MONASH UNIVERSITY

DEPARTMENT OF CIVIL ENGINEERING

CIV3221 Building Structures and Technology
LECTURE NOTES

An introduction to multi-storey structural systems, façade systems, loading and analysis. Design of steel beams and columns, concrete slabs and footings, and composite steel/concrete beams and slabs.

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Cover Photo:

Reflections in the façade of the John Hancock Building, Boston USA (Geoff Taplin 2000)
The John Hancock building (60 stories) has one of the most infamous facades in the world.

(http://www.pubs.asce.org/ceonline/0600feat.html)

LECTURE NOTES

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TOPIC 2: FLOOR FRAMING SYSTEMS

1. General

1.1 Purpose:

to provide a safe and functional working platform

1.2 Factors affecting the selection of a floor system:

1. $ cost
2. $ cost
3. $ cost
4. spacing of supports (walls, columns)
5. serviceability (ie stiffness and vibration)
6. adaptability to future changes of use

………plus safety

2. Types of floor systems

We can classify floor systems in different ways:

by material

- floor slab – concrete
- beams – concrete or steel

by structural action of the slab

- one way spanning slab
- two way spanning slab
- flat slab

by method of construction

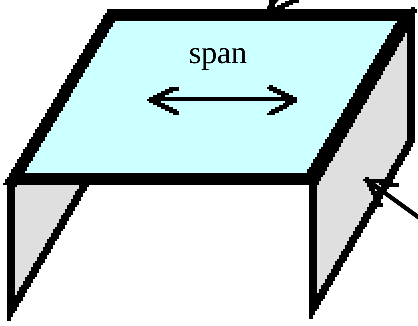
- steel beams with cast-in-situ slab
- steel beams with precast slab
- cast-in-situ concrete beams with cast-in-situ slab
- precast beams with precast slab

The following notes provide some 'rules-of-thumb' for the preliminary sizing of slabs and beams. The thickness of the slab, or the depth of the beam, is given as a span-to-depth ratio, l/d.

note that d is the effective depth – ie you must add cover thickness to the concrete (say 40 mm)
eg,
l/d =30 and slab span = 6 metres υ thickness = 200+40 = 240 mm

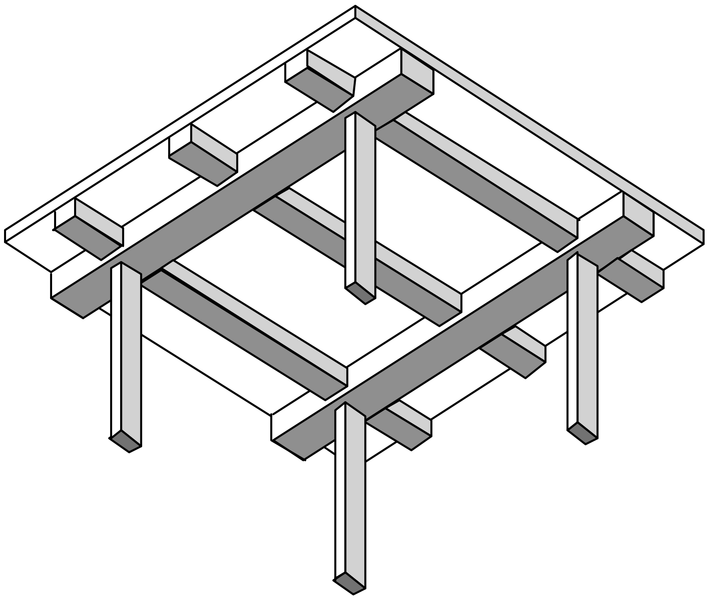
2.1 One way spanning slabs

the slab spans in one direction between beams or walls

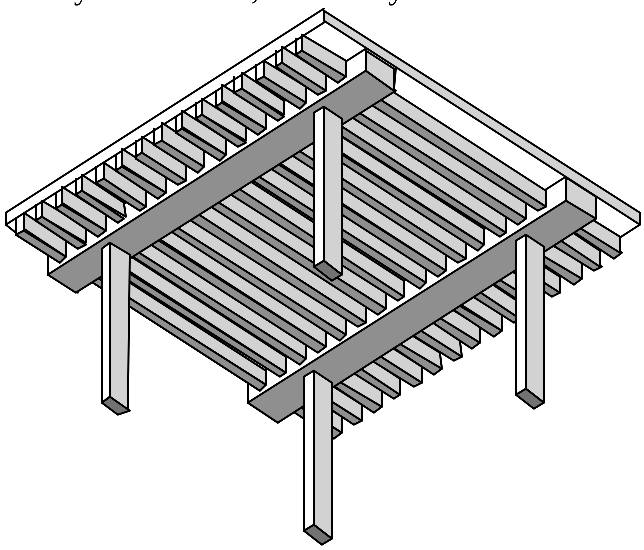


- if the slab is simply-supported (ie one span only), l/d = 24
- if the slab is continuous (ie 2 or more spans), l/d = 28
- if the slab has a cantilever span, l/d = 10
- the slab might be cast on metal decking (usually on steel beams),

- a slab may span one-way onto secondary beams, which in turn span onto primary beams,

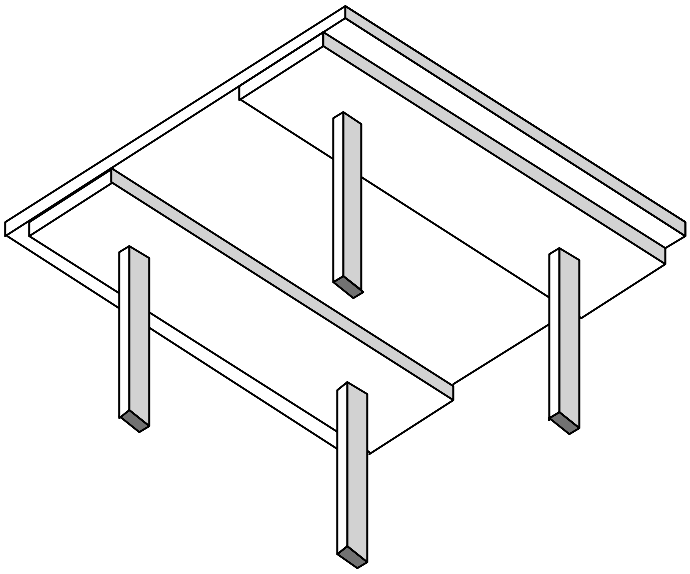


- if the slab is supported by beams on 4 edges, but the *aspect ratio* (ie longer supported length divided by shorter supported length) > 2, treat as a one-way slab
- if the beams are at very close centres, the floor system becomes a ribbed slab,

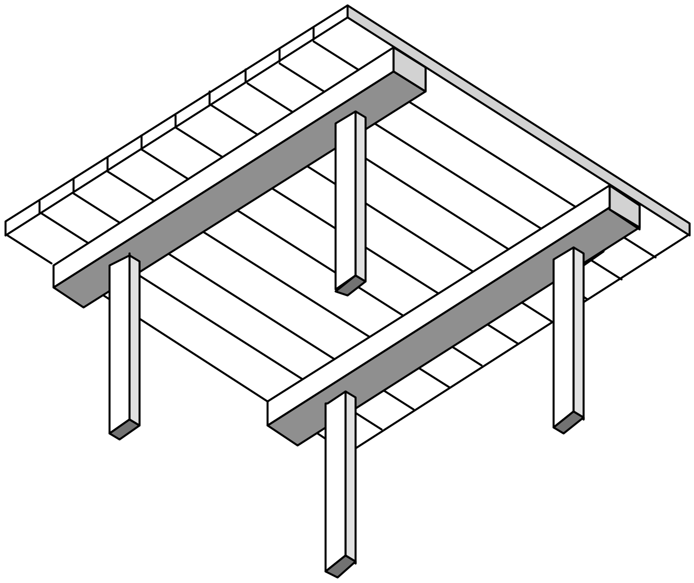


- ribbed slabs are require a lot of labour to build the *formwork* for the slab

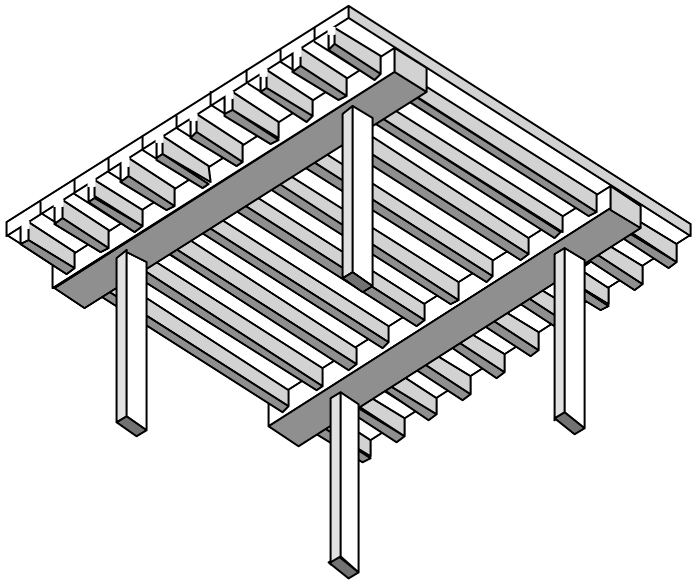
- if wide and shallow concrete beams support the slab, it is called a *band beam* floor system



- band beam systems have simple *formwork*, and are economical with labour
- precast floor systems can be used with one-way slabs
- precast systems are *proprietary systems* – ie they are products which you buy from a manufacturer
- precast planks is one *proprietary system* (Hollowcore brand) – rectangular planks of concrete laid on steel or precast concrete beams

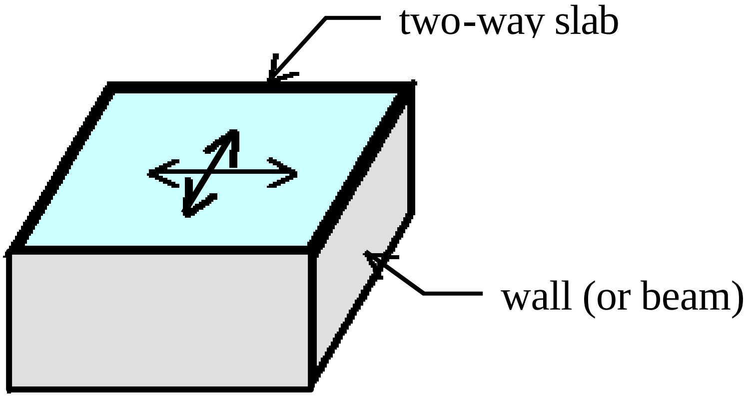


- precast T-beams is another *proprietary system*

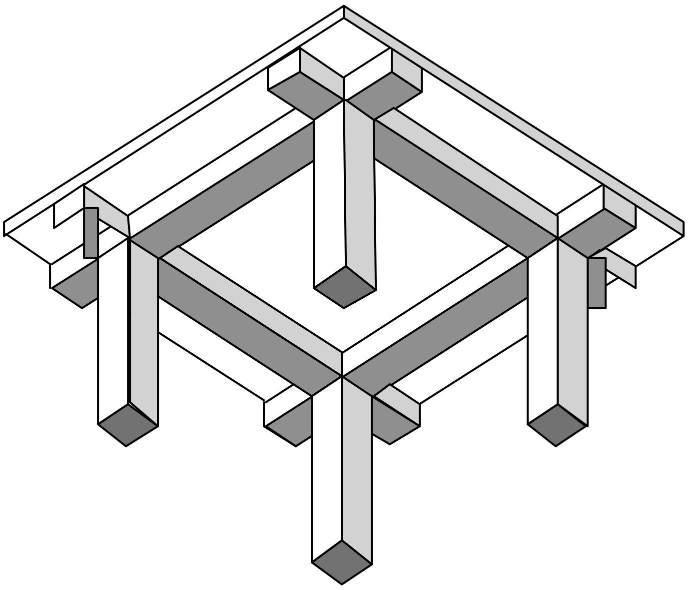


2.2 Two way spanning slabs

When the slab is supported by beams or walls on four sides it is a two-way slab.



- if the slab is simply-supported (ie one span only), l/d = 28
- if the slab is continuous (ie 2 or more spans), l/d = 39
- these values assume square panels
- for aspect ratios between 1:1 and 2:1 interpolate between two-way and one-way values (l is the length of the shorter span)



- if the beams (in two directions) are at very close centres, the floor system becomes a waffle slab
- waffle slabs require a lot of labour to build the *formwork* for the slab

2.3 Beams for one and two way spanning slabs

We now have some ‘rules-of thumb’ for sizing one-way and two-way slabs, but what about the size of the beams that support them?

- if the beam is simply supported (ie one span only, or no moment connection between spans), l/d = 12
- if the beam is continuous (ie 2 or more spans), l/d = 15
- if the beam is a cantilever, l/d = 6

These values can be used for steel or concrete floor beams.

2.4 Flat slabs

What if you do not use beams or walls to support the slab, but instead sit it directly on top of the columns?

These are called *flat slabs*.

- because there are no beams, the *formwork* is very easy
- you must be careful to make sure that there is enough strength at the slab/column junction, and that the slab does not *punch through*

- because there are no beams of walls, flat slabs tend to have large deflections, and for this reason they are not as popular as they were 10 years ago

Flat slabs come in two main types,

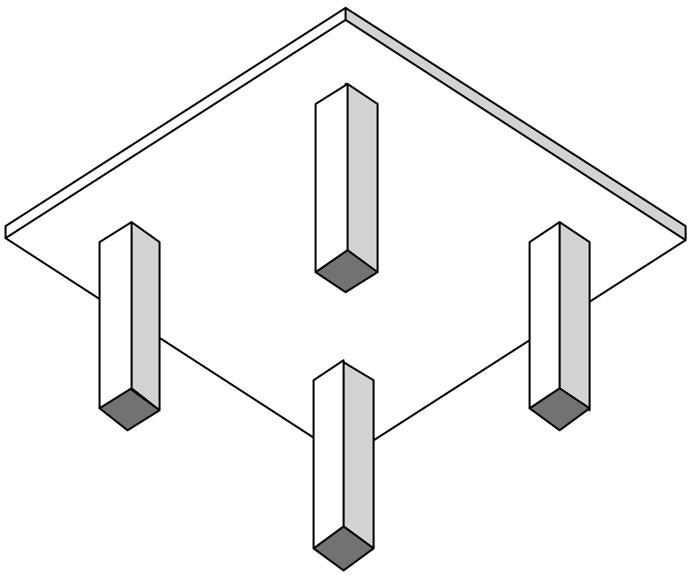
1. flat plate
2. flat slab (yes, 'flat slab' is a type of flat slab!)

A *flat plate* has a completely flat *soffit* (that is the underside)

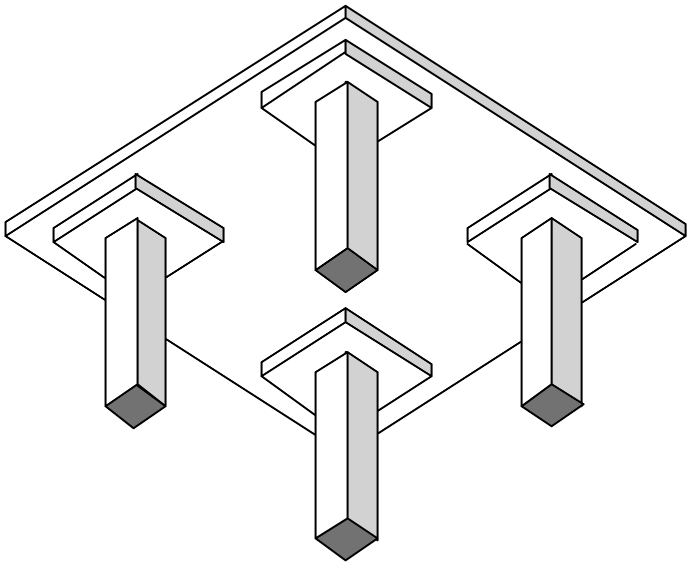
- the slab thickness is usually determined by the need to avoid punch through at the slab/column junction, and then this thickness is used throughout
- it is economical on formwork (labour), but expensive on material

A *flat slab* has a thickening at the slab/column junction

- the thickening can be in the slab – called a drop panel
- the thickening can be at the top of the column – called a *column capital*



flat plate



flat slab (with drop panels)

- ‘rule-of-thumb’ l/d = 33 (based on *longer* span)

3. Prestressed concrete

Prestressing refers to the practice of placing high tensile wires in the concrete, and stretching them once the concrete has hardened.

Prestressed concrete will not be dealt with in this subject, but it is commonly used in floor slabs and beams, and results in thinner slabs and beams and/or longer spans.

4. Choosing an appropriate floor system

Referring to the factors outlined at the beginning,

- cost depends upon local practices, but in developed economies easier formwork leads to lower costs
- less material does not usually lead to lower costs
- flat slabs can give serviceability problems unless carefully designed
- prestressed slabs do not adapt well to future changes of use, because of the critical nature of the prestressing wires

A *guide* to the appropriate span range of the various systems is:

one way slabs – up to 6 metres
one way slabs (prestressed) – up to 9 metres

one way slabs (on metal deck) – up to 3 metres
two way slabs – up to 7.5 metres
ribbed slabs – up to 8 metres
band beams – up to 8 metres
band beams (prestressed) up to 10 metres
flat plates – up to 6 metres
flat slabs – up to 8 metres
precast planks – up to 12 metres
precast T-beams – up to 15 metres

And note, many products in the building industry work on a module of 600 mm (eg ceiling systems, brick and blockwork). Where possible try to use floor spans which are multiples of 600 mm, eg

6.0 m – a useful shorter span
7.2 m – a very common span
8.4 m – quite a common span
10.2 m – a long span

But remember, very often other considerations will prevent a 600 mm module being adopted.

TOPIC 3: FACADES

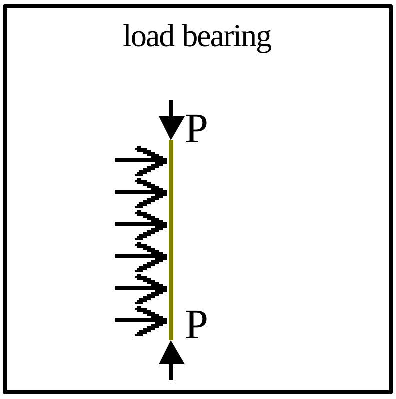
1. What is a façade?

The facade is the *walls* of a building

- control access and security
- reduce temperature effects on the occupants
- admit daylight
- prevent rain penetration
- seal against cold or hot winds
- allow ventilation
- reduce noise penetration
- minimise energy consumption
- be cost effective
- be adequately strong and stiff
- be durable
- easily constructed and maintained

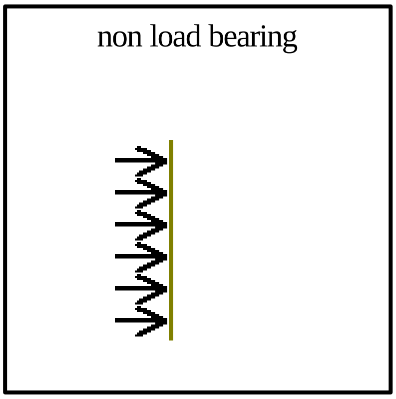
2. Classifying facades

Facades can be either load bearing or non load bearing. Non load bearing facades require a separate structural frame to support the loads.



load bearing

- concrete
- masonry (brick and block)
- timber


non load bearing

- concrete
- masonry (brick and block)
- timber
- metal decking
- glass
- GRC and GRP

3. A facade tour

Views of a range of façade types will be shown in the lecture.

4. Common issues:

- weatherproofing
- allowing for movement
- examples of defects

4.1 Weatherproofing

- impermeable facades
  - glass
  - plastic
  - metal sheeting
- low permeability facades
  - thick concrete or masonry
- cavity wall construction
  - single leaf concrete or masonry

Impermeable facades

- do not allow ventilation - can cause condensation problems
- require careful detailing at junctions
- usually require an extensive framing system

Glass wall systems

- use proprietary components
- aluminum sections are complex extruded shapes
- curtain walls are large assemblies of glass and infill panels supported by a grid of aluminum members

Low permeability facades

- when wet, water penetrates into the façade by capillary action
- when dry, water evaporates from the faces as vapour
- the wall must have sufficient thickness to ensure that the absorbed water can be stored within the façade until the wall dries, without reaching the inside face
- acceptable in tropical climates
- not reliable in temperate climates
- often found in old (pre 20th century) buildings

Precast concrete walling

- can be load bearing or non load bearing
- coatings reduce permeability
- can have a cavity system in critical applications
- sealing of joints between panels is critical

Cavity wall construction

- the wall is constructed as two 'leafs' with a vertical gap between
- water penetrates through the outer leaf of the wall, and drains down the inside of the outer leaf
- 'weepholes' are provided at the base of the cavity to drain the water away
- the inside leaf remains dry
- the wall allows some ventilation

4.2 Allowing for movement

Facades change dimensions and shape due to:

- volumetric change of materials over time
  - concrete and concrete masonry shrinks (300 microstrain)
  - clay masonry expands (500 microstrain)
- thermal expansion and contraction
  - concrete (10 microstrain per ^{o}C)
  - steel (12 microstrain per ^{o}C)
  - aluminum (24 microstrain per ^{o}C)
  - clay masonry (6 microstrain per ^{o}C)
  - glass (10 microstrain per ^{o}C)
- movements in the supporting structure
  - foundation movements
  - deflections in supporting beams

5. Examples of defects

Views of a range of façade defects will be shown in the lecture

TOPIC 4: LOADING

1. Introduction

It is a very important step in the total design process to determine the design loads for the structure. Typical loads for a building are dead load, live load, wind load and earthquake load. Special consideration is sometimes given to impact and fatigue that may occur in vehicles, cranes or machinery.

2. Load Combination

We must consider combinations of various loads that can be imposed on the structure. As the number of loads included in the combination increases, it is customary to introduce a "live load combination factor" (ψ_c), which takes into account the improbability of the maximum value of each load occurring simultaneously. Load combinations are given in Australian Standard 1170.1 –1989. Please also pay attention to the direction of loads.

Different load factors are used for strength limit state design and for serviceability limit state design. Typical examples are given below:

For strength limit state design

$1.25G + 1.5\,Q$
or
$1.25G + W_u + \psi_c Q$
or
$1.25G + 1.6F_{eq} + \psi_c Q$

where
G = dead load
Q = live load
W_u = wind load
F_{eq} = earthquake load

For serviceability limit state design

W_s
or
$\psi_s Q$
or
$G + W_s$
or
$G + \psi_s Q$

where
W_s = wind load for serviceability limit state

3. Dead Load (G)

Dead load consists of the weight of the structure itself plus the weight of permanently installed equipment. It includes the weight of the structural members, floors, ceilings, ductworks, exterior walls, permanent partitions and unusual items such as water in swimming pools. Dead load can usually be estimated with reasonable accuracy. Dead loads are specified in AS1170.1-1989.

4. Live Load (Q)

Live load includes the loads specified by the loading standards for various uses and occupancies of the building. These specified loads cover the occupants, furniture, movable equipment, fixtures, books etc, and are the minimum gravity live loads for which the building can be designed within the jurisdiction of that standard. Live loads are specified in AS1170.1-1989. Live load includes impact and inertia loads. Live load excludes wind, snow and earthquake loads.

Roof:

Case	Q (kPa)
if non-trafficable	0.25
If trafficable	3

Floor: see Table on Pages 411 – 415, SAA HB2.2 –1998
e.g.

Case	Q (kPa)
Parking including driveways and ramps for houses	3
Bedrooms and private rooms in residential and apartment buildings	2
Kitchens	5
Corridors, hallways, passageways, foyers, lobbies, public spaces, stairs and landings in office buildings, subject to crowd loading only	4
Dressing rooms in theatres	2

5. Wind Load (W)

Wind loads are stipulated in AS1170.2-1989. For most structures, wind load can be treated as a static load and is computed with the aid of reference velocity pressures, gust factors, exposure factors and shape factors. Tall, slender buildings must be designed using a dynamic approach to the action of wind gusts or with the aid of experimental methods such as wind tunnel tests.

5.1 Wind load depends on

Wind direction
Wind speed
Structure height

Structure shape
Structure component (wall or roof)
Region (which city)
Topographic condition

5.2 Design wind load

- Ultimate strength limit state design wind load (p_d)
- Serviceability design wind load = p_d x serviceability multiplying factor

serviceability multiplying factor = 0.6 for region A, 0.4 for regions B and C, 0.35 for region D

$p_d = p' \times B_1 \times B_2 \times B_3 \times B_4$

B_1 to B_4 are factors
p' = basic pressure (for wall and roof)

5.3 Factors

B_1 = regional factor

Region	B_1
A = normal	1.0
B= intermediate	1.5
C= tropical cyclone	2.3
D = severe tropical cyclone	3.3

B_2 = terrain and height factor, see Table 2.5.2
e.g. for suburban, sheltered condition, H=10m, B_2 = 0.65

B_3 = topographic factor
For flat area, B_3 = 1.0

B_4 = roof reduction area depending on tributary area, see Table 2.5.4
For wall, B_4 = 1.0
For flat roof, B_4 = 1.0 is taken for simplicity

5.4 Basic pressure (p')

- Needs to be done separately for TWO wind directions
- Needs to be done separately for wall and roof
- Needs to be done separately for external pressure and internal pressure
- Needs to combine external pressure and internal pressure

Terminology:

Dimension in the direction of the wind

External pressure
Internal pressure
Windward sections
Leeward sections
Positive pressure and negative pressure

For roof (external pressure) – see Table 2.4.1.2
Where d = the minimum roof plan dimension in the direction of the wind
e.g. flat roof ($\alpha = 0^o$), if h_t/d <0.5, negative external pressure = -0.95 kPa

For wall (external pressure) – see Table 2.4.1.4
e.g. for normal building, windward side, p'=0.75 kPa

For roof and wall (internal pressure) – see Table 2.4.2
e.g. for normal building with dominant openings, maximum negative pressure = -0.7 kPa
and maximum positive pressure = 0.75 kPa

Combination of external pressure and internal pressure
Get the worst case
Depending on design of individual member or the whole frame

6. Earthquake Load

Conventional earthquake (seismic) design procedures replace the dynamic earthquake loads with equivalent static loads. The earthquake loads which are stipulated are recognised to be much less than the maximum loads possible from a very severe earthquake. However, AS1170.4-1993 attempts to stipulate earthquake loads large enough to prevent structural damage and minimise other damage in moderate earthquakes which occasionally occur, and to avoid collapse or serious damage in severe earthquakes which seldom occur.

6.1 Earthquake load depends on

Physical items	How to take care of it in a design code
Structure types	Types I, II and III (see Appendix A of AS1170.4-1993)
Location of the structure (which city)	Acceleration coefficient (a) in Table 2.3
Soil profile	Site factor (S) ranging from 0.67 (rock) to 2.0 (soft soil), taken as 1.5 if soil condition is unknown

The earthquake design category (A, B, C, D, or E) can be determined using the above three items – see Table 2.6, AS1170.4-1993.

For category A and B – no analysis is needed

For category C, D and E – analysis is needed

6.2 Static analysis

- Determine the total horizontal earthquake base shear force (V)
- Determine the horizontal earthquake force at each level (F_x)

F_x is certain percentage of V depending on the gravity load and height of each level.

6.3 Total base shear force (V)

Physical items	How to take care of it in a design code
Importance of the building	Importance factor (I), I=1 for type I and II, I=1.25 for Type III
Total gravity load	Sum of gravity load G_g for all levels (= $G + \psi_c Q$)
Location of the structure (which city)	Acceleration coefficient (a) in Table 2.3
Soil profile	Site factor (S) ranging from 0.67 (rock) to 2.0 (soft soil), taken as 1.5 if soil condition is unknown
Structural period	T depends on the total height (in metres) of the structure – see clause 6.2.4 of AS1170.4-1993
Structural system	R_f in Table 6.2.6 (a) of AS1170.4-1993

$$V = \frac{1.25 \cdot I \cdot G_g \cdot a \cdot S}{T^{2/3} \cdot R_f}$$

Fundamental period: $T = \frac{h_n}{46}$

Period for the orthogonal direction: $T = \frac{h_n}{58}$

6.4 Horizontal earthquake force at each level (F_x)

F_x depends on the gravity load and height of each level.

$$F_x = C_{vx} \cdot V$$

If the height of the structure is less than 23 meters: $C_{vx} = \frac{G_{gx} \cdot h_x}{\sum_{i=1}^{n} G_{gi} \cdot h_i}$

G_{gx} = portion of gravity load located at level x
h_x = height above the structural base of the structure to level x
G_{gi} = portion of the gravity load located at level i

h_i = height above the structural base of the structure to level i
n = number of levels in structure

7 Example

If V = 125 kN, find the horizontal force at level 3 with the conditions given below.

Level (i)	G_{gi}	h_i
1	1000 kN	3.5 m
2	500 kN	7.5 m
3	250 kN	12.5 m

$$C_{v3} = \frac{G_{g3} \cdot h_3}{\sum_{i=1}^{n} G_{gi} \cdot h_i} = \frac{250 \bullet 12.5}{1000 \bullet 3.5 + 500 \bullet 7.5 + 250 \bullet 12.5} = \frac{3125}{10375} = 0.301 = 30.1\%$$

$F_3 = C_{v3}$ x V = 30.1% x 125 = 37.6 kN

It can also be found that C_{v1} = 33.7% and C_{v2} = 36.1%.

TOPIC 5: LIMIT STATE DESIGN AND METHODS OF ANALYSIS

1. Concept of Limit State Design

1.1 Working Stress Method

There are uncertainties in load, fabrication, material and theoretical models. Two different methods are available to take into account the uncertainties, namely the working stress design method and the limit state design method.

The working stress method can be expressed as:

$$S^* \leq \frac{S}{SF}$$

where

S = Nominal Stress Capacity

SF= Safety Factor

S* = Design Stress

Stress: Normal or Shear

The disadvantages of working stress method are:

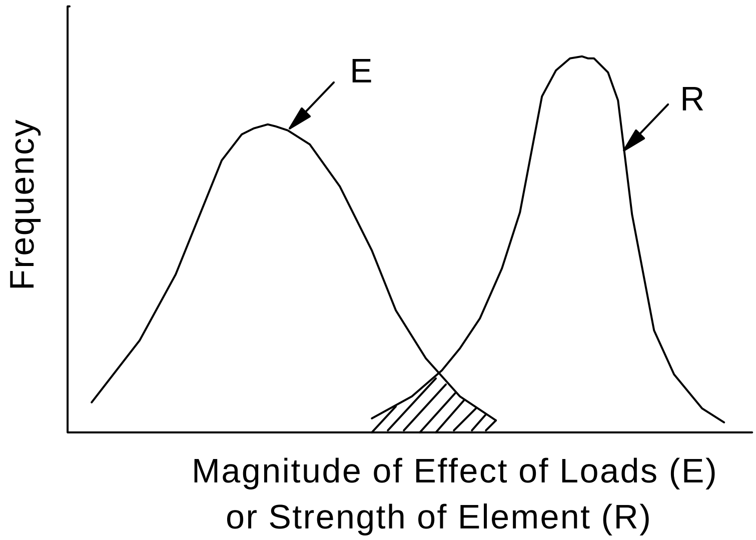
- Not consistently reliable
(One safety factor covers too many cases)
- Too conservative (Wastes material)

1.2 What is Limit State Design?

Limit state design is a design method in which the performance of a structure is checked against various limiting conditions at appropriate load levels. The limiting conditions to be checked in structural steel design are ultimate limit state and serviceability limit state. Ultimate limit states are those states concerning safety, for example, load-carrying capacity, overturning, sliding, and fracture due to fatigue or other causes. Serviceability limit states are those states in which the behavior of the structure under normal operating conditions is unsatisfactory, and these include excessive deflection, excessive vibration, and excessive permanent deformation.

In essence, the designer attempts to ensure that the maximum strength of a structure (or elements of a structure) is greater than the loads that will be imposed upon it, with a reasonable margin against failure. This is the “ultimate limit state” criterion. In addition, the designer attempts to ensure that the structure will fulfill its function satisfactorily where subjected to its service loads. This is the “serviceability limit state” criterion.

The ultimate limit state criterion can be illustrated by the Figure shown below. This figure shows hypothetical frequency distribution curves for the effects of loads on a structural element and the strength, or resistance, of the structural element. Where the two curves overlap, shown by the shaded area, the effect of the loads is greater than the resistance of the element, and the element will fail. The structure must be proportioned so that the overlap of the two curves is small, and hence the probability of failure occurring is small enough to be acceptable.



1.3 Limit State Design (Adopted in AS4100)

The basic equation for checking the limit state condition is:

$$S^* \leq \Phi \cdot R_n$$

where

R_n = Nominal Capacity
Φ= Capacity Factor
S^* = Design Load (action) Effects

Action: axial force, bending moment, shear force, torques

Advantages:

- Consistent Reliability

(different Φ for different cases, calibrated using reliability analysis,
i.e certain reliability index and probability of failure)

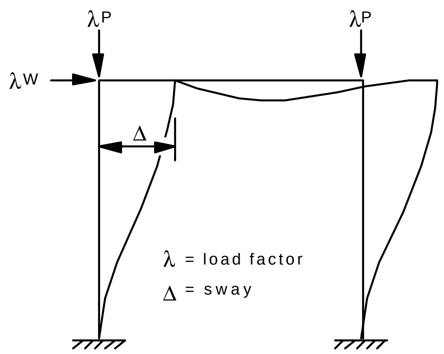
- More Economical

(saves material)

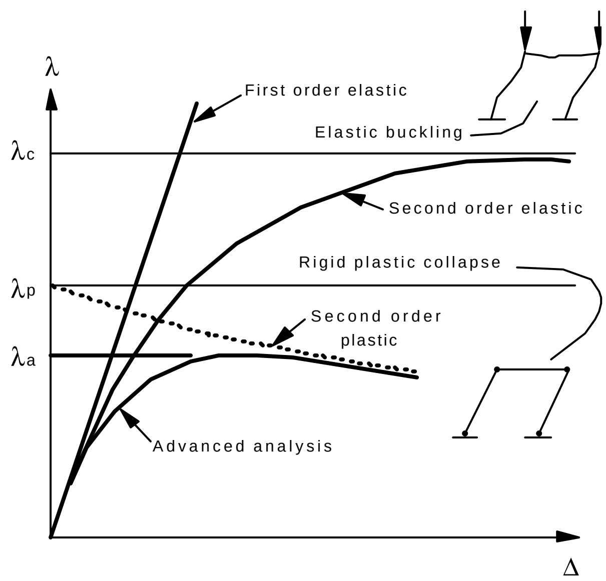
2. Methods of Analysis

The following methods are available as shown in the figure below.

a) 1st order Elastic
b) 2nd order Elastic
c) Plastic
d) Elastic Buckling
e) Advanced



(a) Rigid jointed sway frame



(b) Load deflection responses

2.1 1st order Elastic Analysis

The assumptions made are: geometry of structure remains unchanged and material properties remain unchanged.

Principle of superposition applies.

2.2 2nd order Elastic Analysis

The assumptions made are: geometry of structure keeps changing and material properties remain unchanged.

Principle of superposition does not apply.

2.3 1st Order Plastic Analysis

The assumptions made are: geometry of structure remains unchanged and material properties keep changing.

The design action effects shall be determined using a rigid plastic analysis. It shall be permissible to assume full strength or partial strength connections, provided the capacities of these are used in the analysis. The rotation capacity at none of the hinges in the collapse mechanism must be exceeded.

2.4 Elastic Buckling Analysis

The assumptions made are: only $P\text{-}\delta$ effect is considered and only elastic buckling load is obtained.

2.5 Advanced Analysis

The assumptions made are: geometry of structure keeps changing and material properties keep changing. The analysis includes residual stresses, geometrical imperfections and erection procedures.

For a frame comprising members of compact section with full lateral restraint, an advanced structural analysis may be carried out provided the analysis can be shown to accurately model the behaviour of that class of frame. The analysis shall take into account the relevant material properties, residual stresses, geometrical imperfections, second order effects, erection procedures and interaction with the foundations. For the strength limit state, it shall be sufficient to satisfy the section capacity requirements for the members and the requirements for the connections.

3. Second Order Effects

3.1 Moment Amplification Factor

a) 1st Order Elastic Analysis $\rightarrow M^*_m$

b) 2nd Order Effects

$$M^* = \delta_b \cdot M^*_m$$

$$M^* = \delta_s \cdot M_m^*$$

δ_b = Amplification Factor for braced member
δ_s = Amplification Factor for Sway member

3.2 Braced Member

a) δ_b = 1.0 if axial force is zero or tensile

b) Axial force is compressive

$$\delta_b = \frac{C_m}{1 - \frac{1}{\lambda_m}} \qquad \lambda_m = \frac{N_{omb}}{N^*} \text{ for each member}$$

$$C_m = 0.6 - 0.4 \cdot \beta_m \leq 1.0$$

βm depends on moment distribution
(see Figure 4.4.2.2 of AS4100)

N* = Axial force (1st order analysis)

N_{omb} = Elastic buckling load

$$N_{omb} = \frac{\pi^2 \cdot E \cdot I}{(k_e \cdot L)^2}$$

k_e = Effective length factor which depends on end restraints:

For idealised conditions, use Table 4.6.3.2 of AS4100.
For other conditions, use γ Method, i.e.

$$g = \frac{\sum (\frac{I}{L})_c}{\sum b_e (\frac{I}{L})_b}$$

The quantity $\sum(I/L)_c$ shall be calculated from the sum of the stiffness in the plane of bending of all the compression members rigidly connected at the end of the member under consideration, including the member itself.
The quantity $\sum(I/L)_b$ shall be calculated from the sum of the stiffness 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.

The modifying factor (β_e) which accounts for the conditions at the far ends of the beams are given in the table below.

Fixity conditions 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

There are two special cases:
(a) for a compression member whose base is not rigidly connected to a footing, the γ value shall not be taken as less than 10.
(b) for a compression member whose end is rigidly connected to a footing, the γ value shall not be taken as less than 0.6.

3.3 Sway Members

$$\delta_s = \frac{C_m}{1 - \dfrac{1}{\lambda_{ms}}} \qquad l_{ms} = \frac{\sum \left(\dfrac{N_{oms}}{L}\right)}{\sum \left(\dfrac{N^*}{L}\right)} \quad \text{for each storey}$$

$N_{oms} = \pi^2 EI / (k_e L)^2$ for each column

N^* = axial force in each column with tension taken as negative

Summation includes all columns within a storey

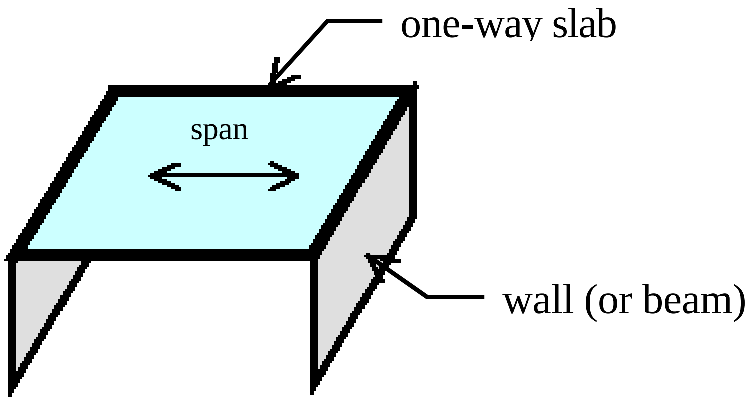
Limitation: $\delta \leq 1.4$

If $\delta > 1.4$ a 2nd Order analysis is required, corresponding $\lambda_{ms} = 3.5$

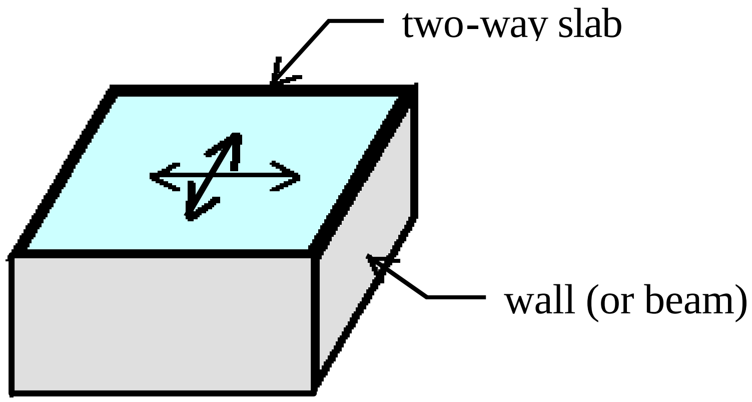
TOPIC 6: FRAME ANALYSIS

TOPIC 7: DESIGN OF REINFORCED CONCRETE FLOOR SLABS

1. Introduction



one way slabs - simply supported (guess l/d = 24)
- continuous (guess l/d = 28)



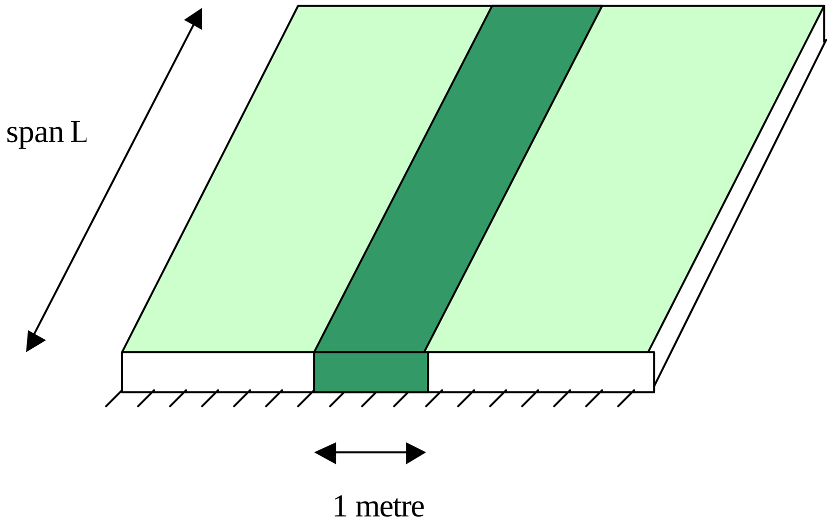
two way slabs - simply supported (guess l/d = 28)
- continuous (guess l/d = 39)

The important considerations for slab design are:
ultimate limit state – bending
serviceability limit state – deflection and cracking

Note that shear capacity is almost never a problem (load intensity is lower than on beams, and the shear capacity is higher because the section is thinner)

2. One-way spanning reinforced concrete slabs

Think of the slab as a beam with a width of one metre.

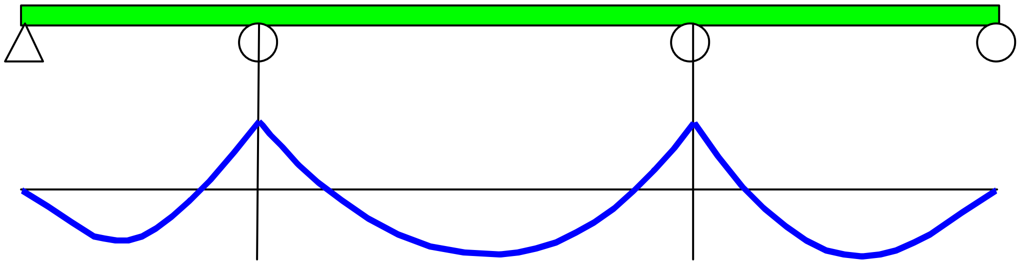


- Calculate M* in kN-m per m width
- Determine bending reinforcement - use the beam bending equation,

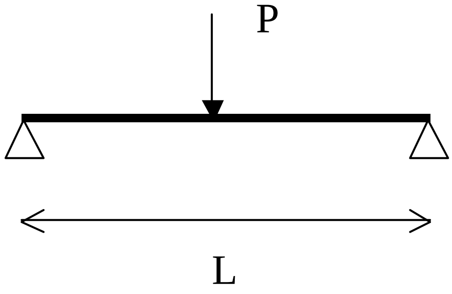
$$\phi M_u = \phi A_{st} f_{sy} d \left(1 - \frac{p f_{sy}}{1.7 f'_c} \right)$$

- Shear is not usually a problem (ligatures not required)
- Deflection (serviceability limit state) is often the critical requirement

If the slab is continuous, design a one metre wide strip as a continuous beam – use matrix analysis to calculate the bending moment diagram,



3. What affects the deflection of slabs?



$$\Delta = \frac{PL^3}{48EI}$$

P ✓ guidance from the loading code

L ✓

E - non linear
time dependent
shrinkage affected

I - varies with the reinforcing and the load

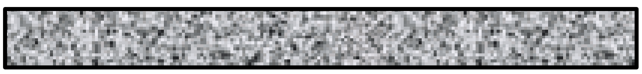
Concrete is not a linear elastic material, *but*, we do give a value to E.

$$E = \rho^{1.5} \times 0.043\sqrt{f_{cm}}$$

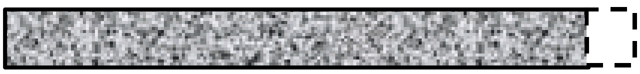
ρ = concrete density
f_{cm} = *mean* compressive strength

3.1 Shrinkage

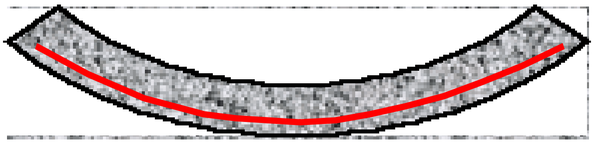
When it dries, this concrete,



shrinks to become this concrete,



and if I restrain *one* face with steel reinforcing bars, it becomes this concrete:



So how do I minimise the effect of shrinkage?

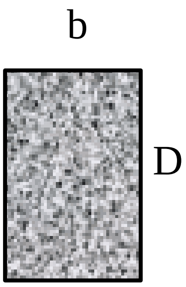
3.2 Creep

What happens if we maintain a load on the concrete?
CREEP happens.

$$\frac{\text{ultimate strain}}{\text{creep strain}} \approx 2.5$$

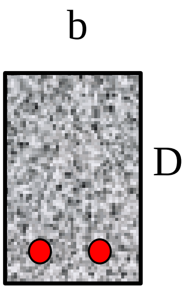
3.3 The value of I

gross section



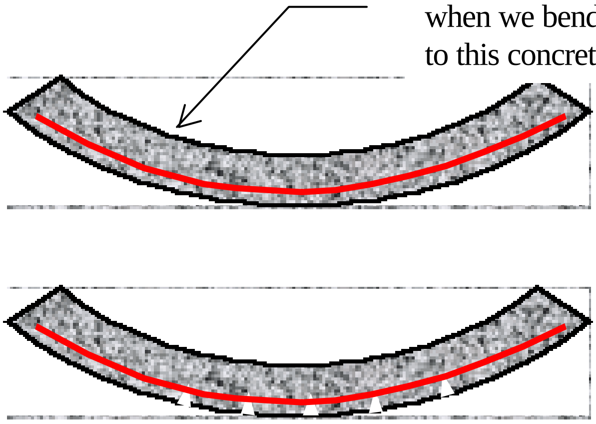
What is I?

uncracked section

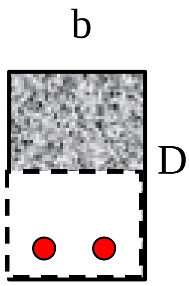


What is I?

when we bend the beam, what to this concrete?



cracked section



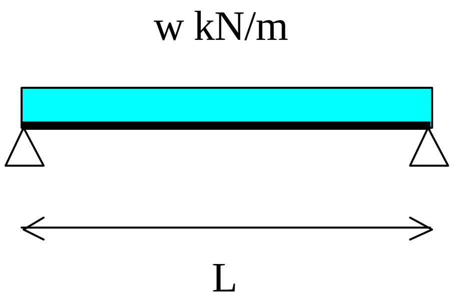
What is I?

I varies ‘unpredictably’ along the beam

3.4 Span to depth ratios

To avoid calculating the non-linear time dependent cracking, creep and shrinkage effects, we can use deemed to comply span to depth ratios.

For a simple beam,



$$\Delta = \frac{5}{384} \times \frac{wL^4}{EI}$$

$$\therefore \frac{\Delta}{L} = \frac{5}{384} \times \frac{wL^3}{E\left(\frac{I}{bd^3}\right)bd^3}$$

$$\therefore \left(\frac{L}{d}\right)^3 = \frac{\left(\frac{I}{bd^3}\right)\left(\frac{\Delta}{L}\right)bE}{\left(\frac{5}{384}\right)w}$$

$\frac{L}{d}$ as calculated must be greater than the actual $\frac{L}{d}$ of the beam, if the SERVICEABILITY LIMIT STATE is to be satisfied.

In AS3600 this is expressed as,

$$\frac{L}{d} = \left[\frac{0.045\left(\frac{\Delta}{L}\right)bE}{k_2 F_{def}}\right]^{\frac{1}{3}}$$ for beams (Cl. 8.5.4)

or

$$\frac{L}{d} = \left[\frac{0.053\left(\frac{\Delta}{L}\right)bE}{k_2 F_{def}}\right]^{\frac{1}{3}}$$ for slabs (Cl. 9.3.4, rearranged)

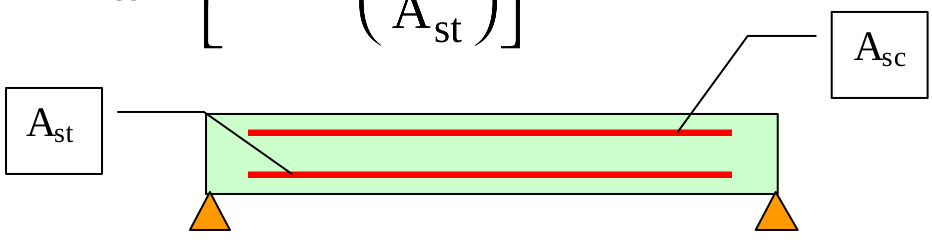
(for an uncracked section, $\frac{I}{bd^3} = \frac{1}{12} = 0.083$)

b	=	width	(one metre for a slab)
k_2	=	5/384	for a simple span
	=	1/185	for an end span
	=	1/384	for an interior span

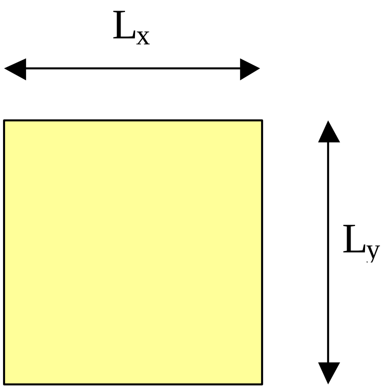
F_{def} = load per metre, including allowance for shrinkage and creep

= $(1.0 + k_{cs})g + (\psi_s + k_{cs}\,\psi_l)q$

where $k_{cs} = \left[2 - 1.2\left(\frac{A_{sc}}{A_{st}}\right)\right] \geq 0.8$



4. Two way spanning reinforced concrete slabs



L_x is the shorter span

Bending moments

spanning in the x direction, $M^*_x = \beta_x \, w \, L_x^2$

spanning in the y direction, $M^*_y = \beta_y \, w \, L_x^2$

note that it is L_x in both cases

(if $L_y/L_x > 2$, treat as 1-way spanning, expect $M^*_x = wL_x^2/8$, $M^*_y = 0$)

The values of β are found in AS3600 Table 7.3.2:

edge condition	short span coefficients (β_x) values of L_y/L_x								β_y for all L_y/L_x
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	≥2.0	
all edges continuous	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
1 short edge discontinuous	0.028	0.032	0.036	0.038	0.041	0.043	0.047	0.050	0.028
1 long edge discontinuous	0.028	0.035	0.041	0.046	0.050	0.054	0.061	0.066	0.028
2 short edges discontinuous	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
2 long edges discontinuous	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
2 adjacent edges discontinuous	0.035	0.041	0.046	0.051	0.055	0.058	0.065	0.070	0.035
1 long edge continuous	0.043	0.049	0.053	0.057	0.061	0.064	0.069	0.074	0.043
1 short edge continuous	0.043	0.054	0.064	0.072	0.078	0.084	0.096	0.105	0.043
all edges discontinuous	0.056	0.066	0.074	0.081	0.087	0.093	0.103	0.111	0.056

-ve moment over the beams = 1.33 x corresponding +ve moment

To check the span/depth ratio for two way slabs, first rearrange the formula,

$$\frac{L}{d} = \left[\frac{0.053\left(\frac{\Delta}{L}\right)bE}{k_2 F_{def}}\right]^{\frac{1}{3}} = \left(\frac{0.053}{k_2}\right)^{\frac{1}{3}}\left[\frac{\left(\frac{\Delta}{L}\right)bE}{F_{def}}\right]^{\frac{1}{3}} = k_4\left[\frac{\left(\frac{\Delta}{L}\right)bE}{F_{def}}\right]^{\frac{1}{3}}$$

where,

$$k_4 = \left(\frac{0.053}{k_2}\right)^{\frac{1}{3}}$$

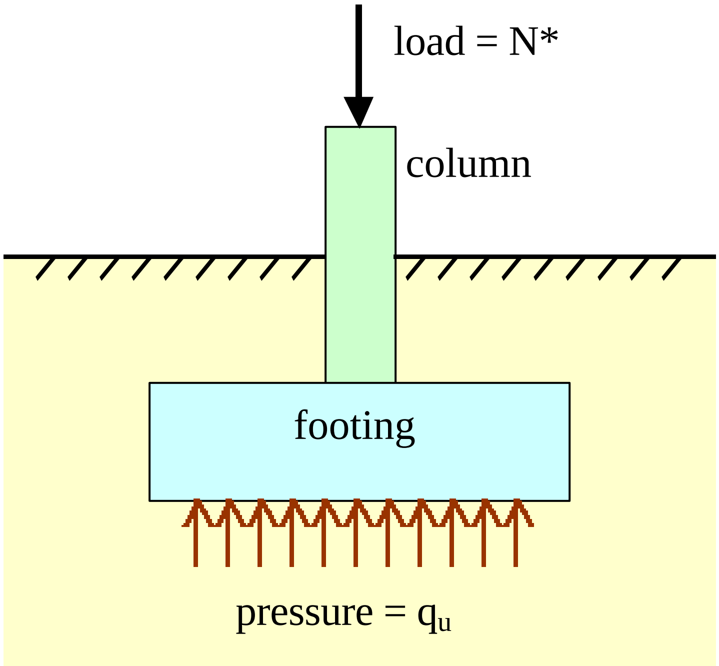
If simply supported, $k_4 = \left(\frac{0.053}{5/384}\right)^{\frac{1}{3}} = 1.6$

To allow for two way spanning slabs, the value of k_4 is adjusted,

edge condition	k_4			
	values of L_y/L_x			
	1.0	1.25	1.5	2.0
all edges continuous	4.00	3.40	3.10	2.75
1 short edge discontinuous	3.75	3.25	3.00	2.70
1 long edge discontinuous	3.75	2.95	2.65	2.30
2 short edges discontinuous	3.55	3.15	2.90	2.65
2 long edges discontinuous	3.55	2.75	2.25	1.80
2 adjacent edges discontinuous	3.25	2.75	2.50	2.20
1 long edge continuous	3.00	2.55	2.40	2.15
1 short edge continuous	3.00	2.35	2.10	1.75
all edges discontinuous	2.50	2.10	1.90	1.70

TOPIC 8: FOOTINGS

1. Introduction

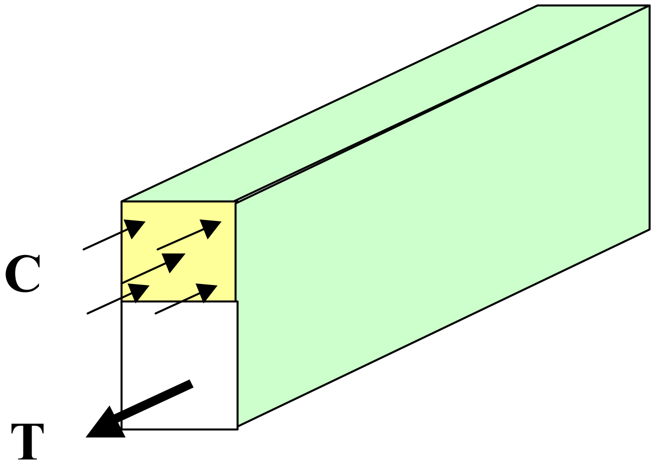


- footings are reinforced concrete elements
- the footing area is determined from the bearing capacity
- they can fail in bending or shear

2. Review reinforced concrete theory

2.1 Bending failure

– moment capacity of under-reinforced sections (ϕ = 0.8 for bending)



Under reinforced means that the steel yields before the concrete crushes – ie there is not *too much* steel

bd	is the area of the cross section
A_{st}	is the area of steel in the cross section
p	= (area of steel)/(area of concrete)
	= A_{st}/bd
d	is the depth to the steel
f_{sy}	is the yield stress of the steel (400 MPa)
f'_c	is the cylinder strength of the concrete (say 32 MPa)
ϕ	is the capacity reduction factor = 0.8

The ultimate moment capacity,

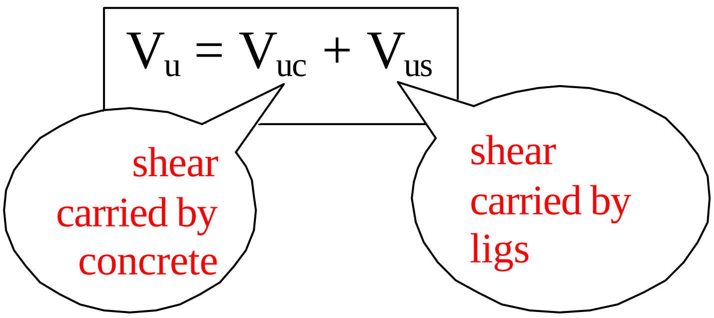
$$\phi M_u = \phi A_{st} f_{sy} d\left(1 - \frac{p f_{sy}}{1.7 f'_c}\right)$$

2.2 Shear failure

(ϕ = 0.7 for shear)

Shear forces are carried by the concrete and the steel (ligatures) acting together,

$$V_u = V_{uc} + V_{us}$$



The shear contribution which can be carried *by the concrete* is,

$$V_{uc} = \beta_1 \beta_2 \beta_3 bd\left(\frac{A_{st} f'_c}{bd}\right)^{\frac{1}{3}}$$

where,

$$b_1 = 1.1\left(1.6 - \frac{d_0}{1000}\right) \geq 1.1,$$ is a depth effect

$\beta_2 = 1$, unless axial tension is large
$\beta_3 = 1$ unless the load is near to the support

After calculating V_{uc},

(a) if $V^* < 0.5\phi V_{uc}$
no ligs required if D<750 mm
minimum ligs required otherwise

(b) if $0.5V_{uc} < V^* < \phi V_{u\,min}$
no ligs required if $V^* < \phi V_{uc}$ and D < 250 mm or B/2
minimum ligs required otherwise

(c) if $V^* > \phi V_{u\,min}$
ligs must be designed

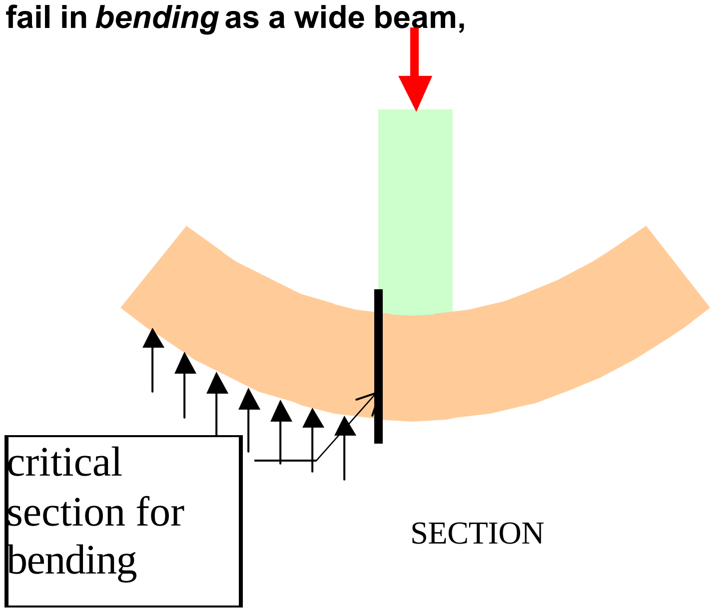
If ligs must be designed, they must provide a shear strength of ϕV_{us}, (found from $V_{us} = V_u - V_{uc}$).

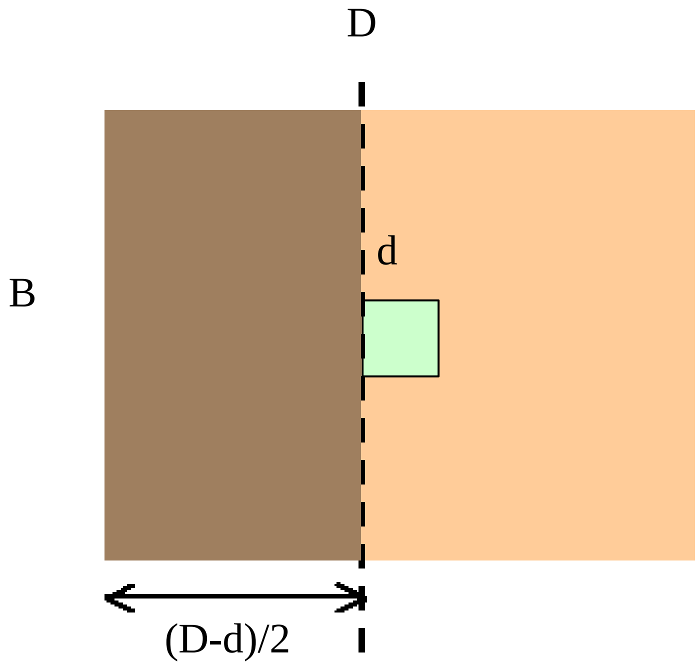
$$\phi V_{us} = 0.7 \times \left(A_{sv} f_{sy.f} \frac{d_0}{s} \right) \cot \theta_v$$

where θ_v varies between 30 and 45 degrees.

3. Apply this to footings?

3.1. Footings can fail in *bending* as a wide beam,

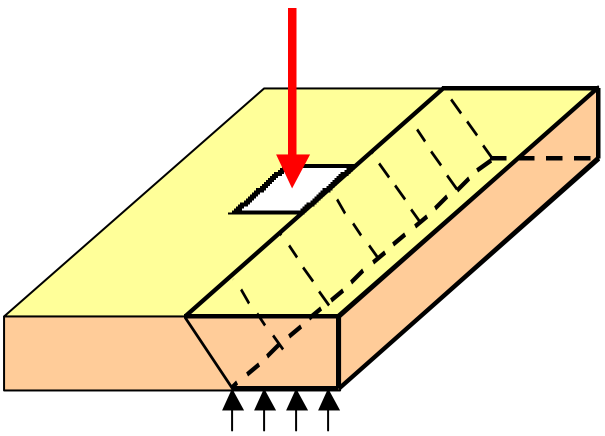




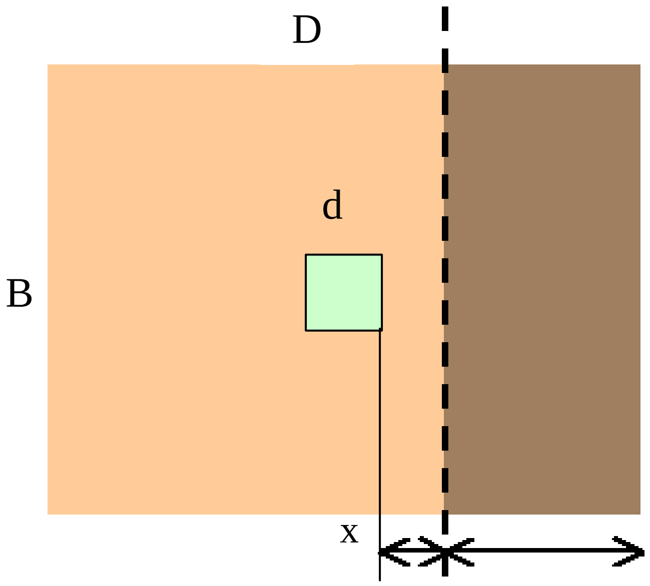
PLAN

$$M^* = \text{force} \times \text{lever arm}$$
$$= q_u\left(\frac{D-d}{2}\right) \times \left[\left(\frac{D-d}{2}\right) \div 2\right]$$

3.2. Footings can fail in *shear* as a wide beam,



The critical section for shear is at 'd' from the face of the column (as for a beam)

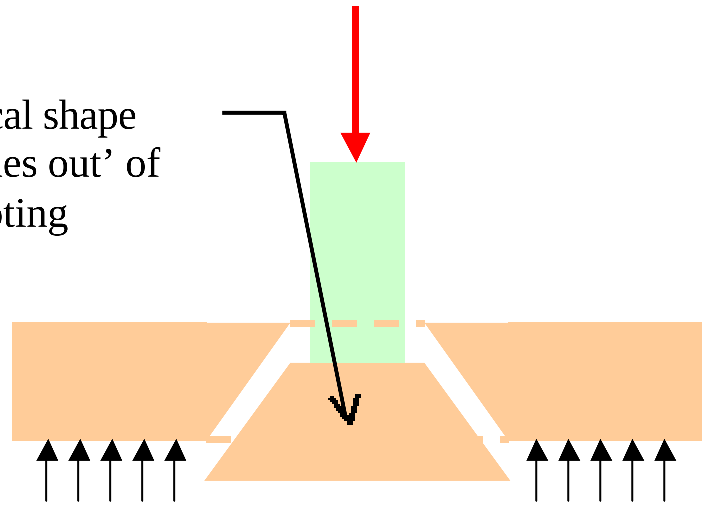


$$V^* = \text{pressure x area}$$

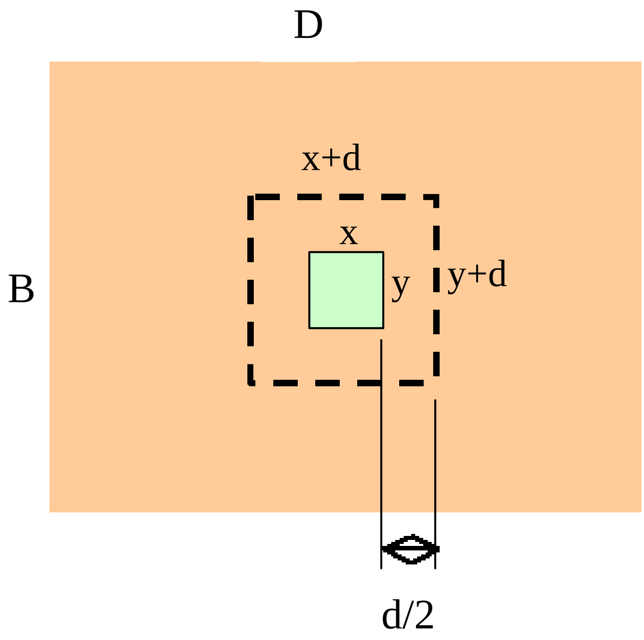
$$= q_u \times B \times \left(\frac{D-d}{2} - d \right)$$

Choose D so that you do not need to use ligatures.

3.3. Footings can fail in a third way, by *punching shear*



The failure cone is approximated as a prism with *vertical sides*, and the sides are taken to be at d/2 from the column face, where d is the effective depth of the footing.



critical perimeter, u = 2(x+d) + 2(y+d)

shear surface = u x d

shear on the shear surface
V* = (total load) - (load on the area inside the critical perimeter)
= N* - q_u x (x+d)(y+d)

shear capacity

$$f V_{uo} = 0.7 \times u \times d \times 0.34 \sqrt{f_c'}$$

Choose D (and hence d) so that V* < ϕV_{uo}

(Punching shear is covered in AS3600 Clause 9.2)

TOPIC 9: COMPOSITE FLOOR SLABS

1. Introduction

Conventional reinforced concrete slabs like this

require a lot of *formwork* to contain the wet concrete until it has gained strength

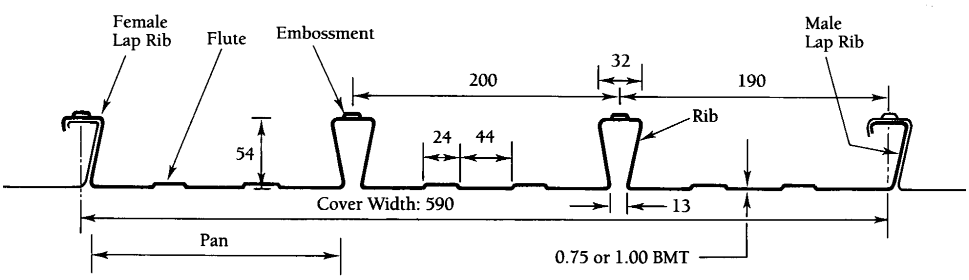
Building and removing formwork is expensive.

Cold formed metal sheeting can be used as *both*

- temporary support for the wet concrete
- permanent steel reinforcing for the slab

In Australia there are 3 manufacturers:

1.Bondek II - BHP Building Products



2. Condeck HP – Stramit Industries

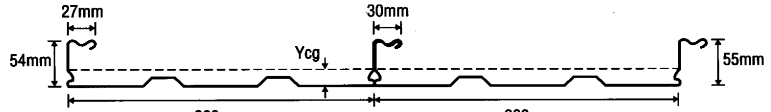


TABLE 3 – Condeck HP Panel Properties per Metre Width

Thickness (Base metal)	Mass	Cross Sectional Area	Y_{cg}	Total Section Properties			I values for calculating deflection		Ultimate moment capacities		Ultimate Shear Capacity	Ultimate Internal Reaction
				I	Z_b	Z_t	I_{sin}	I_{cont}	M_p	M_n	V	R_{int} 100mm bearing
mm	kg/m²	mm²	mm	10^6mm⁴	10^3mm³	10^3mm³	10^6mm⁴	10^6mm⁴	kNm	kNm	kN	kN
0.75	9.89	1211	15.29	0.488	31.89	12.28	0.405	0.340	3.96	5.75	38.93	35.00
0.90	11.80	1456	15.36	0.583	37.97	14.72	0.498	0.408	5.71	7.08	65.98	46.60
1.00	13.07	1620	15.41	0.647	41.98	16.35	0.569	0.486	6.72	7.98	81.78	56.17

3. Comform – Woodroffe Industries

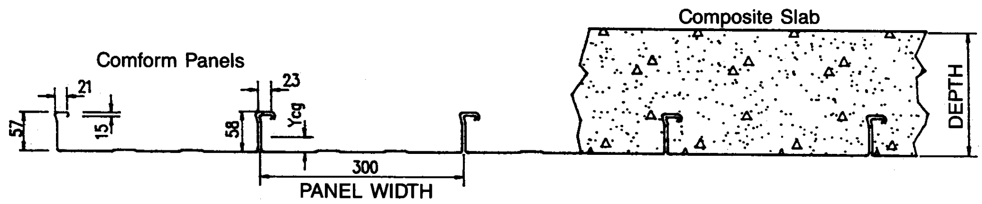
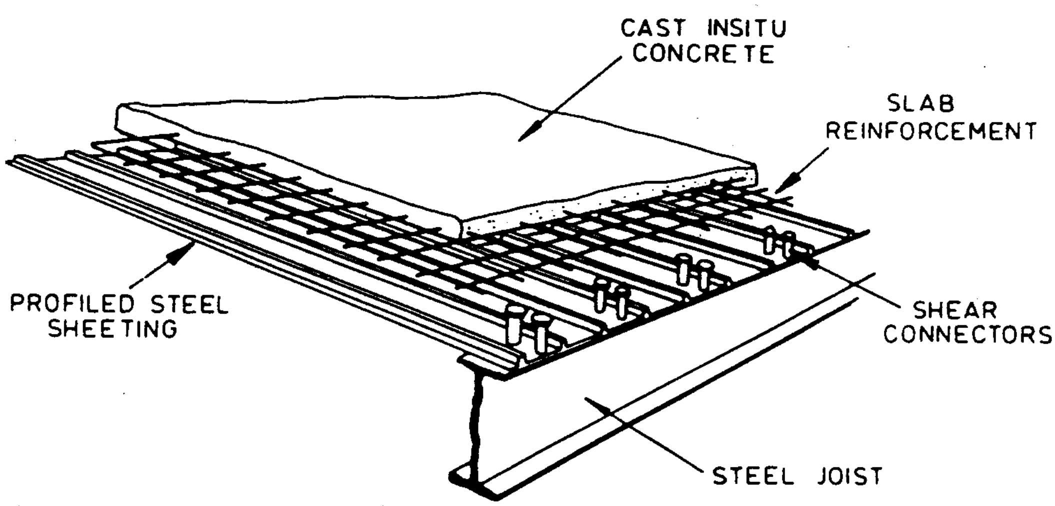


TABLE 1 COMFORM PANEL PROPERTIES PER METRE WIDTH

TCT	MASS	CROSS SECT. AREA	Ycg	POSITIVE BENDING				NEGATIVE BENDING				SAFE END REACTION *	SAFE INTERNAL REACTION *
				Ip	Zpt	Zpc	Mp	In	Znt	Znc	Mn	MAX	MAX
mm	kg/m²	mm²	mm	x 10^6mm⁴	x 10^3mm³	x 10^3mm³	kN.m	x 10^6mm⁴	x 10^3mm³	x 10^3mm³	kN.m	kN	kN
0.79	9.66	1172	14.09	.503	35.63	11.45	3.78	.234	7.46	9.13	2.46	4.40	13.3
1.00	12.13	1485	14.09	.637	45.20	14.50	4.79	.317	10.47	11.87	3.45	7.93	21.6



2. Design issues

No Australian Standard exists yet.
'Design of Composite Slabs for Strength' – BHP Structural Steel, 1998 is the best reference.

- ultimate strength of the bare steel to carry the wet concrete (not considered here)
- ultimate strength of the reinforced concrete slab
    - yielding of the steel (flexure)
    - crushing of the concrete (flexure)
    - slip between the steel and the concrete (longitudinal slip)
- serviceability deflection of the composite slab (treat as for a normal rc slab)

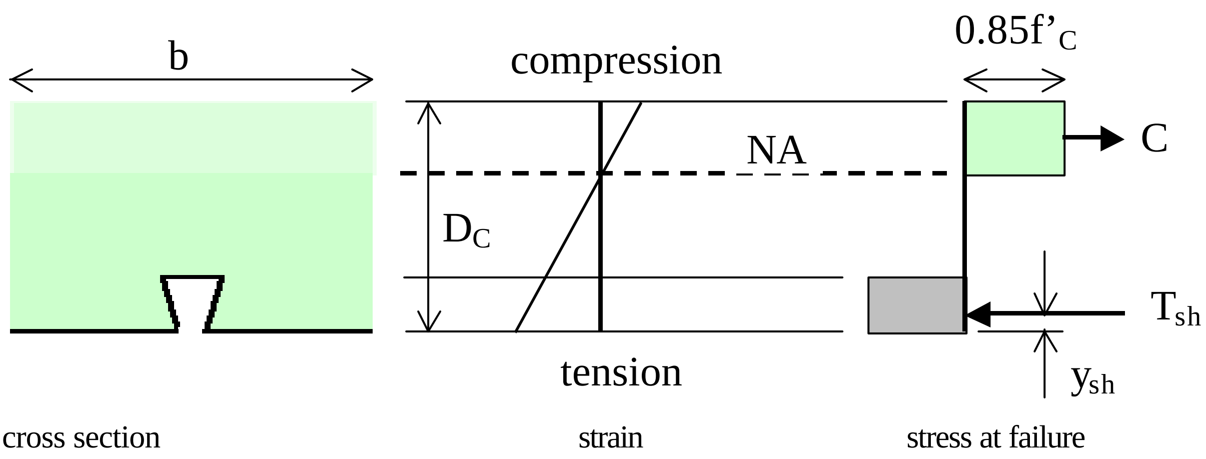
Composite slabs are one-way slabs

3. Flexural theory for composite slabs

3.1 Negative bending

Design for negative bending (hogging) as a normal reinforced concrete slab – ie ignore the metal sheeting.

3.2 Positive bending



b	breadth of the slab
C	compressive force in the concrete
D_C	overall depth of the composite slab (including sheeting)
T_{sh}	tensile force in the sheeting
y_{sh}	height at which the sheeting tensile force acts above the bottom of the slab

$C = 0.85f'_c \times b \times (\text{NA depth})$
from horizontal equilibrium, $C = T_{sh}$

from top of slab to the line of force C = $\dfrac{0.5T_{sh}}{0.85f'_c b}$

lever arm between C and T_{sh} = $D_c - y_{sh} - \dfrac{0.5T_{sh}}{0.85f'_c b}$

therefore, ultimate moment,

$$f\,M_u = f\,T_{sh}\left(D_c - y_{sh} - \frac{0.5T_{sh}}{0.85f'_c b}\right)$$

where $\phi = 0.8$

Assuming the steel has yielded, y_{sh} is tabulated below.

product	y_{sh} (mm)
Condeck HP	12.8
Bondek II	15.5
Conform	13.4

Which leaves us with T_{sh} to find.

Calculation of T_{sh}

There are 3 possibilities for T_{sh} at failure:

1. The sheeting yields (under reinforced – flexural failure)

$$T_{sh} = A_{sh} \times f_{sy.sh}$$

where, A_{sh} = cross section area of sheeting
$f_{sy.sh}$ = design yield stress of sheeting
= 550 MPa for the 3 manufacturers

2. The concrete crushes (over reinforced – flexural failure),

$$T_{sh} = 0.85f'_c(D_c - h_r)$$

where h_r = height of sheeting ribs

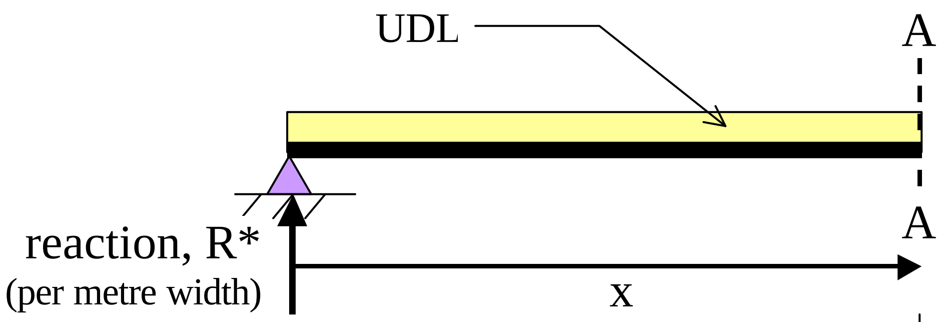
In cases 1 and 2, *complete shear connection* is achieved at failure. In case 3, the concrete and the sheeting do not achieve complete shear connection.

3. The sheeting slips along the concrete/steel interface before either 1 or 2 occurs (longitudinal slip failure),

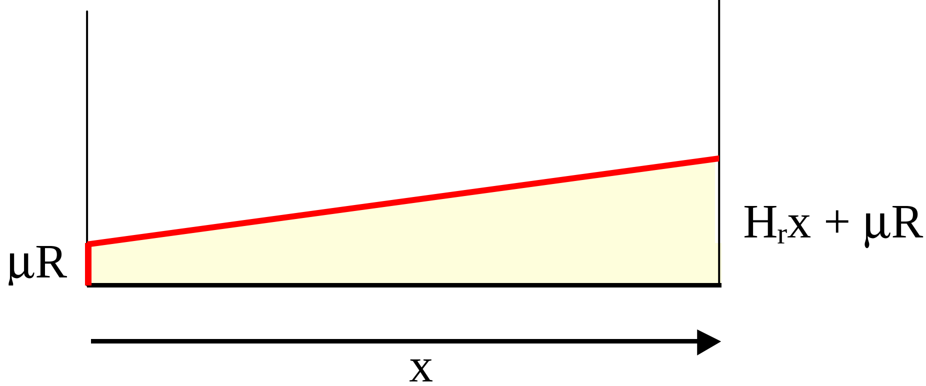
Calculation of the interface shear strength

Shear between the sheeting and the concrete has two components:

1. friction
   - equals vertical reaction at the support (R*, kN) x coefficient of friction (μ)
2. mechanical resistance
   - equals area of sheeting x resistance per unit area of sheeting (H_r, kPa)
   - is due to the geometry of the sheeting cross section



free body diagram



interface shear strength = $H_r x + \mu R$ (per metre width)

So if there is not sufficient distance from the support to the section AA, the cross section will fail by longitudinal slip, rather than by flexural failure.

In summary,

T_{sh} = minimum of

	$T_{sh} = A_{sh} \times f_{sy.sh}$	flexural failure
or,	$T_{sh} = 0.85 f_c'(D_c - h_r)b$	flexural failure
or	$T_{sh} = (H_r x + m R^*)b$	longitudinal slip failure

product	mechanical resistance H_r (kPa)	coefficient of friction μ
Condeck HP	210	0.5
Bondek II 1.00 mm Bondek II 0.75 mm	$88\sqrt{f'_c}$ $76\sqrt{f'_c}$	0.5 0.5
Conform	235	0.5

Note that the strength of composite slabs is a function of the position in the span, as well as the properties of the cross section. This is not the case for reinforced concrete or steel beams.

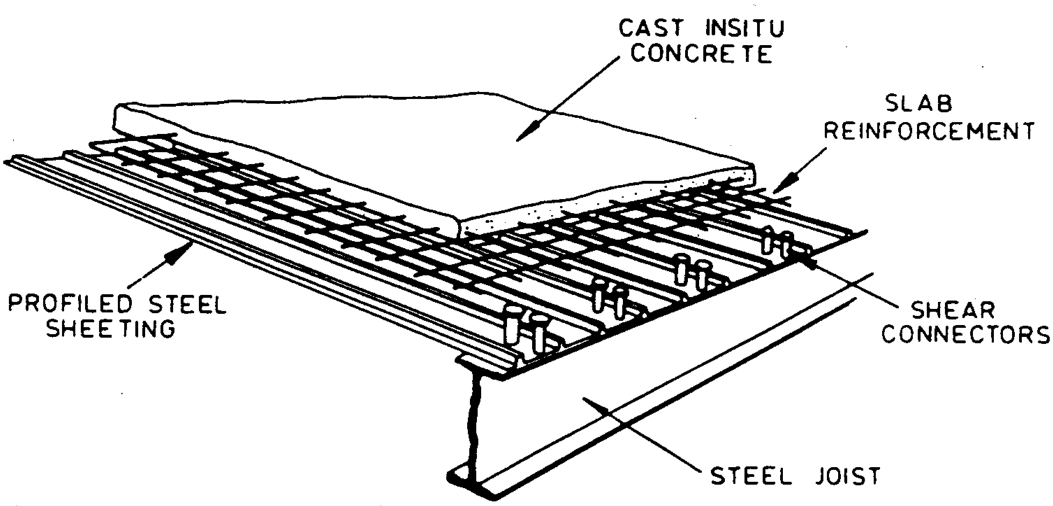
4. Minimum slab thicknesses

It is usual for fire rating and other reasons to maintain a minimum concrete thickness over the ribs of the metal sheeting. This leads to the following minimum slab thicknesses:

product	D_c minimum (mm)
Condeck HP	120
Bondek II	120
Conform	125

TOPIC 10: COMPOSITE STEEL-CONCRETE BEAMS

1. Introduction



Comprise a steel beam and a concrete slab, joined with shear connectors to achieve composite action between the two elements.

In building construction the concrete slab is often (but not always) a composite slab – ie cast on metal sheeting

Composite beam comprise a steel beam connected to a concrete slab with shear connectors, such that the two components act as one beam cross section.

The concrete is in compression and the steel is in tension – this is the best application for each material.

Composite beams are used in buildings and bridges.

2. References

Australian Standard AS2327.1-1996 Composite structures Part 1: Simply supported beams

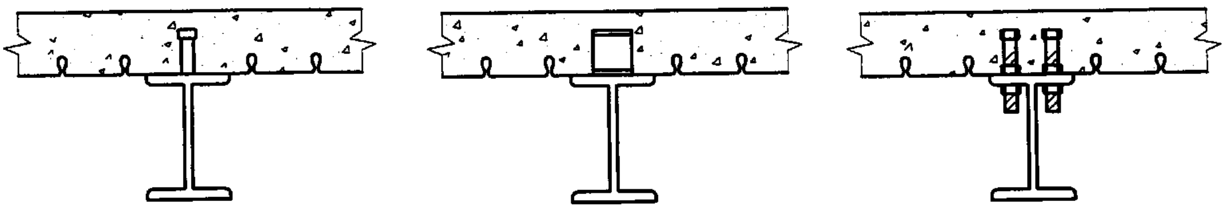
'Design of Simply-Supported Composite Beams for Strength' – BHP Structural Steel, 1998
(624.1771 B575 1998)

The draft *continuous* beam code is still under preparation.

3. Shear connectors

Shear connectors are an essential component of composite beams.

They can be
- welded stud shear connectors (most common)
- welded channels
- high strength bolts



complete shear connection - the strength of the beam cross section is *not limited* by the strength of the shear connection

partial shear connection - the strength of the shear connection *limits* the section capacity

4. Shear strength of welded stud shear connectors, f_{vs}

is the *lesser of* the 2 values below,

If failure occurs in the steel stud,

$$f_{vs} = 0.63 d_{bs}^{2} f_{uc}$$

If failure occurs in the surrounding concrete,

$$f_{vs} = 0.31 d_{bs}^{2} \sqrt{f'_c E_c}$$

where,

- d_{bs} is the stud diameter
- f_{uc} is the characteristic tensile strength of the shear connector material ([500 MPa)
- f'_c is the characteristic compressive strength of the concrete
- E_c is the elastic modulus of the concrete, $E_c = \rho^{1.5} 0.043 \sqrt{f'_c}$

5. Design issues

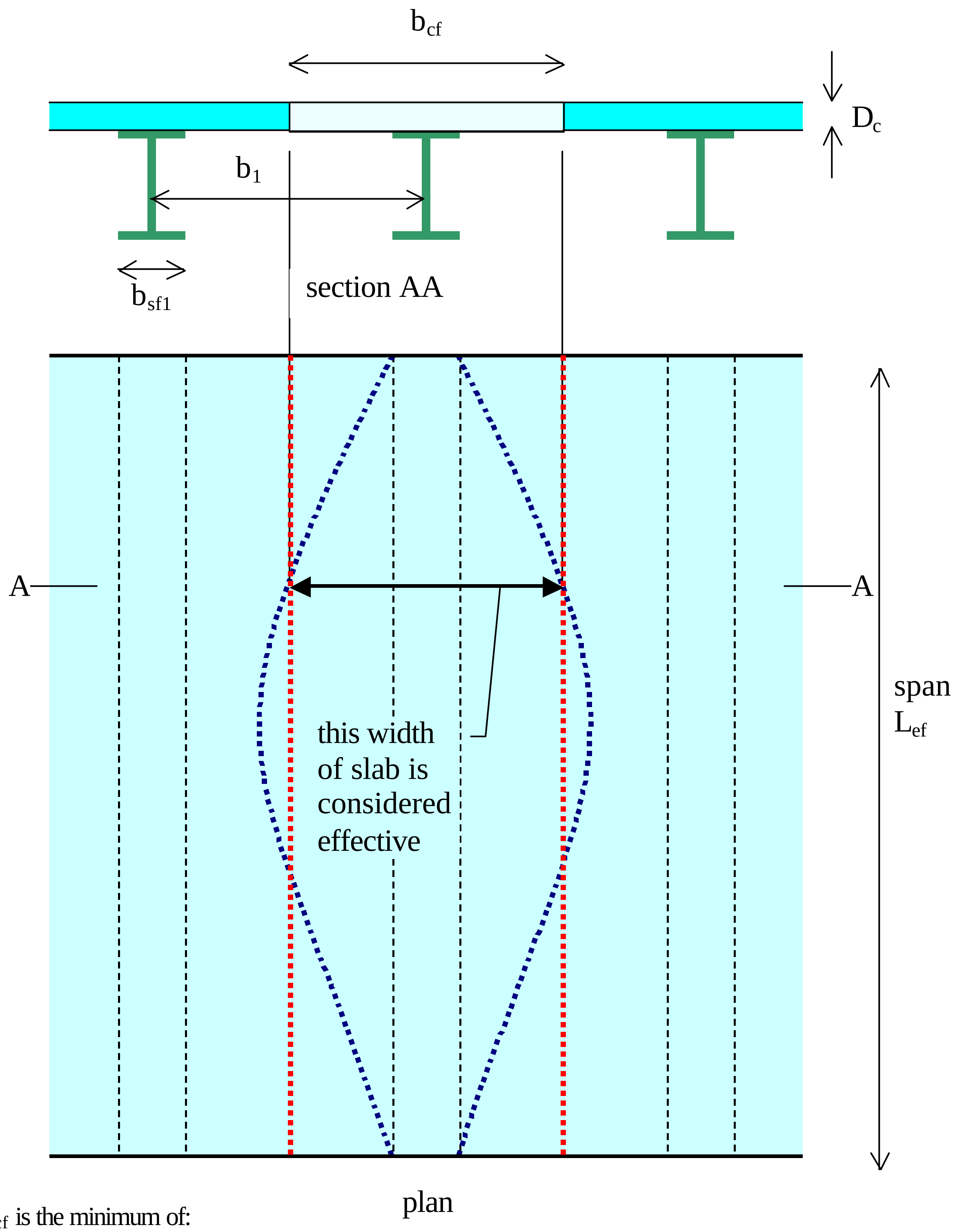
1. ultimate flexural strength of a cross section - complete shear connection
   failure is by yielding of the steel beam in tension, or crushing the compression concrete
2. ultimate flexural strength of a cross section - partial shear connection
   failure occurs in the shear connection
3. longitudinal shear failure within the slab
   a failure plane develops within the slab
4. ultimate shear strength of a cross section
   design for shear as a plain steel beam (ignore the concrete)

5. deflection
   this is complex, because of the deflection caused by the wet concrete on the bare steel beam. and because of creep and shrinkage effects
6. vibration
   must be checked for composite beams

We will look at 1, 2 and 3only, for simply supported beams.

- how do we define the beam cross section?
- how so know if we have complete or partial shear connection?
- how do we calculate the ultimate strength of a cross section?
- how do we check the longitudinal shear strength?

6. Effective width of the slab, b_{cf}



b_{cf} is the minimum of:

- span/4 $= L_{ef}/4$
- beam spacing $= b_1$
- beam width + 8 x slab depth $= b_{sf1} + 8\ D_c$

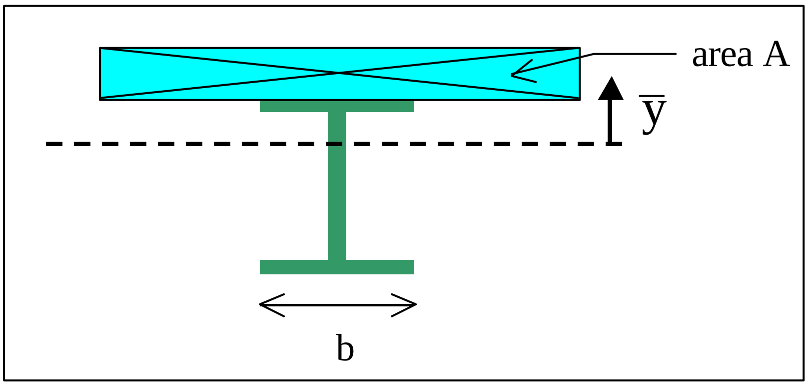
7. Design of the shear connection

There are 2 approaches – one is used for buildings, and another is used for bridges

1. Elastic design of the shear connection (for bridges)

Recalling from CIV2204, the longitudinal shear stress on a cross section,

$$\tau = \frac{VA\bar{y}}{Ib}$$



where
V is the shear force
A is the area of the cross section beyond the shear surface
$\bar{y}$ is the distance to the centroid of the area A
I is the second moment of area of the cross section
b is the width of the shear surface

so, the shear force per unit length along the shear surface,

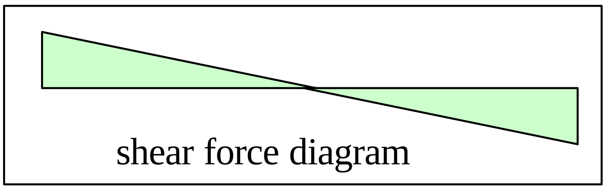
$$\tau b = \frac{VA\bar{y}}{I}$$

For example, if
τb = 200 kN/m
and if,
we use welded shear studs with d_{bs} = 19 mm and f_{uc} = 410 MPa
f'c = 32 MPa, E_c = 28,600 MPa
then f_{vs} = 93 kN.
We must apply a capacity reduction factor ϕ = 0.85 to the shear stud strength, f_{vs}., which reduces it to 79 kN per stud.

So the horizontal spacing of the studs along the beam needs to be,

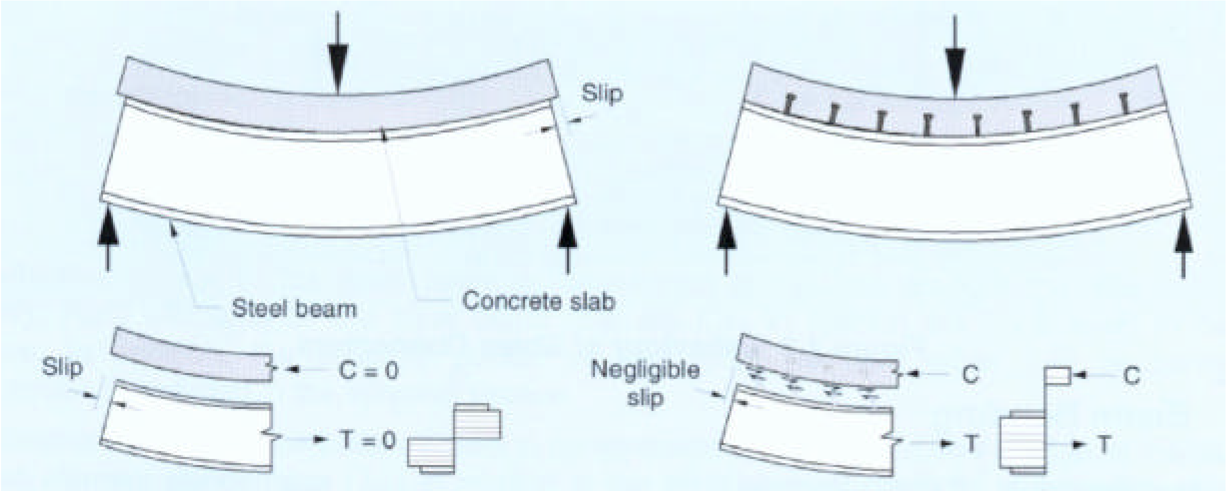
(200 kN/m) + (79 kN per stud) = 2.53 studs per metre,
ie stud spacing = 395 mm.

Because the shear force V is changing constantly along a beam with a UDL, the theoretical spacing of the shear studs is different at every point. In practice we round off the spacings.



1. Plastic design of the shear connection (for buildings)

This uses an ultimate strength approach. The horizontal force that can be transferred at any cross section equals the total strength of the shear connection between that cross section and the free end of the beam.



So if the number of studs between a cross section and the end is n, the horizontal force that can be transferred at that cross section,

$$F = n \times \phi \times f_{vs} \times k_n$$

where ϕ = 0.85, and $k_n = 1.18 - \left(\frac{0.18}{\sqrt{n}}\right)$ allows for the fact that the reliability of the connection increases with increasing number of studs.

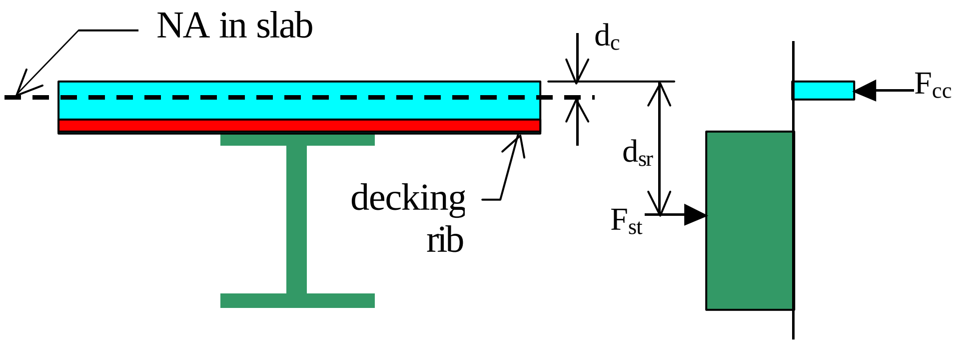
Using this method, the studs can be equally spaced if the beam has a UDL.

We will use this method from now on.

8. Ultimate strength of the cross section

We will only consider

- cross sections with complete shear connection
- cases where the neutral axis of the composite section is in the slab



cross section ultimate stress distribution

The concrete below the neutral axis (in tension) is ignored.

If the neutral axis in is the slab, the steel beam is all yielded, but not all of the concrete has crushed.

$$F_{cc} = F_{st}$$

where,

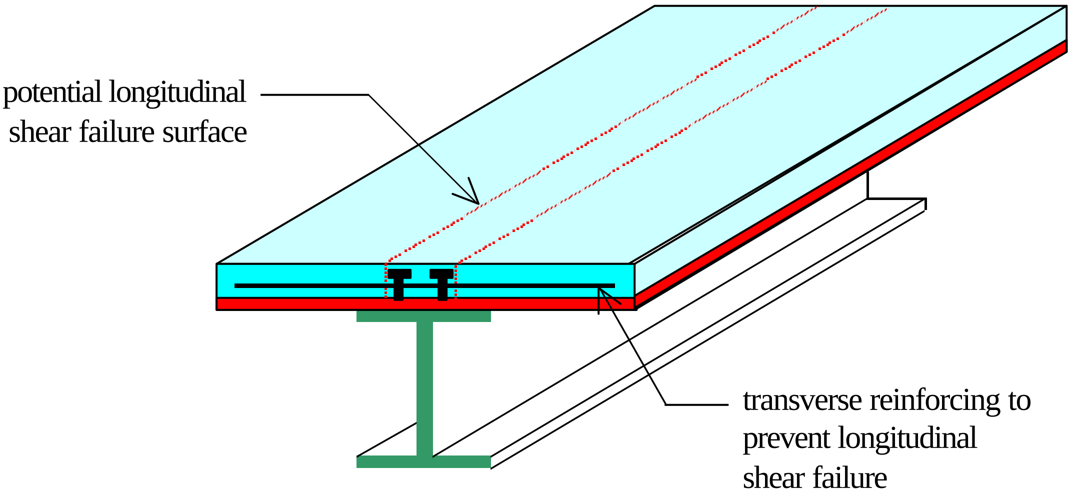
$$F_{st} = A_s f_y$$
$$F_{cc} = 0.85 f'_c b_{cf} \times d_c$$

so solve for d_c.

Moment carried by the cross section,

$$\phi M_{bc} = 0.9 \times F_{cc}\left(d_{sr} - \frac{d_c}{2}\right)$$

9.Longitudinal shear in the concrete



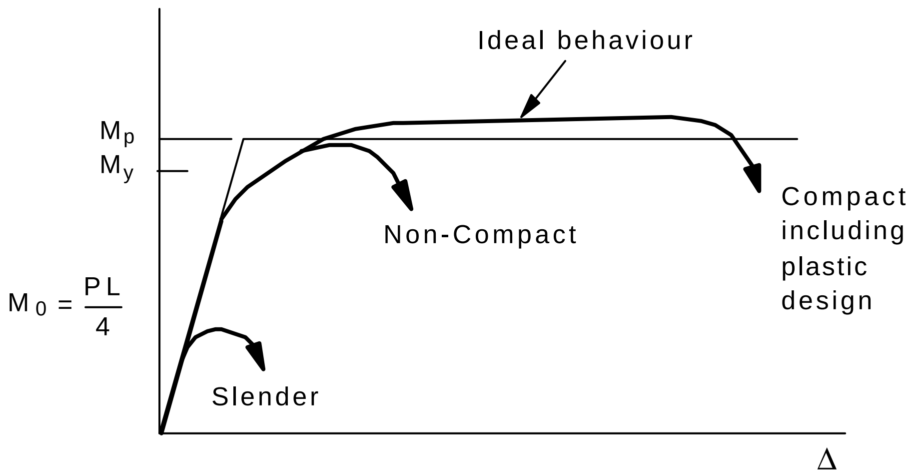
- identify potential longitudinal shear failure planes
- calculate the shear force on each shear plane
- calculate the strength of each shear plane
- if necessary include additional steel reinforcing to prevent longitudinal shear

We will not look at how to do these calculations in this course.

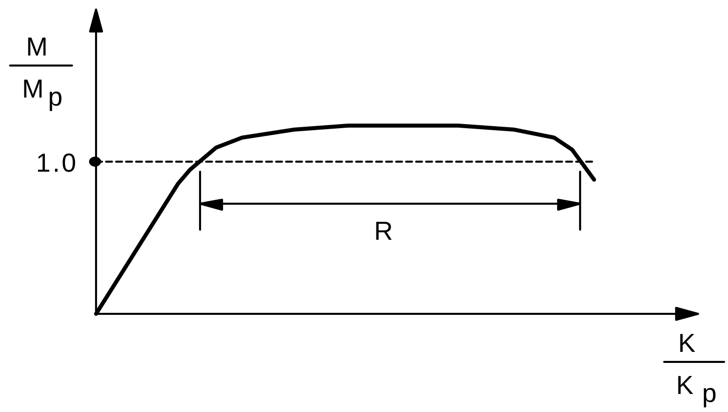
TOPIC 11: STEEL BEAMS

1. Section Classification

Three typical load deflection curves are shown in the figure below. Three types of sections are used in AS4100 to classify the behaviour, namely the compact section, non-compact section and slender section.



The compact section must not only develop a moment resistance equal to the plastic capacity of the member but maintain this resistance through relatively large inelastic deformations. This will enable the complete structure to redistribute bending moments and reach the load-carrying capacity anticipated on the basis of a plastic analysis. The width-to-thickness ratio below which a certain rotation capacity (R defined in the figure below) can be achieved is called the plasticity width-to-thickness limit. The corresponding plate slenderness is called the plate plasticity slenderness limit (λ_{ep}).



The width-to-thickness limits for non-compact sections are less restrictive than those for compact sections. The plates composing the cross-section should be capable of allowing the member to develop a moment resistance equal to the yield moment (M_y), and in this condition the stress in the extreme fibre will be equal to the yield stress f_y. In general the plate will behave elastically at this stage although some deterioration due to the large compressive residual stresses in the flange tips may be expected.

For slender sections the plates composing the section is not capable to achieve the yield moment. The width-to-thickness ratio beyond which the yield moment can not be achieved is called the yield width-to-thickness limit. The corresponding plate slenderness is called the plate yield slenderness limit (λ_{ey}).

The concept of compact, non-compact and slender sections are adopted in the Australian Standard for Steel Structures AS4100-1998 and the American Institute of Steel Construction LRFD Specification –1993. A similar approach has been adopted in Eurocode 3 –1992, British Standard BS5950 Part 1- 1990, Japanese Standard AIJ-1990 and the Canadian Standard CSA-S16.1-M89 where a section can be classified from class 1 to class 4, ie. from a plastic (deformation) section to a slender (buckling) section. Compact or Class 1 sections can form a plastic hinge with the rotation capacity required for plastic design. Slender or Class 4 sections can not reach first yield moment due to local buckling effect. Non-compact sections include Class 2 and Class 3. A Class 2 section can develop the fully plastic moment, but have limited rotation capacity. A Class 3 section can reach the first yield moment, but local buckling prevents the development of the fully plastic moment.

The following rules can be used to determine the section classification:

Compare λs with λ_{sy} and λ_{sp}

$\lambda_s < \lambda_{sp}$ Compact section
$\lambda_s > \lambda_{sy}$ Slender section
$\lambda_{sp} \le \lambda_s \le \lambda_{sy}$ Non-compact section

λ_s= Section Slenderness (see CIV2222)
λ_{sy} , λ_{sp} = Section Yield and Plasticity Slenderness Limits

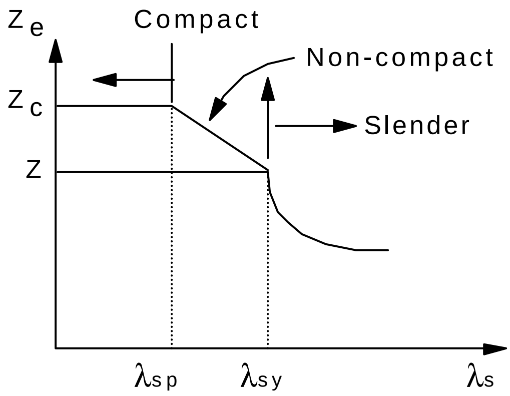
2. Section Capacity

$$M_s = \Phi\, Z_e\, f_y$$

Capacity Factor $\Phi = 0.90$
f_y = Yield Stress
Z_e = Effective Section Modulus

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

where $Z_c = \text{Min}\{S, 1.5Z\}$, Z is the elastic section modulus, S is the plastic section modulus.



λ_{sy} can be determined by using stub column tests (see Lecture notes on Steel Columns).
λ_{sp} can be determined by using pure bending tests. Plot rotation capacity (R) versus slenderness λ_s .

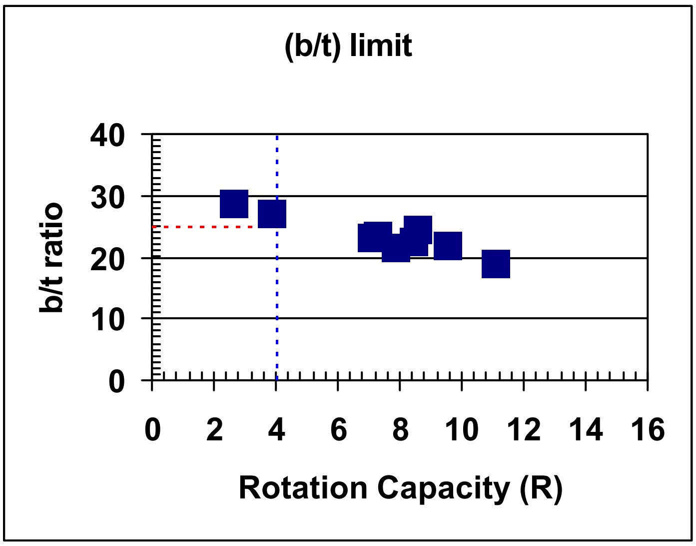
Example 1.

A rotation requirement of R = 4 is adopted in developing the b/t limits in AS4100 for compact sections. Table 1 gives results of a series of tests carried out for C350 SHS (square hollow sections) under pure bending. Determine the b/t limit for compact section for C350 SHS based on the test results. What is the plate slenderness limit (λ_p) corresponding to the experimentally determined b/t limit? The yield stress is taken as 350 MPa.

[Hint: plate slenderness $l = (\frac{b}{t}) \cdot \sqrt{\frac{s_y}{250}}$]

Table 1Test Results

b/t	Rotation capacity (R)
29.1	2.6
24.7	8.5
22.2	9.5
23.9	7.2
23.4	7.0
19.2	11.0
22.7	8.4
21.8	7.8
27.3	3.8



Solution: for R=4, lower bound $(b/t)_{limit} = 25$, $\lambda_p = (\frac{b}{t})_{limit} \cdot \sqrt{\frac{\sigma_y}{250}} = 25 \cdot \sqrt{\frac{350}{250}} = 30$

3. Effective Length

See Section 8.3 and Section 8.4 of CIV 2222 Steel Framed Structures Lecture Notes

$$L_e = k_t \cdot k_l \cdot k_r \cdot L$$

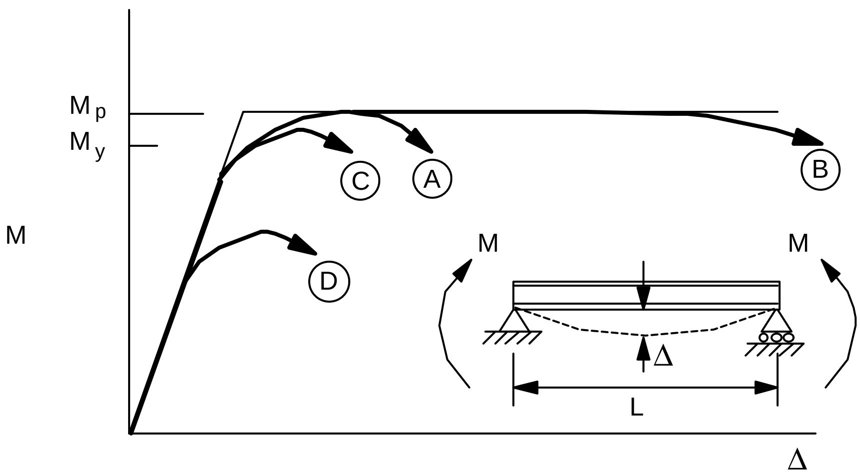
k_t = Twist restraint factor
k_l = Load height factor
k_r= Lateral rotation restraint factor
see Clause 5.6.3 of AS4100-1998

4. Member Capacity

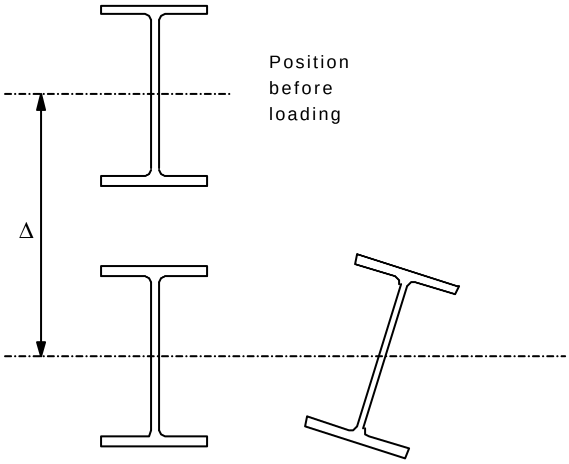
4.1 Behaviour

It has been assumed so far that the strength of the beam is dependent on the local buckling of its plate elements. In most cases this assumption is valid. However, if the beam is laterally unsupported the strength may be governed instead by lateral buckling of the complete member.

A plot of the relationship between the applied moment (M) and the resulting mid-span deflection (Δ) for a member of length (L) is shown in the figure below. The member, shown in the insert of the figure, is subjected to end moments producing a uniform bending moment distribution over the length of the span. Lateral supports are assumed to be present at the ends of the member so that the laterally unbraced length is equal to the span.



At low values of M, the member will respond elastically. However, as the moment is increased yielding will occur due to the strains produced by the applied moment and the residual strains in the cross-section. Further increase in the applied moment will result in general yielding over the cross-section as the moment approaches M_p. The movement of the cross-section during the loading process can be shown in the figure below. As the member is loaded the cross-section movers vertically from its initial position. At some stage of loading, however, the cross-section may twist and bend about its weak axis, ie. lateral buckling has occurred. Lateral buckling may occur at any stage during the loading history, e.g. after the member has reached M_p as shown by curves A and B above, between M_y and M_p as shown by curve C and even at moments below M_y as shown by curve D. The lateral buckling capacity of the member depends on its unbraced length and on a variety of cross-sectional properties.



4.2 Capacity

The resistance of the member to lateral bending depends on the weak axis bending stiffness of the cross-section (EI_y). The resistance to a twisting motion can be broken into two portions. One portion is termed the St Venant resistance and is a function of the stiffness term, GJ where G represents the shear modulus or the modulus of torsional regidity and J is the St Venant torsional constant for the section. The second portion of resistance to twisting is the warping resistance that is developed by cross-bending of the flanges. For rectangular hollow sections the warping of the section may be ignored.

If full lateral restraint (FLR) is provided to a beam the member capacity of the beam is the same as the section capacity. The length below which the section capacity can be achieved is called FLR (Full Lateral Restraint) length.
The calculation of FLR length has been described in 8.4 of CIV 2222 Steel Framed Structures Lecture Notes. If full lateral restraint is not provided to a beam there is an interaction of yielding and buckling. To account the interaction in the presence of residual stresses and geometric imperfections, AS4100 uses a beam design curve which combines the beam section moment capacity (M_s) with the elastic buckling moment (M_o). The beam design curve is expressed in the form of a slender reduction factor (α_s) given by

$$a_s = 0.6\{\sqrt{(\frac{M_s}{M_{oa}})^2 + 3} - (\frac{M_s}{M_{oa}})\}$$

such that the nominal member moment capacity (M_b) is given by

$M_b = \Phi\, \alpha_m\, \alpha_s\, M_s$

where α_m is called the Moment Modification Factor that accounts for non-uniform moment as given in Table 5.6.1 of AS4100 for segments fully or partially restrained at both ends and Table 5.6.2 of AS4100 for segments unrestrained at one end. M_{oa} is the Elastic buckling moment given by

$$M_{oa} = \sqrt{\left(\frac{p^2 EI_y}{L_e^2}\right)\left(GJ + \frac{p^2 EI_w}{L_e^2}\right)}$$

where

E = Young's modulus of elasticity
G = shear modulus of elasticity
I_y = second moment of area about the cross-section minor principal y-axis
J = torsion constant for a cross-section
I_w = warping constant for a cross-section
L_e = effective length of a laterally unrestrained member

Different approaches were used in determining the beam curves in different codes. Most of the beam curves were derived from test results of I-sections. A lower bound approach was adopted in AS4100. A summary of various beam curves are given in the figure below. Recent research on RHS beams showed that the beam curve in AS4100 is conservative when applied to RHS beams. An RHS under bending test is shown below.

The proposed beam curve is also shown in the figure below, with the actual expression give by

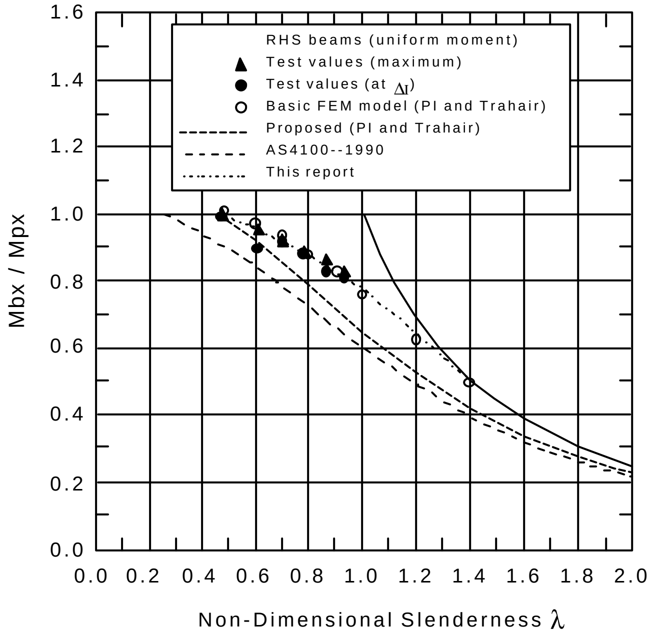
$$M_{bx} = (1.056 - 0.278\lambda^2) \cdot M_{px} \qquad \text{for } 0.45 \le \lambda \le 1.40$$

$$M_{bx} = M_{yz} \qquad \text{fro } \lambda > 1.40$$

where $$\lambda = \sqrt{\frac{M_{px}}{M_{yz}}}$$

$$M_{yz} = \frac{\pi}{L}\sqrt{EI_y GJ}$$

The beam length corresponding to λ of 0.45 is called the plastic buckling length (L_p) while the beam length corresponding to λ of 1.40 is called the elastic buckling length (L_e) for RHS beams.



Example 2

Determine the plastic buckling length (L_p) and elastic buckling length (L_e) for C350 Cold-Formed RHS 75x25x2.5.

Solution:

Set $\lambda_p = 0.45$,

$$\lambda_p = \sqrt{\frac{M_{px}}{M_{yz}}}$$

where $M_{yz} = \frac{\pi}{L_p}\sqrt{EI_y GJ}$

therefore

$$L_p = \frac{\pi\sqrt{EI_y GJ}}{M_{px}} \cdot \lambda_p^2$$

when E = 200,000 MPa, I_y = 0.0487 x 10^6 mm^4, G = 80,000 MPa, J = 0.144 x 10^6 mm^4, $M_{px} = S_x f_y$ = 10.1 x 10^3 mm^3 x 350 MPa = 3,535 x 10^6 Nmm

L_p = 1906 mm

Set λ_e = 1.40,

$$\lambda_e = \sqrt{\frac{M_{px}}{M_{yz}}}$$

where $M_{yz} = \frac{\pi}{L_e}\sqrt{EI_y GJ}$

therefore

$$L_e = \frac{\pi\sqrt{EI_y GJ}}{M_{px}} \cdot \lambda_e^2$$

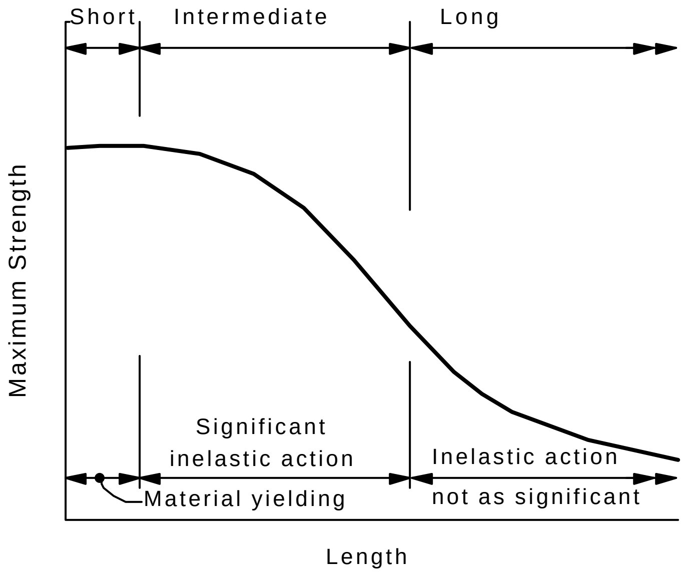
when E = 200,000 MPa, I_y = 0.0487 x 10^6 mm^4, G = 80,000 MPa, J = 0.144 x 10^6 mm^4, $M_{px} = S_x f_y$ = 10.1 x 10^3 mm^3 x 350 MPa = 3,535 x 10^6 Nmm

L_p = 18,451 mm

TOPIC 12: STEEL COLUMNS

1. Form Factor

The maximum strength of a steel column depends, to a large degree, on the member length. Steel columns can be normally classified as short, intermediate or long members. Each range has associated with it a characteristic type of behaviour, and therefore different techniques must be used to assess the maximum strength. The figure below shows schematically the relationship between the maximum strength of a column and its length.



A short (stub) column may be defined as a member which can resist a load equal to the section capacity (N_s). The effective cross-sectional area of a section (A_e) will be less than the gross cross-section (A_g) for cross-sections with very slender plate elements. In AS4100, the ratio of the effective area to the gross area of the cross-section is termed the form factor (k_f):

$k_f = A_e/A_g$

In other words when the component plates comprising a short column are sufficiently slender, the cross-section strength will never attain the yield capacity (N_s) due to the onset of local buckling. The effective width (b_e) concept is used to approximate the strength of a plate undergo local buckling (see Section 10.2 of CIV2222 Lecture Notes).

$$b_e = b(\frac{\lambda_{ey}}{\lambda_e}) \leq b$$

λ_{ey} = Plate Yield Slenderness Limit (Table 6.2.4 of AS4100)

$$l_e = \frac{b}{t}\sqrt{\frac{f_y}{250}}$$

b = clear width

The effective area (A_e) used to calculate the form factor is based on the effective width of each element, i.e.

$$A_e = \sum_{plate\_elements} b_e \cdot t$$

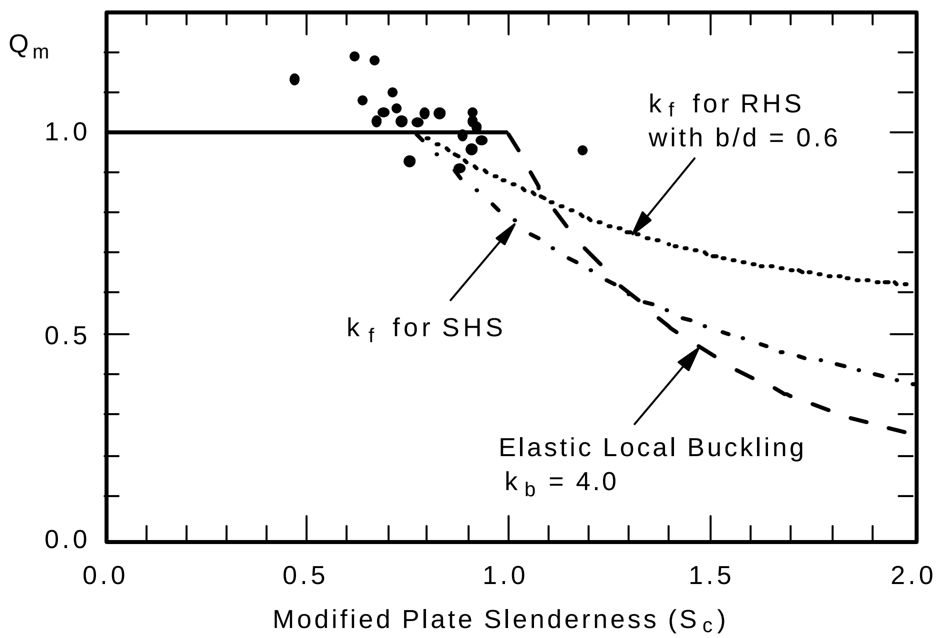
λ_{ey} can be determined experimentally from the results of stub column tests as demonstrated in Example 1, when plotting the ratio Q_m (= P_{ult}/P_{yield}) versus modified plate slenderness S_c, which is defined as:

$$S_c = \frac{b}{t} \cdot \sqrt{\frac{f_y \cdot 12 \cdot (1-\nu^2)}{E \cdot \pi^2 \cdot k_b}}$$ where $\nu = 0.3$, E = 200,000 MPa, $k_b = 4.0$.

The term S_c can be rewritten as: $$S_c = \frac{b}{t}\sqrt{\frac{f_y}{250}} \cdot \sqrt{\frac{250 \cdot 12 \cdot (1-\nu^2)}{E \cdot \pi^2 \cdot k_b}} = \frac{\lambda_e}{54}$$

Example 1

The ratio Q_m (= P_{ult}/P_{yield}) is plotted against the modified plate slenderness S_c for C350 and C450 RHS (rectangular hollow sections) based on test results. Determine the yield slenderness λ_{ey}.



A value of S_c = 0.74 is obtained for the transition, which produces a limit of λ_{ey} = 40.

2. Section Capacity

The section capacity is defined as

$$N_s = \Phi \, k_f \, A_n \, f_y$$

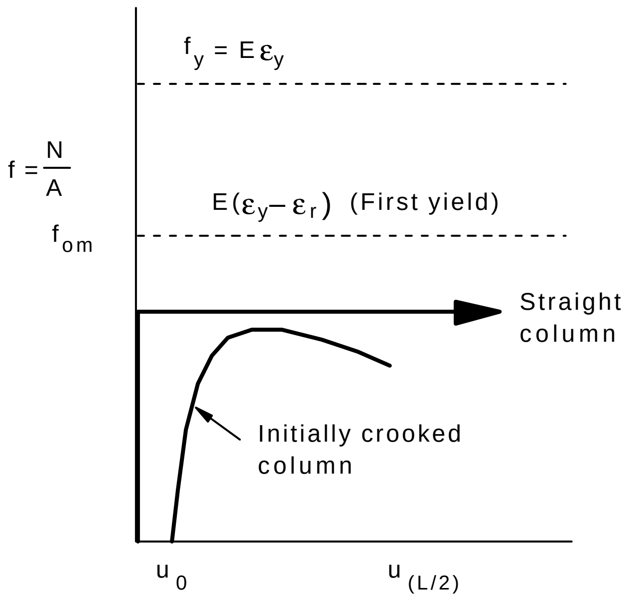
where Capacity Factor $\Phi = 0.90$, k_f is the Form Factor, f_y is the Yield Stress and A_n is the net area of cross-section. Section 6.2.1. of AS4100 states that for sections with penetrations or unfilled holes that reduce the section area by less than $100[1-f_y/(0.85f_u)]\%$, the gross area A_g may be used in lieu of the net area A_n.

All of the BHP 300PLUS UC sections and the Grade 300 WC sections have been specially tailored so that the gross area is effective in compression (i.e. k_f = 1). On the other hand, the deep web of UB sections means that, for the majority of the BHP-300PLUS UB sections and all the Grade 300 WB sections, the web is not fully effective and thus $k_f < 1.0$.

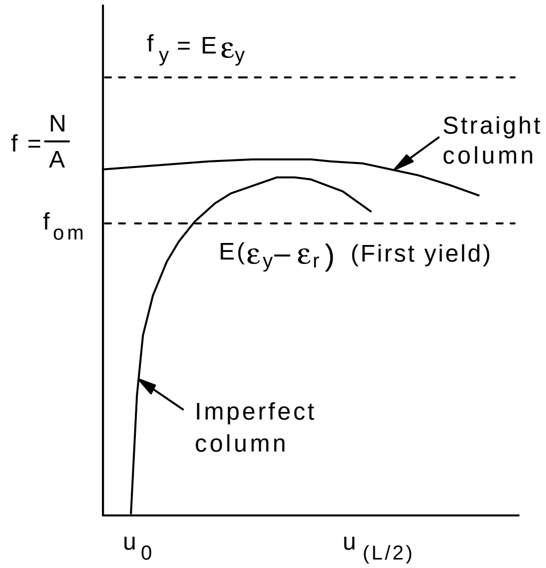
3. Member Capacity

3.1 Behaviour

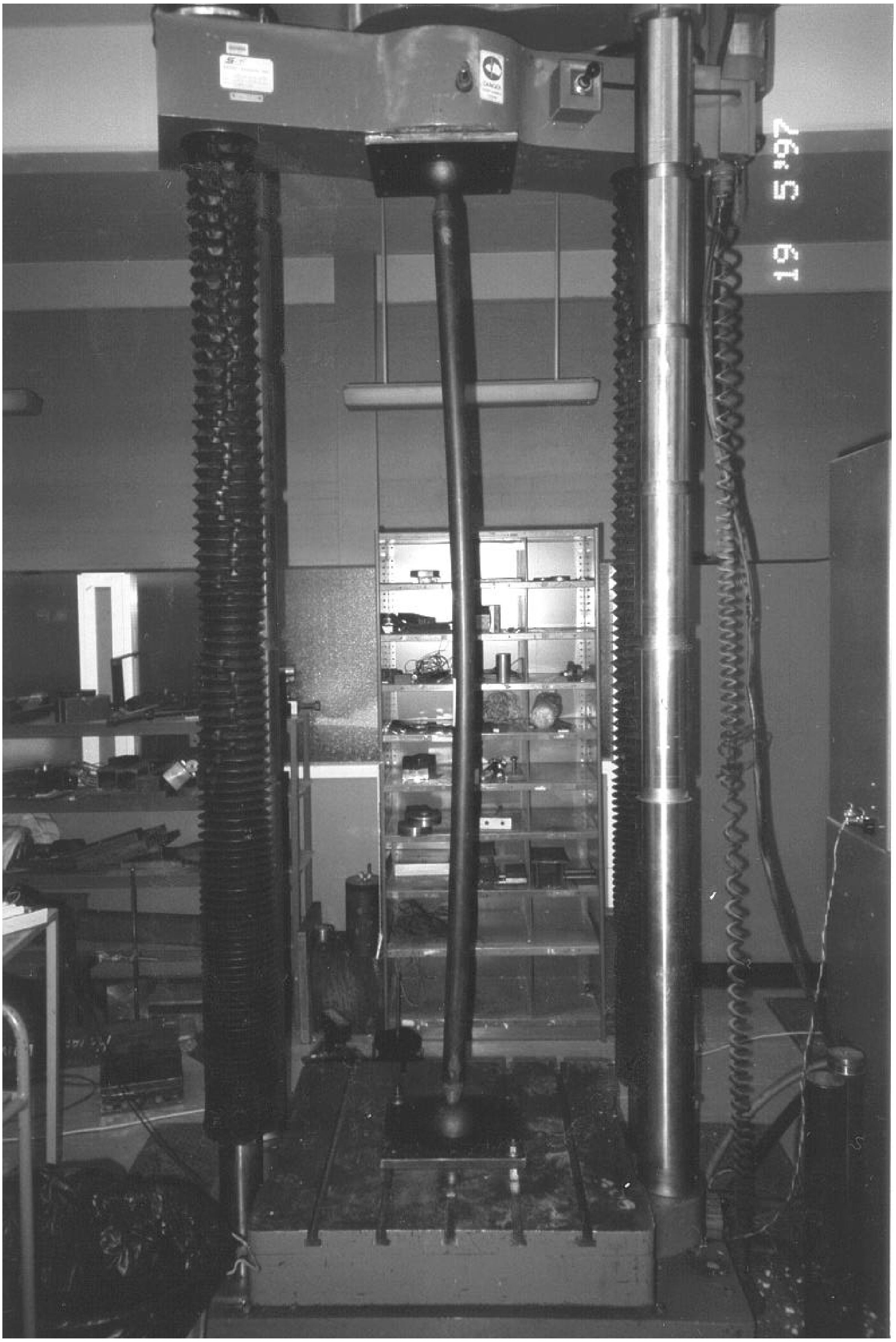
For longer columns, failure is accompanied by a rapid increase in the lateral deflection. If the member is extremely slender, the load at which this increased deflection takes place is not sufficient to significantly yield the member. Thus the maximum load is not a function of the material strength but rather depends on the bending stiffness of the member (EI), and its length (L). The failure of a long column may occur at stress levels below that required to initiate local buckling of slender plate elements. The load deflection relationship for a long column is schematically shown below.



Columns falling into the intermediate range are more complex to analyse but also are the most common in steel structures. For intermediate length columns, failure is also characterised by a rapid increase in the lateral deflection, but only after some portions of the column cross-section have yielded and local buckling of slender plate elements has occurred. Yielding is initiated first in those portions of the cross-section which have large compressive residual stresses. The failure in this case is called inelastic instability and the maximum strength of the column depends not only on the bending stiffness and length but also on the yield stress of the steel, the distribution of residual stress over the cross-section, the cross-section slenderness, and the magnitude of the initial imperfections in columns and component plates of the cross-section. The load deflection relationship for an intermediate column is schematically shown below.



A column under testing in compression is shown below.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3.2 Capacity

Column capacity can be calculated using Perry-Robertson equation:

$$N_c = N_s \cdot [\frac{(N_s / N_{om}) + 1 + \eta}{2 \cdot (N_s / N_{om})}] \cdot \sqrt{1 - \frac{1}{(N_s / N_{om})} \cdot [\frac{2 \cdot (N_s / N_{om})}{(N_s / N_{om}) + 1 + \eta}]^2}$$

where N_{om} is the elastic buckling load of column and η is the "imperfection parameter".

The above expression does not consider the influence of residual stresses on the column strength and behaviour. Since residual stresses are in effect another kind of imperfection, a simple way of considering them is to empirically adjust the imperfection parameter η so that the strength prediction shown above is in reasonable agreement with test results. In principle, this is the approach used in AS4100 since the column strength equations used therein are based on the Perry-Robertson equation.

AS4100 also places limitations on the actual permissible initial imperfection for columns (<L/1000 or 3mm). For very slender members, the maximum load carrying capacity is not greatly reduced by the presence of initial geometric imperfections and residual stresses. However, for columns of intermediate length the situation is more serious as it is in this region where the sensitivity to imperfection is greatest. Since the initial strains corresponding to bending are trigger by initial imperfections, the imperfection of these two variables (residual strain pattern and magnitude of initial imperfection) results in a wide scatter in column strengths for intermediate columns.

The Member Capacity can be calculated according to AS4100 as

$$N_c = \Phi\ \alpha_c\ N_s = \Phi\ \alpha_c k_f\ A_n\ f_y$$

where Capacity Factor $\Phi = 0.90$, k_f is the form factor, A_n is the net area of cross-section, f_y is the Yield Stress and α_c is the member slenderness reduction factor.

α_c depends on λ_n and α_b , see Table 6.3.3 (3) of AS4100

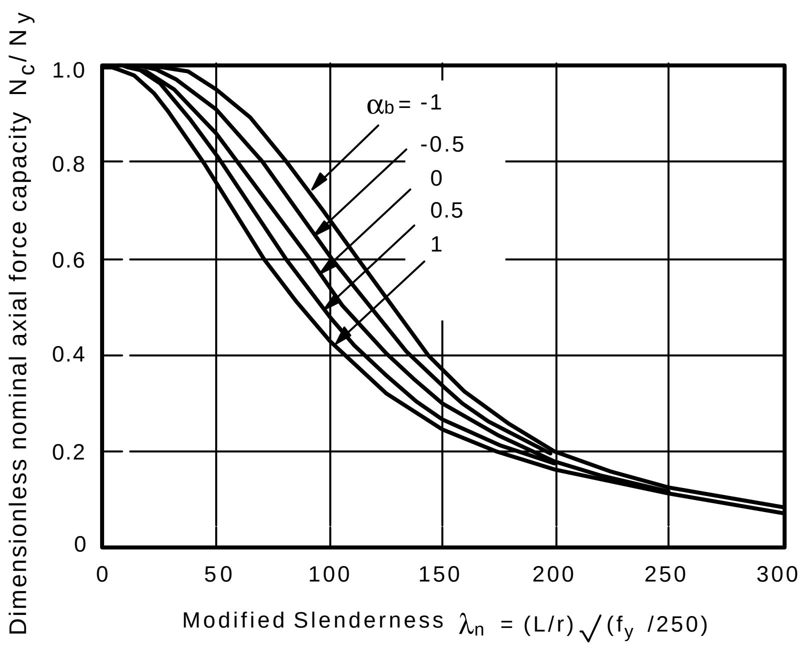
$$\lambda_n = \left(\frac{L_e}{r}\right) \cdot \sqrt{k_f} \cdot \sqrt{\frac{f_y}{250}}$$

L_e = Effective Length of Column
$r = r_x$ or r_y (Radius of Gyration)

Member Section Constant α_b are given in Table 6.3.3 of AS4100, some of the values are summarised below.

Section Description	Compression Member Section Constant (α_b)
Hot-formed RHS and CHS Cold-formed (stress relieved) RHS and CHS	-1.0
Cold-formed (non-stress relieved) RHS and CHS	-0.5
Hot-rolled UB and UC sections (flange thickness up to 40 mm)	0.0
Hot-rolled channels	0.5
Hot-rolled UB and UC sections (flange thickness over 40 mm)	1.0

The design provision of AS4100 can be shown in the figure below where the member compressive strength (N_c) divided by the section compressive strength (N_s) is plotted against the slenderness ratio of the member. It can be seen that multiple columns are used corresponding to different values of α_b. The provisions depicted in the figure are based on the assumption that failure will involve bending about one of the major axes of the cross-section. This will be the axis associated with the larger slenderness ratio, i.e. the larger of (L_{ex}/r_x) and (L_{ey}/r_y). In sections having only one axis of symmetry, or in sections with no axes of symmetry, the possibility also exists that failure will be accompanied by both bending and twisting of the cross-section and may occur at a reduced load. Buckling of this kind is called flexural-torsional buckling, and for sections which may buckle in this manner, the compressive strength should be based on a consideration of the actual failure modes.



Example 2

Member capacity

Compare the member capacities of columns with length of 3m, pin - ended, cold - formed (stress - relieved) tubes:
C350 (350 MPa) SHS 100x100x3
C350 (450 MPa) SHS 100x100x3
C350 (350 MPa) SHS 100x100x6
C350 (350 MPa) SHS 100x100x6

Solutions:

1. SHS 100x100x3 350MPa (= 0.350 kN/mm^2)

$(b/t)_{limit}$ = 33.8

$$b/t = \frac{B-2t}{t} = \frac{100-2\times 3}{3} = 31.33 \quad <33.8$$

$\therefore k_f = 1.0$

$N_s = A\sigma_y$ = 1140x0.350 = 399 kN

$$l_n = (\frac{l_e}{r_y})\sqrt{k_f}\,(\sqrt{\frac{s_y}{250}}) = \frac{3000}{39.4}\sqrt{\frac{350}{250}} = 90$$

From Table 6.3.3 (3), α_b = -1.0

α_c = 0.737

$N_c = \alpha_c\ N_s$ = 0.737 x 399 = 294kN

2. 100x100x3 SHS 450MPa

$(b/t)_{limit}$ = 29.8

$$b/t = \frac{B-2t}{t} = \frac{100-2\times 3}{3} = 31.3 > 29.8$$

$$l_e = \frac{b}{t}\sqrt{\frac{s_y}{250}} = 42$$

$$k_f \langle 1.0, \qquad b_e = b\frac{l_{ey}}{l_e} = 94(\frac{40}{42}) = 89.5 \text{ mm}$$

$$k_f = \frac{4b_e t + 4t^2}{4bt + 4t^2} = \frac{1110}{1164} = 0.95$$

$N_s = k_f A\sigma_y$ = 0.95 x1140x0.450 = 487 kN

$$l_n = (\frac{l_e}{r_y})\sqrt{k_f}\sqrt{\frac{s_y}{250}} = \frac{3000}{39.3}\sqrt{0.95}\sqrt{\frac{450}{250}} = 99.8$$

From Table 6.3.3(3), α_b = -0.5

α_c = 0.60

$N_c = \alpha_c \ N_s$ = 0.60 x 487 = 292 kN

3. 100 x 100 x 6 SHS, 350MPa

similarly, k_f = 1.0, N_s = 746 kN, N_c = 522 kN

4. 100 x 100 x 6SHS, 450 MPa

similarly, k_f = 1.0, N_s = 959 kN, N_c = 584 kN

Comparison

	100 x 100 x3 (350MPa)	100 x 100 x 3 (450MPa)	$\frac{N450}{N350}$
N_s	399kN	487kN	22% difference
α_c	0.737	0.60	
N_c	294kN	292kN	-0.7% difference

	100 x 100 x 6 (350MPa)	100 x 100 x 6 (450MPa)	$\frac{N450}{N350}$
N_s	746kN	959kN	29% difference
α_c	0.70	0.609	
N_c	522kN	584kN	12% difference

Difference in N_c is small especially for thin sections.

TOPIC 13: STEEL BEAM-COLUMNS

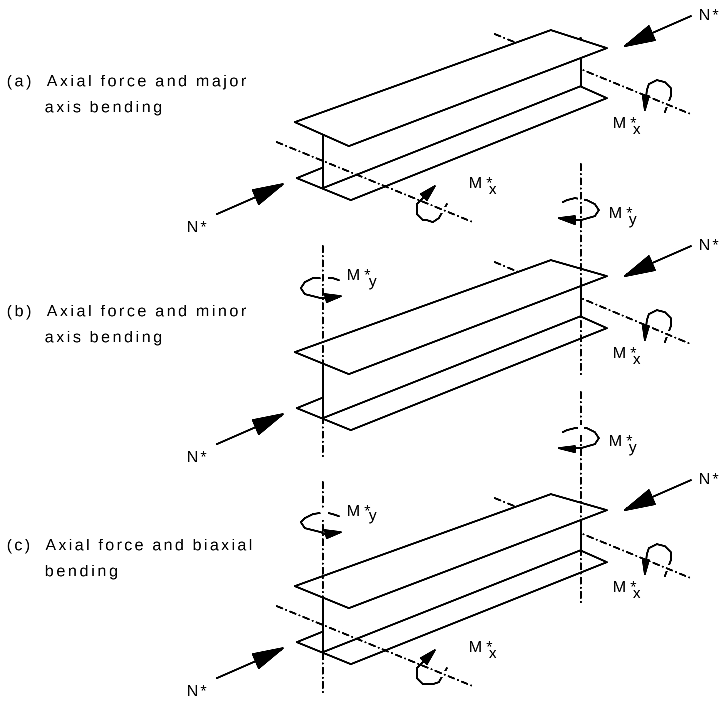
1. Combined Actions

The following loading and behaviour scenarios are possible for beam-columns:

a) The member is subjected to axial compression N* and uniaxial bending M_x* about the major principal x-axis of the cross-section, as shown in the figure a) below. If the member is prevented from deflecting laterally, its behaviour is confined to the plane of bending; the strength of the member may be limited by a local cross-sectional strength criterion or an overall in-plane member strength criterion relating to major-axis failure. The latter failure mode is related to the in-plane bending of beams and flexural buckling of compression members about the major axis. If the member is not completely restrained from deflecting laterally (i.e. it does not have full lateral restraint), then it may buckle prematurely out of the plane of bending by deflecting laterally and twisting; this action is related to the lateral and lateral-torsional buckling of beams discussed in Section 7. The beam-column strength may thus be governed by an out-of-plane member strength criterion.

b) The member is subjected to axial compression N* and uniaxial bending M_y* about the minor principal y-axis of the cross-section, as shown in the figure b) below. In this case, there is no possibility that the member will fail in an out-of-plane mode because it is already deflecting in its weak plane. The strength of the member may be limited by a local cross-sectional strength criterion or an overall in-plane member strength criterion involving bending and flexural buckling about the minor-axis.

c) The member is subjected to axial compression N* and biaxial M_x* and M_y* about both principal axes of the cross-section, as shown in the figure c) below. The member bends and deflects in both planes and may also twist. The member strength may be governed by a cross-section strength criterion, an in-plane member strength criterion, or an out-of-plane member strength criterion.



2. Section Capacity

a) Symbols Used

Actions:

N^* = Axial force

M^*_x , M^*_y = Bending moment about x, y axes

Capacities:

N_s = Column section capacity

M_{sx}, M_{sy} = Beam section capacity about x, y axes

Reduced capacities:

M_{rx} , M_{ry} = Reduced beam section capacity

b) Format

The design rules in AS4100 for members subjected to combined bending moments and axial forces have a two tier approach. For section capacity rules, simple linear interaction formulae are specified. However, for doubly symmetric compact I-sections and cold-formed square hollow sections, more advanced interaction rules are specified as higher tiers in AS4100.

(i) Compression (N*) and M^*_x

- General formula

$$M^*_x \leq \Phi M_{rx} = \Phi M_{sx}(1 - \frac{N^*}{\Phi N_s})$$

- For special I-sections and compact RHS

$$M^*_x \leq \Phi M_{rx} = \Phi 1.18 M_{sx}(1 - \frac{N^*}{\Phi N_s})$$

(ii) Compression (N*) and M^*_y

- General formula

$$M^*_y \leq \Phi M_{ry} = \Phi M_{sy}\ (1 - \frac{N^*}{\Phi N_s})$$

- For special I-sections

$$M^*_y \leq \Phi M_{ry} = \Phi 1.19\ M_{sy}\ [1 - (\frac{N^*}{\Phi N_s})^2]$$

- For compact RHS

$$M^*_y \leq \Phi M_{ry} = \Phi 1.18\ M_{sy}\ (1 - \frac{N^*}{\Phi N_s})$$

(iii) N* and M^*_x and M^*_y

- General formula

$$\frac{N^*}{\Phi N_s} + \frac{M^*_x}{\Phi M_{sx}} + \frac{M^*_y}{\Phi M_{sy}} \leq 1$$

- For special I-sections and compact RHS

$$(\frac{M^*_x}{\Phi M_{rx}})^r + (\frac{M^*_y}{\Phi M_{ry}})^r \leq 1$$

where

$$\gamma = 1.4 + \left(\frac{N^*}{\phi N_s}\right) \leq 2.0$$

Example 1

1. Determine the design <u>major</u> (x) axis section moment capacity of a 200UC52.2 of Grade 250 steel which has a design axial compression force of N* = 112 kN.
2. Determine the design <u>minor</u> (y) axis section moment capacity of a 200UC52.2 of Grade 250 steel which has a design axial compression force of N* = 112 kN

Given:
f_y = 250 MPa, capacity factor ϕ = 0.9, form factor k_f = 1.0, compact section cross-section area A_n = 6640 mm^2, plastic section modulus S_x = 568,000 mm^3, S_y = 261,000 mm^3

Solution:

1. about major (x) axis

- using general formula

$M_{sx} = S_x f_y = 568{,}000 \text{ x } 250 = 142 \text{ x } 10^6$ Nmm = 142 kNm

$N_s = k_f A_n f_y = 1.0 \text{ x } 6640 \text{ x } 250 = 1660 \text{ x } 10^3$ N = 1660 kN

$\phi M_{rx} = \Phi M_{sx}(1 - \frac{N*}{\Phi N_s})$ = 0.9 x 142 x [1 – 112/(0.9x1660)] = 118 kNm

- using formula for special I-section

$M_{sx} =$ 142 kNm

$N_s =$ 1660 kN

$\phi M_{rx} = 1.18 \text{ x } 118 = 139$ kNm

2. about minor (y) axis

- using general formula

$M_{sy} = S_y f_y = 261{,}000 \text{ x } 250 = 65.3 \text{ x x } 10^6$ Nmm = 65.3 kNm

$N_s =$ 1660 kN

$\phi M_{ry} = \Phi M_{sy}\ (1 - \frac{N^*}{\Phi N_s})$ =0.9 x 65.3 x [1 – 112/(0.9x1660)] = 54.4 kNm

- using formula for special I-section

$M_{sy} =$ 65.3 kNm

$N_s =$1660 kN

$\phi M_{ry} = \Phi 1.19\ M_{sy}[1 - (\frac{N^*}{\Phi N_s})^2]$ =0.9 x 1.19 x 65.3 x [1–(112/(0.9x1660))^2]=69.5kNm

$\phi M_{ry} = \phi M_{sy}$ = 0.9 x 65.3 = 58.8 kNm

3. Member Capacity

a) Symbols Used

Actions:
N^* = Axial force
M^*_x , M^*_y = Bending moment about x, y axes

Capacities:
M_{bx} = Beam member capacity about x axis
$M_{bxo} = M_{bx}$ with α_m = 1.0
N_{cx} = Column member capacity about principal (x) axis with k_e = 1.0
N_{cy} = Column member capacity about y axis

Reduced capacities:
M_i = Reduced in-plane member moment capacity
M_{ox} = Reduced out-of-plane member moment capacity

β_m = Ratio of end moments
Assume x-axis is the principal axis

b) Formulae for beams

Formulae are given for the following three cases: beams with FLR, beams without FLR and beams under biaxial bending.

(i) With FLR

- General formula

$$M^*_x \le \Phi M_i = \Phi M_{sx}(1 - \frac{N^*}{\Phi N_{cx}})$$

- For special I-sections and compact RHS

$$M^*_x \le \Phi M_i = \Phi M_{sx}\{[1-(\frac{1+\beta_m}{2})^3](1-\frac{N^*}{\Phi N_{cx}}) + 1.18(\frac{1+\beta_m}{2})^3\sqrt{1-\frac{N^*}{\phi N_{cx}}}\}$$

(ii) Without FLR

- General formula

$$M^*_x \le \Phi M_{ox} = \Phi M_{bx}\left(1-\frac{N^*}{\phi N_{cy}}\right)$$

- For special I-sections

$$M^*_x \le \Phi M_{ox} = \Phi \alpha_{bc} M_{bxo}\sqrt{(1-\frac{N^*}{\phi N_{cy}})(1-\frac{N^*}{\phi N_{oz}})}$$

$$N_{oz} = \frac{GJ + \pi^2 EI_w / L_z^2}{(I_x + I_y)/A} =$$ Elastic torsional buckling capacity

$$\frac{1}{\alpha_{bc}} = \frac{1-\beta_m}{2} + (\frac{1+\beta_m}{2})^3 (0.4 - 0.23\frac{N^*}{\phi N_{cy}})$$

where

E, G = the elastic moduli
A, I_w, I_x, I_y and J = the section constant
L_z = the distance between partial or full torsional restraints

(iii) Biaxial Bending

$$(\frac{M_x^*}{\Phi M_{cx}})^{1.4} + (\frac{M_y^*}{\Phi M_{iy}})^{1.4} \le 1$$

where

Mcx = Min {M_{ix} , M_{ox}}
Miy = Mi about y axis

Example 2

Check the in-plane member capacity of the 200UC52.2 beam-column of Grade 250 steel, which is subjected to a combined moment M_x* of 105 kNm and a compression force of 112 kN. Assume FLR is provided.

Given:
f_y = 250 MPa, capacity factor ϕ = 0.9, form factor k_f = 1.0, compact section.
Column length L = 5000 mm, effective length factor k_e = 1.0, radius of gyration r_y = 89 mm, α_b = 0

Solution:

- using general formula

$$\lambda_n = (\frac{k_e L}{r_y})\sqrt{k_f} \cdot \sqrt{\frac{f_y}{250}} = (\frac{1.0 \cdot 5000}{89}) \cdot 1.0 \cdot \sqrt{\frac{250}{250}} = 56$$

α_c = 0.84 [from Table 6.3.3(3)]

N_s = 1660 kN [from Example 1]

ϕN_c = 0.9 x 0.84 x 1660 = 1255 kN

M_{sx} = 142 kNm [from Example 1]

$$\Phi M_i = \Phi M_{sx}(1 - \frac{N^*}{\Phi N_{cx}}) = 0.9 \cdot 142 \cdot (1 - \frac{112}{1255}) = 116 \text{ kNm}$$

- using formula for special I-section

$\lambda_n = 56$

$\alpha_c =$ 0.84 [from Table 6.3.3(3)]

$N_s =$ 1660 kN [from Example 1]

$\phi N_c = 1255$ kN

$M_{sx} =$ 142 kNm [from Example 1]

$\beta_m = 1.0$ (double curvature bending)

$$\phi M_i = \Phi M_{sx}\{[1 - (\frac{1+\beta_m}{2})^3](1 - \frac{N^*}{\Phi N_{cx}}) + 1.18(\frac{1+\beta_m}{2})^3 \sqrt{1 - \frac{N^*}{\phi N_{cx}}}\}$$

$$= 0.9 \cdot 142 \left\{0 + 1.18 \cdot \sqrt{1 - \frac{112}{1255}}\right\} = 144 \text{ kNm}, \ \phi M_i \le \phi M_{rx} = 139 \text{ kNm (from Example 1).}$$

TOPIC 14: STEEL CONNECTIONS

1. Type of Connections

AS4100 allows three forms of construction which relates to the bahaviour of the connections. It then requires that the design of the connections be such that the structure is capable of resisting all design actions, calculated by assuming that the connections are appropriate to the form of construction of the structure or structural part. The design of the connections is to be consistent with the form of construction assumed. The three forms of construction are:

(1) Rigid construction – For rigid construction, the connections are assumed to have sufficient rigidity to hold the original angles between the members unchanged. The joint deformations must be such that they have no significant influence on the distribution of the action effects nor on the overall deformation of the frame.

(2) Semi-rigid construction – For semi-rigid construction, the connections may not have sufficient rigidity to hold the original angles between the members unchanged, but are required to have the capacity to furnish dependable and known degree of flexural restraint. The relationship between the degree of flexural restraint and the level of the load effects is required to be established by methods based on test results. This is outside the scope of CIV3221.

(3) Simple construction – For simple construction, the connections at the ends of members are assumed not to develop bending moments. Connections between members in simple construction must be capable of deforming to provide the required rotation at the connection. The connections are required to not develop a level of restraining bending moment which adversely affects any part of the structure. The rotation capacity of the connection must be provided by the detailing of the connection and must have been demonstrated experimentally. The connection is then required to be considered as subject to reaction shear forces acting at an eccentricity appropriate to the connection detailing.

Typical types of connections for simple construction (also called flexible connections) are:

- Angle seat
- Bearing pad
- Flexible end plate
- Angle cleat
- Web side plate
- Stiff seat
- Bracing cleat

Typical types of connections for rigid construction (also called rigid connections) are:

- moment connection (welded)

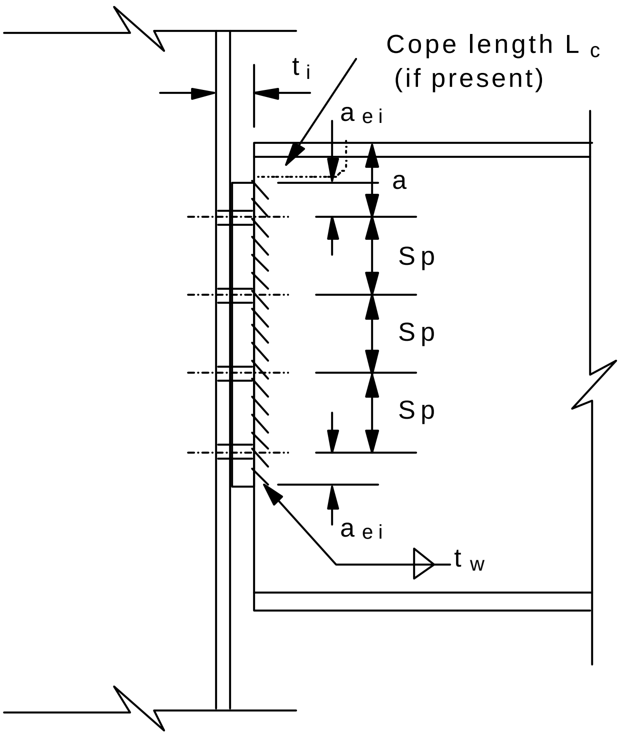
- moment connection (bolted end plate)

This section will only deal with flexible end plate connection (shear connection) and moment connection (bolted end plate)

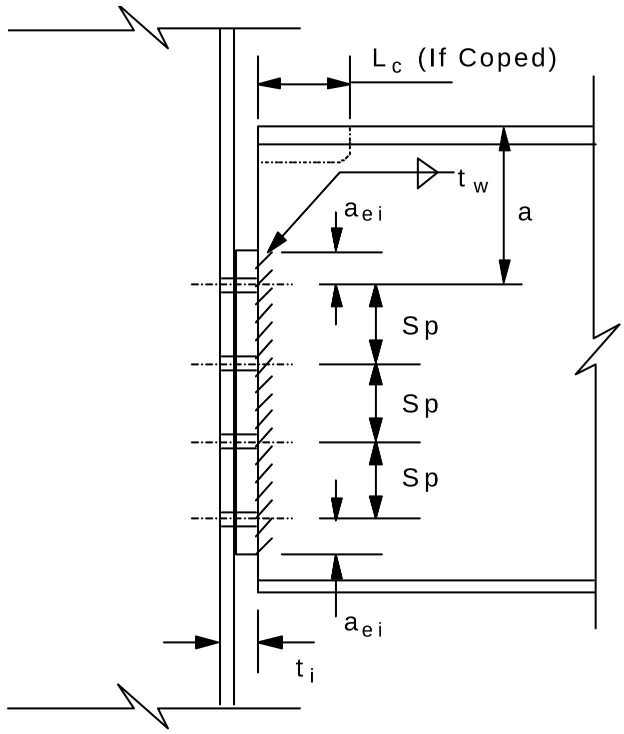
2. Shear Connections (Flexible End Plate)

2.1 Connection Details

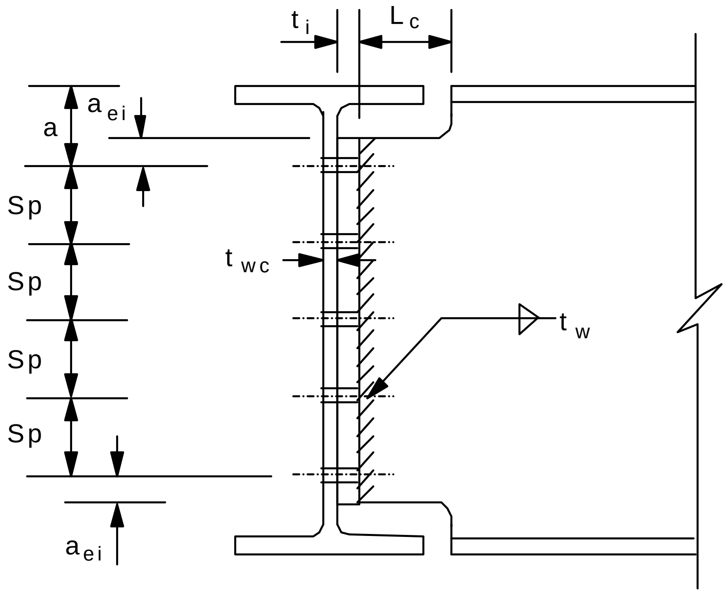
Typical flexible end plate shear connections are shown in the figure below. Fabrication of this type of connection requires close control in cutting the beam to length and adequate consideration must be given to squaring the beam ends such that both end plates are parallel and the effect of beam camber does not result in out-of-square end plates that makes erection and field fit-up difficult. Shims may be required on runs of beams in a line in order to compensate for mill and shop tolerance. The use of this connection for two sided beam-to-beam connections should be considered carefully. Installation of bolts in the end plates can cause difficulties in this case. When unequal sized beams are used, special coping of the bottom flange of the smaller beam may be required to prevent it fouling the bolts. Since the end plate is intended to behave flexibly, damage of the end plate during transport is not normally of concern and may be rectified on site. For coped beams, the top of the end plate and the bottom of the cope cut should coincide. Curvature of the end plate due to welding can usually be pulled out when installing the bolts. Check end plate component width to ensure that it will fit between fillets of column section when connecting to column web.



uncoped and single web coped beams (end plate located towards top of beam)



uncoped and single web coped beams (end plate located towards bottom of beam)



double web coped beams

2.2 Possible failure modes

The acting force in the connection is shear force V*. Possible failure modes include weld failure along the web, bolt failure in shear, end plate component failure in shear, beam web failure in shear at end plate, coped section failure in shear near connection and coped section failure in bending near connection.

2.3 Design capacities

Design is based on determining V_{des}, the design capacity of the connection, which is the minimum of the design capacity $\{V_a, V_b, V_c, V_d, V_e, V_f\}$. The design requirement is then $V_{des} \geq V^*$. Each of the design capacity is given below.

(a) Weld to web $V_a = \phi\, v_w\, 2d_i$

The design capacity of the weld of the end plate to the beam web (V_a) is based on the assumption of vertical shear only on the fillet weld.

(b) Bolts in end plate $V_b = n_b\,(\phi V_{df})$
where $n_b = 2n_p$, n_p is the number of bolts in a line at pitch s_p and ϕV_{df} is the design capacity of a single bolt in shear.

The design capacity of the bolts in the end plate (V_b) is based on the assumption of vertical shear acting at the bolt group centroid. Possible failure modes of bolt shear, local bearing failure and end plate tearout are considered. For economy, either 4.6/S or 8.8/S bolting category is preferred. 8.8/TB category is uneconomic since it has the same design capacity as 8.8/S and requires tensioning. The use of 8.8/TF bolting category in this connection is not recommended since 8.8/TF is designed on a "no-slip" basis. While this may be desirable in certain restricted instances in order to maintain beam levels, it also rstricts the horizontal slipping of the end plate, which is an inherent part of the connection's "flexible" behaviour. This may result in the development of high levels of restraint moment at the support.

(c) End plate component in shear $V_c = \phi\,(0.5\, f_{yi}\, t_i\, 2\, d_i)$
where $\phi = 0.9$

The design capacity of the end plate in shear (V_c) assumes that failure, if it occurs, takes place on each side of the weld/web interface and that it occurs by shear yielding.

(d) Beam web in shear at end plate $V_d = \phi\,(0.6\, f_{yw}\, t_{wb}\, d_i)$
where $\phi = 0.9$

The expression for the shear capacity of the web at the end plate/web interface (V_d) has been derived by assuming that a near uniform stress distribution applies at the interface and that therefore, the nominal capacity is given by Clauses 5.11.2 and 5.11.4 of AS4100.

(e) Coped section in shear near connection $V_e = \phi V_u$

(f) Coped section in bending near connection $V_f = \phi M_s/e_v$

The expression for V_e and V_f deal with the strength of the section remaining after coping of the supported member.

The following additional design considerations are worth noting. Flexible end plate connections will exhibit a wide range of connection flexibility depending on the connection parameters such as plate thickness, plate depth, bolt category, web thickness. Rotational flexibility in the connection is required if the connection is to meet the requirements of AS4100 for simple construction. This is provided by the use of a relatively thin end plate which deforms out of plane under applied rotation, the use of snug-tightened bolts which allows the end plate to slip horizontally and detailing a wide gauge between lines of bolts.

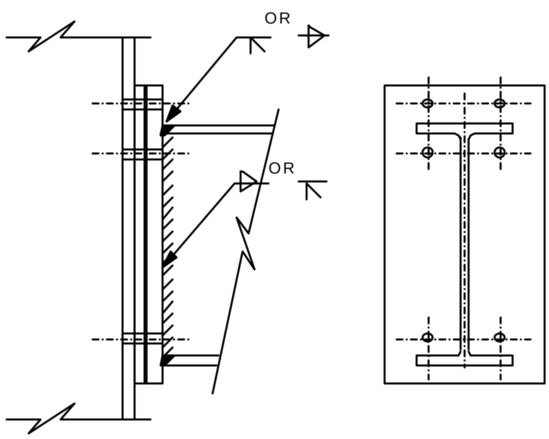
Example 1

See section 4.3.3 of AISC DSC/04-1994 Hogan, T.J. and Thomas, I.R. (1994), Design of Structural Connections, 4th Edition, Australian Institute of Steel Construction, Sydney, page 58 to page 59

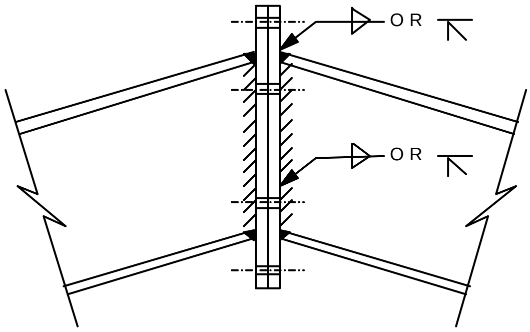
3. Moment Connections (Bolted End Plate)

3.1 Connection Details

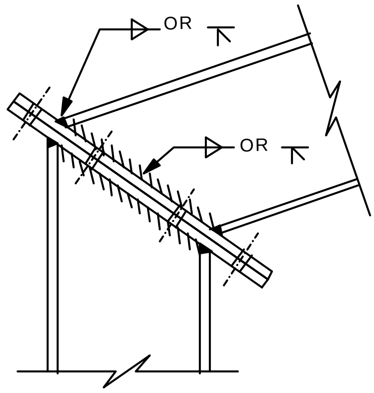
Typical bolted end plate moment connections are shown in the figure below. Fabrication of this type of connection requires close control in cutting the beam to length and adequate consideration must be given to squaring the beam ends such that end plates at each end are parallel and the effect of any beam camber does not result in out-of-square end plates which make erection and field fit-up difficult. Shims may be required to compensate for mill and shop tolerances. 8.8/T (fully tensioned) bolt category can be used. Holes are 2mm larger than the nominal bolt diameter.



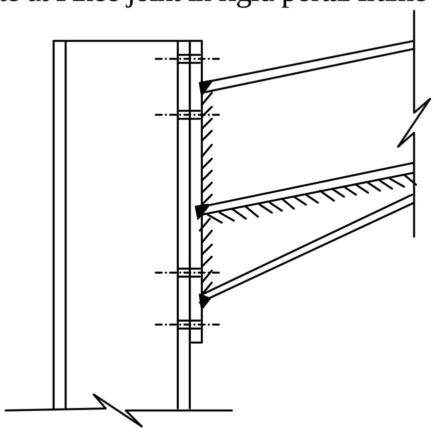
end plate at right angle to column



end plate at apex in rigid portal frame

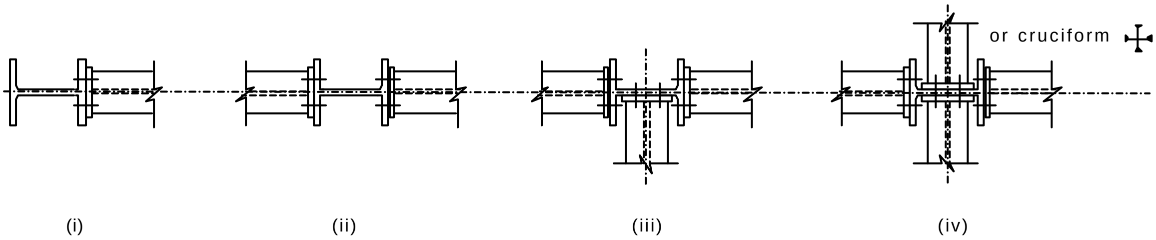


end plate at Knee joint in rigid portal frame



end plate at Knee joint with or without haunch in rigid portal frame
-incoming member inclined to column

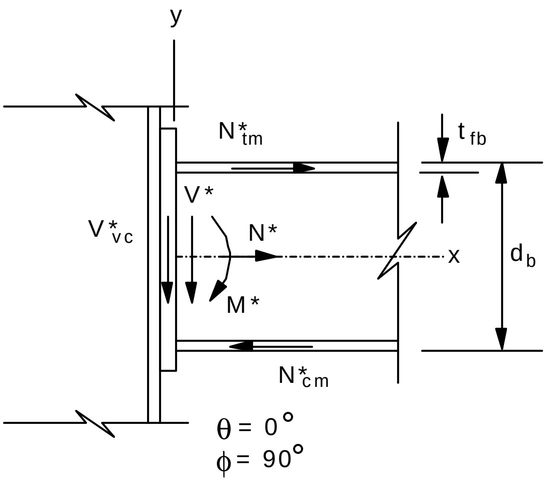
This type of connection may be used in the following variations as a beam-to-column connection (see the figure below):
(i) one sided beam-to-column flange
(ii) two sided beam-to-column flange
(iii) two way, two sided beam-to-column flange plus one sided beam-to-column web
(iv) four ways, two sided beam-to-column flange plus two sided beam-to-column web



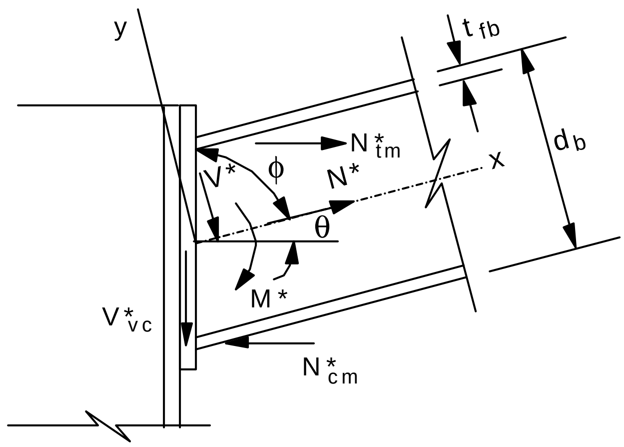
3.2 Actions and possible failure modes

This types of connection is considered to be a rigid connection wherein the original angles between the members remain unchanged during loading and the connection would be used in a frame where rigid construction was the assumed form of construction.

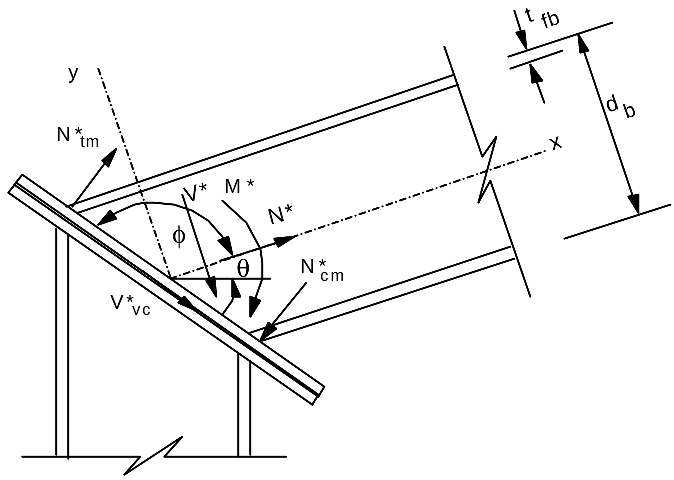
The design action effects at the connection could be determined from either a first order elastic analysis with moment amplification or a second order elastic analysis. Applied actions at a connection are assumed to be bending moment M*, shear force V* and axial force N* as shown in the figure below.



Type A



Type B



Type C

The following assumptions are made in determining actions in various components:

(i) The flanges transmit design flange forces due to moment M*, these comprising N_{tm}* (tension flange) and N_{cm}* (compression flange). For the design of the flange and web welds, the assumption is made that the proportion of the bending moment transmitted by the web is k_{mw} while the proportion of the bending moment transmitted by the flange is (1-k_{mw}). The proportion of the bending moment transmitted by the web is given by

$$k_{mw} = \frac{l_w}{l_w + l_f}$$

However for the assessment of the loads on the bolts, and on the end plate and for the assessment of the necessity for stiffeners and the design of the stiffeners, it is conventional practice to assume that all the force above and below the neutral axis is concentrated at the flanges which is equivalent to assuming that all the bending moment is transmitted through the flange area.

(ii) The web transmits the design shear force V*.
(iii) For the design of welds, it is assumed that the flanges and web transmit a share of the axial force N*, the proportion taken by each being proportional to their contribution to the total section area.

For the design of bolts and the end plate, it is assumed that the flanges transmitted all of the design axial force N*, the proportion taken by each being proportional to their contribution to the total section area. This assumption is made because the bolts, which must transmit the axial force into the column, are concentrated at the flanges.

The design actions can be summarized as follows:

- For the **flange welds** connecting the beam to the end plate:

  $$N^*_{fw} = \frac{M^*}{(d_f - t_{fb})}(1 - k_{mw}) + N^*\left(\frac{A_t}{A}\right)$$

  Where,
  k_{mw} = proportion of the bending moment transmitted by the web (see attached)
  A_t = area of tension flange, A = total area of the section

- For the **web welds** connecting the beam to the end plate:

  Axial force component $N_w^* = k_w N^*$

  Moment component $M_w^* = k_{mw} M^*$

  Shear force component $V_w^* = V^*$

  Where,
  k_w = (area of the web)/(total cross-sectional area), k_{mw} as defined above.

- For the **bolts, end plate and stif**feners:

  Total design force in tension flange: $N^*_{ft} = \dfrac{M^*}{(d_b - t_{fb})} + \dfrac{N^*}{2}$

  Total design force in compression flange: $N^*_{fc} = \dfrac{M^*}{(d_b - t_{fb})} - \dfrac{N^*}{2}$

  Total design shear force at end plate/column interface: $V_{vc}^* = V^*$

Clause 9.1.4 of AS4100 requires that this type of connection be designed for the following minimum design actions:

bending moment = 0.5 times the member moment capacity

No minimum requirement is placed on the simultaneously applied shear force or axial force. It is suggested that the following requirement is placed on the simultaneously applied shear force or axial force. It is suggested that the following minimum values might be used simultaneously with the above minimum design bending moment:

Shear force = 40 kN
Axial force = 0

The intention of the AS4100 provision is that connections have a guaranteed minimum design capacity with some inherent robustness.

Possible failure modes include flange welds failure, web welds failure, bolts failure, end plate failure and stiffeners failure.

3.3 Design capacities

Design capacities are given for each possible failure mode in the connection.

- Flange welds

Check $N_{fw}^* < \phi N_w = \phi f_{fy} b_{fb} t_{fb}$ for full penetration butt welds
where ϕ = 0.9 for SP (structural purpose) weld, f_{fy} = yield stress of beam flange, b_{bf} = width of beam flange, t_{fb} = beam flange thickness.

Check $N_{fw}^* < \phi N_w = 2 L_w (\phi v_w)$ for fillet welds
where L_w =weld length across flange, usually b_f and (ϕv_w) = design capacity of fillet weld per unit length of weld

- Web welds

Check $\sqrt{v_z^{*2} + v_y^{*2}} \leq \phi v_w$

$$v_z^* = \frac{N_w^*}{2L_w} + \frac{3M_w^*}{L_w^2}$$

$$v_y^* = \frac{V^*}{2L_w}$$

where v is strength per unit length, L_w = weld length along web = $d_b - 2t_{fb}$

- Bolts

Check $\frac{N^*_{ft}}{n_t} \leq \phi N_{tf}$

Check $\frac{V^*_{vc}}{n} \leq \phi V_{fx} \text{or} \phi V_{fn}$

where n_t is the number of bolts in tension, n is the number of bolts in shear, N_{tf}, V_{fx} and V_{fn} are capacities for a single bolt.

Check bolt group

$$(\frac{1.2 \cdot N^*_{ft} / n_t}{\phi N_{tf}})^2 + (\frac{N^*_{vc} / n}{\phi N_{fx} \text{or} \phi N_{fn}})^2 \leq 1$$

- End plate

Flexure

Check $N^*_{ft} \leq \frac{\phi \cdot f_{yi} \cdot b_i \cdot t_i^2}{a_{fe}}$

where f_{yi} = yield stress of end plate, b_i = width of end plate, t_i = end plate thickness, a_{fe} = distance between the bolt center and the top flange of the beam.

Shear

Check $V^*_{vc} \leq 2 \cdot \phi \cdot 0.5 \cdot f_{yi} \cdot d_i \cdot t_i$
and $N^*_{ft} \leq 2 \cdot \phi \cdot 0.5 \cdot f_{yi} \cdot b_i \cdot t_i$
and $N^*_{fc} \leq 2 \cdot \phi \cdot 0.5 \cdot f_{yi} \cdot b_i \cdot t_i$
where d_i = depth of end plate

- Stiffeners

Check necessity of column stiffeners

If $\frac{N^*_{ft}}{\phi \cdot f_{yc} \cdot t_{wc}} > \frac{d_b - t_{fb}}{2}$, stiffeners are required.

where f_{yc} = yield stress of column, t_{wc} = thickness of column web, d_b = depth of beam, t_{fb} = thickness of beam flange.

Example 2

See section 4.8.4 of AISC DSC/04-1994 Hogan, T.J. and Thomas, I.R. (1994), Design of Structural Connections, 4th Edition, Australian Institute of Steel Construction, Sydney, page 115 to page 118