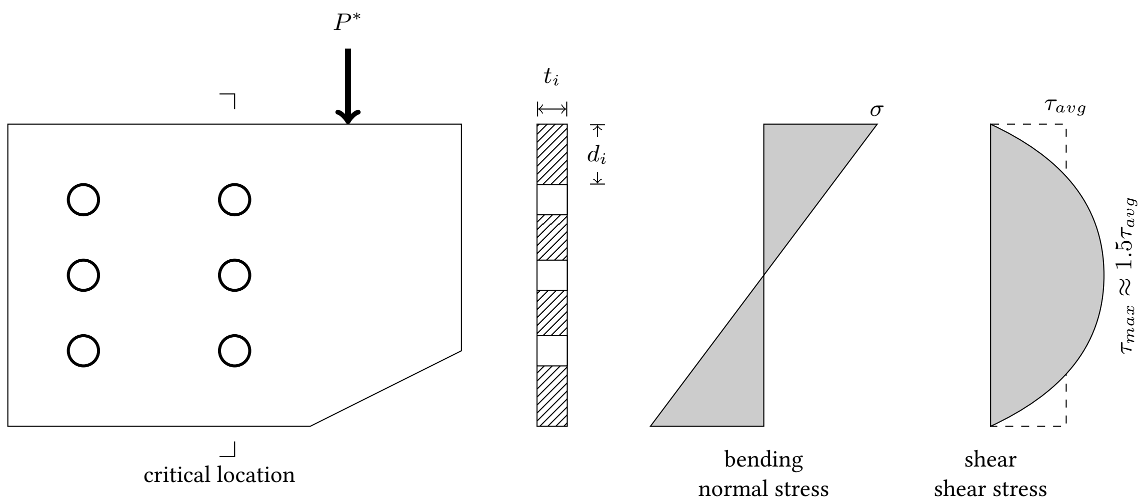


Figure 8.3: Stress distribution on reduced section

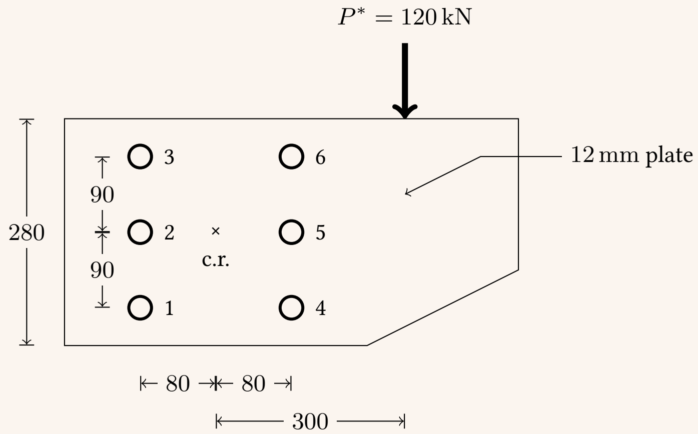
$$M^* \leqslant \phi M_n = \phi Z_x f_y. \tag{8.11}$$

In which Z_x accounts for the holes in plate.

$$V^* \leqslant \phi V_n = \phi A_v f_y = 0.9 \cdot \sum d_t t_i \cdot \frac{2}{3} \cdot 0.6 f_y. \tag{8.12}$$

The factor $2/3$ accounts for averaged shear stress.

Example 8.1 *Determine the size of Grade 8.8/N/S bolts in the bearing type connection below using Grade 300 steel.*



Solution 8.1

Bolt No.	x_i (mm)	y_i (mm)	r_i (mm)	r_i^2 (mm^2)	V_y (kN)	T_i (kN)	$T_{i,x}$ (kN)	$T_{i,y}$ (kN)	B_i (kN)
1	-80	-90	120.42	14500	20	61.23	-45.76	-40.68	50.22
2	-80	0	80.00	6400	20	40.68	0.00	-40.68	20.68
3	-80	90	120.42	14500	20	61.23	45.76	-40.68	50.22
4	80	-90	120.42	14500	20	61.23	-45.76	40.68	76.00
5	80	0	80.00	6400	20	40.68	0.00	40.68	60.68
6	80	90	120.42	14500	20	61.23	45.76	40.68	76.00
			$\sum r_i^2 =$	70800					

The critical bolts are are bolt 4 and 6. A M20/8.8N bolt has a capacity of 92.6 kN (see Table 6.7). Thus use six M20/8.8N bolts.

Example 8.2 *Find the maximum factored load, P^* and the maximum service load, P, that can be carried in the connection in the previous example.*

Solution 8.2

- Factored force P^*
  Since the behaviour is linear,

$$P^* = \frac{92.6\,\text{kN}}{76.0\,\text{kN}} \times 120\,\text{kN} = 146.2\,\text{kN}.$$

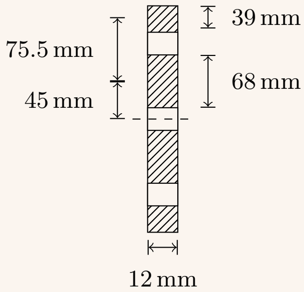
- Service load P
  Similarly,

$$P = \frac{35.5\,\text{kN}}{76.0\,\text{kN}} \times 120\,\text{kN} = 56.1\,\text{kN}.$$

This method is quite conservative to compute P, since it assumes that bolt force increases linearly with distance from the centroid. For friction bolts (/TF and /TB), all bolts may be at the proof force at the same time.

Example 8.3 *Check the plate strength in bending and shear in the previous example.*

Solution 8.3



Using parallel axis theorem,

$$\begin{aligned} A_v &= 2 \times (12 \times 39 + 12 \times 68) \\ &= 2568\,\text{mm}^2, \\ I_x &= 2 \times \left(\frac{12 \times 39^3}{12} + 12 \times 39 \times 120.5^2 + \frac{12 \times 68^3}{12} + 12 \times 68 \times 45^2 \right) \\ &= 17\,643\,256\,\text{mm}^4, \\ Z_x &= \frac{I_x}{y_{max}} = \frac{17\,643\,256\,\text{mm}^4}{140\,\text{mm}} \\ &= 1.26 \times 10^5\,\text{mm}^3. \end{aligned}$$

The moment applied on the section is

$$\begin{aligned} M^* &= Pe \\ &= 120\,\text{kN} \times (300\,\text{mm} - 80\,\text{mm}) \\ &= 26.4\,\text{kN m}. \end{aligned}$$

The moment capacity is

$$\begin{aligned} \phi M_n &= \phi Z_x f_y \\ &= 0.9 \times 1.26 \times 10^5\,\text{mm}^3 \times 310\,\text{MPa} \\ &= 35.2\,\text{kN m} > M^* = 26.4\,\text{kN m}. \end{aligned}$$

The shear capacity is

$$\begin{aligned} \phi V_n &= \phi A_v f_y = 0.9 \cdot \sum d_t t_i \cdot \frac{2}{3} \cdot 0.6 f_y \\ &= 0.9 \times 2568\,\text{mm}^2 \times \frac{2}{3} \times 0.6 \times 310\,\text{MPa} \\ &= 286.6\,\text{kN} > V^* = 120\,\text{kN}. \end{aligned}$$

It is necessary to check other possible failure modes, for example,

- bolt bearing
- bolt tear out
- edge distance
- block tearing, etc.

8.2.2 Ultimate Strength Method

Many countries use the ultimate strength method to estimate the strength of a connection. This method uses the concept of the instantaneous centre of rotation (ICR).

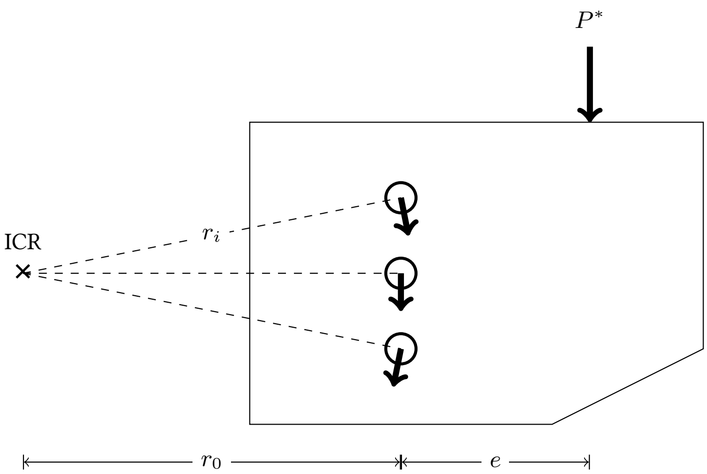


Figure 8.4: Illustration of ICR

According to this method, the deformation of each fastener (or part of a weld) is assumed to be proportional to its distance from the ICR. The connection fails when on of the fasteners reaches the maximum deformation Δ_{max}. Accordingly, the maximum resistance is denoted as R_{ult}.

It is also necessary to know the force deformation relationship for each bolt. A linear relationship can be expressed as

$$R_i = R_{ult}\frac{\Delta_i}{\Delta_{max}}, \tag{8.13}$$

where R_{ult} is the ultimate dependable resistance of a bolt in shear.

This relationship does not have to be linear. For example, according to AISC tables, the deformation relationship is given by the following exponential equation.

$$R_i = R_{ult}\left(1 - e^{-0.394\Delta_i}\right)^{0.55}. \tag{8.14}$$

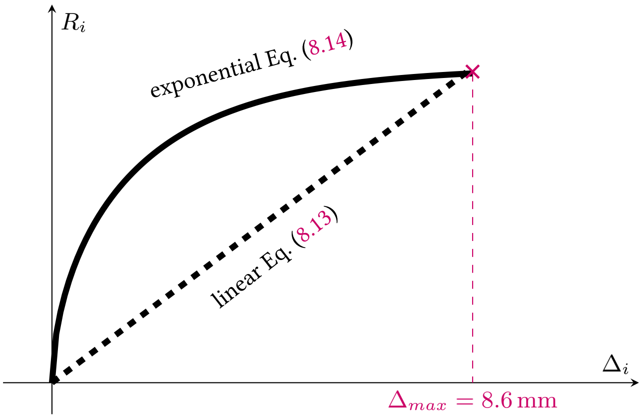


Figure 8.5: Linear and nonlinear responses

The procedure is iterative.

- Assume location of ICR (r_0)
- Check equilibrium
  For vertical P only, there are

$$\sum F_x = 0, \quad \longrightarrow \quad \sum R_i \frac{y_i}{r_i} = 0, \quad \text{Since there is only vertical applied load.} \tag{8.15}$$

$$\sum F_y = 0, \quad \longrightarrow \quad \sum R_i \frac{x_i}{r_i} = P_{u1}, \tag{8.16}$$

$$\sum M = 0, \quad \longrightarrow \quad \sum R_i \frac{r_i}{e + r_0} = P_{u2}, \tag{8.17}$$

  where $R_i = R_i\left(\Delta_i\right)$ is Eq. (8.14) (or Eq. (8.13)) and $\Delta_i = \dfrac{r_i}{r_{max}} \Delta_{max}$, in which

$$r_i = \text{distance from ICR to bolt } i$$
$$r_{max} = \text{distnace to the farthest bolt}$$

  This assumes the farthest bolt(s) would reach the maximum deformation Δ_{max}.

- Iterate on r_0 until $P_{u1} = P_{u2}$
  Such iteration can be tedious so tables are often made for standard connections.

For friction–grip (slip critical) connections, the force R_i may be assumed to be constant for all fasteners. It is the dependable friction resistance per bolt.

Example 8.4 *Ultimate Strength Method on Bolt Group*

Use ultimate strength method to analyse the previous example.

Solution 8.4

From the previous example, the demand causes the maximum shear force 76 kN while the capacity per bolt is 92.6 kN, thus the maximum P^* is

$$P^* = 120\,\text{kN} \times \frac{92.6\,\text{kN}}{76\,\text{kN}} = 146.2\,\text{kN}.$$

Knowing $\Delta_{max} = 8.6\,\text{mm}$, $e = 300\,\text{mm}$ and $R_{ult} = 92.6\,\text{kN}$.

Try $r_0 = 39.33\,\text{mm}$, using **linear** relationship Eq. (8.13),

Bolt No.	About c.r. x_i mm	y_i mm	About ICR x_i mm	y_i mm	r mm	Δ_i mm	R_i kN	P_{u1} kN	P_{u2} kN
1	-80	-90	-40.67	-90	98.76	5.68	61.19	-25.19	17.81
2	-80	0	-40.67	0	40.67	2.34	25.19	-25.19	3.02
3	-80	90	-40.67	90	98.76	5.68	61.19	-25.19	17.81
4	80	-90	119.33	-90	149.47	8.60	92.60	73.93	40.79
5	80	0	119.33	0	119.33	6.87	73.93	73.93	26.00
6	80	90	119.33	90	149.47	8.60	92.60	73.93	40.79
				r_{max}	149.47		$\sum$	146.21	146.21

It can be noted that this method is identical to standard elastic method since bolt deformation is linear elastic.

Try $r_0 = 62.76\,\text{mm}$, using **exponential** relationship Eq. (8.14),

Bolt No.	About c.r. x_i mm	y_i mm	About ICR x_i mm	y_i mm	r mm	Δ_i mm	R_i kN	P_{u1} kN	P_{u2} kN
1	-80	-90	-17.24	-90	91.64	4.67	84.20	-15.84	21.27
2	-80	0	-17.24	0	17.24	0.88	47.11	-47.11	2.24
3	-80	90	-17.24	90	91.64	4.67	84.20	-15.84	21.27
4	80	-90	142.76	-90	168.76	8.60	90.87	76.87	42.27
5	80	0	142.76	0	142.76	7.27	89.66	89.66	35.29
6	80	90	142.76	90	168.76	8.60	90.87	76.87	42.27
				r_{max}	168.76		$\sum$	164.61	164.61

For exponential relationship, $P_u = 164.61\,\text{kN}$ corresponds to $12\,\%$ increase in strength due to nonlinearity. Inelastic ICR method can be used for welds too.

8.3 Eccentric Bolted Tension Shear Connections

These connections have complex behaviour and several methods have been proposed for design as described by Smith and Smith (1996). We consider proof loaded high strength bolts here and we will use the method (Salmon et al., 2009) where high strength bolt forces are used to prevent lift-off.

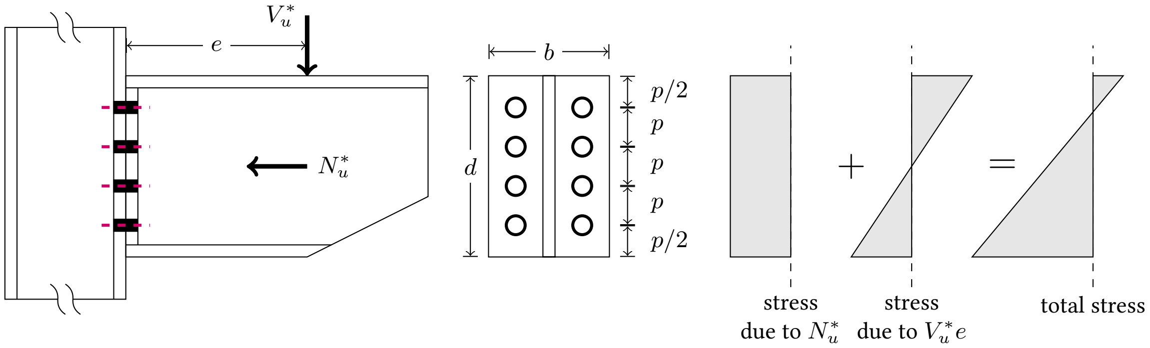


Figure 8.6: Stress components of eccentrically loaded bolted connection

The compressive stress due to $N^* = \sum N_{tf}^*$ can be computed as

$$f_{bolt} = \frac{\sum N_{tf}^*}{bd}. \tag{8.18}$$

The tensile stress on the **top** bolt due to $M^* = V_u^* e$ can be computed as

$$f_{plate} = \frac{M^*}{I} y = \frac{V_u^* e}{bd^3/12}\left(\frac{d}{2} - \frac{p}{2}\right) = 6V_u^* e\frac{d-p}{bd^3}. \tag{8.19}$$

Bolt forces only increase significantly after the applied stress becomes greater than f_{bolt} and lift-off occurs. The limit is

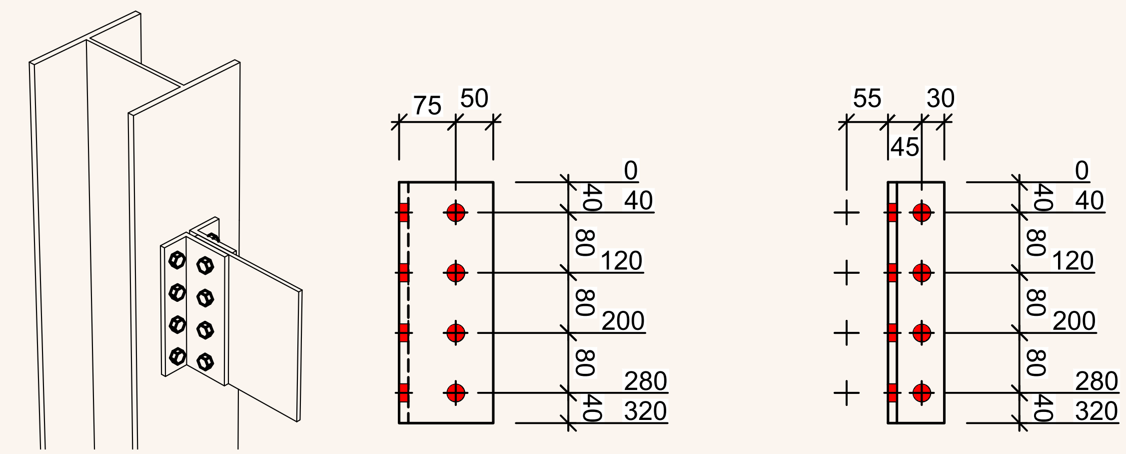
$$f_{bolt} \geqslant f_{plate} \qquad \longrightarrow \qquad \frac{\sum N_{tf}^*}{bd} \geqslant 6V_u^* e\frac{d-p}{bd^3}.$$

Thus,

$$M^* = V_u^* e \leqslant \frac{\sum N_{tf}^*}{bd}\frac{1}{6}\frac{bd^3}{d-p}, \qquad \longrightarrow \qquad M^* = V_u^* e \leqslant \frac{\sum N_{tf}^*}{6}\frac{d^2}{d-p}. \tag{8.20}$$

Noting that due to the presence of shear force V_u^*, N_{tf}^* shall be reduced by considering combined action (see § 6.11.8 and § 6.12.3).

Example 8.5 *Find P^* considering M20X/8.8/S bolts. Two 125×75×12UA angles are used in the connection with short leg connected to beam and long leg connected to plate. A vertical force P^* is applied on bolt line connecting plate (75 mm from column surface).*



Solution 8.5

$$M^* = P^* e \leqslant \frac{\sum N_{tf}^*}{6}\frac{d^2}{d-p},$$

$$P^* \leqslant \frac{1}{75\,\text{mm}} \times \frac{8N_{tf}^*}{6} \times \frac{320\,\text{mm} \times 320\,\text{mm}}{320\,\text{mm} - 80\,\text{mm}} = 7.59N_{tf}^*.$$

Or, $N_{tf}^* \geqslant 0.132P^*$.

$$V_f^* = \frac{P^*}{n} = 0.125P^*.$$

For M20X/8.8/S bolts,

$$\phi N_{tf} = 163\,\text{kN}, \qquad \phi V_f = 129\,\text{kN}.$$

Consider combined action,

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leqslant 1.0,$$

$$\left(\frac{0.125P^*}{129\,\text{kN}}\right)^2 + \left(\frac{0.132P^*}{163\,\text{kN}}\right)^2 \leqslant 1.0,$$

$$P^* \leqslant 792.3\,\text{kN}.$$

Using a friction–type connection (/TB), assume $k_h = 1.0$, consider combined action,

$$\frac{V_f^*}{\phi V_f} + \frac{N_{tf}^*}{\phi N_{tf}} \leqslant 1.0,$$

$$\frac{0.125P^*}{35.5\,\text{kN}} + \frac{0.132P^*}{101.5\,\text{kN}} \leqslant 1.0,$$

$$P^* \leqslant 207.5\,\text{kN}.$$

8.4 Welded Eccentric Shear Connections

8.4.1 Elastic Method

It is assumed that the connector plate is rigid, the welds do not deform, the forces result from direct shear and torsion. Rotation is assumed to occur about the weld centroid.

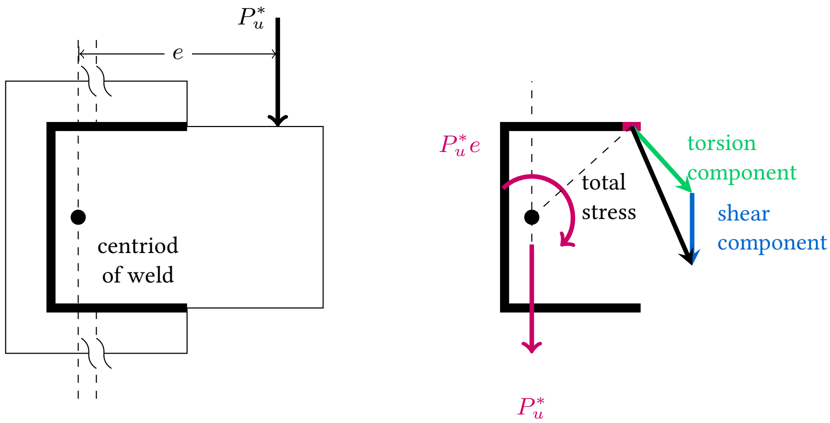


Figure 8.7: Stress components of weld element

Analysis Procedure

- Determine centroid of weld

$$\bar{x} = \frac{\int x \mathrm{d}A}{\int \mathrm{d}A} = \frac{\sum Ax}{\sum A}, \qquad \bar{y} = \frac{\int y \mathrm{d}A}{\int \mathrm{d}A} = \frac{\sum Ay}{\sum A}.$$

- Calculate polar moment of inertia J

$$J = \int r^2 \mathrm{d}A = \int \left(x_i^2 + y_i^2\right) \mathrm{d}A = \int x_i^2 \mathrm{d}A + \int y_i^2 \mathrm{d}A = I_x + I_y,$$

  where
  $x_i = x - \bar{x} = x$ distance measured from centroid
  $y_i = y - \bar{y} = y$ distance measured from centroid

  By parallel axis theorem,

$$J = \sum I_{x0} + \sum I_{y0} + \sum Ax_i^2 + \sum Ay_i^2,$$

  where I_{x0} and I_{y0} are moment of inertia about the element's own axes.

- Calculate components of stress due to direct shear

$$f_x' = \frac{P_{u,x}^*}{A} = \frac{P_{u,x}^*}{t_e l_w}, \qquad f_y' = \frac{P_{u,y}^*}{A} = \frac{P_{u,y}^*}{t_e l_w}.$$

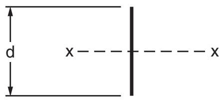
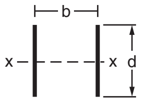
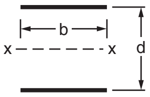
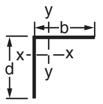
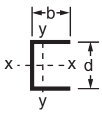
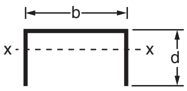
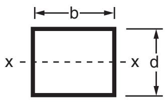
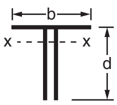
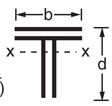
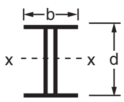
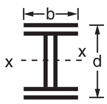
Outline of Welded Joint b = Width d = Depth	**Bending (About Horizontal Axis $x - x$), in²**	**Twisting, in³**
	$S_w = \frac{d^2}{6}$	$J_w = \frac{d^3}{12}$
	$S_w = \frac{d^2}{3}$	$J_w = \frac{d(3b^2+d^2)}{6}$
	$S_w = bd$	$J_w = \frac{b^3+3bd^2}{6}$
 $N_y = \frac{b^2}{2(b+d)}$ $N_x = \frac{d^2}{2(b+d)}$	$S_w = \frac{4bd+d^2}{6} = \frac{d^2(4b+d)}{6(2b+d)}$ top bottom	$J_w = \frac{(b+d)^4 - 6b^2d^2}{12(b+d)}$
$N_y = \frac{b^2}{2b+d}$ 	$S_w = bd + \frac{d^2}{6}$	$J_w = \frac{(2b+d)^3}{12} - \frac{b^2(b+d)^2}{2b+d}$
$N_x = \frac{d^2}{b+2d}$ 	$S_w = \frac{2bd+d^2}{3} = \frac{d^2(2b+d)}{3(b+d)}$ top bottom	$J_w = \frac{(b+2d)^3}{12} - \frac{d^2(b+d)^2}{b+2d}$
	$S_w = bd + \frac{d^2}{3}$	$J_w = \frac{(b+d)^3}{6}$
$N_y = \frac{d^2}{b+2d}$ 	$S_w = \frac{2bd+d^2}{3} = \frac{d^2(2b+d)}{3(b+d)}$ top bottom	$J_w = \frac{(b+2d)^3}{12} - \frac{d^2(b+d)^2}{b+2d}$
$N_y = \frac{d^2}{2(b+d)}$ 	$S_w = \frac{4bd+d^2}{3} = \frac{4bd^2+d^3}{6b+3d}$ top bottom	$J_w = \frac{d^3(4b+d)}{6(b+d)} + \frac{b^3}{6}$
	$S_w = bd + \frac{d^2}{3}$	$J_w = \frac{b^3+3bd^2+d^3}{6}$
	$S_w = 2bd + \frac{d^2}{3}$	$J_w = \frac{2b^3+6bd^2+d^3}{6}$

Figure 8.8: Treating welds as lines (Roark, 2012)

- Calculate components of stress due to torsion

$$f''_x = P^*_u e \frac{y_i}{J}, \qquad f''_y = P^*_u e \frac{x_i}{J}.$$

- Calculate components of stress due to any potential out-of-plane moment M^*

$$f_z = M^* \frac{y_i}{I_x} + M^* \frac{x_i}{I_y}.$$

- Calculate total stress

$$f_u^* = \sqrt{(f_x' + f_x'')^2 + \left(f_y' + f_y''\right)^2 + f_z^2}.$$

- Find critical point(s) on the weld and ensure

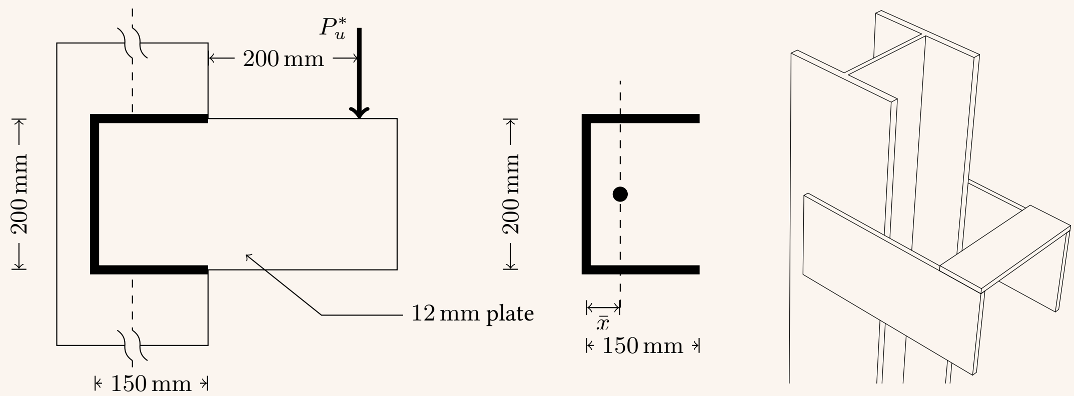
$$f_{u,max}^* \leqslant \begin{cases} \phi_{weld} 0.6 f_{weld} & \text{weld strength} \\ \phi_{steel} 0.85 \dfrac{t_p}{t_t} 0.6 f_u & \text{plate block tearing} \end{cases}$$

Design Procedure

- Select weld process, strength and weld lines
- Analyse the connection to find the required throat thickness t_t to resist P^*
- Calculate the weld leg size t_w
- Check shear and tension stresses in the base metal

Example 8.6 *Worksheet Eccentrically Loaded Weld*

A bracket is to be connected to both flanges of a 250UC89.5 Grade 300 column to transit a vertical factored force of 200 kN *that is* 200 mm *from the column. Assume plate satisfies all necessary requirements. Use E48XX GP electrodes and find weld size.*



Solution 8.6

Welds are subject to shear and torsion.

Since there are two sides, each side experiences $200\,\text{kN}/2 = 100\,\text{kN}$ force.

- Determine centroid of weld

$$\bar{x} = \frac{\sum Ax}{\sum A} = \frac{2 \times 150\,\text{mm} \times \dfrac{150\,\text{mm}}{2}}{200\,\text{mm} + 2 \times 150\,\text{mm}} = 45\,\text{mm}.$$

- Calculate polar moment of inertia J, ignoring any high order terms of t_t,

$$
\begin{aligned}
J &= 2 \times \underbrace{150\,\text{mm} \times t_t \times (100\,\text{mm})^2}_{Ay_i^2 \text{ of horizontal elements}} + \underbrace{\frac{t_t \times (200\,\text{mm})^3}{12}}_{I_x \text{ of vertical element}} \\
&\quad + 2 \times \underbrace{\frac{t_t \times (150\,\text{mm})^3}{12}}_{I_y \text{ of horizontal elements}} + 2 \times \underbrace{150\,\text{mm} \times t_t \times (75\,\text{mm} - 45\,\text{mm})^2}_{Ax_i^2 \text{ of horizontal elements}} \\
&\quad + \underbrace{200\,\text{mm} \times t_t \times (45\,\text{mm})^2}_{Ax_i^2 \text{ of vertical element}} \\
&= \frac{14712500}{3} t_t \approx 4904166.7 t_t.
\end{aligned}
$$

  The factor 4904166.7 can alternatively be computed via the formula provided in Fig. 8.8.

- Calculate components of stress due to direct shear

$$
f'_y = \frac{P_{u,y}}{A} = \frac{200\,\text{kN}/2}{500 t_t} = \frac{0.2}{t_t}.
$$

- Calculate components of stress due to torsion
  The critical points are top and bottom right ends.

$$
\begin{aligned}
f''_x &= P_u e \frac{y_{i,max}}{J} = 200\,\text{kN}/2 \times (200\,\text{mm} + 105\,\text{mm}) \times \frac{100\,\text{mm}}{4904166.7 t_t} = \frac{0.622}{t_t}, \\
f''_y &= P_u e \frac{x_{i,max}}{J} = 200\,\text{kN}/2 \times (200\,\text{mm} + 105\,\text{mm}) \times \frac{105\,\text{mm}}{4904166.7 t_t} = \frac{0.653}{t_t}.
\end{aligned}
$$

- Calculate total stress

$$
\begin{aligned}
f^*_u &= \sqrt{(f''_x)^2 + \left(f'_y + f''_y\right)^2} \\
&= \sqrt{\left(\frac{0.622}{t_t}\right)^2 + \left(\frac{0.2}{t_t} + \frac{0.653}{t_t}\right)^2} \\
&= \frac{1.056\,\text{kN}\,\text{mm}^{-1}}{t_t}.
\end{aligned}
$$

- Check capacity

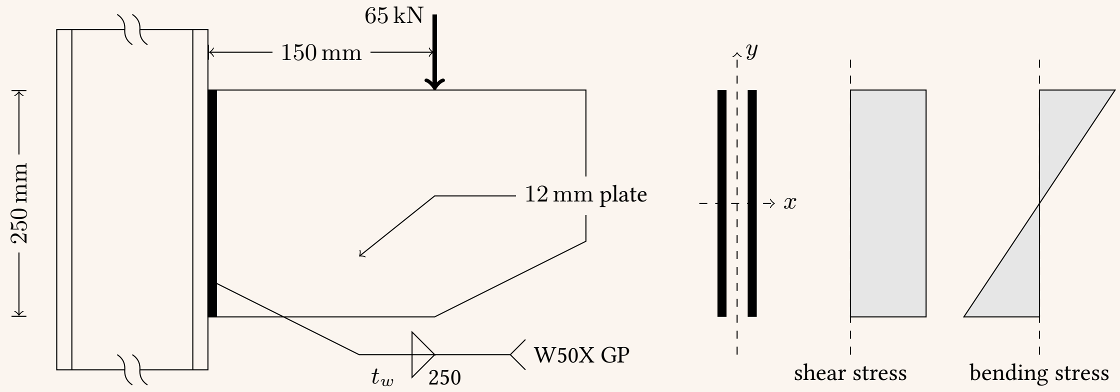
$$
\frac{1.056\,\text{kN}\,\text{mm}^{-1}}{t_t} \leqslant \phi_{weld} 0.6 f_{weld}, \quad \longrightarrow \quad t_t \geqslant \frac{1.056\,\text{kN}\,\text{mm}^{-1}}{0.6 \times 0.6 \times 480\,\text{MPa}} = 6.11\,\text{mm}.
$$

- Check block tearing in plate

$$
\begin{aligned}
\frac{1.056\,\text{kN}\,\text{mm}^{-1}}{t_t} &\leqslant \phi_{steel} 0.85 \frac{t_p}{t_t} 0.6 f_u \\
\frac{1.056\,\text{kN}\,\text{mm}^{-1}}{t_t} &\leqslant 0.9 \times 0.85 \times \frac{12\,\text{mm}}{t_t} \times 0.6 \times 440\,\text{MPa} = \frac{2.424\,\text{kN}\,\text{mm}^{-1}}{t_t}.
\end{aligned}
$$

Thus required weld size is $t_w = \sqrt{2} \times 6.11\,\text{mm} = 8.64\,\text{mm}$, try 10 mm weld. The minimum size is 5 mm. The maximum size is 11 mm. Use 10 mm E48XX GP fillet welds.

Example 8.7 *Compute required size of W50X fillet weld. Assume that column and bracket do no govern. Assume the shear stress is uniform.*



Solution 8.7

Compute stress due to shear force, assume shear stress is uniformly distributed.

$$f'_y = \frac{N^*}{A} = \frac{65\,\text{kN}}{2 \times 250 \times t_t} = \frac{0.13\,\text{kN}\,\text{mm}^{-1}}{t_t}.$$

Compute stress due to moment.

$$I_x = 2 \times \frac{t_t \times (250\,\text{mm})^3}{12} = 2604166.7 t_t.$$

Thus,

$$f_{z,max} = M^* \frac{y_{max}}{I_x} = 65\,\text{kN} \times 150\,\text{mm} \times \frac{125\,\text{mm}}{2604166.7 t_t} = \frac{0.468\,\text{kN}\,\text{mm}^{-1}}{t_t}.$$

The total stress is then

$$f_u^* = \sqrt{\left(\frac{0.13\,\text{kN}\,\text{mm}^{-1}}{t_t}\right)^2 + \left(\frac{0.468\,\text{kN}\,\text{mm}^{-1}}{t_t}\right)^2} = \frac{0.486\,\text{kN}\,\text{mm}^{-1}}{t_t}.$$

Check capacity.

$$\frac{0.486\,\text{kN}\,\text{mm}^{-1}}{t_t} \leqslant \phi_{weld} 0.6 f_{weld}, \quad \longrightarrow \quad t_t \geqslant \frac{0.486\,\text{kN}\,\text{mm}^{-1}}{0.6 \times 0.6 \times 480\,\text{MPa}} = 2.81\,\text{mm}.$$

Thus, $t_w = \sqrt{2} \times 2.81\,\text{mm} = 3.98\,\text{mm}$. The minimum size is 5 mm. Use 5 mm W50X GP fillet welds.

Part III

Structural Steel Weld Specification and Implementation

Fabricators generally only stock a few types of welding consumable so it is recommended that the following be used in practice.

A. Specification for Engineers for Construction Documents

For welds to Grade 300 and 350 structural steel:

1. Weld metal shall be designated as $f_{uw} = 490\,\mathrm{MPa}$.
2. Welding consumables shall be matched with the steel types in accordance with AS/NZS 1554.1 (2014) Table 4.6.1(A).
3. All welding, including qualification of welding procedures, shall comply with AZ/NZS 1554.1 (2004; Am. 1, 2015; Am. 2, 2017).
4. Welds subject to earthquake loads or effects shall comply with AS/nZS 5131 (2016; Am., 2020) Section 7.5.17 and welding consumables shall have a Ship Classification Societies Grade 3 approval.
5. eld category specified for seismic structural applications, and for all construction to Construction Category (CC) 3, shall be SP to AS/NZS 1554.1 (2004; Am. 1, 2015; Am. 2, 2017).
6. Fabrication shall comply with the Construction Category (CC) in accordance with AS/NZS 5131 (2016; Am., 2020).
7. Weld inspection shall follow AZ/NZS 5131 (2016; Am., 2020) Appendix I.

B. Fabricator Actions in Response to Above Specification

1. Welding consumables shall be used within the welding parameter ranges specified by the manufacturer, and as per any relevant Ship Classification Societies approval.
2. Fabricator shall comply with the Construction Category (CC) specified for the job.

C. Commentary

1. The recommendations above were developed together with John Jones Steel and the HERA Welding Centre (June, 2021). They were developed to simplify weld specification.

2. Welding inspection can be performed as a combination of in-house welding inspection and by a third–party inspector if required. The standard is not specific as to who should perform the inspection. The type and quantity of inspection needs to be specified by the engineer. It can be performed by the fabricator (in-house), and/or by a third–party inspection company.

3. Fillet welds should be specified where possible because butt welds are generally more expensive.

4. Storage of welding consumables will comply with manufacturer's recommendations (to avoid cold cracking, porosity and other weld defects) as the fabricator meets the Construction Category (CC) requirements.

LIBERTY

Hot Rolled and Structural Steel Products

Eighth Edition

Contents

Foreword

This edition of Liberty Steel's Hot Rolled and Structural Steel Product Catalogue incorporates the following changes from the previous edition.

- The depths and widths of Universal Beams (UBs) and Columns (UCs) were previously provided to three significant figures. For consistency with AS/NZS 3679.1 *Structural Steel – Hot rolled bars and sections*, these measurements are now provided to one decimal place. The dimensions for UBs and UCs were converted from imperial to metric units of measure in the mid 1970s and resulted in dimensions that were not whole millimetres. Until this edition they were rounded to three significant figures. The other sections in the Catalogue are metric and therefore in whole millimetres. The section properties for all sections in this version and the previous versions have used depths and widths correct to one decimal place to calculate the tabulated values presented to three significant figures. These values are unchanged from the previous edition.
- The inclusion of tolerance tables for each of the products listed. These values are consistent with AS/NZS 3679.1.
- The inclusion of tables providing the allowable camber and sweep of sections consistent with AS/NZS 3679.1.

Introduction

Liberty Steel owns facilities which have a long and significant presence in the Australian steel industry. These facilities which produce steel and finished steel products, date back to the establishment of steelmaking in Newcastle in 1915 and continues to the present day.

Liberty Steel's major manufacturing facilities for hot rolled products are located in Whyalla, South Australia; in Melbourne, Victoria and in Newcastle and western Sydney, New South Wales. Together they are considered Australia's premier manufacturer of steel long products. These products include structural sections, rail, sleepers, rod, bar, and wire.

This catalogue, which demonstrates Liberty Steel's ongoing commitment to the Australian construction and manufacturing industry, has been produced to provide general information on a range of hot rolled structural steel products.

Commitment to Quality

Liberty Steel supplies products that are compliant to the relevant Australian Standards or its own high quality standards. Liberty Steel's aim is to supply a consistent high quality product which delivers benefits to our customers by minimising variation and reducing waste.

The quality of products is constantly checked in NATA accredited testing laboratories, by skilled technical staff using proven equipment. Strict metallurgical control is maintained, from receipt of raw materials to despatch of the finished product. Products are rigorously tested and certified, with test certificates providing assurance that Liberty Steel sections meet all required specifications. These are made available free of charge via our EzyCommerce® website.

At its manufacturing sites Liberty Steel has third party accreditation to Quality Management System ISO 9001 and Environmental Management System ISO 14001.

Test Certificates – EzyCommerce

NATA accredited test certificates are available for all AS/NZS 3679.1 products. The Steel Structures Design Standard – AS4100, acknowledges these certificates provide designers and certifiers with sufficient evidence that they are acceptable steels for use in designs to AS4100. Our test certificates also comply with EN10204 Type 3.1.

Fabricators can ensure they receive a copy of the relevant certificate covering the steel ordered and delivered by requesting them at the time of order. The certificates can be provided manually, electronically or customers can access these via Liberty Steel's EzyCommerce® website at https://ezycommerce.libertygfg.com

All distributors of Liberty Steel AS/NZS 3679.1 products have access to certificates via EzyCommerce® – this is a free service that offers the ability to access and retrieve this information anytime.

Access to EzyCommerce® Online is free to approved customers of Liberty Steel – all you need is a login name and password – please refer to www/libertygfg.com/steel/ezycommerce for more information on obtaining access to the website.

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For more information:

Ezycommerce, https://libertygfg.com/steel/ezycommernce

ACRS - Third Party Certification

In addition to our quality systems and NATA endorsed laboratories, Liberty Steel's range of AS/NZS 3679.1 hot rolled products are all produced at mills with ACRS certification.

Copies of our ACRS accreditation can be viewed at the Liberty Steel website: www.libertygfg.com

For more information:

Liberty Steel website: www.libertygfg.com

ACRS: www.steelcertification.com

ACRS Independent Third Party Australasian Standards Certification & Verification of Reinforcing, Prestressing & Structural Steels Compliance
www.steelcertification.com

Commitment to Quality

Test Certificate sample

LIBERTY

TEST CERTIFICATE

Page 1 of 2
Certificate No.: W971841
Transmission Date: 28/11/17

Customer:	Supplier: OneSteel Manufacturing Pty Limited
	Whyalla, SA - 5600, Australia
	A.B.N. 42 004 651 325
	Sales Order No: B7093
Ship To:	Printed on: 28/11/2018

NATA

Accredited for compliance with ISO/IEC 17025 - Testing. This document shall not be reproduced except in full.

Sampling undertaken byOneSteel Whyalla15352
Approved Signatory - P. Rawnsley
Chemical results as identified are fromBureau Veritas Minerals Pty Ltd, Whyalla0834
Approved Signatory - K. Barsby
Mechanical results as identified are fromBureau Veritas Minerals Pty Ltd, Whyalla0794
Approved Signatory - I. Harrison

STEELMAKING: Basic Oxygen - Slab Cast
SPECIFICATION: ***AS/NZS3679.1-300PLUS/S0***
PRODUCT: ***310UB40.4***

INSPECTION: Supplier
CERTIFICATION: Supplier

ITEMS COVERED BY THIS TEST CERTIFICATE

Item No	Heat No	Customer Order	Length
2260C	571984	7505648987	10.500
2260C	571985	7505648987	10.500
2260C	571986	7505648987	10.500
2289C	571973	7505649607	18.000
2289C	571984	7505649607	18.000

CHEMICAL ANALYSIS

Percentage of element by mass (L=Cast, P=Product, -S=Soluble, -T=Total, CF=Chemical Formula, n=Min, x=Max)

Item No	Heat / Unit No	NATA Lab	L/P	C	P	Mn	Si	S	Ni	Cr	Mo	Cu	Sn	Al
2260C	571984	0834	L	.188	.018	1.32	.150	.006	.008	.022	.005	.008	.002	.012
2260C	571985	0834	L	.184	.016	1.33	.140	.008	.007	.022	.005	.008	.002	.022
2260C	571986	0834	L	.188	.013	1.34	.130	.007	.007	.022	.005	.008	.001	.023
2289C	571973	0834	L	.157	.016	1.53	.150	.010	.008	.024	.006	.009	.002	.022
2289C	571984	0834	L	.188	.018	1.32	.150	.006	.008	.022	.005	.008	.002	.012

Item No	Heat / Unit No	NATA Lab	L/P	Nb	Ti	B	V	N	Ca	Zr	CF1
2260C	571984	0834	L	.003	.001	.0005	.002	.0042	.0001	.002	.41
2260C	571985	0834	L	.003	.001	.0005	.002	.0050	.0001	.002	.41
2260C	571986	0834	L	.003	.001	.0006	.002	.0044	.0001	.002	.42
2289C	571973	0834	L	.004	.001	.0005	.002	.0060	.0001	.003	.42
2289C	571984	0834	L	.003	.001	.0005	.002	.0042	.0001	.002	.41

CF1=C+Mn/6 + (Cr+Mo+V)/5 + (Ni+Cu)/15

MECHANICAL TESTING

Tensile

Item No	Heat No	Tested Unit	NATA Lab	Test Report	ReH MPa	Rm MPa	ELONGN %
2260C	571984	571984	0794	57196	380	520	37
2260C	571984	571984	0794	57196	365	500	36
2260C	571985	571985	0794	57197	350	500	36
2260C	571985	571985	0794	57197	350	490	36
2260C	571986	571986	0794	57197	355	490	36
2260C	571986	571986	0794	57197	355	500	39
2289C	571973	571973	0794	57196	360	500	38
2289C	571973	571973	0794	57196	345	490	38
2289C	571973	571973	0794	57196	360	510	34
2289C	571984	571984	0794	57196	380	520	37
2289C	571984	571984	0794	57196	365	500	36

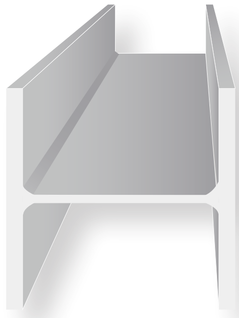
Yield Strength - determined in accordance with requirements of nominated product standard

Availability

Structural Steel Sections

Hot Rolled Products

Hot Rolled Structural Steel Sections produced by Liberty Steel are manufactured in accordance with the requirements of Australian Standard AS/NZS 3679.1 Structural steel – hot rolled bars and sections.



Grade Availability

300PLUS® Steel is the standard product manufactured by Liberty Steel for hot rolled Structural Steel Sections for Australia.

300PLUS® Steel for hot rolled products is produced to exceed the minimum requirements of AS/NZS 3679.1 grade 300.

For further information contact Liberty Steel Sales.

The following AS/NZS 3679.1 grades are also available by enquiry and will depend on the section and quantity required.

Table 1: Additional Grades Available

Additional Grades Available
300PLUS® L0 – Exceeds the requirements of AS/NZS 3679.1 – 300L0
300PLUS® L15 – Exceeds the requirements of AS/NZS 3679.1 – 300L15
AS/NZS 3679.1 – 350
AS/NZS 3679.1 – 350L0
AS/NZS 3679.1 – 350L15

Length Availability

The majority of Structural Steel Sections produced by Liberty Steel are available in standard length and bundle configurations.

We would recommend that attention be given to the standard lengths produced by Liberty Steel as they are more readily available than other lengths. Table 2 (page 6) indicates the standard lengths produced by Liberty Steel in Structural Steel Sections. For other lengths (including those in excess of 18 metres) please contact Liberty Steel Sales for further details.

Availability

Table 2 Standard Lengths

Section	Length (m) 6.0	7.5	9.0	10.5	12.0	13.5	14.0	15.0	16.5	18.0	20.0*
Universal Beams											
610 UB, 530 UB, 460 UB, 410 UB, 360 UB			●	●	●	●		●	●	●	●
310 UB 46.2, 40.4			●	●	●	●		●	●	●	●
310 UB 32.0			●	●	●	●		●		●	
250 UB			●	●	●	●		●	●	●	
200 UB 29.8, 25.4, 22.3			●	●	●	●		●	●	●	
200 UB 18.2			●	●	●	●		●			
180 UB, 150 UB			●	●	●	●		●	●		
Universal Columns											
310 UC 158, 137, 118			●	●	●	●		●	●	●	
310 UC 96.8			●	●	●	●		●	●	●	●
250 UC			●	●	●	●		●	●	●	●
200 UC, 150 UC			●	●	●	●		●	●	●	
100 UC			●		●			●			
Tapered Flange Beams											
125 TFB, 100 TFB		●	●		●		●	●			
Parallel Flange Channels											
380 PFC, 300 PFC, 250 PFC, 230 PFC, 200 PFC, 180 PFC			●	●	●	●		●	●	●	
150 PFC			●	●	●	●		●			
125 PFC, 100 PFC, 75 PFC	●		●		●						
Universal Bearing Piles											
310 UBP, 200 UBP							By enquiry				
Equal Angles											
200 EA, 150 EA, 125 EA			●	●	●	●		●			
100 EA, 90 EA **	†	†	●		●						
75 EA, 65 EA, 55 EA, 50 EA **	†	†	●		†						
45 EA, 40EA, 30 EA, 25 EA	†	†	†		†						
Unequal Angles											
150 x 100 UA, 150 x 90 UA			●	●	●	●		●			
125 x 75 UA, 100 x 75 UA	†	†	†		†						
75 x 50 UA, 65 x 50 UA	†	†	†		†						

● The Section/Length combination is available in Standard Bundle configurations.

* By enquiry – delivery to capital cities only.

** Certain thicknesses may not be available in both lengths. Confirm availability with Liberty Steel.

† By enquiry.

Availability

Merchant Bar Sections

Rounds, Squares and Flats

Availability

Merchant bar rounds, squares and flats are available in a variety of steel grades and sizes.

Due to process limitations not all grades are available in all sizes. For new applications we recommend you confirm product availability with a Liberty Steel Sales Office at an early stage of design. Other specifications and sizes may also be available on enquiry.

Specifications

Merchant bar sections are available in the following standards:

- 300PLUS® and AS/NZS 3679.1 – Structural Steel – Hot rolled bars and sections.
- AS 1442 – Carbon Steels and Carbon Manganese Steels – Hot rolled bars and semifinished products.
- AS 1444 – Wrought Alloy Steels Standard, Hardenability (H) Series and Hardened and Tempered to Designated Mechanical Properties.
- AS 1447 – Hot-rolled spring steels.
- Liberty Steel grades (based on AISI-SAE nomenclature).

Table 3 Rounds – Size Availability and Mass

Diameter (mm)	Mass (kg/m)
10	0.616
12	0.887
13	1.04
14	1.21
15	1.39
16	1.58
17	1.78
18	1.99
19	2.23
20	2.46
22	2.98
24	3.55
27	4.49
30	5.55
33	6.71
36	7.99
39	9.38
42	10.9
45	12.5
48	14.2
50	15.4
56	19.3
60	22.2
65	26.0
75	34.7
90	49.9

Standard Length: 6 metres

Table 4 Squares – Size Availability and Mass

Thickness (mm)	Mass (kg/m)
10*	0.790
12	1.13
16	2.01
20	3.14
25	4.91
40	12.5

Standard Length: 6 metres

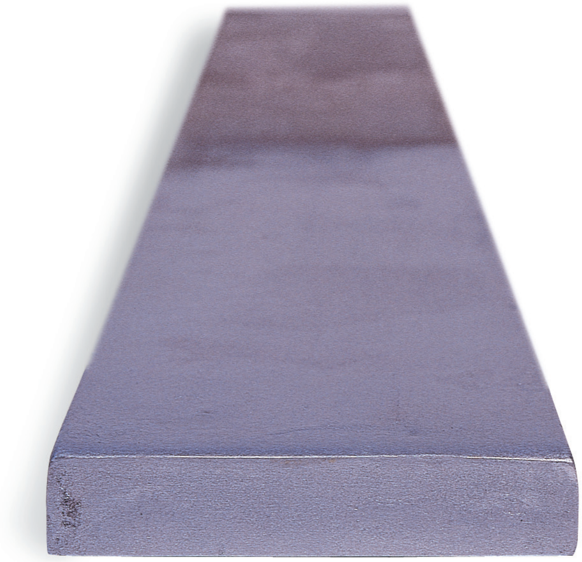
* Confirm availability.

Availability

Table 5 Flats – Size Availability and Mass (kg/m)

	Thickness (mm)							
Width (mm)	5	6	8	10	12	16	20	25
20				1.57				
25	0.981	1.18	1.57	1.96	2.36			
32	1.26	1.51	2.01	2.51	3.01			
40	1.57	1.88	2.51	3.14	3.77	5.02	6.28	
50	1.96	2.36	3.14	3.93	4.71	6.28	7.85	9.81
65	2.55	3.06	4.08	5.10	6.12	8.16	10.2	
75	2.94	3.53	4.71	5.89	7.07	9.42	11.8	14.7
90		4.24	5.65	7.07	8.48			
100	3.93	4.71	6.28	7.85	9.42	12.6	15.7	19.6
110				8.64				
130			8.16	10.2	12.2	16.3	20.4	25.5
150			9.42	11.8	14.1	18.8	23.6	29.4

Standard Length: 6 metres



Availability

Table 6 Merchant Bar Sections – Regular Grade

Steel Type	Standard	Grades Available
Structural Steels	Liberty Steel AS/NZS 3679.1	300PLUS® 350
Carbon and Carbon-Manganese Steels	AS 1442	1016 1022 1045
Spring Steels	AS 1447	XK5160S XK9258S XK9261S
Liberty Steel Grades	Liberty Steel	X4K92M61S

Note
Liberty Steel 300PLUS® exceeds the requirements of AS/NZS 3679.1 Grade 300.
Grade availability can vary with section.

Rods and Light Billets

Rods and light billets are available in a wide range of Liberty Steel grades, and selected grades from AS 1442, AS 1444 and AS 1447 specifications.

These sections are not available in structural grades 300PLUS® or 350 grade.

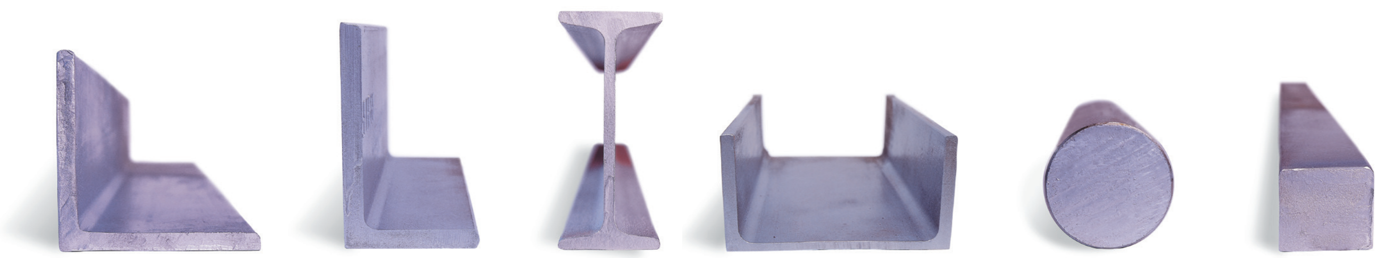
Due to process limitations not all grades are available in all sizes. Confirm product availability with a Liberty Steel Sales Office at an early stage of design.

Table 7 Rods – Size Availability

Diameter (mm)
5.5 6.5 7.0 8.0 9.0 10.0 11.2 12.5 13.0 14.0
15.0 16.0 17.0 18.0

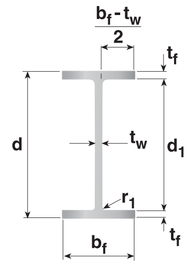
Table 8 Light Billets – Size Availability

Sizes Available (mm x mm)
45 x 45
50 x 50
63 x 63
75 x 75



Universal Beams

AUSTRALIAN MADE



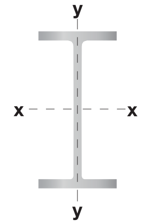


Table 9 Universal Beams – Dimensions and Properties

Designation	Depth of Section	Flange Width	Flange Thickness	Web Thickness	Root Radius	Depth Between Flanges	$\frac{d_1}{t_w}$	$\frac{(b_f-t_w)}{2t_f}$	Gross Area of Cross Section	About x-axis				About y-axis				Torsion Constant	Warping Constant	Designation
	d	b_f	t_f	t_w	r_1	d_1			A_g	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	J	I_w	
kg/m	mm	mm	mm	mm	mm	mm			mm^2	10^6mm^4	10^3mm^3	10^3mm^3	mm	10^6mm^4	10^3mm^3	10^3mm^3	mm	10^3mm^4	10^9mm^6	
610 UB 125	611.6	229.0	19.6	11.9	14.0	572.4	48.1	5.54	16000	986	3230	3680	249	39.3	343	536	49.6	1560	3450	610 UB 125
113	607.0	228.0	17.3	11.2	14.0	572.4	51.1	6.27	14500	875	2880	3290	246	34.3	300	469	48.7	1140	2980	113
101	602.0	228.0	14.8	10.6	14.0	572.4	54.0	7.34	13000	761	2530	2900	242	29.3	257	402	47.5	790	2530	101
530 UB 92.4	533.0	209.0	15.6	10.2	14.0	501.8	49.2	6.37	11800	554	2080	2370	217	23.8	228	355	44.9	775	1590	530 UB 92.4
82.0	528.2	209.0	13.2	9.6	14.0	501.8	52.3	7.55	10500	477	1810	2070	213	20.1	193	301	43.8	526	1330	82.0
460 UB 82.1	460.4	191.0	16.0	9.9	11.4	428.4	43.3	5.66	10500	372	1610	1840	188	18.6	195	303	42.2	701	919	460 UB 82.1
74.6	457.4	190.0	14.5	9.1	11.4	428.4	47.1	6.24	9520	335	1460	1660	188	16.6	175	271	41.8	530	815	74.6
67.1	453.8	190.0	12.7	8.5	11.4	428.4	50.4	7.15	8580	296	1300	1480	186	14.5	153	238	41.2	378	708	67.1
410 UB 59.7	406.4	178.0	12.8	7.8	11.4	380.8	48.8	6.65	7640	216	1060	1200	168	12.1	135	209	39.7	337	467	410 UB 59.7
53.7	402.6	178.0	10.9	7.6	11.4	380.8	50.1	7.82	6890	188	933	1060	165	10.3	115	179	38.6	234	394	53.7
360 UB 56.7	358.6	172.0	13.0	8.0	11.4	332.6	41.6	6.31	7240	161	899	1010	149	11.0	128	198	39.0	338	330	360 UB 56.7
50.7	355.6	171.0	11.5	7.3	11.4	332.6	45.6	7.12	6470	142	798	897	148	9.60	112	173	38.5	241	284	50.7
44.7	352.0	171.0	9.7	6.9	11.4	332.6	48.2	8.46	5720	121	689	777	146	8.10	94.7	146	37.6	161	237	44.7
310 UB 46.2	307.2	166.0	11.8	6.7	11.4	283.6	42.3	6.75	5930	100	654	729	130	9.01	109	166	39.0	233	197	310 UB 46.2
40.4	304.0	165.0	10.2	6.1	11.4	283.6	46.5	7.79	5210	86.4	569	633	129	7.65	92.7	142	38.3	157	165	40.4
32.0	298.0	149.0	8.0	5.5	13.0	282.0	51.3	8.97	4080	63.2	424	475	124	4.42	59.3	91.8	32.9	86.5	92.9	32.0
250 UB 37.3	256.2	146.0	10.9	6.4	8.9	234.4	36.6	6.40	4750	55.7	435	486	108	5.66	77.5	119	34.5	158	85.2	250 UB 37.3
31.4	251.6	146.0	8.6	6.1	8.9	234.4	38.4	8.13	4010	44.5	354	397	105	4.47	61.2	94.2	33.4	89.3	65.9	31.4
25.7	248.0	124.0	8.0	5.0	12.0	232.0	46.4	7.44	3270	35.4	285	319	104	2.55	41.1	63.6	27.9	67.4	36.7	25.7
200 UB 29.8	207.0	134.0	9.6	6.3	8.9	187.8	29.8	6.65	3820	29.1	281	316	87.3	3.86	57.5	88.4	31.8	105	37.6	200 UB 29.8
25.4	203.2	133.0	7.8	5.8	8.9	187.6	32.3	8.15	3230	23.6	232	260	85.4	3.06	46.1	70.9	30.8	62.7	29.2	25.4
22.3	201.6	133.0	7.0	5.0	8.9	187.6	37.5	9.14	2870	21.0	208	231	85.5	2.75	41.3	63.4	31.0	45.0	26.0	22.3
18.2	198.0	99.0	7.0	4.5	11.0	184.0	40.9	6.75	2320	15.8	160	180	82.6	1.14	23.0	35.7	22.1	38.6	10.4	18.2
180 UB 22.2	179.0	90.0	10.0	6.0	8.9	159.0	26.5	4.20	2820	15.3	171	195	73.6	1.22	27.1	42.3	20.8	81.6	8.71	180 UB 22.2
18.1	175.0	90.0	8.0	5.0	8.9	159.0	31.8	5.31	2300	12.1	139	157	72.6	0.975	21.7	33.7	20.6	44.8	6.80	18.1
16.1	173.0	90.0	7.0	4.5	8.9	159.0	35.3	6.11	2040	10.6	123	138	72.0	0.853	19.0	29.4	20.4	31.5	5.88	16.1
150 UB 18.0	155.0	75.0	9.5	6.0	8.0	136.0	22.7	3.63	2300	9.05	117	135	62.8	0.672	17.9	28.2	17.1	60.5	3.56	150 UB 18.0
14.0	150.0	75.0	7.0	5.0	8.0	136.0	27.2	5.00	1780	6.66	88.8	102	61.1	0.495	13.2	20.8	16.6	28.1	2.53	14.0

Table 10 Universal Beams – Properties for Assessing Section Capacity

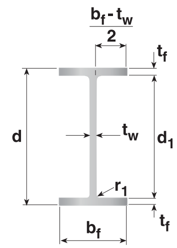
Designation	Yield Stress Flange f_y	Yield Stress Web f_y	Form Factor k_f	About x-axis Compactness	About x-axis Z_{ex}	About y-axis Compactness	About y-axis Z_{ey}	Yield Stress Flange f_y	Yield Stress Web f_y	Form Factor k_f	About x-axis Compactness	About x-axis Z_{ex}	About y-axis Compactness	About y-axis Z_{ey}	Designation
	MPa	MPa			10^3mm^3		10^3mm^3	MPa	MPa			10^3mm^3		10^3mm^3	
	300PLUS® *							AS/NZS 3679.1-350							
610 UB 125	280	300	0.950	C	3680	C	515	340	340	0.916	C	3680	C	515	610 UB 125
113	280	300	0.926	C	3290	C	451	340	340	0.891	C	3290	C	451	113
101	300	320	0.888	C	2900	C	386	340	360	0.867	C	2900	C	386	101
530 UB 92.4	300	320	0.928	C	2370	C	342	340	360	0.907	C	2370	C	342	530 UB 92.4
82.0	300	320	0.902	C	2070	C	289	340	360	0.880	C	2070	C	289	82.0
460 UB 82.1	300	320	0.979	C	1840	C	292	340	360	0.956	C	1840	C	292	460 UB 82.1
74.6	300	320	0.948	C	1660	C	262	340	360	0.926	C	1660	C	262	74.6
67.1	300	320	0.922	C	1480	C	230	340	360	0.901	C	1480	C	230	67.1
410 UB 59.7	300	320	0.938	C	1200	C	203	340	360	0.918	C	1200	C	203	410 UB 59.7
53.7	320	320	0.913	C	1060	C	173	360	360	0.894	N	1050	N	172	53.7
360 UB 56.7	300	320	0.996	C	1010	C	193	340	360	0.974	C	1010	C	193	360 UB 56.7
50.7	300	320	0.963	C	897	C	168	340	360	0.943	C	897	C	168	50.7
44.7	320	320	0.930	N	770	N	140	360	360	0.911	N	762	N	139	44.7
310 UB 46.2	300	320	0.991	C	729	C	163	340	360	0.972	C	729	C	163	310 UB 46.2
40.4	320	320	0.952	C	633	C	139	360	360	0.936	N	629	N	138	40.4
32.0	320	320	0.915	N	467	N	86.9	360	360	0.898	N	462	N	85.7	32.0
250 UB 37.3	320	320	1.00	C	486	C	116	360	360	1.00	C	486	C	116	250 UB 37.3
31.4	320	320	1.00	N	395	N	91.4	360	360	0.991	N	392	N	90.3	31.4
25.7	320	320	0.949	C	319	C	61.7	360	360	0.932	C	319	C	61.7	25.7
200 UB 29.8	320	320	1.00	C	316	C	86.3	360	360	1.00	C	316	C	86.3	200 UB 29.8
25.4	320	320	1.00	N	259	N	68.8	360	360	1.00	N	257	N	68.0	25.4
22.3	320	320	1.00	N	227	N	60.3	360	360	1.00	N	225	N	59.4	22.3
18.2	320	320	0.990	C	180	C	34.4	360	360	0.970	C	180	C	34.4	18.2
180 UB 22.2	320	320	1.00	C	195	C	40.7	360	360	1.00	C	195	C	40.7	180 UB 22.2
18.1	320	320	1.00	C	157	C	32.5	360	360	1.00	C	157	C	32.5	18.1
16.1	320	320	1.00	C	138	C	28.4	360	360	1.00	C	138	C	28.4	16.1
150 UB 18.0	320	320	1.00	C	135	C	26.9	360	360	1.00	C	135	C	26.9	150 UB 18.0
14.0	320	320	1.00	C	102	C	19.8	360	360	1.00	C	102	C	19.8	14.0

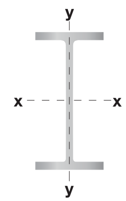
* 300PLUS® replaced Grade 250 as the base grade for these sections in 1994.
300PLUS® hot rolled sections are produced to exceed the minimum requirements of AS/NZS 3679.1-300.

Notes

1. For 300PLUS® sections the tensile strength (f_u) is 440 MPa.
2. For Grade 350 sections the tensile strength (f_u) is 480 MPa.
3. C: Compact Section; N: Non-compact Section; S: Slender Section.

AUSTRALIAN MADE





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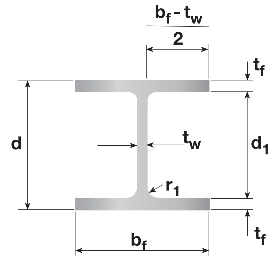
Universal Columns



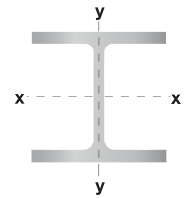


Table 11 Universal Columns – Dimensions and Properties

Designation	Depth of Section	Flange		Web Thickness	Root Radius	Depth Between Flanges	d_1	(b_f-t_w)	Gross Area of Cross Section	About x-axis				About y-axis				Torsion Constant	Warping Constant	Designation
		Width	Thickness																	
	d	b_f	t_f	t_w	r_1	d_1	t_w	$2t_f$	A_g	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	J	I_w	
kg/m	mm	mm	mm	mm	mm	mm			mm²	10⁶mm⁴	10³mm³	10³mm³	mm	10⁶mm⁴	10³mm³	10³mm³	mm	10³mm⁴	10⁹mm⁶	
310 UC 158	327.2	311.0	25.0	15.7	16.5	277.2	17.7	5.91	20100	388	2370	2680	139	125	807	1230	78.9	3810	2860	310 UC 158
137	320.6	309.0	21.7	13.8	16.5	277.2	20.1	6.80	17500	329	2050	2300	137	107	691	1050	78.2	2520	2390	137
118	314.6	307.0	18.7	11.9	16.5	277.2	23.3	7.89	15000	277	1760	1960	136	90.2	588	893	77.5	1630	1980	118
96.8	308.0	305.0	15.4	9.9	16.5	277.2	28.0	9.58	12400	223	1450	1600	134	72.9	478	725	76.7	928	1560	96.8
250 UC 89.5	260.0	256.0	17.3	10.5	14.0	225.4	21.5	7.10	11400	143	1100	1230	112	48.4	378	575	65.2	1040	713	250 UC 89.5
72.9	253.8	254.0	14.2	8.6	14.0	225.4	26.2	8.64	9320	114	897	992	111	38.8	306	463	64.5	586	557	72.9
200 UC 59.5	209.8	205.0	14.2	9.3	11.4	181.4	19.5	6.89	7620	61.3	584	656	89.7	20.4	199	303	51.7	477	195	200 UC 59.5
52.2	206.4	204.0	12.5	8.0	11.4	181.4	22.7	7.84	6660	52.8	512	570	89.1	17.7	174	264	51.5	325	166	52.2
46.2	203.4	203.0	11.0	7.3	11.4	181.4	24.8	8.90	5900	45.9	451	500	88.2	15.3	151	230	51.0	228	142	46.2
150 UC 37.2	161.8	154.0	11.5	8.1	8.9	138.8	17.1	6.34	4730	22.2	274	310	68.4	7.01	91.0	139	38.5	197	39.6	150 UC 37.2
30.0	157.6	153.0	9.4	6.6	8.9	138.8	21.0	7.79	3860	17.6	223	250	67.5	5.62	73.4	112	38.1	109	30.8	30.0
23.4	152.4	152.0	6.8	6.1	8.9	138.8	22.8	10.7	2980	12.6	166	184	65.1	3.98	52.4	80.2	36.6	50.2	21.1	23.4
100 UC 14.8	97.0	99.0	7.0	5.0	10.0	83.0	16.6	6.71	1890	3.18	65.6	74.4	41.1	1.14	22.9	35.2	24.5	34.9	2.30	100 UC 14.8

Universal Columns

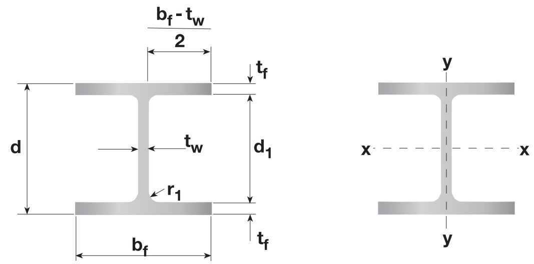
Table 12 Universal Columns – Properties for Assessing Section Capacity

Designation	Yield Stress		Form Factor	About x-axis		About y-axis		Yield Stress		Form Factor	About x-axis		About y-axis		Designation
	Flange	Web		Compactness		Compactness		Flange	Web		Compactness		Compactness		
	f_y	f_y	k_f		Z_{ex}		Z_{ey}	f_y	f_y	k_f		Z_{ex}		Z_{ey}	
	MPa	MPa			10^3mm^3		10^3mm^3	MPa	MPa			10^3mm^3		10^3mm^3	
			300PLUS® *								AS/NZS 3679.1-350				
310 UC 158	280	300	1.00	C	2680	C	1210	340	340	1.00	C	2680	C	1210	310 UC 158
137	280	300	1.00	C	2300	C	1040	340	340	1.00	C	2300	C	1040	137
118	280	300	1.00	C	1960	C	882	340	340	1.00	N	1950	N	878	118
96.8	300	320	1.00	N	1560	N	694	340	360	1.00	N	1550	N	684	96.8
250 UC 89.5	280	320	1.00	C	1230	C	567	340	360	1.00	C	1230	C	567	250 UC 89.5
72.9	300	320	1.00	N	986	N	454	340	360	1.00	N	977	N	448	72.9
200 UC 59.5	300	320	1.00	C	656	C	299	340	360	1.00	C	656	C	299	200 UC 59.5
52.2	300	320	1.00	C	570	C	260	340	360	1.00	N	569	N	260	52.2
46.2	300	320	1.00	N	494	N	223	340	360	1.00	N	490	N	219	46.2
150 UC 37.2	300	320	1.00	C	310	C	137	340	360	1.00	C	310	C	137	150 UC 37.2
30.0	320	320	1.00	C	250	C	110	360	360	1.00	N	248	N	109	30.0
23.4	320	320	1.00	N	176	N	73.5	360	360	1.00	N	174	N	72.3	23.4
100 UC 14.8	320	320	1.00	C	74.4	C	34.4	360	360	1.00	C	74.4	C	34.4	100 UC 14.8

* 300PLUS® replaced Grade 250 as the base grade for these sections in 1994.
300PLUS® hot rolled sections are produced to exceed the minimum requirements of AS/NZS 3679.1-300.

Notes

1. For 300PLUS® sections the tensile strength (f_u) is 440 MPa.
2. For Grade 350 sections the tensile strength (f_u) is 480 MPa.
3. C: Compact Section; N: Non-compact Section; S: Slender Section.



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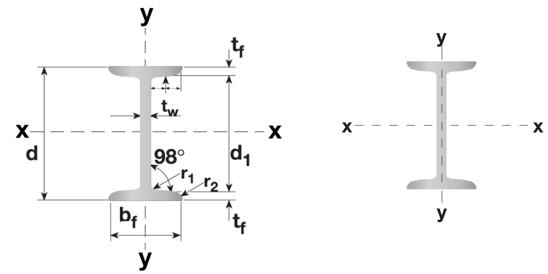


Tapered Flange Beams

Table 13 Tapered Flange Beams – Dimensions and Properties

Designation	Mass per metre	Depth of Section	Flange Width	Flange Thickness	Web Thickness	Radii Root	Radii Toe	Depth Between Flanges	$\frac{d_1}{t_w}$	$\frac{(b_f-t_w)}{2t_f}$	Gross Area of Cross Section	About x-axis				About y-axis				Torsion Constant	Warping Constant	Designation
		d	b_f	t_f	t_w	r_1	r_2	d_1			A_g	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	J	I_w	
	kg/m	mm	mm	mm	mm	mm	mm	mm			mm^2	10^6mm^4	10^3mm^3	10^3mm^3	mm	10^6mm^4	10^3mm^3	10^3mm^3	mm	10^3mm^4	10^9mm^6	
125 TFB	13.1	125	65.0	8.5	5.0	8.0	4.0	108	21.6	3.53	1670	4.34	69.4	80.3	50.9	0.337	10.4	17.2	14.2	40.2	1.14	125 TFB
100 TFB	7.20	100	45.0	6.0	4.0	7.0	3.0	88	22.0	3.42	917	1.46	29.2	34.1	39.9	0.0795	3.53	6.00	9.31	11.6	0.176	100 TFB

Table 14 Tapered Flange Beams – Properties for Assessing Section Capacity

Designation	Yield Stress Flange	Yield Stress Web	Form Factor	About x-axis Compactness	About x-axis	About y-axis Compactness	About y-axis	Yield Stress Flange	Yield Stress Web	Form Factor	About x-axis Compactness	About x-axis	About y-axis Compactness	About y-axis	Designation
	f_y	f_y	k_f		Z_{ex}		Z_{ey}	f_y	f_y	k_f		Z_{ex}		Z_{ey}	
	MPa	MPa			10^3mm^3		10^3mm^3	MPa	MPa			10^3mm^3		10^3mm^3	
	300PLUS® *							**AS/NZS 3679.1-350**							
125 TFB	320	320	1.00	C	80.3	C	15.6	360	360	1.00	C	80.3	C	15.6	125 TFB
100 TFB	320	320	1.00	C	34.1	C	5.30	360	360	1.00	C	34.1	C	5.30	100 TFB

* 300PLUS® replaced Grade 250 as the base grade for these sections in 1997.
300PLUS® hot rolled sections are produced to exceed the minimum requirements of AS/NZS 3679.1-300.

Notes

1. For 300PLUS® sections the tensile strength (f_u) is 430 MPa.
2. For Grade 350 sections the tensile strength (f_u) is 480 MPa.
3. C: Compact Section; N: Non-compact Section; S: Slender Section.

Parallel Flange Channels

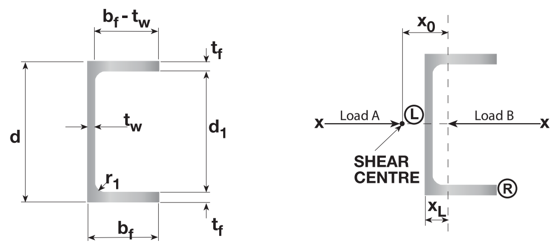
Table 15 Parallel Flange Channels – Dimensions and Properties

Designation	Mass per metre	Depth of Section	Flange Width	Flange Thickness	Web Thickness	Root Radius	Depth Between Flanges	$\frac{d_1}{t_w}$	$\frac{(b_f - t_w)}{t_f}$	Gross Area of Cross Section	Coordinate of Centroid	Coordinate of Shear Centre	About x-axis				About y-axis					Torsion Constant	Warping Constant	Designation
		d	b_f	t_f	t_w	r_1	d_1			A_g	X_L	X_O	I_x	Z_x	S_x	r_x	I_y	Z_{yR}	Z_{yL}	S_y	r_y	J	I_w	
	kg/m	mm	mm	mm	mm	mm	mm			mm²	mm	mm	10⁶mm⁴	10³mm³	10³mm³	mm	10⁶mm⁴	10³mm³	10³mm³	10³mm³	mm	10³mm⁴	10⁹mm⁶	
380 PFC	55.2	380	100	17.5	10.0	14.0	345	34.5	5.14	7030	27.5	56.7	152	798	946	147	6.48	89.4	236	161	30.4	491	151	380 PFC
300 PFC	40.1	300	90	16.0	8.0	14.0	268	33.5	5.13	5110	27.2	56.1	72.4	483	564	119	4.04	64.4	148	117	28.1	304	58.2	300 PFC
250 PFC	35.5	250	90	15.0	8.0	12.0	220	27.5	5.47	4520	28.6	58.5	45.1	361	421	99.9	3.64	59.3	127	107	28.4	248	35.9	250 PFC
230 PFC	25.1	230	75	12.0	6.5	12.0	206	31.7	5.71	3200	22.6	46.7	26.8	233	271	91.4	1.76	33.6	77.8	61.0	23.5	112	15.0	230 PFC
200 PFC	22.9	200	75	12.0	6.0	12.0	176	29.3	5.75	2920	24.4	50.5	19.1	191	221	80.9	1.65	32.7	67.8	58.9	23.8	105	10.6	200 PFC
180 PFC	20.9	180	75	11.0	6.0	12.0	158	26.3	6.27	2660	24.5	50.3	14.1	157	182	72.9	1.51	29.9	61.5	53.8	23.8	84.5	7.82	180 PFC
150 PFC	17.7	150	75	9.5	6.0	10.0	131	21.8	7.26	2250	24.9	51.0	8.34	111	129	60.8	1.29	25.7	51.6	46.0	23.9	56.6	4.59	150 PFC
125 PFC	11.9	125	65	7.5	4.7	8.0	110	23.4	8.04	1520	21.8	45.0	3.97	63.5	73.0	51.1	0.658	15.2	30.2	27.2	20.8	23.8	1.64	125 PFC
100 PFC	8.33	100	50	6.7	4.2	8.0	86.6	20.6	6.84	1060	16.7	33.9	1.74	34.7	40.3	40.4	0.267	8.01	16.0	14.4	15.9	13.6	0.424	100 PFC
75 PFC	5.92	75	40	6.1	3.8	8.0	62.8	16.5	5.95	754	13.7	27.2	0.683	18.2	21.4	30.1	0.120	4.56	8.71	8.20	12.6	8.42	0.106	75 PFC

Table 16 Parallel Flange Channels – Properties for Assessing Section Capacity

Designation	Yield Stress Flange	Yield Stress Web	Form Factor	About x-axis	About y-axis Load A	About y-axis Load B	Yield Stress Flange	Yield Stress Web	Form Factor	About x-axis	About y-axis Load A	About y-axis Load B	Designation
	f_y	f_y	k_f	Z_{ex}	Z_{ey}	Z_{ey}	f_y	f_y	k_f	Z_{ex}	Z_{ey}	Z_{ey}	
	MPa	MPa		10³mm³	10³mm³	10³mm³	MPa	MPa		10³mm³	10³mm³	10³mm³	
	300PLUS® *						**AS/NZS 3679.1-350**						
380 PFC	280	320	1.00	946	115	134	340	360	1.00	946	104	134	380 PFC
300 PFC	300	320	1.00	564	82.3	96.6	340	360	1.00	564	77.2	96.6	300 PFC
250 PFC	300	320	1.00	421	88.7	89.0	340	360	1.00	421	84.9	89.0	250 PFC
230 PFC	300	320	1.00	271	45.1	50.4	340	360	1.00	271	42.6	50.4	230 PFC
200 PFC	300	320	1.00	221	46.7	49.1	340	360	1.00	221	44.5	49.1	200 PFC
180 PFC	300	320	1.00	182	44.9	44.8	340	360	1.00	182	44.1	44.8	180 PFC
150 PFC	320	320	1.00	129	38.5	38.5	360	360	1.00	129	38.5	38.5	150 PFC
125 PFC	320	320	1.00	72.8	22.8	22.8	360	360	1.00	72.0	22.5	22.8	125 PFC
100 PFC	320	320	1.00	40.3	12.0	12.0	360	360	1.00	40.3	12.0	12.0	100 PFC
75 PFC	320	320	1.00	21.4	6.84	6.84	360	360	1.00	21.4	6.84	6.84	75 PFC

* 300PLUS® replaced Grade 250 as the base grade for these sections in 1994.
300PLUS® hot rolled sections are produced to exceed the minimum requirements of AS/NZS 3679.1-300.



Notes

1. For 300PLUS® sections the tensile strength (f_u) is 440 MPa.
2. For Grade 350 sections the tensile strength (f_u) is 480 MPa.
3. C: Compact Section; N: Non-compact Section; S: Slender Section. 3

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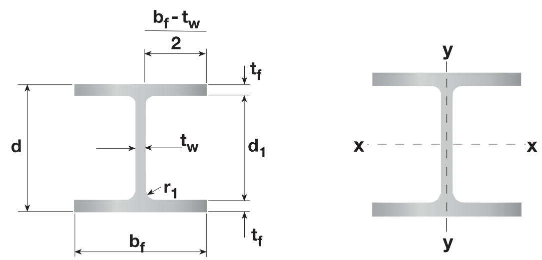


Universal Bearing Piles (refer Note 4)

Table 17 Universal Bearing Piles – Dimensions and Properties

Designation	Depth of Section	Flange Width	Flange Thickness	Web Thickness	Root Radius	Depth Between Flanges	$\frac{d_1}{t_w}$	$\frac{(b_f-t_w)}{2t_f}$	Gross Area of Cross Section	About x-axis				About y-axis				Torsion Constant	Warping Constant	Designation
	d	b_f	t_f	t_w	r_1	d_1			A_g	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	J	I_w	
kg/m	mm	mm	mm	mm	mm	mm			mm^2	10^6mm^4	10^3mm^3	10^3mm^3	mm	10^6mm^4	10^3mm^3	10^3mm^3	mm	10^3mm^4	10^9mm^6	
310UBP149	318	316	20.6	20.5	16.5	277	13.5	7.14	19000	330	2080	2370	132	109	691	1070	75.8	2970	2410	310UBP149
110	308	311	15.4	15.3	16.5	277	18.1	9.57	14000	236	1530	1720	130	76.6	494	759	73.9	1240	1640	110
78.8	299	306	11.1	11.1	16.5	277	24.9	13.3	10100	165	1100	1220	128	53.1	347	530	72.5	484	1100	78.8
200UBP122	230	220	25.0	25.0	11.4	180	7.20	3.90	15600	129	1120	1340	91.0	44.6	406	635	53.5	3540	469	200UBP122

Table 18 Universal Bearing Piles – Properties for Assessing Section Capacity

Designation	Yield Stress Flange f_y	Yield Stress Web f_y	Form Factor k_f	About x-axis Compactness	About x-axis Z_{ex}	About y-axis Compactness	About y-axis Z_{ey}	Yield Stress Flange f_y	Yield Stress Web f_y	Form Factor k_f	About x-axis Compactness	About x-axis Z_{ex}	About y-axis Compactness	About y-axis Z_{ey}	Designation
	MPa	MPa			10^3mm^3		10^3mm^3	MPa	MPa			10^3mm^3		10^3mm^3	
	300PLUS® *							AS/NZS 3679.1-350							
310 UBP 149	280	280	1.00	C	2370	C	1040	340	340	1.00	C	2370	C	1040	310 UBP 149
110	300	300	1.00	N	1680	N	718	340	340	1.00	N	1660	N	708	110
78.8	300	300	1.00	N	1130	N	460	340	340	1.00	N	1110	N	450	78.8
200 UBP 122	280	280	1.00	C	1340	C	609	340	340	1.00	C	1340	C	609	200 UBP 122

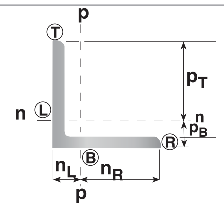
* 300PLUS® hot rolled sections are produced to exceed the minimum requirements of AS/NZS 3679.1-300.

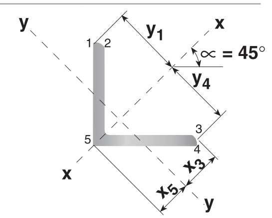
Notes

1. For 300PLUS® sections the tensile strength (f_u) is 440 MPa.
2. For Grade 350 sections the tensile strength (f_u) is 480 MPa.
3. C: Compact Section; N: Non-compact Section; S: Slender Section.
4. These sections are generally not stocked and are available for project orders only subject to enquiry from your nearest Liberty Steel Sales Office.

Table 19 Equal Angles – x-axis and y-axis – Dimensions and Properties

Designation	Nominal Thickness	Mass per metre	Actual Thickness	Radii			Gross Area of Cross Section	Coordinate of Centroid		About x-axis					About y-axis							Torsion Constant	Designation
Leg-size				Root	Toe	$(b_1\text{-}t)$		$n_L=$	$n_R=$		$y_1=$	$Z_{x1}=$										
$b_1 \times b_1$			t	r_1	r_2	t	A_g	p_B	p_T	I_x	y4	Z_{x4}	S_x	r_x	I_y	x_3	Z_{y3}	x_5	Z_{y5}	S_y	r_y	J	
mm mm mm		kg/m	mm	mm	mm		mm^2	mm	mm	10^6mm^4	mm	10^3mm^3	10^3mm^3	mm	10^6mm^4	mm	10^3mm^3	mm	10^3mm^3	10^3mm^3	mm	10^3mm^4	
200 x 200 x 26 EA		76.8	26.0	18.0	5.0	6.69	9780	59.3	141	56.8	141	402	643	76.2	14.9	73.9	202	83.8	178	329	39.0	2250	200 x 200 x 26 EA
20 EA		60.1	20.0	18.0	5.0	9.00	7660	57.0	143	45.7	141	323	511	77.2	11.8	72.9	162	80.6	147	260	39.3	1060	20 EA
18 EA		54.4	18.0	18.0	5.0	10.1	6930	56.2	144	41.7	141	295	464	77.6	10.8	72.6	149	79.5	136	236	39.4	778	18 EA
16 EA		48.7	16.0	18.0	5.0	11.5	6200	55.4	145	37.6	141	266	417	77.9	9.72	72.3	135	78.4	124	212	39.6	554	16 EA
13 EA		40.0	13.0	18.0	5.0	14.4	5090	54.2	146	31.2	141	221	344	78.3	8.08	71.9	112	76.6	105	176	39.8	304	13 EA
150 x 150 x 19 EA		42.1	19.0	13.0	5.0	6.89	5360	44.2	106	17.6	106	166	265	57.2	4.60	54.9	83.8	62.6	73.5	135	29.3	657	150 x 150 x 19 EA
16 EA		35.4	15.8	13.0	5.0	8.49	4520	43.0	107	15.1	106	142	225	57.8	3.91	54.3	71.9	60.8	64.2	115	29.4	386	16 EA
12 EA		27.3	12.0	13.0	5.0	11.5	3480	41.5	108	11.9	106	112	175	58.4	3.06	53.7	56.9	58.7	52.1	89.3	29.6	174	12 EA
10 EA		21.9	9.5	13.0	5.0	14.8	2790	40.5	109	9.61	106	90.6	141	58.7	2.48	53.4	46.4	57.3	43.3	72.0	29.8	88.9	10 EA
125 x 125 x 16 EA		29.1	15.8	10.0	5.0	6.91	3710	36.8	88.2	8.43	88.4	95.4	153	47.7	2.20	45.4	48.5	52.1	42.3	77.8	24.4	313	125 x 125 x 16 EA
12 EA		22.5	12.0	10.0	5.0	9.42	2870	35.4	89.6	6.69	88.4	75.7	120	48.3	1.73	44.7	38.6	50.1	34.5	60.8	24.5	141	12 EA
10 EA		18.0	9.5	10.0	5.0	12.2	2300	34.4	90.6	5.44	88.4	61.6	96.5	48.7	1.40	44.4	31.5	48.7	28.8	49.0	24.7	71.9	10 EA
8 EA		14.9	7.8	10.0	5.0	15.0	1900	33.7	91.3	4.55	88.4	51.5	80.2	48.9	1.17	44.2	26.5	47.7	24.5	40.8	24.8	40.6	8 EA
100 x 100 x 12 EA		17.7	12.0	8.0	5.0	7.33	2260	29.2	70.8	3.29	70.7	46.6	74.5	38.2	0.857	35.8	23.9	41.3	20.8	37.9	19.5	110	100 x 100 x 12 EA
10 EA		14.2	9.5	8.0	5.0	9.53	1810	28.2	71.8	2.70	70.7	38.2	60.4	38.6	0.695	35.4	19.6	39.9	17.4	30.7	19.6	56.2	10 EA
8 EA		11.8	7.8	8.0	5.0	11.8	1500	27.5	72.5	2.27	70.7	32.0	50.3	38.8	0.582	35.2	16.5	38.9	14.9	25.6	19.7	31.7	8 EA
6 EA		9.16	6.0	8.0	5.0	15.7	1170	26.8	73.2	1.78	70.7	25.2	39.3	39.1	0.458	35.0	13.1	37.9	12.1	20.0	19.8	14.8	6 EA
90 x 90 x 10 EA		12.7	9.5	8.0	5.0	8.47	1620	25.7	64.3	1.93	63.6	30.4	48.3	34.5	0.500	31.9	15.7	36.4	13.8	24.6	17.6	50.5	90 x 90 x 10 EA
8 EA		10.6	7.8	8.0	5.0	10.5	1350	25.0	65.0	1.63	63.6	25.6	40.4	34.8	0.419	31.7	13.2	35.4	11.8	20.5	17.6	28.6	8 EA
6 EA		8.22	6.0	8.0	5.0	14.0	1050	24.3	65.7	1.28	63.6	20.1	31.6	35.0	0.330	31.5	10.5	34.3	9.62	16.1	17.8	13.4	6 EA
75 x 75 x 10 EA		10.5	9.5	8.0	5.0	6.89	1340	22.0	53.0	1.08	53.0	20.4	32.8	28.4	0.282	26.6	10.6	31.1	9.09	16.8	14.5	41.9	75 x 75 x 10 EA
8 EA		8.73	7.8	8.0	5.0	8.62	1110	21.3	53.7	0.913	53.0	17.2	27.5	28.7	0.237	26.4	8.99	30.1	7.87	14.0	14.6	23.8	8 EA
6 EA		6.81	6.0	8.0	5.0	11.5	867	20.5	54.5	0.722	53.0	13.6	21.6	28.9	0.187	26.2	7.15	29.0	6.44	11.0	14.7	11.2	6 EA
5 EA		5.27	4.6	8.0	5.0	15.3	672	19.9	55.1	0.563	53.0	10.6	16.7	29.0	0.147	26.1	5.62	28.1	5.22	8.61	14.8	5.28	5 EA
65 x 65 x 10 EA		9.02	9.5	6.0	3.0	5.84	1150	19.6	45.4	0.691	46.0	15.0	24.3	24.5	0.183	23.7	7.71	27.7	6.60	12.5	12.6	35.1	65 x 65 x 10 EA
8 EA		7.51	7.8	6.0	3.0	7.33	957	19.0	46.0	0.589	46.0	12.8	20.5	24.8	0.154	23.4	6.56	26.8	5.73	10.5	12.7	20.0	8 EA
6 EA		5.87	6.0	6.0	3.0	9.83	748	18.3	46.7	0.471	46.0	10.2	16.2	25.1	0.122	23.1	5.26	25.8	4.71	8.25	12.8	9.37	6 EA
5 EA		4.56	4.6	6.0	3.0	13.1	581	17.7	47.3	0.371	46.0	8.08	12.7	25.3	0.0959	23.0	4.18	25.0	3.83	6.46	12.9	4.36	5 EA
55 x 55 x 6 EA		4.93	6.0	6.0	3.0	8.17	628	15.8	39.2	0.278	38.9	7.14	11.4	21.0	0.0723	19.6	3.69	22.3	3.24	5.82	10.7	7.93	55 x 55 x 6 EA
5 EA		3.84	4.6	6.0	3.0	11.0	489	15.2	39.8	0.220	38.9	5.66	8.93	21.2	0.0571	19.4	2.94	21.5	2.66	4.57	10.8	3.71	5 EA
50 x 50 x 8 EA		5.68	7.8	6.0	3.0	5.41	723	15.2	34.8	0.253	35.4	7.16	11.7	18.7	0.0675	18.1	3.73	21.5	3.14	6.00	9.66	15.2	50 x 50 x 8 EA
6 EA		4.46	6.0	6.0	3.0	7.33	568	14.5	35.5	0.205	35.4	5.79	9.30	19.0	0.0536	17.8	3.01	20.5	2.61	4.76	9.71	7.21	6 EA
5 EA		3.48	4.6	6.0	3.0	9.87	443	13.9	36.1	0.163	35.4	4.61	7.32	19.2	0.0424	17.6	2.40	19.7	2.15	3.75	9.78	3.38	5 EA
3 EA		2.31	3.0	6.0	3.0	15.7	295	13.2	36.8	0.110	35.4	3.11	4.90	19.3	0.0289	17.6	1.65	18.7	1.55	2.53	9.90	1.01	3 EA
45 x 45 x 6 EA		3.97	6.0	5.0	3.0	6.50	506	13.3	31.7	0.146	31.8	4.59	7.41	17.0	0.0383	16.0	2.39	18.8	2.04	3.79	8.71	6.32	45 x 45 x 6 EA
5 EA		3.10	4.6	5.0	3.0	8.78	394	12.7	32.3	0.117	31.8	3.66	5.84	17.2	0.0303	15.8	1.91	18.0	1.68	2.99	8.76	2.96	5 EA
3 EA		2.06	3.0	5.0	3.0	14.0	263	12.0	33.0	0.0790	31.8	2.48	3.92	17.3	0.0206	15.7	1.31	17.0	1.21	2.02	8.85	0.875	3 EA
40 x 40 x 6 EA		3.50	6.0	5.0	3.0	5.67	446	12.0	28.0	0.0997	28.3	3.53	5.75	15.0	0.0265	14.3	1.86	17.0	1.55	2.95	7.71	5.60	40 x 40 x 6 EA
5 EA		2.73	4.6	5.0	3.0	7.70	348	11.5	28.5	0.0801	28.3	2.83	4.55	15.2	0.0209	14.0	1.49	16.2	1.29	2.33	7.75	2.63	5 EA
3 EA		1.83	3.0	5.0	3.0	12.3	233	10.8	29.2	0.0545	28.3	1.93	3.06	15.3	0.0142	13.9	1.02	15.3	0.933	1.58	7.82	0.785	3 EA
30 x 30 x 6 EA		2.56	6.0	5.0	3.0	4.00	326	9.53	20.5	0.0387	21.2	1.83	3.06	10.9	0.0107	10.7	0.993	13.5	0.790	1.59	5.72	4.16	30 x 30 x 6 EA
5 EA		2.01	4.6	5.0	3.0	5.52	256	8.99	21.0	0.0316	21.2	1.49	2.45	11.1	0.00839	10.5	0.799	12.7	0.660	1.26	5.72	1.98	5 EA
3 EA		1.35	3.0	5.0	3.0	9.00	173	8.30	21.7	0.0218	21.2	1.03	1.67	11.2	0.00573	10.3	0.554	11.7	0.488	0.862	5.76	0.605	3 EA
25 x 25 x 6 EA		2.08	6.0	5.0	3.0	3.17	266	8.28	16.7	0.0210	17.7	1.19	2.03	8.89	0.00600	8.97	0.669	11.7	0.513	1.07	4.75	3.44	25 x 25 x 6 EA
5 EA		1.65	4.6	5.0	3.0	4.43	210	7.75	17.3	0.0173	17.7	0.980	1.65	9.07	0.00469	8.73	0.537	11.0	0.428	0.849	4.72	1.66	5 EA
3 EA		1.12	3.0	5.0	3.0	7.33	143	7.07	17.9	0.0121	17.7	0.685	1.13	9.22	0.00319	8.56	0.373	9.99	0.319	0.583	4.73	0.515	3 EA





Equal Angles

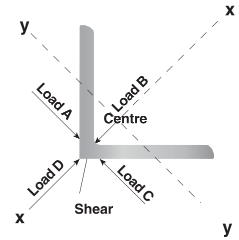


Table 20 Equal Angles – x-axis and y-axis – Properties for Assessing Section Capacity

Designation	Yield Stress	Form Factor	About x-axis	About y-axis		Yield Stress	Form Factor	About x-axis	About y-axis		Designation
			Load A or C	Load B	Load D			Load A or C	Load B	Load D	
	f_y	k_f	Z_{ex}	Z_{ey}	Z_{ey}	f_y	k_f	Z_{ex}	Z_{ey}	Z_{ey}	
mm mm mm	MPa		10^3mm^3	10^3mm^3	10^3mm^3	MPa		10^3mm^3	10^3mm^3	10^3mm^3	
	300PLUS® *					**AS/NZS 3679.1-350**					
200 x 200 x 26 EA	280	1.00	602	267	267	340	1.00	602	267	267	200 x 200 x 26 EA
20 EA	280	1.00	479	218	220	340	1.00	469	214	220	20 EA
18 EA	280	1.00	427	196	204	340	1.00	417	192	204	18 EA
16 EA	300	1.00	369	172	186	340	1.00	362	169	186	16 EA
13 EA	300	1.00	285	136	158	340	0.956	278	132	158	13 EA
150 x 150 x 19 EA	280	1.00	248	110	110	340	1.00	248	110	110	150 x 150 x 19 EA
16 EA	300	1.00	212	95.7	96.3	340	1.00	209	94.5	96.3	16 EA
12 EA	300	1.00	155	72.3	78.1	340	1.00	152	70.9	78.1	12 EA
10 EA	320	0.958	114	54.5	64.9	360	0.906	111	53.1	64.9	10 EA
125 x 125 x 16 EA	300	1.00	143	63.4	63.4	340	1.00	143	63.4	63.4	125 x 125 x 16 EA
12 EA	300	1.00	110	50.3	51.7	340	1.00	109	49.6	51.7	12 EA
10 EA	320	1.00	83.2	38.9	43.1	360	1.00	81.6	38.1	43.1	10 EA
8 EA	320	0.943	64.3	30.7	36.8	360	0.892	62.7	29.9	36.8	8 EA
100 x 100 x 12 EA	300	1.00	69.9	31.1	31.1	340	1.00	69.9	31.1	31.1	100 x 100 x 12 EA
10 EA	320	1.00	55.1	25.2	26.1	360	1.00	54.4	24.8	26.1	10 EA
8 EA	320	1.00	43.7	20.4	22.4	360	1.00	42.9	20.0	22.4	8 EA
6 EA	320	0.906	30.9	14.8	18.1	360	0.856	30.0	14.4	18.1	6 EA
90 x 90 x 10 EA	320	1.00	45.0	20.4	20.6	360	1.00	44.5	20.1	20.6	90 x 90 x 10 EA
8 EA	320	1.00	36.0	16.7	17.8	360	1.00	35.4	16.4	17.8	8 EA
6 EA	320	1.00	25.9	12.4	14.4	360	0.954	25.3	12.1	14.4	6 EA
75 x 75 x 10 EA	320	1.00	30.5	13.6	13.6	360	1.00	30.5	13.6	13.6	75 x 75 x 10 EA
8 EA	320	1.00	25.4	11.6	11.8	360	1.00	25.1	11.5	11.8	8 EA
6 EA	320	1.00	18.7	8.85	9.66	360	1.00	18.4	8.70	9.66	6 EA
5 EA	320	0.927	13.2	6.47	7.82	360	0.876	12.8	6.30	7.82	5 EA
65 x 65 x 10 EA	320	1.00	22.5	9.90	9.90	360	1.00	22.5	9.90	9.90	65 x 65 x 10 EA
8 EA	320	1.00	19.2	8.59	8.59	360	1.00	19.2	8.59	8.59	8 EA
6 EA	320	1.00	14.7	6.76	7.07	360	1.00	14.5	6.66	7.07	6 EA
5 EA	320	1.00	10.6	5.05	5.75	360	1.00	10.4	4.94	5.75	5 EA
55 x 55 x 6 EA	320	1.00	10.7	4.84	4.86	360	1.00	10.5	4.78	4.86	55 x 55 x 6 EA
5 EA	320	1.00	7.88	3.70	3.98	360	1.00	7.75	3.64	3.98	5 EA
50 x 50 x 8 EA	320	1.00	10.7	4.71	4.71	360	1.00	10.7	4.71	4.71	50 x 50 x 8 EA
6 EA	320	1.00	8.69	3.92	3.92	360	1.00	8.69	3.92	3.92	6 EA
5 EA	320	1.00	6.60	3.08	3.22	360	1.00	6.50	3.03	3.22	5 EA
3 EA	320	0.907	3.82	1.90	2.32	360	0.858	3.71	1.85	2.32	3 EA
45 x 45 x 6 EA	320	1.00	6.88	3.06	3.06	360	1.00	6.88	3.06	3.06	45 x 45 x 6 EA
5 EA	320	1.00	5.39	2.47	2.52	360	1.00	5.32	2.44	2.52	5 EA
3 EA	320	1.00	3.19	1.55	1.81	360	0.954	3.12	1.52	1.81	3 EA
40 x 40 x 6 EA	320	1.00	5.29	2.33	2.33	360	1.00	5.29	2.33	2.33	40 x 40 x 6 EA
5 EA	320	1.00	4.25	1.93	1.93	360	1.00	4.22	1.92	1.93	5 EA
3 EA	320	1.00	2.59	1.25	1.40	360	1.00	2.54	1.23	1.40	3 EA
30 x 30 x 6 EA	320	1.00	2.74	1.19	1.19	360	1.00	2.74	1.19	1.19	30 x 30 x 6 EA
5 EA	320	1.00	2.23	0.990	0.990	360	1.00	2.23	0.990	0.990	5 EA
3 EA	320	1.00	1.50	0.714	0.732	360	1.00	1.48	0.705	0.732	3 EA
25 x 25 x 6 EA	320	1.00	1.78	0.769	0.769	360	1.00	1.78	0.769	0.769	25 x 25 x 6 EA
5 EA	320	1.00	1.47	0.642	0.642	360	1.00	1.47	0.642	0.642	5 EA
3 EA	320	1.00	1.03	0.479	0.479	360	1.00	1.03	0.479	0.479	3 EA

* 300PLUS® replaced Grade 250 as the base grade for 125 x 125 x 8 equal angles and larger in 1994.
300PLUS® replaced Grade 250 as the base grade for 100 x 100 x 12 equal angles and smaller in 1997.
300PLUS® hot rolled sections are produced to exceed the minimum requirements of AS/NZS 3679.1-300.

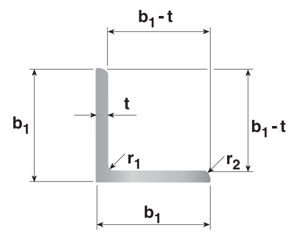
Notes

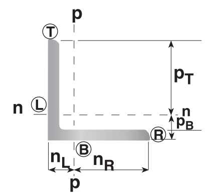
1. For 300PLUS® sections the tensile strength (fu) is 440 MPa.
2. For Grade 350 sections the tensile strength (fu) is 480 MPa.

Equal Angles

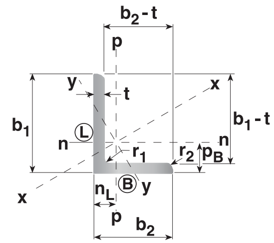
Table 21 Equal Angles – n-axis and p-axis – Properties

Designation	About n-axis and p-axis							Product of 2nd Moment of Area	Designation
	$I_n=I_p$	$n_L=p_B$	$Z_{nB}=Z_{pL}$	$n_R=p_T$	$Z_{nT}=Z_{pR}$	$S_n=S_p$	$r_n=r_p$	I_{np}	
mm mm mm	10^6mm^4	mm	10^3mm^3	mm	10^3mm^3	10^3mm^3	mm	10^6mm^4	
200 x 200 x 26 EA	35.8	59.3	605	141	255	460	60.5	-20.9	200 x 200 x 26 EA
20 EA	28.8	57.0	505	143	201	363	61.3	-16.9	20 EA
18 EA	26.3	56.2	467	144	183	330	61.5	-15.5	18 EA
16 EA	23.7	55.4	427	145	164	296	61.8	-14.0	16 EA
13 EA	19.7	54.2	363	146	135	243	62.2	-11.6	13 EA
150 x 150 x 19 EA	11.1	44.2	250	106	105	189	45.4	-6.48	150 x 150 x 19 EA
16 EA	9.48	43.0	220	107	88.7	160	45.8	-5.58	16 EA
12 EA	7.46	41.5	180	108	68.8	124	46.3	-4.40	12 EA
10 EA	6.04	40.5	149	109	55.2	99.9	46.6	-3.56	10 EA
125 x 125 x 16 EA	5.32	36.8	144	88.2	60.3	109	37.9	-3.11	125 x 125 x 16 EA
12 EA	4.21	35.4	119	89.6	47.0	85.0	38.3	-2.48	12 EA
10 EA	3.42	34.4	99.4	90.6	37.8	68.4	38.6	-2.02	10 EA
8 EA	2.86	33.7	84.9	91.3	31.3	56.8	38.8	-1.69	8 EA
100 x 100 x 12 EA	2.08	29.2	71.1	70.8	29.3	53.2	30.3	-1.22	100 x 100 x 12 EA
10 EA	1.70	28.2	60.1	71.8	23.6	42.9	30.6	-1.00	10 EA
8 EA	1.42	27.5	51.7	72.5	19.6	35.7	30.8	-0.842	8 EA
6 EA	1.12	26.8	41.8	73.2	15.3	27.8	31.0	-0.661	6 EA
90 x 90 x 10 EA	1.22	25.7	47.3	64.3	18.9	34.4	27.4	-0.716	90 x 90 x 10 EA
8 EA	1.02	25.0	40.9	65.0	15.7	28.7	27.6	-0.604	8 EA
6 EA	0.805	24.3	33.2	65.7	12.3	22.4	27.7	-0.475	6 EA
75 x 75 x 10 EA	0.681	22.0	31.0	53.0	12.8	23.4	22.6	-0.399	75 x 75 x 10 EA
8 EA	0.575	21.3	27.0	53.7	10.7	19.6	22.7	-0.338	8 EA
6 EA	0.455	20.5	22.1	54.5	8.35	15.3	22.9	-0.268	6 EA
5 EA	0.355	19.9	17.9	55.1	6.44	11.8	23.0	-0.208	5 EA
65 x 65 x 10 EA	0.437	19.6	22.3	45.4	9.62	17.4	19.5	-0.254	65 x 65 x 10 EA
8 EA	0.371	19.0	19.6	46.0	8.07	14.6	19.7	-0.218	8 EA
6 EA	0.296	18.3	16.2	46.7	6.34	11.5	19.9	-0.175	6 EA
5 EA	0.234	17.7	13.2	47.3	4.94	8.97	20.1	-0.138	5 EA
55 x 55 x 6 EA	0.175	15.8	11.1	39.2	4.46	8.11	16.7	-0.103	55 x 55 x 6 EA
5 EA	0.139	15.2	9.12	39.8	3.48	6.34	16.8	-0.0814	5 EA
50 x 50 x 8 EA	0.160	15.2	10.5	34.8	4.61	8.38	14.9	-0.0928	50 x 50 x 8 EA
6 EA	0.129	14.5	8.90	35.5	3.64	6.63	15.1	-0.0756	6 EA
5 EA	0.103	13.9	7.36	36.1	2.85	5.19	15.2	-0.0602	5 EA
3 EA	0.0694	13.2	5.25	36.8	1.89	3.46	15.3	-0.0405	3 EA
45 x 45 x 6 EA	0.0922	13.3	6.93	31.7	2.91	5.30	13.5	-0.0538	45 x 45 x 6 EA
5 EA	0.0734	12.7	5.76	32.3	2.28	4.16	13.6	-0.0432	5 EA
3 EA	0.0498	12.0	4.14	33.0	1.51	2.77	13.8	-0.0292	3 EA
40 x 40 x 6 EA	0.0631	12.0	5.24	28.0	2.26	4.12	11.9	-0.0366	40 x 40 x 6 EA
5 EA	0.0505	11.5	4.39	28.5	1.77	3.24	12.0	-0.0296	5 EA
3 EA	0.0344	10.8	3.19	29.2	1.18	2.17	12.2	-0.0201	3 EA
30 x 30 x 6 EA	0.0247	9.53	2.59	20.5	1.21	2.22	8.71	-0.0140	30 x 30 x 6 EA
5 EA	0.0200	8.99	2.22	21.0	0.951	1.76	8.83	-0.0116	5 EA
3 EA	0.0138	8.30	1.66	21.7	0.635	1.18	8.93	-0.00804	3 EA
25 x 25 x 6 EA	0.0135	8.28	1.63	16.7	0.807	1.49	7.13	-0.00750	25 x 25 x 6 EA
5 EA	0.0110	7.75	1.42	17.3	0.638	1.19	7.23	-0.00632	5 EA
3 EA	0.00765	7.07	1.08	17.9	0.426	0.802	7.33	-0.00446	3 EA





Unequal Angles



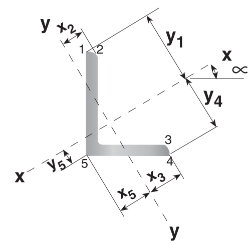


Table 22 Unequal Angles – x-axis and y-axis – Dimensions and Properties

Designation: Leg-size, Nominal Thickness $b_1 \times b_2$	Mass p/ metre	Actual Thickness t	Radii: Root r_1	Radii: Toe r_2	$\frac{b_1-t}{t}$	$\frac{b_2-t}{t}$	Gross Area of Cross Section A_g	Coordinate of Centroid p_B	Coordinate of Centroid n_L	About x-axis I_x	y_1	Z_{x1}	y_4	Z_{x4}	y_5	Z_{x5}	S_x	r_x	About y-axis I_y	x_2	Z_{y2}	x_3	Z_{y3}	x_5	Z_{y5}	S_y	r_y	Torsion Constant J	Tan Alpha	Designation
mm mm mm	kg/m	mm	mm	mm			mm^2	mm	mm	10^6mm^4	mm	10^3mm^3	mm	10^3mm^3	mm	10^3mm^3	10^3mm^3	mm	10^6mm^4	mm	10^3mm^3	mm	10^3mm^3	mm	10^3mm^3	10^3mm^3	mm	10^3mm^4		
150 x 100 x 12 UA	22.5	12.0	10.0	5.0	11.5	7.33	2870	49.1	24.3	7.51	102	73.5	75.3	99.7	35.2	213	127	51.2	1.35	27.6	48.8	52.9	25.5	42.0	32.1	51.7	21.7	141	0.438	150 x 100 x 12 UA
10 UA	18.0	9.5	10.0	5.0	14.8	9.53	2300	48.1	23.3	6.11	103	59.5	74.9	81.5	34.6	177	102	51.6	1.09	26.9	40.7	53.0	20.6	40.7	26.9	41.8	21.8	71.9	0.441	10 UA
150 x 90 x 16 UA	27.9	15.8	10.0	5.0	8.49	4.70	3550	52.5	22.7	8.80	99.5	88.4	71.9	122	41.9	210	154	49.8	1.32	24.6	53.8	49.9	26.5	38.9	34.0	55.9	19.3	300	0.353	150 x 90 x 16 UA
12 UA	21.6	12.0	10.0	5.0	11.5	6.50	2750	51.0	21.2	6.97	100	69.4	71.3	97.8	40.8	171	120	50.4	1.04	23.4	44.5	50.1	20.8	37.2	28.0	43.8	19.5	136	0.360	12 UA
10 UA	17.3	9.5	10.0	5.0	14.8	8.47	2200	50.0	20.2	5.66	101	56.1	70.7	80.1	40.1	141	96.6	50.7	0.847	22.6	37.4	50.4	16.8	36.1	23.5	35.4	19.6	69.0	0.363	10 UA
8 UA	14.3	7.8	10.0	5.0	18.2	10.5	1820	49.2	19.6	4.73	101	46.7	70.3	67.3	39.5	120	80.1	51.0	0.710	22.1	32.2	50.6	14.0	35.2	20.2	29.5	19.7	39.0	0.364	8 UA
125 x 75 x 12 UA	17.7	12.0	8.0	5.0	9.42	5.25	2260	43.3	18.4	3.91	83.2	47.0	59.7	65.5	34.6	113	81.4	41.6	0.585	19.9	29.3	41.4	14.1	31.9	18.4	29.7	16.1	110	0.356	125 x 75 x 12 UA
10 UA	14.2	9.5	8.0	5.0	12.2	6.89	1810	42.3	17.5	3.20	83.8	38.2	59.3	53.9	33.9	94.4	65.8	42.0	0.476	19.2	24.9	41.6	11.4	30.7	15.5	24.1	16.2	56.2	0.360	10 UA
8 UA	11.8	7.8	8.0	5.0	15.0	8.62	1500	41.5	16.8	2.68	84.2	31.8	58.9	45.5	33.3	80.4	54.6	42.2	0.399	18.6	21.5	41.8	9.55	29.9	13.3	20.1	16.3	31.7	0.363	8 UA
6 UA	9.16	6.0	8.0	5.0	19.8	11.5	1170	40.7	16.0	2.10	84.7	24.8	58.5	36.0	32.8	64.1	42.4	42.5	0.315	18.0	17.5	42.1	7.47	29.0	10.8	15.7	16.4	14.8	0.364	6 UA
100 x 75 x 10 UA	12.4	9.5	8.0	5.0	9.53	6.89	1580	31.8	19.4	1.89	69.2	27.3	54.5	34.6	18.6	101	46.5	34.6	0.401	22.3	18.0	36.4	11.0	32.2	12.5	21.2	16.0	49.1	0.546	100 x 75 x 10 UA
8 UA	10.3	7.8	8.0	5.0	11.8	8.62	1310	31.1	18.7	1.59	69.4	22.9	54.3	29.2	18.2	87.0	38.7	34.8	0.337	21.8	15.4	36.4	9.26	31.3	10.7	17.8	16.0	27.8	0.549	8 UA
6 UA	7.98	6.0	8.0	5.0	15.7	11.5	1020	30.3	17.9	1.25	69.7	17.9	54.0	23.1	17.9	70.0	30.1	35.1	0.265	21.4	12.4	36.5	7.27	30.3	8.75	13.9	16.2	13.0	0.551	6 UA
75 x 50 x 8 UA	7.23	7.8	7.0	3.0	8.62	5.41	921	25.2	12.8	0.586	50.8	11.5	37.8	15.5	18.0	32.5	20.0	25.2	0.106	14.2	7.46	26.4	4.01	21.7	4.88	8.19	10.7	19.5	0.430	75 x 50 x 8 UA
6 UA	5.66	6.0	7.0	3.0	11.5	7.33	721	24.4	12.1	0.468	51.2	9.15	37.5	12.5	17.6	26.7	15.8	25.5	0.0842	13.6	6.17	26.5	3.18	20.8	4.04	6.48	10.8	9.21	0.435	6 UA
5 UA	4.40	4.6	7.0	3.0	15.3	9.87	560	23.8	11.5	0.370	51.5	7.17	37.2	9.93	17.2	21.5	12.3	25.7	0.0666	13.2	5.03	26.6	2.50	20.1	3.32	5.09	10.9	4.32	0.437	5 UA
65 x 50 x 8 UA	6.59	7.8	6.0	3.0	7.33	5.41	840	21.1	13.6	0.421	44.9	9.37	36.3	11.6	11.6	36.4	16.1	22.4	0.0936	15.6	6.00	23.9	3.91	22.3	4.20	7.49	10.6	17.6	0.570	65 x 50 x 8 UA
6 UA	5.16	6.0	6.0	3.0	9.83	7.33	658	20.4	12.9	0.338	45.2	7.48	36.1	9.35	11.2	30.2	12.7	22.7	0.0743	15.1	4.91	23.9	3.11	21.4	3.48	5.93	10.6	8.29	0.575	6 UA
5 UA	4.02	4.6	6.0	3.0	13.1	9.87	512	19.8	12.4	0.267	45.4	5.89	35.9	7.43	10.9	24.5	9.92	22.8	0.0587	14.8	3.97	23.9	2.46	20.6	2.85	4.66	10.7	3.87	0.577	5 UA

Unequal Angles

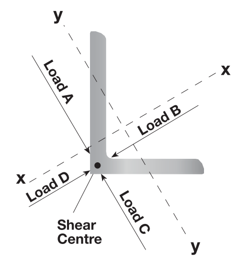
Table 23 Unequal Angles – x-axis and y-axis – Properties for Assessing Section Capacity

Designation	Yield Stress	Form Factor	About x-axis		About y-axis		Yield Stress	Form Factor	About x-axis		About y-axis		Designation
			Load A	Load C	Load B	Load D			Load A	Load C	Load B	Load D	
	f_y	k_f	Z_{ex}	Z_{ex}	Z_{ey}	Z_{ey}	f_y	k_f	Z_{ex}	Z_{ex}	Z_{ey}	Z_{ey}	
mm mm mm	MPa		10^3mm^3	10^3mm^3	10^3mm^3	10^3mm^3	MPa		10^3mm^3	10^3mm^3	10^3mm^3	10^3mm^3	
300PLUS® *							**AS/NZS 3679.1-350**						
150 x 100 x 12 UA	300	1.00	102	110	35.3	38.2	340	1.00	100	110	34.7	38.2	150 x 100 x 12 UA
10 UA	320	0.975	74.8	81.7	26.0	30.9	360	0.943	73.0	78.9	25.3	30.9	10 UA
150 x 90 x 16 UA	300	1.00	132	133	39.5	39.8	340	1.00	130	133	39.0	39.8	150 x 90 x 16 UA
12 UA	300	1.00	96.3	104	28.8	31.1	340	1.00	94.6	104	28.3	31.1	12 UA
10 UA	320	0.973	70.6	81.8	21.2	25.2	360	0.940	68.8	79.5	20.6	25.2	10 UA
8 UA	320	0.863	53.1	60.3	15.9	21.0	360	0.836	51.2	57.9	15.4	21.0	8 UA
125 x 75 x 12 UA	300	1.00	68.6	70.5	20.6	21.2	340	1.00	67.6	70.5	20.3	21.2	125 x 75 x 12 UA
10 UA	320	1.00	51.6	57.2	15.5	17.2	360	1.00	50.6	57.2	15.2	17.2	10 UA
8 UA	320	0.964	39.8	46.0	11.9	14.3	360	0.931	38.8	44.7	11.6	14.3	8 UA
6 UA	320	0.824	26.8	30.1	8.07	11.2	360	0.799	25.8	28.7	7.75	11.2	6 UA
100 x 75 x 10 UA	320	1.00	39.4	40.9	15.9	16.6	360	1.00	38.8	40.9	15.7	16.6	100 x 75 x 10 UA
8 UA	320	1.00	31.2	33.1	12.6	13.9	360	1.00	30.6	32.1	12.4	13.9	8 UA
6 UA	320	0.946	22.0	21.8	8.93	10.9	360	0.917	21.4	20.7	8.68	10.9	6 UA
75 x 50 x 8 UA	320	1.00	17.0	17.3	5.93	6.02	360	1.00	16.8	17.3	5.85	6.02	75 x 50 x 8 UA
6 UA	320	1.00	12.6	13.7	4.37	4.77	360	1.00	12.4	13.7	4.30	4.77	6 UA
5 UA	320	0.956	8.89	9.65	3.10	3.75	360	0.926	8.66	9.30	3.02	3.75	5 UA
65 x 50 x 8 UA	320	1.00	14.1	14.1	5.86	5.86	360	1.00	14.1	14.1	5.86	5.86	65 x 50 x 8 UA
6 UA	320	1.00	10.7	11.2	4.46	4.67	360	1.00	10.6	11.2	4.40	4.67	6 UA
5 UA	320	1.00	7.76	7.92	3.23	3.68	360	1.00	7.59	7.64	3.17	3.68	5 UA

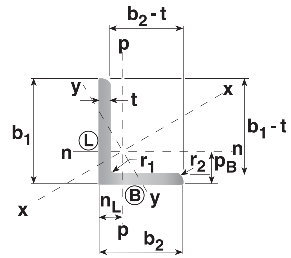
* 300PLUS® replaced Grade 250 as the base grade for 150 x 90 x 8 unequal angles and larger in 1994.
300PLUS® replaced Grade 250 as the base grade for 125 x 75 x 12 unequal angles and smaller in 1997.
300PLUS® hot rolled sections are produced to exceed the minimum requirements of AS/NZS 3679.1-300.

Notes

1. For 300PLUS® sections the tensile strength (fu) is 440 MPa.
2. For Grade 350 sections the tensile strength (fu) is 480 MPa.



Unequal Angles



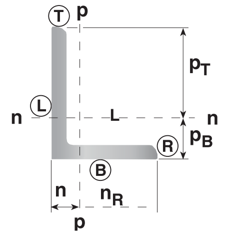


Table 24 Unequal Angles – n-axis and p-axis – Dimensions and Properties

Designation	About n-axis							About p-axis							Product of 2nd Moment of Area	Designation
	I_n	p_B	Z_{nB}	p_T	Z_{nT}	S_n	r_n	I_p	n_L	Z_{pL}	n_R	Z_{pR}	S_p	r_p	I_{np}	
mm mm mm	10^6mm^4	mm	10^3mm^3	mm	10^3mm^3	10^3mm^3	mm	10^6mm^4	mm	10^3mm^3	mm	10^3mm^3	10^3mm^3	mm	10^6mm^4	
150 x 100 x 12 UA	6.52	49.1	133	101	64.6	117	47.7	2.34	24.3	96.2	75.7	30.9	56.0	28.6	-2.27	150 x 100 x 12 UA
10 UA	5.29	48.1	110	102	51.9	94.0	48.0	1.91	23.3	81.9	76.7	24.9	44.7	28.8	-1.85	10 UA
150 x 90 x 16 UA	7.97	52.5	152	97.5	81.7	145	47.4	2.15	22.7	94.9	67.3	32.0	59.5	24.6	-2.35	150 x 90 x 16 UA
12 UA	6.29	51.0	123	99.0	63.5	114	47.8	1.72	21.2	81.0	68.8	25.0	45.7	25.0	-1.89	12 UA
10 UA	5.10	50.0	102	100	51.0	91.5	48.2	1.41	20.2	69.5	69.8	20.2	36.5	25.3	-1.54	10 UA
8 UA	4.26	49.2	86.6	101	42.3	76.0	48.4	1.18	19.6	60.4	70.4	16.8	30.1	25.5	-1.29	8 UA
125 x 75 x 12 UA	3.54	43.3	81.8	81.7	43.3	77.3	39.6	0.958	18.4	52.0	56.6	16.9	31.4	20.6	-1.05	125 x 75 x 12 UA
10 UA	2.88	42.3	68.2	82.7	34.9	62.5	39.9	0.789	17.5	45.2	57.5	13.7	25.1	20.9	-0.867	10 UA
8 UA	2.41	41.5	58.1	83.5	28.9	52.0	40.1	0.664	16.8	39.6	58.2	11.4	20.7	21.0	-0.731	8 UA
6 UA	1.89	40.7	46.5	84.3	22.5	40.6	40.3	0.524	16.0	32.7	59.0	8.89	16.0	21.2	-0.575	6 UA
100 x 75 x 10 UA	1.55	31.8	48.6	68.2	22.6	41.3	31.3	0.743	19.4	38.3	55.6	13.4	24.3	21.7	-0.625	100 x 75 x 10 UA
8 UA	1.30	31.1	41.8	68.9	18.8	34.4	31.5	0.626	18.7	33.5	56.3	11.1	20.2	21.9	-0.528	8 UA
6 UA	1.02	30.3	33.7	69.7	14.6	26.9	31.7	0.494	17.9	27.5	57.1	8.67	15.7	22.0	-0.416	6 UA
75 x 50 x 8 UA	0.511	25.2	20.3	49.8	10.3	18.5	23.6	0.181	12.8	14.1	37.2	4.86	8.96	14.0	-0.174	75 x 50 x 8 UA
6 UA	0.407	24.4	16.7	50.6	8.05	14.6	23.8	0.145	12.1	12.0	37.9	3.84	6.98	14.2	-0.140	6 UA
5 UA	0.321	23.8	13.5	51.2	6.27	11.4	23.9	0.115	11.5	10.0	38.5	3.00	5.41	14.3	-0.111	5 UA
65 x 50 x 8 UA	0.341	21.1	16.2	43.9	7.75	14.1	20.1	0.174	13.6	12.7	36.4	4.78	8.74	14.4	-0.141	65 x 50 x 8 UA
6 UA	0.272	20.4	13.4	44.6	6.10	11.1	20.3	0.140	12.9	10.8	37.1	3.77	6.85	14.6	-0.114	6 UA
5 UA	0.215	19.8	10.9	45.2	4.75	8.70	20.5	0.111	12.4	8.96	37.6	2.95	5.32	14.7	-0.0903	5 UA

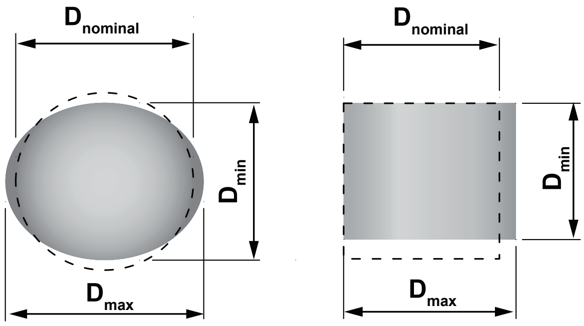
Tolerances

Rounds and Squares

Table 25 Permissible variations in cross-sectional dimensions for Rounds and Squares

Nominal Dimension	Permissible Variation	Permissible out-of-round or out-of-square
$D_{nominal}$		$D_{max} - D_{min}$
mm	mm	mm
≤25	±0.25	0.40
> 25 ≤ 30	±0.30	0.45
> 30 ≤ 40	±0.40	0.60
> 40 ≤ 50	±0.50	0.75
> 50 ≤ 60	±0.60	0.90
> 60 ≤ 70	±0.70	1.05
> 70 ≤ 80	±0.80	1.20
> 80 ≤ 100	±0.90	1.35
> 80* ≤ 100*	+2.45 to –0*	1.85*

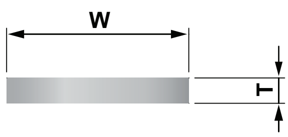
Note: * indicates alternative for material produced as primary-rolled product.



Flats

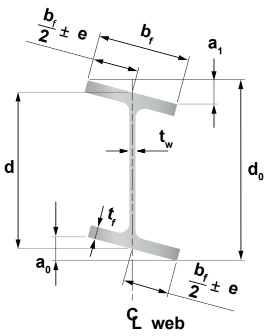
Table 26 Permissible variations in cross-sectional dimensions for Flats

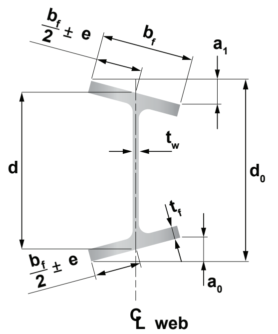
Nominal Width		Width Tolerance	Thickness Tolerance				
W			T				
mm		mm	mm				
			<6	≥6 ≤12	>12 ≤25	>25 ≤50	>50
≤25		±0.40	±0.20	±0.20	±0.25	–	–
>25	≤50	±0.80	±0.20	±0.30	±0.40	±0.80	–
>50	≤100	+1.60 to –0.80	±0.20	±0.40	±0.50	±0.80	±1.20
>100	≤150	+2.40 to –1.60	±0.25	±0.40	±0.50	±0.80	±1.60



Universal Beam

Table 27 Universal Beam Tolerances

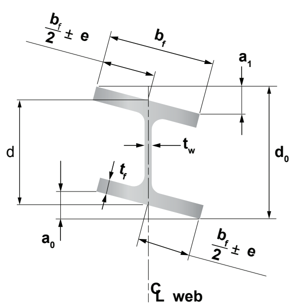


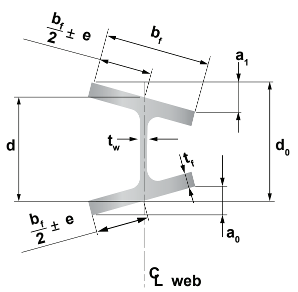


	Permissible variation of depth	Permissible variation of flange width	Permissible variation of flange thickness	Permissible variation of web thickness	Maximum difference of flange over four flanges	Permissible out-of-square on each flange	Permissible total out-of-square	Permissible web off-centre	Permissible overall depth over specified depth
	d	b_f	t_f	t_w		(a_1 or a_0)	(a_1 + a_0)	e	(d_0 - d)
Designation	mm	mm	mm	mm	mm	mm	mm	mm	mm
610UB125	±3.0	+6.0 to –5.0	±1.5	±0.7	1.5	5.0	8.0	5.0	6.0
610UB113	±3.0	+6.0 to –5.0	±1.5	±0.7	1.5	5.0	8.0	5.0	6.0
610UB101	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
530UB92.4	±3.0	+6.0 to –5.0	±1.5	±0.7	1.5	5.0	8.0	5.0	6.0
530UB82.0	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
460UB82.1	±3.0	+6.0 to –5.0	±1.5	±0.7	1.5	5.0	8.0	5.0	6.0
460UB74.6	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
460UB67.1	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
410UB59.7	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
410UB53.7	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
360UB56.7	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
360UB50.7	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
360UB44.7	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
310UB46.2	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
310UB40.4	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
310UB32.0	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
250UB37.3	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
250UB31.4	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
250UB25.7	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
200UB29.8	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
200UB25.4	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
200UB22.3	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
200UB18.2	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
180UB22.2	+2.5 to -1.5	±3.0	±1.0	±0.7	1.0	2.0	2.5	2.5	4.0
180UB18.1	+2.5 to -1.5	±3.0	±1.0	±0.7	1.0	2.0	2.5	2.5	4.0
180UB16.1	+2.5 to -1.5	±3.0	±1.0	±0.7	1.0	2.0	2.5	2.5	4.0
150UB18.0	+2.5 to -1.5	±3.0	±1.0	±0.7	1.0	1.5	2.5	2.5	4.0
150UB14.0	+2.5 to -1.5	±3.0	±1.0	±0.7	1.0	1.5	2.5	2.5	4.0

Universal Column

Table 28 Universal Column Tolerances



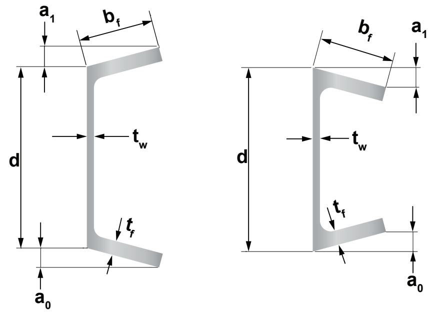


	Permissible variation of depth	Permissible variation of flange width	Permissible variation of flange thickness	Permissible variation of web thickness	Maximum difference of flange over four flanges	Permissible out-of-square on each flange	Permissible total out-of-square	Permissible web off-centre	Permissible overall depth over specified depth
	d	b_f	t_f	t_w		(a_1 or a_0)	(a_1 + a_0)	e	(d_0 -d)
Designation	mm	mm	mm	mm	mm	mm	mm	mm	mm
310UC158	±3.0	+6.0 to –5.0	±1.5	±1.0	1.5	5.0	8.0	5.0	6.0
310UC137	±3.0	+6.0 to–5.0	±1.5	±0.7	1.5	5.0	8.0	5.0	6.0
310UC118	±3.0	+6.0 to –5.0	±1.5	±0.7	1.5	5.0	8.0	5.0	6.0
310UC96.8	±3.0	+6.0 to –5.0	±1.5	±0.7	1.5	5.0	8.0	5.0	6.0
250UC89.5	±3.0	+6.0 to –5.0	±1.5	±0.7	1.5	4.0	6.0	5.0	6.0
250UC72.9	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
200UC59.5	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
200UC52.2	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
200UC46.2	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
150UC37.2	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
150UC30.0	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
150UC23.4	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0
100UC14.8	±3.0	+6.0 to –5.0	±1.0	±0.7	1.0	4.0	6.0	5.0	6.0

Parallel Flange Channels

Table 29 Parallel Flange Channel Tolerances

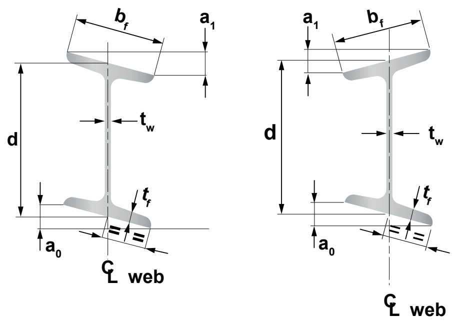
	Permissible variation of depth	Permissible variation of flange width	Permissible variation of flange thickness	Permissible variation of web thickness	Permissible out-of-square on each flange	Permissible total out-of-square
	d	b_f	t_f	t_w	(a_1 or a_0)	($a_1 + a_0$)
Designation	mm	mm	mm	mm	mm	mm
380PFC	+5.0 to −3.0	+3.0 to −4.0	±1.0	±1.0	2.0	3.0
300PFC	+3.0 to −1.5	±3.0	±1.0	±1.0	1.5	2.7
250PFC	+3.0 to −1.5	±3.0	±1.0	±1.0	1.5	2.7
230PFC	+3.0 to −1.5	±3.0	±1.0	±1.0	1.5	2.3
200PFC	+3.0 to −1.5	±3.0	±1.0	±1.0	1.5	2.3
180PFC	+3.0 to −1.5	±3.0	±1.0	±1.0	1.5	2.3
150PFC	+3.0 to −1.5	±3.0	±1.0	±1.0	1.5	2.3
125PFC	+3.0 to −1.5	±3.0	±1.0	±1.0	1.5	2.0
100PFC	+3.0 to −1.5	±3.0	±0.7	±0.7	1.0	1.5
75PFC	+3.0 to −1.5	±3.0	±0.7	±0.7	1.0	1.2



Tapered Flange Beam

Table 30 Tapered Flange Beam Tolerances

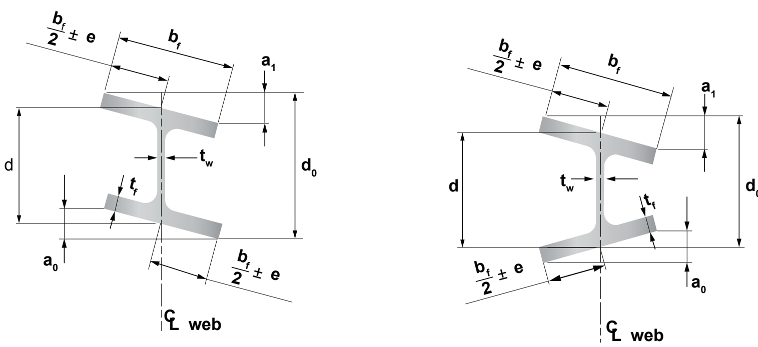
	Permissible variation of depth	Permissible variation of flange width	Permissible variation of flange thickness	Permissible variation of web thickness	Permissible out-of-square on each flange	Permissible total out-of-square
	d	b_f	t_f	t_w	(a_1 or a_0)	($a_1 + a_0$)
Designation	mm	mm	mm	mm	mm	mm
125TFB	+2.5 to −1.5	±3.0	±0.7	±0.7	1.5	2.0
100TFB	+2.5 to −1.5	±3.0	±0.7	±0.7	1.5	1.4



Universal Bearing Piles

Table 31 Universal Bearing Pile Tolerances

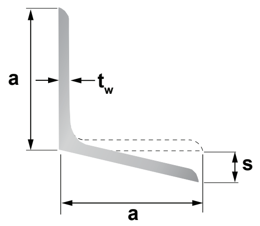
	Permissible variation of depth	Permissible variation of flange width	Permissible variation of flange thickness	Permissible variation of web thickness	Maximum difference of flange over four flanges	Permissible out-of-square on each flange	Permissible total out-of-square	Permissible web off-centre	Permissible overall depth over specified depth
	d	b_f	t_f	t_w		(a_1 or a_0)	($a_1 + a_0$)	e	(d_0 -d)
Designation	mm	mm	mm	mm	mm	mm	mm	mm	mm
310UBP149	+3.0 to −2.0	±4.0	±1.5	±0.7	1.5	4.0	6.3	3.5	6.0
310UBP110	+3.0 to −2.0	±4.0	±1.5	±0.7	1.5	4.0	6.2	3.5	6.0
310UBP78.8	+3.5 to −3.5	+6.5 to −5.4	±1.0	±0.7	1.0	5.0	8.0	5.0	6.0
200UBP122	+3.4 to −3.4	+6.5 to −5.4	±1.5	±1.0	1.5	4.0	6.0	5.0	6.0

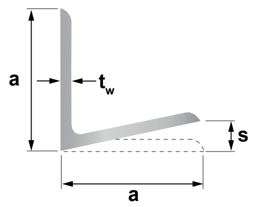


Tolerances

Equal Angle

Table 32 Equal Angle Tolerances



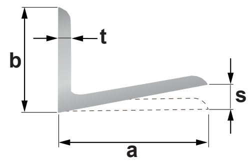


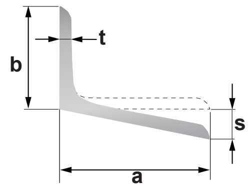
Designation	Permissible variation of leg length	Permissible variation of thickness	Permissible out-of-square
	a	t_w	s
	mm	mm	mm
200x200x26 EA	+5.0 to −3.0	±1.5	±5.0
200x200x20 EA	+5.0 to −3.0	±1.0	±5.0
200x200x18 EA	+5.0 to −3.0	±1.0	±5.0
200x200x16 EA	+5.0 to −3.0	±1.0	±5.0
200x200x13 EA	+5.0 to −3.0	±0.7	±5.0
150x150x19 EA	±3.0	±1.0	±4.0
150x150x16 EA	±3.0	±1.0	±4.0
150x150x12 EA	±3.0	±0.7	±4.0
150x150x10 EA	±3.0	±0.5	±4.0
125x125x16 EA	±3.0	±1.0	±3.0
125x125x12 EA	±3.0	±0.7	±3.0
125x125x10 EA	±3.0	±0.5	±3.0
125x125x8 EA	±3.0	±0.5	±3.0
100x100x12 EA	±3.0	±0.7	±3.0
100x100x10 EA	±3.0	±0.5	±3.0
100x100x8 EA	±3.0	±0.5	±3.0
100x100x6 EA	±3.0	±0.5	±3.0
90x90x10 EA	±3.0	±0.5	±3.0
90x90x8 EA	±3.0	±0.5	±3.0
90x90x6 EA	±3.0	±0.5	±3.0
75x75x10 EA	+2.5 to −1.5	±0.5	±2.0
75x75x8 EA	+2.5 to −1.5	±0.5	±2.0
75x75x6 EA	+2.5 to −1.5	±0.5	±2.0
75x75x5 EA	+2.5 to −1.5	±0.5	±2.0
65x65x10 EA	+2.5 to −1.5	±0.5	±2.0
65x65x8 EA	+2.5 to −1.5	±0.5	±2.0
65x65x6 EA	+2.5 to −1.5	±0.5	±2.0
65x65x5 EA	+2.5 to −1.5	±0.5	±2.0
55x55x6 EA	+2.5 to −1.5	±0.5	±2.0
55x55x5 EA	+2.5 to −1.5	±0.5	±2.0
50x50x8 EA	+2.5 to −1.5	±0.5	±2.0
50x50x6 EA	+2.5 to −1.5	±0.5	±2.0
50x50x5 EA	+2.5 to −1.5	±0.5	±2.0
50x50x3 EA	+2.5 to −1.5	±0.5	±2.0
45x45x6 EA	+2.5 to −1.5	±0.5	±2.0
45x45x5 EA	+2.5 to −1.5	±0.5	±2.0
45x45x3 EA	+2.5 to −1.5	±0.5	±2.0
40x40x6 EA	+2.5 to −1.5	±0.5	±1.0
40x40x5 EA	+2.5 to −1.5	±0.5	±1.0
40x40x3 EA	+2.5 to −1.5	±0.5	±1.0
30x30x6 EA	+2.5 to −1.5	±0.5	±1.0
30x30x5 EA	+2.5 to −1.5	±0.5	±1.0
30x30x3 EA	+2.5 to −1.5	±0.5	±1.0
25x25x6 EA	+2.5 to −1.5	±0.5	±1.0
25x25x5 EA	+2.5 to −1.5	±0.5	±1.0
25x25x3 EA	+2.5 to −1.5	±0.5	±1.0

Unequal Angle

Table 33 Unequal Angle Tolerances

	Permissible variation of leg length – Long Leg	Permissible variation of leg length – Short Leg	Permissible variation of thickness	Permissible out-of-square
	a	b	t_w	s
Designation	mm		mm	mm
150x100x12 UA	±3.0	±3.0	±0.7	±4.0
150x100x10 UA	±3.0	±3.0	±0.5	±4.0
150x90x16 UA	±3.0	±3.0	±1.0	±4.0
150x90x12 UA	±3.0	±3.0	±0.7	±4.0
150x90x10 UA	±3.0	±3.0	±0.5	±4.0
150x90x8 UA	±3.0	±3.0	±0.5	±4.0
125x75x12 UA	±3.0	+2.5 to −1.5	±0.7	±3.0
125x75x10 UA	±3.0	+2.5 to −1.5	±0.5	±3.0
125x75x8 UA	±3.0	+2.5 to −1.5	±0.5	±3.0
125x75x6 UA	±3.0	+2.5 to −1.5	±0.5	±3.0
100x75x10 UA	±3.0	+2.5 to −1.5	±0.5	±3.0
100x75x8 UA	±3.0	+2.5 to −1.5	±0.5	±3.0
100x75x6 UA	±3.0	+2.5 to −1.5	±0.5	±3.0
75x50x8 UA	+2.5 to −1.5	+2.5 to −1.5	±0.5	±2.0
75x50x6 UA	+2.5 to −1.5	+2.5 to −1.5	±0.5	±2.0
75x50x5 UA	+2.5 to −1.5	+2.5 to −1.5	±0.5	±2.0
65x50x8 UA	+2.5 to −1.5	+2.5 to −1.5	±0.5	±2.0
65x50x6 UA	+2.5 to −1.5	+2.5 to −1.5	±0.5	±2.0
65x50x5 UA	+2.5 to −1.5	+2.5 to −1.5	±0.5	±2.0





LIBERTY

Straightness

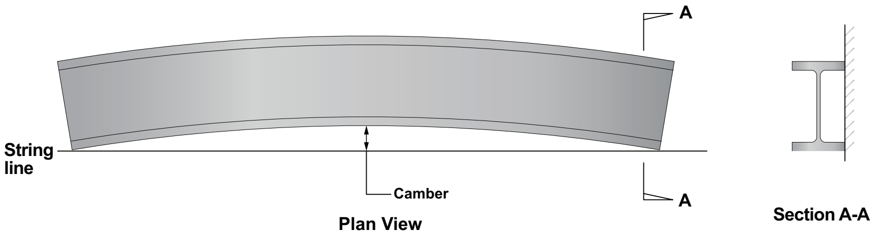
Universal Sections

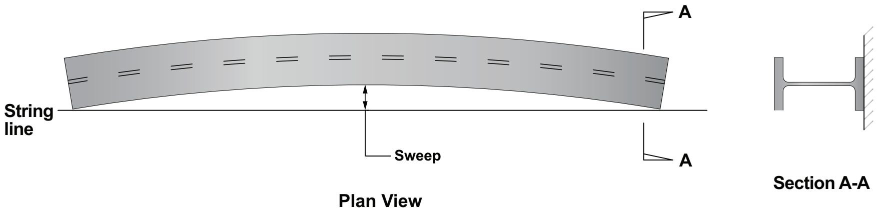
Table 34 Permissable Variations in Straightness for Universal Sections

Section	Camber (mm)	Sweep (mm)
Beams with flange b_f < 150mm	$\frac{\text{Length (mm)}}{1000}$	$\frac{\text{Length (mm)}}{500}$
Beams with flange $b_f \geq$ 150mm	$\frac{\text{Length (mm)}}{1000}$	(See Note 2)
Columns ≤ 14000mm long	$\frac{\text{Length (mm)}}{1000}$ but no more than 10mm	(See Note 2)
Columns > 14000mm long	$10\text{mm} + \frac{\text{Length (mm)} - 14000}{10000}$	(See Note 2)

Notes:

1. Measuring of the camber and sweep shall be in accordance with the figure below.
2. Owing to the extreme variation in the elastic flexibility of these sections about the y axis, difficulty may be experienced in obtaining reproducible sweep measurements.





Non-universal Sections

Table 35 Permissible Variations in Straightness for Channels, Taper Flange Beams and Angles

Section	Camber (mm)	Sweep (mm)
Channels	$\frac{\text{Length (mm)}}{500}$	(See Note 2)
Taper Flange Beams		
Angles		

Notes:

1. For angles having a combined leg length of greater than 150mm this is the straightness tolerance.
2. Owing to the extreme variation in flexibility of these sections about the y axis, straightness tolerances are as specified by the purchaser for the individual sections involved. .

Standard Specifications

Structural Steel – Hot Rolled Bars and Sections – Standard: AS/NZS 3679.1

Table 36 Chemical Composition – Bars and Sections

Grade (see Note 1)	Cast analysis (max.) (See Notes 2 and 3) %							
	C	Si	Mn	P	S	B	Micro-alloying elements (see Note 4)	CE (see Note 5)
300PLUS®, 300PLUS® L0, 300PLUS® L15, 300PLUS® S0	0.25	0.50	1.60	0.040	0.040	<0.0008	(see Note 6)	0.44
350, 350L0, 350 L15, 350 S0	0.22	0.50	1.60	0.040	0.040	<0.0008	(see Note 7)	0.45

Notes

1. The use of sulfide modification steel making techniques for these grades is permitted.
2. Grain refining elements, i.e. aluminium and titanium, may be added, provided that the total content does not exceed 0.15%. Limits are for total or soluble aluminium.
3. The following elements may be present to the limits stated, subject to a maximum total of 1.00%:
   (a) Copper 0.50%
   (b) Nickel 0.50%
   (c) Chromium 0.30%
   (d) Molybdenum 0.10%
4. For grade 300PLUS, the following are not considered as micro-alloying elements:
   (a) Titanium 0.040% maximum
   (b) Niobium 0.020% maximum
   (c) Vanadium 0.030% maximum
   (d) Niobium plus vanadium 0.030% maximum
5. Carbon equivalent (CE) is calculated from the following equation:
   $$CE = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15}$$
6. Micro-alloying elements are not permitted in grade 300 except for thicknesses greater than or equal to 15mm, where the following apply:
   (a) the maximum combined micro-alloying element content is 0.15%
   (b) where micro-alloying elements are used, the percentage of each element is to be shown on certificates.
7. For grade 350, micro-alloying elements niobium, vanadium and titanium may be added, provided that their total combined content does not exceed 0.15%.

Table 37 Tensile Properties – Flat Bars and Sections – Standard: AS/NZS 3679.1

Grade	Minimum yield stress, MPa Thickness (see Note 1) mm			Minimum tensile strength MPa	Minimum elongation on a gauge length of $5.65\sqrt{S_0}$ % (see Note 2)
	< 11	≥ 11 to ≤ 17	> 17 to < 40		
300PLUS®, 300PLUS®L0, 300PLUS® L15	320	300	280	440	22
350, 350L0, 350L15	360	340	340	480	20

Table 38 Tensile Properties – Round and Square Bars – Standard: AS/NZS 3679.1

Grade	Minimum yield stress, MPa Thickness mm			Minimum tensile strength MPa	Minimum elongation on a gauge length of $5.65\sqrt{S_0}$ %
	≤ 50	> 50 to < 100	≥ 100		
300PLUS®	300	290	280	440	22
350	340	330	320	480	20

Notes (apply to tables 37 and 38)

1. For a section, the term 'thickness' refers to the nominal thickness of the part from which the sample is taken.
2. S_0 is the cross-sectional area of the test piece before testing.
3. For precise details of properties reference should be made to the latest edition of AS/NZS 3679.1 or the latest Liberty Steel specification.
4. 300PLUS® steel is produced to exceed the latest requirements for grade 300 in AS/NZS 3679.1.

Table 39 Charpy V-Notch Impact Test Requirements – Bars and Sections – Standard: AS/NZS 3679.1

Grade	Test Temperature °C	Minimum Absorbed Energy, J Size of Test Piece					
		10mm x 10mm		10mm x 7.5mm		10mm x 5mm	
		Average of 3 Tests	Individual Test	Average of 3 Tests	Individual Test	Average of 3 Tests	Individual Test
300PLUS®L0, 350L0*	0	27	20	22	16	18	13
300PLUS®L15, 350L15	-15	27	20	22	16	18	13

Notes

This does not cover impact tested grades for thickness less than 7mm.
*Impact testing is not available for bars and is only available for some sections by enquiry.

Standard Specifications

Merchant Bar Sections

Table 40 Chemical Composition – For Liberty Steel Merchant Bar Sections – Regular Grades – AS 1442

Steel Type	Grade	C		Si		Mn		P		S	
		Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Carbon and Carbon Manganese Steels	1016	0.13	0.18	0.10	0.35	0.60	0.90	*	0.040	*	0.040
	1022	0.18	0.23	0.10	0.35	0.70	1.00	*	0.040	*	0.040
	1045	0.43	0.50	0.10	0.35	0.60	0.90	*	0.040	*	0.040

Table 41 Chemical Composition – For Liberty Steel Merchant Bar Sections – Regular Grades – AS 1447

Steel Type	Grade	C		Si		Mn		P		S		Cr	
		Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Spring Steels	5160	0.55	0.65	0.10	0.35	0.70	1.00	*	0.040	*	0.040	0.70	0.90
	9258	0.50	0.65	1.60	2.20	0.70	1.05	*	0.040	*	0.040	*	*
	9261	0.55	0.65	1.80	2.20	0.70	1.00	*	0.040	*	0.040	0.10	0.25

Table 42 Liberty Steel Grades

Steel Type	Grade	C		Si		Mn		P		S		Cr		V	
		Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Liberty Steel	X4K92M61S*	0.55	0.65	1.60	1.90	0.70	1.00	*	0.040	*	0.040	0.10	0.25	0.15	0.25

Table 43 Heat Treatment Limitations

Grade	Maximum Recommended Cross Section*		
	Rounds	Squares	Flats
5160	40mm	36mm	28mm
9261	27mm	25mm	19mm
9258			16mm

* The recommendations are based on the criterion that, at the maximum dimensions, a hardness of 50 HRC can be achieved in the centre of the quenched section.

The actual properties obtained are dependent on both grade and heat treatment process control. As Liberty Steel has no control over the springmakers' heat treatment process, the above recommendations cannot be guaranteed. However, springmakers with efficient heat treatment facilities will be able to achieve a hardness value of 50 HRC as recommended.

Customer Technical Service

MORE INFORMATION

Further information on Liberty Steel products, services and other publications can be found at: www.libertygfg.com

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