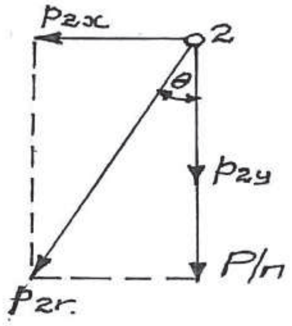
Re-arranging the **torsion equation** results in:

$$\tau A = p_n = \frac{T\rho}{\sum_{i=1}^{n} y_i^2 + \sum_{i=1}^{n} x_i^2} \qquad (10.8)$$

The **critical nail force** will occur on the nail which has:

- components of p_{in}, i.e.p_{inx} and p_{iny} **additive** to the **components** of **P**, in this case **P/n**

Hence, **nail 2** in Figure 10.5 will be the **worst loaded nail.** The components of nail load will be:



where:

$$p_{2x} = \frac{T\bar{y}}{I_p}$$

$$p_{2y} = \frac{T\bar{x}}{I_p}$$

$$p_{2r} = \sqrt{(p_{2x})^2 + (p_{2y} + P_y/n)^2} \qquad (10.9)$$

There is **no reason** why this approach **cannot be used** in practice provided a suitable **computer program** was developed.

Alternative Methods are available for determining the **moment capacity** of **rotational joints** such as that shown in FIGURE 10.6.

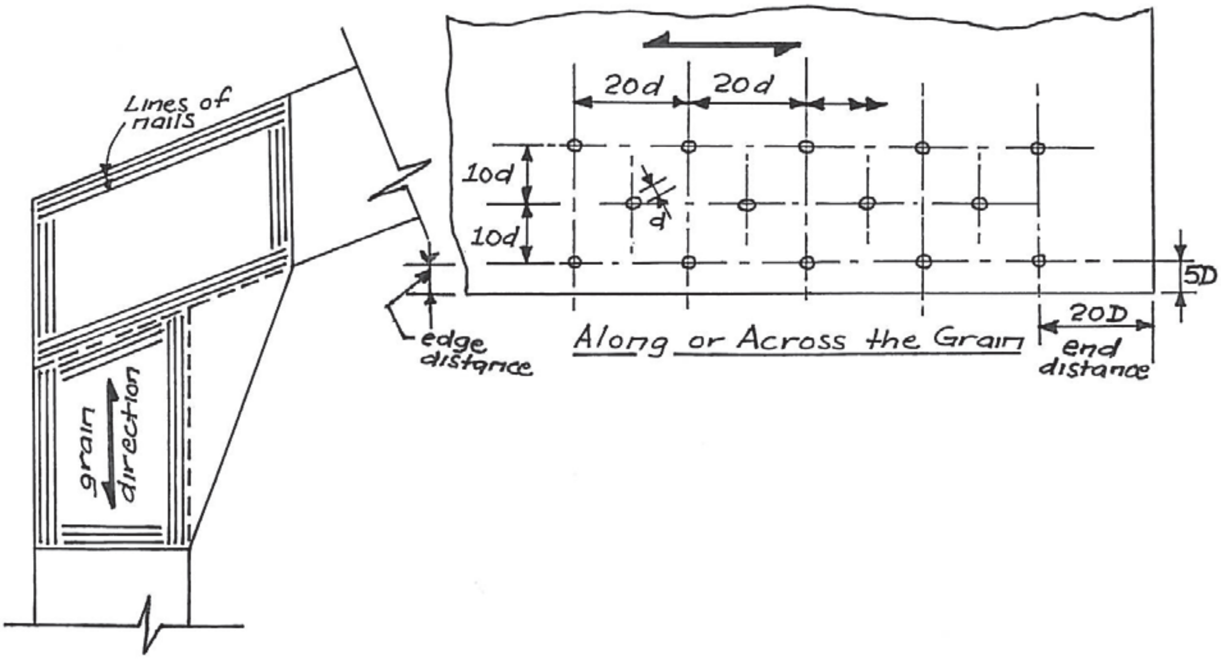


FIGURE 10.6: Typical nailing pattern and an idealised line representation

AS1720.1-1997 suggests the application of the relationship:

$$\varphi M_j = \varphi k_1 k_{13} k_{14} k_{16} k_{17} r_{max} Q_k \left[\sum_{i=1}^{n} \left(\frac{r_i}{r_{max}} \right)^{3/2} \right] \qquad (10.10)$$

An **alternative, simpler** but more **conservative method** of determining the **moment capacity** of a nail group is that recommended by **Hutchings, as described below.** This procedure assumes the nails to be **smeared as lines** whose **width (w)** is proportional to the **nailing density.**

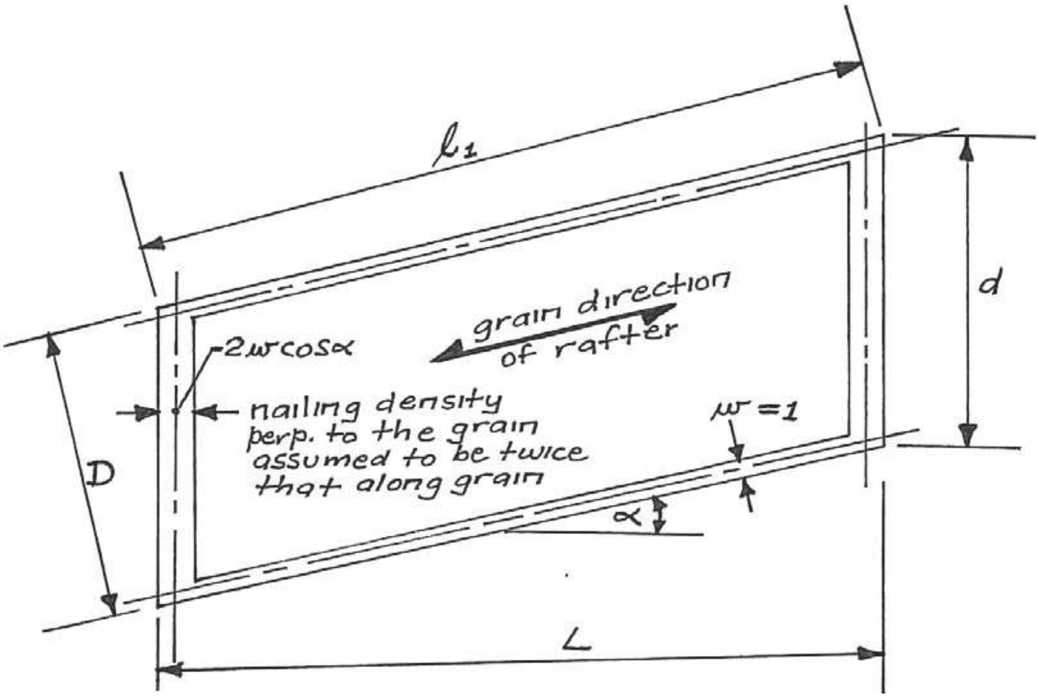


FIGURE 10.7: Nails smeared as a continuous line

To determine the **polar moment of area** of a line of **width w and length** ℓ about a **point O** as shown in FIGURE 10.8 can be shown by application of the **parallel axes theorem,** to be given by:

$$I_{po} = \frac{w\ell^3}{12} + w\ell r^2$$

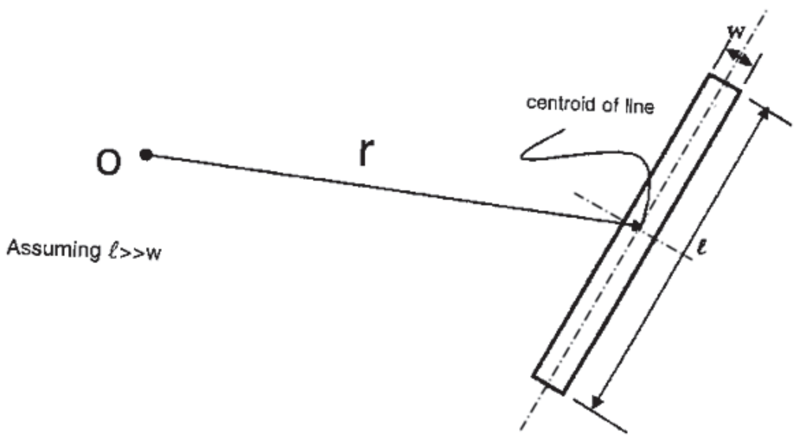


FIGURE 10.8: Polar moment of a line width w

For **each nail ring**, assume the **width w of** the **line of nails parallel to** the **grain is unity**, then the **width, w** of the **vertical line** becomes:

$$w_v = \frac{\text{nail spacing parallel to grain}}{\text{nail spacing perpendicular to grain}} \cos\alpha$$

where: α = the angle of the roof pitch.

The **polar moment of area (I_p)** of each of the nail rings with respect to the **nail group centroid** may then be calculated by substituting appropriate values of ℓ, **d and w_v into the following equation.**

$$I_p = \frac{2}{s}\left[\frac{\ell^3}{12} + \frac{w_v d^3}{12} + \ell\left(\frac{d}{2}\right)^2 + w_v d\left(\frac{\ell}{2}\right)^2\right] \qquad (10.11)$$

where: s = nail spacing along the grain.

The **polar moment (I_p)** for the joint group will then be the **sum of the polar moments of the individual rings of nails.**

10.4 Plywood / LVL Gusseted Joints – Methodology

The **steps** involved **in the design of a plywood / LVL gusseted joint**, (for the nomenclature refer Figues 10.2 and 10.3) are as follows:

i. Determine the portal frame **moments, shears and axial forces** from a rigid frame analysis. Obtain a **preliminary size** of the **column / rafter** by application of the **flexure formula:**

$$Z = M/f_b$$

For portals spans to about 20m, assume a member breadth of b = 60 to 100mm

ii. Determine the **length** of the **gusset** which should be **1.5 to 2 times depth of the column / rafter.**

iii. Determine the **effective depth h** for a **mitred internal knee** or **ridge gusset** or d_e for an **internal or external haunch gusset** at the **critical stress line.**

$$h = 2D \text{ or}$$

$$d_e = \frac{L - D}{1 + \left(1 - \frac{D}{2L}\right)\tan\propto} + D \text{ or}$$

$$d_e = L - y$$

where $\propto$ = roof slope

Depth of rafter/ column D (mm)	Rafter Pitch in degrees							
	5		10		15		20	
	d_e (mm)		d_e (mm)		d_e (mm)		d_e (mm)	
	L=1.5D	L=2D	L=1.5D	L=2D	L=1.5D	L=2D	L=1.5D	L=2D
200	295	390	290	380	285	370	280	360
300	445	585	435	565	430	550	425	540
400	590	776	580	755	570	735	565	715
600	885	1165	870	1130	855	1100	845	1075
800	1180	1555	1160	1510	1140	1470	1130	1430
1000	1475	1940	1450	1885	1425	1835	1405	1790
1200	1770	2330	1740	2260	1710	2200	1685	2145

TABLE 10.1: Effective depths (d_e) for internal haunch gussets

iv. Determine a **preliminary thickness** for the **plywood** or **LVL gusset**. The thickness of **parallel plies (t)** required each side of the joint is:

$$t \geq \frac{6M^*_p}{2(\Phi k_{19}\, g_{19}) f'_b\, d_e^2}$$

where:
M^*_p = **design in-plane moment** on joint
ϕ = **capacity factor** for plywood / LVL
k_1 = **duration of load strength** modification **factor**
k_{19} = **moisture condition** strength modification **factor**

g_{19} = **plywood assembly** modification **factor**
f'_b = **characteristic bending strength**
d_e = **effective depth** of gusset **at critical section**

v. Determine $\mathbf{I_p}$, the **polar moment of area** for each ring of nails, and sum to find $\mathbf{I_{p(total)}}$.

The procedure followed herein is that proposed by **Hutchings** and described in the **Nail Joint Action** section.

vi. Determine the **moment capacity** of the joint such that:

$$\phi M \geq M^*$$

AS1720.1-1997 approach for determining **rotational joint capacity** requires application of Equation 10.10:

The **Hutching Method,** which applies the **classical torsion equation** in which **nail force** is directly **proportional** to **distance** from the **nail group centroid**, will be used. That is:

$$\phi M = \phi . k_1 . k_{13} . k_{14} . k_{16} . k_{17} . Q_k \, [I_p / r_m] \qquad (10.12)$$

where: M^* = **design action effect** on joint (in-plane moment)
ϕ = **capacity factor** for a nailed joint — **AS1720.1-1997 Clause 2.3**
k_1 = the **factor** for **duration** of **load** for **joints** — **AS1720.1-1997, Clause 2.4.1.1**
k_{13} = 1.0 for nails in **side grain**
= 0.6 for nails in **end grain**

k_{14} = 1.0 for nails in **single shear**
= 2.0 for nails in **double shear**

k_{16} = 1.2 for nails driven through **close fitting holes** in **metal side plates**
= 1.1 for nails driven through **plywood gussets**
= 1.0 otherwise

k_{17} =factor for **multiple nailed joints** given in AS1720.1-1997 Table 4.3(B) forType 1 joints to resist in-plane moments
r_m = the **maximum** value of $\mathbf{r_i}$
r_i = the **distance** from the **ith nail** to the **centroid** of the nail group
Q_k = **characteristic capacity** given in — **AS1720.1-1997 Table 4.1(A) and 4.1(B)**

I_p = **polar moment** of **inertia**

vii. Check capacity of **worst loaded nail.**

Design Example – Plywood Gusseted Portals

Assume the **materials** to be used to be:

- **Gusset: F11 structural pine** 2400 x 1200mm plywood panels;
- **2.9mm** diameter machine driven nails;
- **600 x 63mm LVL** for columns and rafters with a **joint strength group JD3**

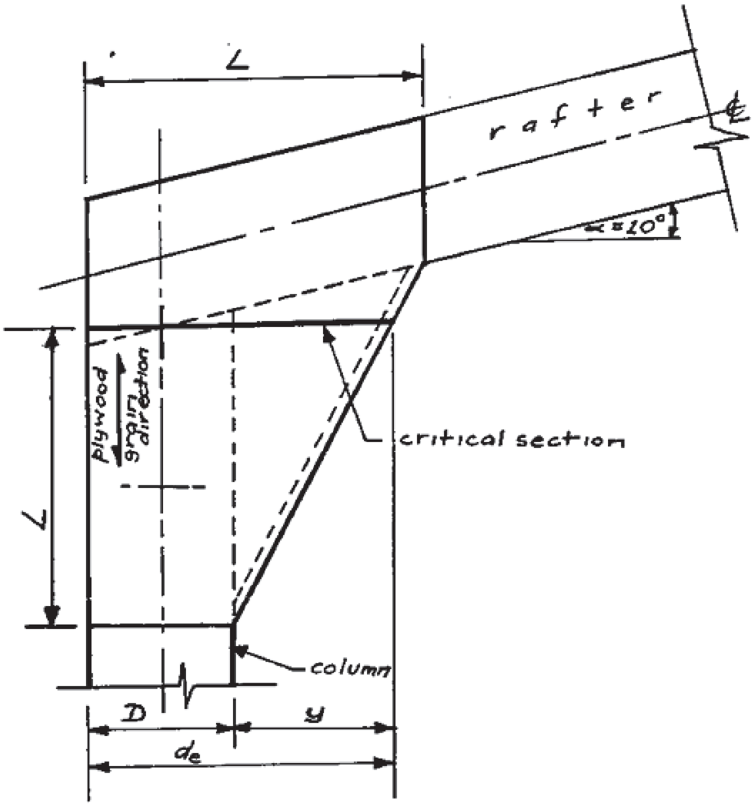


FIGURE 10.9: Defines the major dimensions of the knee joint

Gusseted Joints – Worked Example

1. Loading

Assume the **worst loading condition** on the gusset to be due to a **combination of wind and dead load** resulting in:

column moment, M*	**= 144kNm;**
column axial force	**= 55kN;**
column shear force	**= 20kN**

NOTE:
These member forces are **typical** for a **portal span** of **18m**, an **eaves height** of **6m**, **frames** at **6m spacing**, a roof slope $\propto$ = 10° and a wind speed of 41m/s.

2. Sizing of Gusset:

The **length (L)** of the gusset should be **1.5 to 2 times the depth of the column/rafter**. Choose L = 2D, hence for 600mm deep column/rafter members:

L = 1200mm

3. Determine the **effective depth (d_e)** of the internal gusset:

$$\mathbf{d_e} = \frac{L-D}{1+(1-D/2L)\tan\alpha} + D$$

$$= \frac{1200-600}{1+(1-600/2.1200)\tan 10^\circ} + 600$$

$$\mathbf{= 1130mm}$$

4. Determine **required gusset thickness:**

$$t \geq \frac{6M^*_p}{(\phi k_1 k_{19} g_{19}) f'_b d_e^2}$$

For F11 structural plywood:

f'_b = 35 MPa (F11 structural plywood)		**Table 5.1**
M^* = design action effect		
= 144 kNm		
ϕ = capacity factor for plywood = 0.8		**Table 2.6**
k_1 = 1.15 (wind gust)		**Table 2.7**
k_{13} = 1.0 (moisture content <15%)		**Table 5.2(A)**
g_{19} = 1.0		**Table 5.3**

Required thickness of parallel plies per side:

$$t = \frac{1}{2}\left(\frac{6 \times 144 \times 10^6}{0.8 \times 1.15 \times 1.0 \times 1.0 \times 35 \times 1130^2}\right)$$

$$= 10.5\,mm$$

Choose 18-30-7 ($t_{//}$ = 10.8mm) F11 DD 2400 x 1200 structural plywood

5. Nail Joint Design

The **most important**, and **time consuming** task associated with the design of the nailed joint, is the determination of the **polar moment of area (I_p)** of the nail group.

For economy of calculation it is usual to have the **same nailing pattern** for both the **rafter** and **column connections** to the gussets. The **design moment** used in the joint design is conservatively taken as the moment determined at the rafter/column centre lines intersection. The **actual design moment** effective at the nail **group centroid** is typically **smaller** than that determined by the computer analysis which is at the rafter/column centre lines intersection.

For convenience of reference, restating **Equation 10.11** allows the determination of (I_P) for a **smeared single ring** (rectangle) **of nails** as shown in Figure 10.7 and results in:

$$I_p = \frac{2}{S_{II}}\left[\frac{\ell_1^3}{12} + \frac{wd^3}{12} + \ell_1\left(\frac{d}{2}\right)^2 + wd\left(\frac{\ell_1}{2}\right)^2\right]$$

In this example the **nail centres** for the **LVL** will be:

- **edge** distance – 5D
  = 5 x 2.9 ≅ 15 say 20 mm.

- **parallel** to grain – 20D
  = 20 x 2.9 ≅ 60 mm.

- **perpendicular** to grain – 10D
  = 10 x 2.9 ≅ 30 mm.

- **assuming width of lines parallel to grain is unity, then width of lines** perpendicular to grain will be:

$$\mathbf{w_v} = \frac{\mathbf{60}}{\mathbf{30}}\mathbf{\cos 10^\circ}$$
$$= \mathbf{1.97}$$

The **dimensions** of the **first ring of nails** are shown in FIGURE 10.10 as being $2(\ell_1 + d_1)$ and can be evaluated from:

$$\ell_1 = \frac{1200}{\cos 10^\circ} - 60 - 20$$

plywood edge distance
LVL edge distance

ℓ_1 **= 1140 (*which is equally divisible by 60*)**

$$\mathbf{d_1} = \frac{\mathbf{D}}{\mathbf{\cos 10^\circ}} - \mathbf{2 \, x \, 20}$$

$\mathbf{d_1}$ = **570 (which is equally divisible by 30)**

Therefore for the first ring of nails:

$$\mathbf{I_{p1}} = \frac{2}{60}\left[\frac{1140^3}{12} + \frac{1.97 \text{ x } 570^3}{12} + 1140\left(\frac{570}{2}\right)^2 + 1.97 \text{ x } 570\left(\frac{1140}{2}\right)^2\right]$$

$$\mathbf{I_{p1}} = \mathbf{20.38 \text{ x } 10^6 \text{ mm}^2}$$

For the second and third rings of nails:

ℓ_2 = 1140 – 60
= 1080;

d_2 = 570 – 60
= 510

ℓ_3 = 1080 – 60
= 1020;

d_3 = 510 – 60
= 450

$$\mathbf{I_{p2}} = \frac{2}{60}\left[\frac{1080^3}{12} + \frac{1.97 \text{ x } 510^3}{12} + 1080\left(\frac{510}{2}\right)^2 + 1.97 \text{ x } 510\left(\frac{1080}{2}\right)^2\right]$$

$$\mathbf{I_{p2}} = \mathbf{16.33 \text{ x } 10^6 \text{ mm}^2}$$

$$\mathbf{I_{p3}} = \frac{2}{60}\left[\frac{1020^3}{12} + \frac{1.97 \text{ x } 450^3}{12} + 1020\left(\frac{450}{2}\right)^2 + 1.97 \text{ x } 450\left(\frac{1020}{2}\right)^2\right]$$

$$\mathbf{I_{p3}} = \mathbf{11.13 \text{ x } 10^6 \text{ mm}^2}$$

To determine the **number of nails per ring:**

$$n = \frac{2}{s_{//}}\left(\ell_n + \frac{s_{//}}{s_\perp} dn\right)$$

where:

$s_{//}$ = Nail spacing **parallel** to the grain;
$s_\perp$ = Nail spacing **perpendicular** to grain;

ℓn = Length of the **nth nail ring;**
dn = **Height** of the **nth nail ring.**

Nail Ring Number	$I_p(mm^4)$	Nails/ring	Total nails/gusset
1	20.38×10^6	76	76
2	16.33×10^6	70	146
3	11.13×10^6	64	210
I_p(total)	47.84×10^6		

Co-ordinates of **extreme nail from** the **centroid** as **defined** by $\mathbf{x_m}$ and $\mathbf{y_m}$ in **Figure 10.10.**

$$X_m = \frac{L}{2} - 60$$
$$= 600 - 60$$
$$X_m = 540\,mm$$
$$Y_m = \frac{d_1}{2} + \frac{\ell_1}{2}\sin 10^\circ$$
$$= 285 + 99$$
$$Y_m = 384\,mm$$
$$\rho = \sqrt{(540)^2 + (384)^2}$$
$$\rho = 663\,mm$$

6. **Joint Capacity – Moment Joint Design**

From AS1720.1-1997 the capacity of the nailed moment joint is given by:

$$\varphi M_j = \varphi . k_1 . k_{13} . k_{14} . k_{16} . k_{17} . Q_k \left[\frac{I_p}{r_m}\right] \geq M^*$$

where:

ϕ = capacity factor = **0.8**
M_j = **moment capacity of** the **nailed joint**
M^* = **design action effect on** the **joint**, i.e the calculated moment to be resisted
k_1 = duration of load factor for joints
= **1.3 in this case**

k_{13} = 1.0 for nails in side grain
= 0.6 for nails in end grain
= **1.0 in this case**

k_{14} = 1.0 for nails in single shear
= 2.0 for nails in double shear
= **1.0 in this case**

k_{16} = 1.2 for nails driven through close fitting holes in metal side plates
= 1.1 for nails driven through plywood gussets
= 1.0 otherwise
= **1.1 in this case**

k_{17} = factor for multiple nailed joints for Type 1 joints resisting in-plane moments
= **1.2 in this case**

hence:

$$\Phi M_j = (0.8 \times 1.3 \times 1.0 \times 1.0 \times 1.1 \times 1.2 \times 706)\left[\frac{I_p}{r_m}\right]$$

$$= 962\left[\frac{I_p}{r_m}\right] 2 \text{ (gusset each side)}$$

where:

I_p = 47.84 x 10^6 mm^2

r_m = 637 mm

$$\Phi M_j = \left(\frac{962.2 \times 47.84}{637}\right) \times 2 \text{kNm}$$

$$= 145 \text{kNm}$$

Design Action Effect

From the relationship:

ϕM_j	≥ **M***
145 kNm	**> 144 kNm,** hence the **joint is OK**

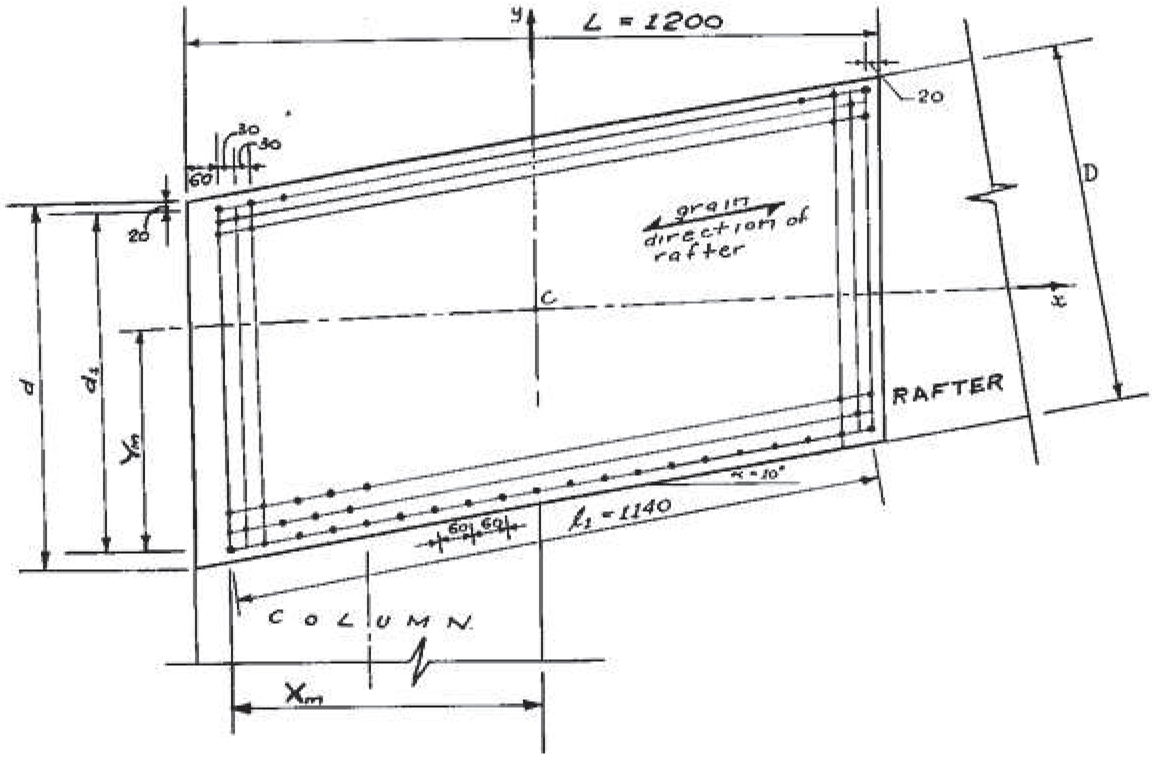


FIGURE 10.10: Shows nailing pattern for nail group in column or rafter

7. **Worst Loaded Nail**

The nailing pattern has been determined for the **rafter member** in this instance. The **maximum forces** acting on the **worst loaded nail** however, are given as being developed in the **column element.** Since the nailing pattern for each primary structural member is identical the co-ordinates of the worst loaded nail will be the same. **More** precisely, (ρ) will be the same, rather than $\mathbf{x}_m$ and $\mathbf{y}_m$.

To evaluate the **force** on the **worst loaded nail:**

$$p_{ix} = \frac{T \times Y_m}{2 \times I_p} + \frac{S}{n}$$

$$= \frac{144 \times 10^6 \times 384}{2 \times 47.84 \times 10^6} + \frac{20 \times 10^3}{2 \times 210}$$

$$= 578 + 48$$

$$p_{ix} = 626\,N$$

$$p_{iy} = \frac{T \times X_m}{2 \times I_p} + \frac{P}{n}$$

$$= \frac{144 \times 10^6 \times 540}{2 \times 47.84 \times 10^6} + \frac{55 \times 10^3}{2 \times 210}$$

$$= 813 + 131$$

$$p_{iy} = 944\,N$$

$$p_{ir} = \sqrt{(626)^2 + (944)^2}$$

$$p_{ir} = 1132\,N$$

8. Nail Capacity

The **design capacity (ϕN_j)** of a **2.9 ϕ nail** driven into seasoned timber of strength group JD3 is:

$$\phi N_j = \phi . k_1 . k_{13} . k_{14} . k_{16} . k_{17} . n . Q_k$$

where:

Q_k from interpolation		= 989N
For nails in primary structural elements other than houses	ϕ	= 0.8
For wind gusts	k_1	= 1.14
For nails inside grain	k_{13}	= 1.0
For nails in single shear	k_{14}	= 1.0
For nails through plywood gussets	k_{16}	= 1.1
For multiple nailed joints	k_{17}	= 1.0
Number of nails	n	= 1.0

Hence:

ϕN_j = (0.8 x 1.14 x 1 x 1 x 1.1 x 1 x 989)N

ϕN_j = **983 N <1132N** so **not OK.**

Since the **nail capacity** is **less than required** a **design decision** has to be made. This will not be pursued further herein. However, the example does demonstrate, yet again, the iterative nature of the design process, leaving the designer needing to assess the options available. The obvious ones in this instance are:

- to **add** an **extra ring** of **nails;**
- **increase** the **nail diameter**

10.5 Photograhs

Appendix A10 provides some examples of portal frames, moment joints and pin joints.

A10 Chapter 10 Appendix

Photographs of Portals, Moment and Pin Joints

Plate 1

Plate 2

Plate 3

Plate 4

Plate 5

Plate 6

Plate 7

Plate 8

Plate 9(a)

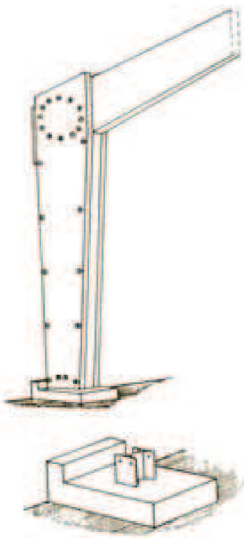


Plate 9(b)

Plate 10

Plate 11

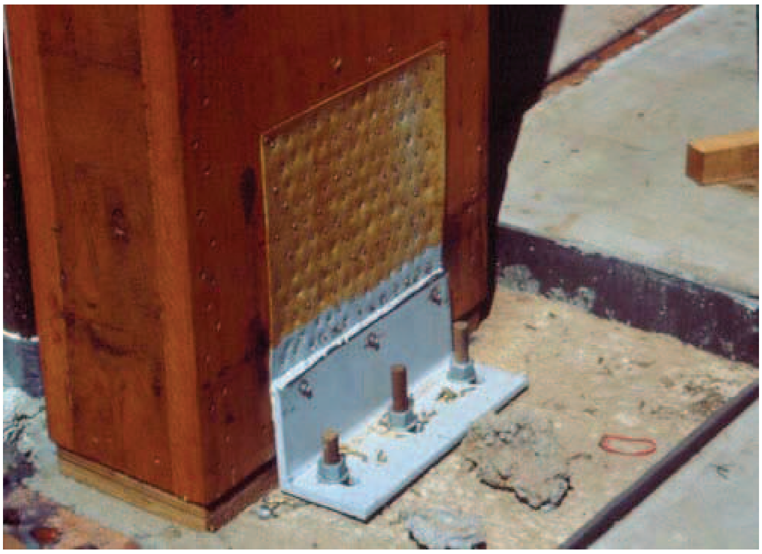


Plate 12

REFERENCES CITED:

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2. **Investigation of Plywood Gusseted Joints**, 4th Year Civil engineering Project Report, Central Queensland University (CIAE), 1983.

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4. **Portal Frames**, B. Walford, Section B-2, Timber Use Manual, New Zealand Timber Industry Federation Incorporated.

5. **Portal Frame Design Example**, B. Hutchings, TRADAC Timber Engineering Workshop, Brisbane, April, 1989.

6. **Glue Laminated Timber Portal Framed Industrial Warehouse Building**, D. Wheeler, Paper prepared for National Association of Forest Industries.

7. **AS1720.1-1997 – Timber Structures, Part 1 : Design Methods.**

11 Plywood Stressed Skin Panels

11.1 Introduction

Because of its **lightness**, **directional strength properties and inherent stiffness** plywood is an excellent material to fix to timber beam (stringer) elements to produce a composite construction. Such a structural system can have the **plywood skins** affixed to **one** or **both sides** of the stringers. **Structurally,** the function of the plywood skins is to develop the **flexure stresses** as in-plane **tension** and **compressive** stresses as a result of loading the panel perpendicular to its surface.

To be categorised as a **stressed skin panel** as shown in FIGURE 11.1 the **plywood sheathing must be glued** to the stringers. The necessary **pressure** required to effect curing of the adhesive can be applied by **nailing, screwing** or **stapling.** If plywood/stringer interconnection is sought through **mechanical fasteners** only (no adhesive) **full composite action** will not be attained and a stressed skin system as referred to herein will not result.

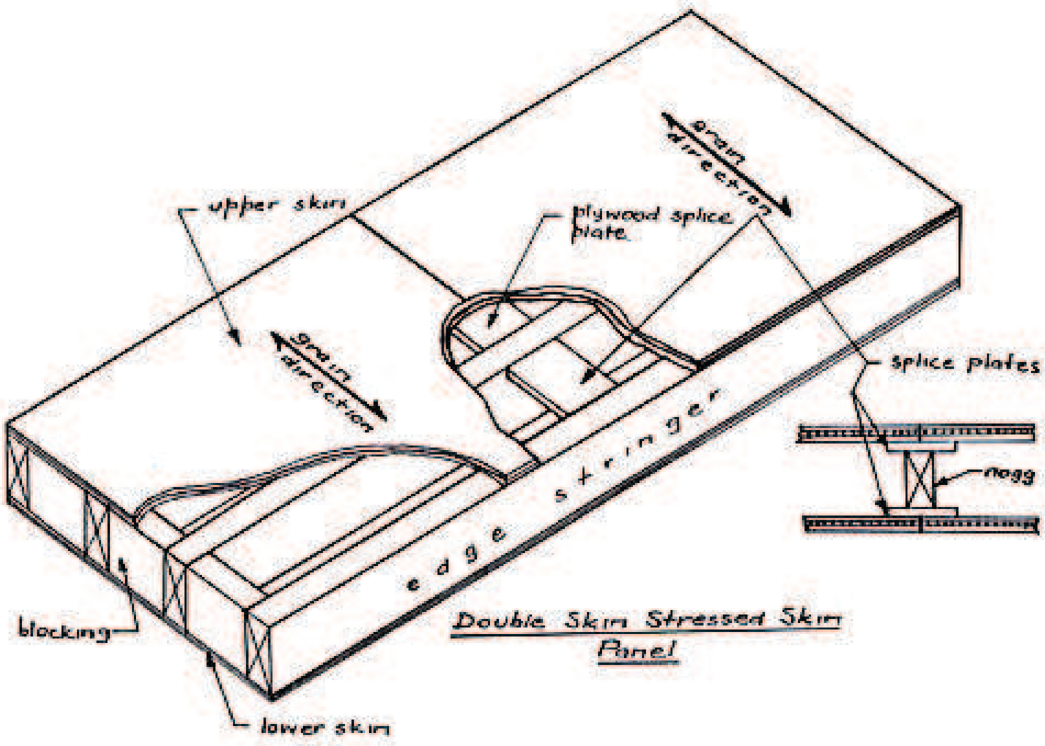


FIGURE 11.1: Component parts of a stressed skin panel

The **plywood skins** of **roof, wall or floor panels** fulfil a number of important functions, e.g. they:

- develop **I** or **tee beam action** thus **minimising stringer size** for a given span;
- provide a **trafficable surface** for floors or roofs which can be covered by other materials such as tiles, vinyl, etc. or sanded and suitably coated to provide an aesthetically pleasing floor;
- provide a **feature ceiling**;
- develop **diaphragm action** to resist in-plane horizontal **wind** or **earthquake loads;**
- provide a **void** between skins which can be filled with **insulation.**

The **plywood stressed skin panel** is also highly amenable to **prefabrication** thus allowing **process control procedures** to be implemented therefore ensuring the quality of the glue bonds.

Maximum spans of simple stringer members is generally constrained to the 7 to 9m range, however with the **availability of LVL** this range can now be extended.

11.2 Materials

Plywood

Plywood used in the construction of stressed skin panels designed to the specifications stated herein, shall be **EWPAA product certified structural plywood,** manufactured to AS/NZS 2269-2004 : Plywood - Structural.

Plywood panels can be joined by **scarf jointing** provided: the scarf is;

- not steeper than **1 in 8** in the **tension** skin;
- not steeper than **1 in 5** in the **compression** skin.

Butt joints in the plywood skins shall be backed with **plywood splice plates** centred over the joint and glued over the full contact area. The width of splice plates shall be **25 x thickness** of the plywood skin.

At the time of **gluing** the plywood **moisture content** must be within the limits specified by the glue manufacturer.

Stringers

Stringers of **LVL** must be EWPAA stamped with a **stress grade** or an **identification mark** associated with a **defined mechanical property**.

At the time of **gluing** the **moisture content** of the stringers must be within the limits specified by the glue manufacturer. Stringer surfaces to be glued must be **clean** and free from **oil** and other **foreign matter** likely to inhibit the gluing process.

Glue

Stress skin plywood systems in which the components have been interconnected using **thermo-setting resins,** the **shear resistance** is **fully dependent upon the adhesive,** the nail contribution being discounted. The reason for this is attributed to the **greater rigidity of the adhesive** compared to the nails whose main function is to apply the necessary pressure to effect curing.

Adhesives used in this application must be **room temperature setting** with a modicum of **gap filling properties** in the event mating surfaces are not smooth and even. The preferred adhesive for this application will therefore be of the **resorcinol family,** unless otherwise specified by the designer. Because nailing applies permanent pressure to the glueline **curing time is not critical.**

Nails

Nails should be **minimum of 2.8mm diameter** for all thicknesses of plywood with a depth of **penetration** into the stringers of **not less than 2.5 x plywood thickness or 20mm.** Nail spacing should be:

- not greater than **100mm along the framing members**,
- a **single row** on stringers up to 50mm thick and **two rows** on stringers greater than 50mm wide up to 100mm wide.

Nailing may commence at **any point** but must progress to an end or ends.

Unless otherwise stated **panel edge straightness, squareness, width and length** shall not vary outside the limits set for these parameters for a **plywood panel.**

Insulation and/or Vapour-Barrier Material

Insulation and vapour-barrier material must be securely fastened to the structural assembly in such a manner as to **not interfere** with the gluing of the plywood skins to the stringers. **Ventilation** requirements should be incorporated as seen necessary by the designer.

11.3 Application

Although only the design of **flat panels** will be considered in this Manual **curved panels** for roof construction are also a viable proposition. **Uses** for plywood stressed skin panels can be found in:

- **prefabricated housing** for walls, floors and roofs;
- **folded plate roofs;**
- **curved roofs** for domestic, commercial and industrial buildings;
- **concrete formwork**
- a range of applications dependent upon the **designers ingenuity.**

11.4 Stressed Skin Panel Design – Panel Action

Simplistically, the flat panel with plywood skins rigidly fixed to either side of timber stringers, performs structurally as a **series of composite I-beams.** The **plywood skins** develop most of the **normal stress due to bending** of the panel whilst the **shear stresses** are taken by the **stringers.**

Shear Lag

A stress resultant phenomena resulting from loading thin walled structures, and to which the **elementary flexure theory** does not directly apply **due to the influence of shear deformations**, is termed **shear lag.**

The **normal stress distribution** across the flange of a stressed skin panel subjected to bending is **non-uniform** as shown in FIGURE 11.2.

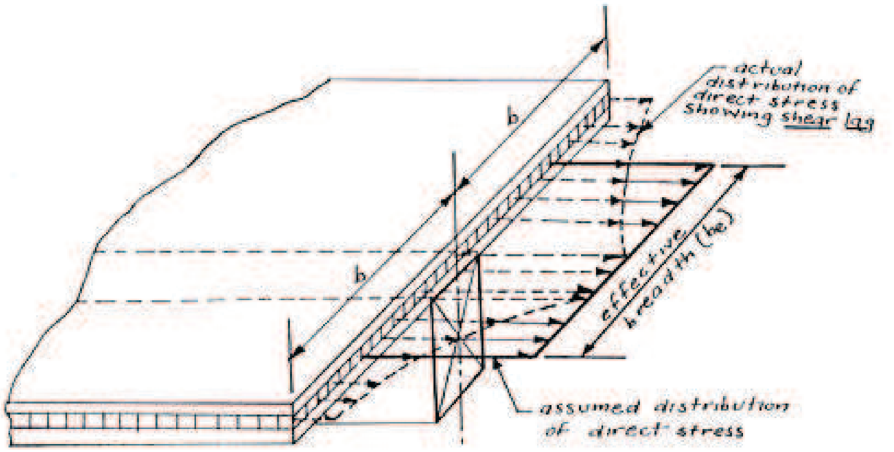
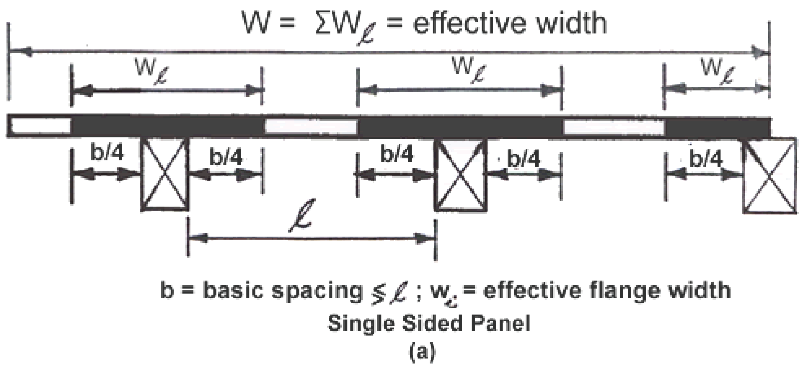


FIGURE 11.2: Distribution of flange stresses

Therefore, to apply the **simple flexure formula** to this **non-uniform stress distribution** requires using a **reduced or effective flange width (b_e)** rather than the **actual width (2b).** This **reduced width can be evaluated** if the stress distribution shown in FIGURE 11.2 by the broken lines is known. To determine b_e then becomes a matter of making the area of the **rectangle defined by the solid lines equal** to the **area of the actual stress distribution.**

North American approach is to use the **basic spacing (b)** shown in FIGURE 11.3 **as** the **design parameter.**



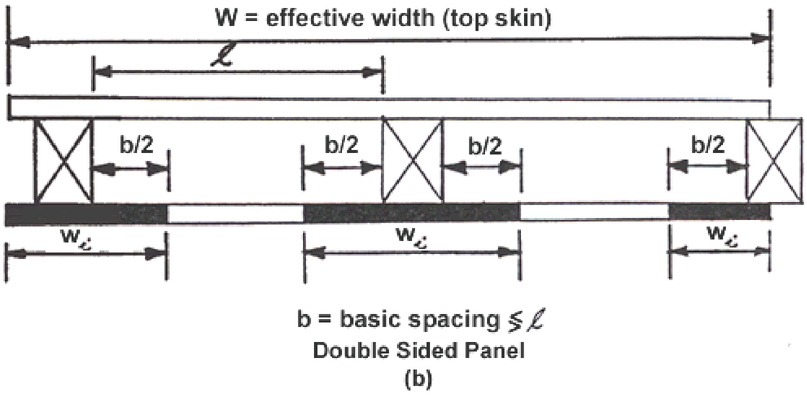


FIGURE 11.3: Effective widths of plywood

By choosing the **clear spacing (ℓ)** between stringers to be **less than (b)**, for which values have been determined for a range of **plywood thicknesses and surface smoothness's, no reductions** are necessary to compensate for:

- **shear lag;**
- **buckling** of the compressions skin;
- **dishing** of thin plywood skins between stringers, towards the panel neutral axis

Choosing a value of **(b)** equal to **45 times the plywood thickness,** and ensuring **(ℓ) less than (b)** will satisfy the above requirements for **plywood face grain parallel** to the **longitudinal members.**

If **(ℓ)** is less than (b) in either skin, then a correspondingly **reduced length of skin** as shown for the bottom skin in FIGURE 11.3, is effective in resisting the **applied bending moment.**

> **NOTE:**
> The **full length** of both skins are included in determining the **panel section properties** for **deflection calculations.**

For the case where the **face grain direction** of the plywood is **perpendicular** to the **longitudinal members** make **(b)** equal to **50 times the plywood thickness.** To determine the **effective width** of the plywood skins follow the same procedure as described for **face grain parallel to longitudinal members.**

Rolling Shear

Rolling Shear is a structural response in which **shearing forces tend to roll the wood fibres across the grain** and is of particular significance in certain plywood applications. One such instance occurs with stressed skin panels in which full surface contact of a stringer, with the face/back veneer of the plywood skins, is effected by rigid gluing of the interfaces.

FIGURE 11.4shows the location of the **critical plane** for the case of the **face grain of the plywood panel parallel to** the direction of spanning of the **stringers** which is the **generally preferred option**.

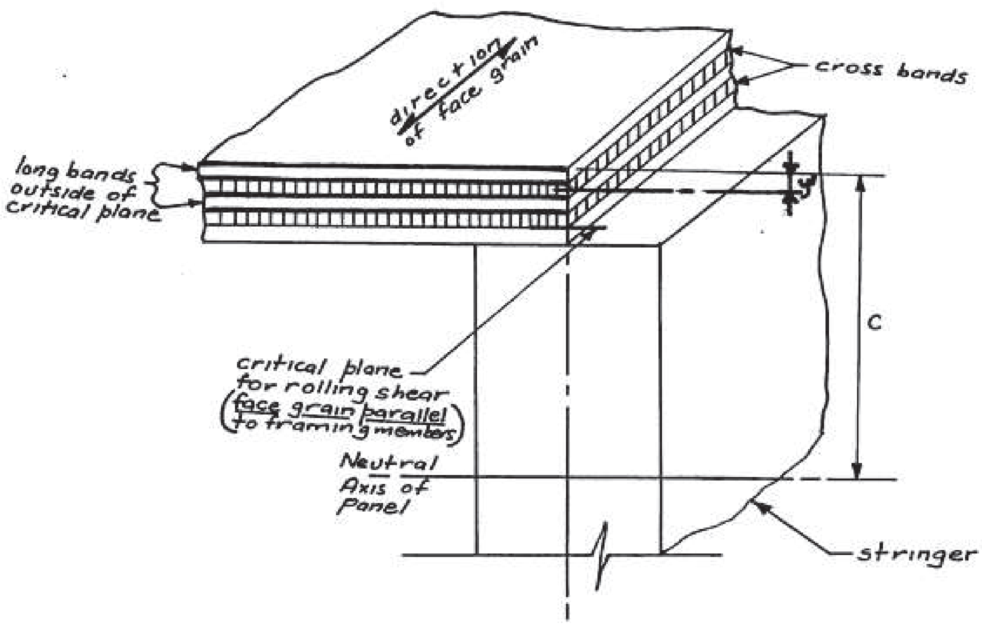


FIGURE 11.4: Position of critical plane for rolling shear

To determine the **magnitude of the rolling shear at the critical interface** requires application of the formula:

$$\tau = \frac{SQ_s}{1.b_s} \qquad \textbf{(11.1)}$$

where:

S = applied shear force in Newtons;
Q_s = first moment of the area of the parallel to stringer plies outside the critical plane as shown in FIGURE 11.4;
I = gross second moment of area of the panel in mm^4;
b_s = sum of widths of stringer glued surfaces in mm;

11.5 Panel Design – Methodology

The design method **presented in this Manual is** based on **the** approach **given** by **the** Engineered Wood Products Association of USA **(formerly APA).**

Trial Section

Choose a **trial section** based on experience or by taking a **single beam element** as a model. If the latter method is chosen keep in mind the final element will be 1200 mm wide and stiffened by top and bottom skins. Be mindful of the following **design parameters** when choosing the trial section:

- **maximum stringer spacing 600mm;**
- **minimum thickness of tension skin 7mm;**
- **basic spacing (b) between stringers should** be **equal** to **45** or **50 x plywood thickness;**
- for **effective width** of plywood **to be full width (b)** ≥ **(ℓ)**, the **clear spacing** between stringers

Transformed Section

Since the **plywood skins** and the **timber stringers** will generally be of **different species** it is necessary to reduce them to a **common basis** by computing the **transformed section.** This procedure entails:

- transforming the **actual stringer widths** to an **equivalent width of a skin** through the ratio:

$$\frac{\text{stringerMoE}}{\text{skinMoE}} \text{ x stringer width}$$

- for **skins of differing species,** performing a similar transformation to the above, on the skin **not initially** chosen.

Panel Deflection – Section Properties

To determine the relevant **panel section properties**, i.e the **neutral axis** and the **panel flexural rigidity (EI_g)** is best done using the tabular layouts shown in TABLE 11.1 and TABLE 11.2.

Element	MoE (N/mm)	A_{II} (mm^2)	$A_{II}E$ (N x 10^6)	y (mm)	$A_{II}Ey$ (N.mm x 10^6)
Top Skin Stringers Bottom Skin					
			$\sum A_{11}E=$		$\sum A_{11}Ey=$

TABLE 11.1: Layout to determine neutral axis

$$\bar{y} = \frac{\sum A_{II}Ey}{\sum A_{II}E}$$

The **EI_o values** for the **top and bottom skins** about their own neutral axes is very small compared with the other values and can therefore be disregarded without undue effect on the accuracy of (EI_g).

Item	$A_{II}E$ (N x 10^6)	I_o	EI_o (N.mm^2 x 10^6)	d (mm)	$A_{II}Ed^2$ (N.mm^2 x 10_6)	$EI_o + A_{II}Ed^2$ (N.mm^2 x 10^6)
Top Skin Stringers Bottom Skin						
						$\sum EI_g =$

TABLE 11.2: Layout to determine panel flexural rigidity

Flexural deflection can be determined from the familiar relationship:

$$\Delta b = \frac{5\,wL^4}{384\,EI_g} \qquad \textbf{(11.3)}$$

where
w = panel load in kPa
L = panel span in mm
EI_g = flexural rigidity of the panel in N-mm2

Shear deflection can be determined from the less familiar relationship for uniform or quarter-point loading:

$$\Delta_S = \frac{1.8\,PL}{AG} \qquad \textbf{(11.4)}$$

where P = total load on panel (N)

L = panel span (mm)

A = cross-sectional area of all stringers and T flanges (mm^2)

G = modulus of rigidity of stringers (N/mm^2)

Top skin deflection for plywood panels with **skins each side**, resulting in the **top skin** functioning as a **fixed ended beam** when **spanning across** stringers:

$$\Delta = \frac{4 w \ell^4}{384 E_t I} \quad (11.5)$$

where w = **panel load** in kPa

ℓ = **clear span between stringers** (mm);

I = second moment of area of a **unit width of top skin perpendicular** to the direction of spanning of the stringers;

E_t = **modulus of elasticity** for **top skin** (MPa).

Bending Stresses

To check the **bending** capacity of the panel may require:

re-evaluation of panels **section properties** if the **clear distance** (ℓ) between stringers is >**b** (see FIGURE 11.3) for either or both skins thus **requiring** a **reduction** in the **effective width of skin/s;**

for **single skin/panels**, if ℓ>**b** the **effective width** will be the **sum** of the **stringer widths plus 0.25b on each side**.

$$F_{b,a} = \frac{MyE}{EI_g} \quad (11.6)$$

where M = the **bending moment** on the panel at the section considered;

y = the **distance from the neutral axis** to the fibre under consideration;

E = **MoE** of the **element being considered**;

EI_g = the **flexural rigidity of the panel**.

Where such information is available, and if it is applicable, the necessary **increase in maximum stress** in the **stringers and plywood skins** should be made to account for **shear lag.**

FIGURE 11.5 shows a **typical stress distribution** for the plywood skins and stringers of a stressed skin panel.

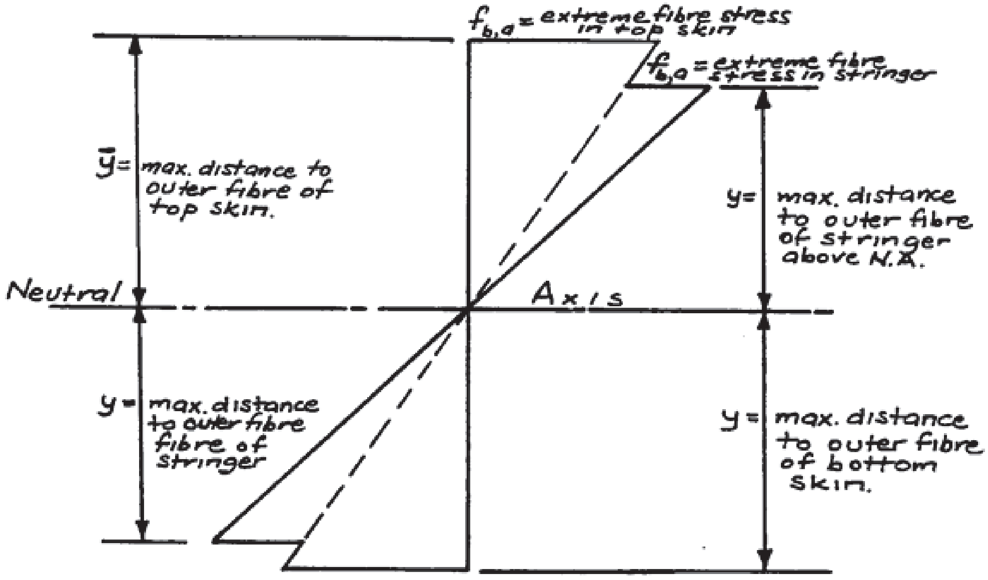


FIGURE 11.5: Bending stresses in stressed skin panel

The values of $\bar{y}$ and the y's shown in FIGURE 11.5, when substituted in Equation 11.6 for y, will on solution result in the evaluation of **extreme fibre stresses $f_{b,a}$.**

Splice Plate Check

From Equation 11.6 :

$$f_b = \frac{MyE}{EI_g} \text{ for the full panel width;}$$

$$M = \frac{wL^2}{8} \text{ for maximum moment under u.d. loading;}$$

$$F_{sp} = \text{splice force}$$

$$= f_b \left[\frac{W_{sp}}{W_p}\right] A_{sp}$$

where:

$$\frac{W_{sp}}{W_p} = \text{total width of splice plate / total panel width}$$

$$\frac{F_{sp}}{A_{sp}} = f_{sp} = \left[\frac{wl^2}{8} x \frac{E_{sp}}{EI_g} x\, y\, x \frac{W_{sp}}{W_p} x\, 10^3\right] MPa \quad (11.7)$$

where f_{sp} = splice stress
w = uniformly distributed load (kN/m)
E_{sp} = modulus of elasticity of splice material
EI_g = stiffness factor (N-mm2) from TABLE 11.2
y = distance from neutral axis to the extreme tension or compression fibre (mm)

NOTE:
The above **f_{sp}** is for the splice plate at the point of **maximum moment.** If this controls the design the splice can be **relocated** in a new area of lower moment.

Rolling Shear Stress

From FIGURE 11.4:

d_s = c-y'
Q_s = A.d_s

where:

A = area of plywood **veneers parallel to the stringers** and outside the critical zone.

The **rolling shear stress** will be, from **Equation 11.1:**

$$\tau_r = \frac{SQ_s}{Ib}$$

Horizontal Shear Stress

Q_H will **not be** the **same** as Q_s because it will be the **first moment** of **all veneers parallel** to the **stringers above** (or below) the **neutral axis.**

To account for **differences** in **modulus of elasticity** a **transformed section** is required thus:

$$Q_H = Q_{stringer} + \frac{E_{skin}}{E_{stringers}} x Q_{skin} \tag{11.8}$$

Hence

$$\tau_H = \frac{S.Q_H.E_{ST}}{(EI_g)b} \tag{11.9}$$

where

S = total shear force
Q_H = as defined
E_{ST} = modulus of elasticity of stringers (N/mm2)
EI_g = stiffness factor (N-mm2)
b = total width of stringers intersected by the neutral axis (mm).

11.6 Design Example – Stressed Skin Panels

Design a floor panel to span 5m for the following unfactored loading and deflection requirements:

Uniformly distributed live load	= 2kPa
Uniformly distributed dead load	= 0.5kPa
Deflection limitation for live load	= span/360
Deflection limitation for dead and live load	= span/240

Stressed Skin Panel – Worked Example

The solution will follow the **Design Methodology** previously discussed in Section 11.5 of this chapter.

Trial Section

Assume as a trial section the panel having the material specifications and dimensions shown in FIGURE 11.6.

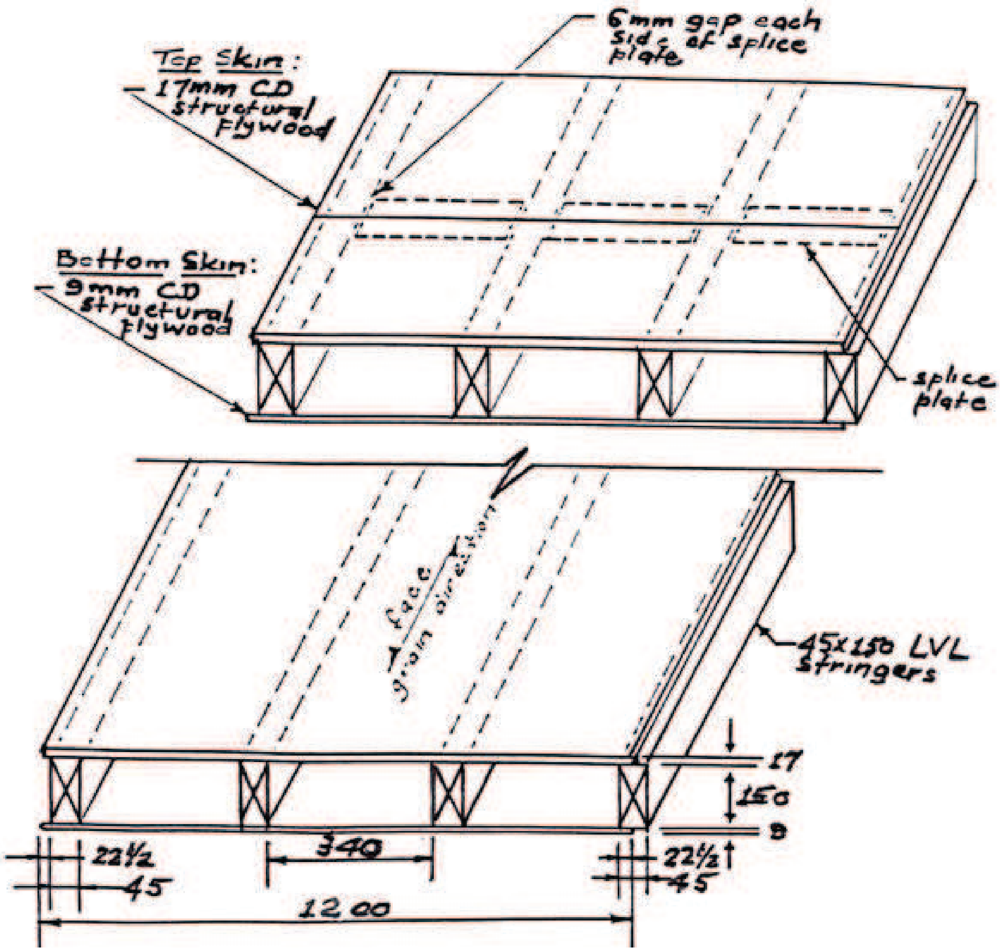


FIGURE 11.6: Stressed skin panel trial section

Design data for the **structural components is:**

F11 x 17 – 24 – 7 plywood	I_{II}	= 285mm^4/mm;
	$I_{\perp}$	= 120mm^4/mm;
	A_{II}	= (4x2.4x10^{-3}x1)
		= 0.0096 m^2/m or 9600mm^2/m
	E	= 10,500MPa
F11 x 9 – 30 – 3 plywood	I_{II}	= 60mm^4/mm;
	$I_{\perp}$	= 4mm^4/mm;
	A_{II}	= (2x3x10^{-3}x1)
		= 0.006 m^2/m or 6000mm^2/m
	E	= 10,500MPa
Laminated Veneer Lumber -	45 x 150mm;	
	I	= bd^3/12 = 12.66 x 10^{6}mm^4;
	E	= 13,200MPa
	A	= 6750mm^2
	G	= 880MPa

Basic Spacing

Clear distance between stringers (ℓ)	= (1200 – (2x22.5)-(3x45)) / 3
ℓ	= 340mm
Total splice plate width (S_w)	= 3(340 – 12)
S_w	= 984mm
For 17mm thick plywood (b)	= (17 x 45)mm
b	**= 765mm >ℓ = 340mm**

For 9mm thick plywood (b) = (9 x 45)mm

b = 405mm >ℓ = 340mm

Deflection - Section Properties

Element	MoE	A_{II}	$A_{II}E$	Y	$A_{II}Ey$
Top Skin	10500	1.2 x 9600 = 11520	121×10^6	167.5	20.3×10^9
Stringer	13200	4 x 6750 x 1.26 = 34020	449×10^6	84	37.7×10^9
Bottom Skin	10500	1.2 x 6000 = 7200	75.6×10^6	4.5	0.34×10^9
		Σ	645.6×10^6	Σ	58.34×10^9

TABLE 11.3: Gives procedure for determining section centroid

$$\bar{y} = \frac{\Sigma A_D x E_{xy}}{\Sigma A_D x E} = \frac{58.34 \times 10^9}{645.6 \times 10^6}$$

$$\bar{y} = 90.4\text{mm}$$

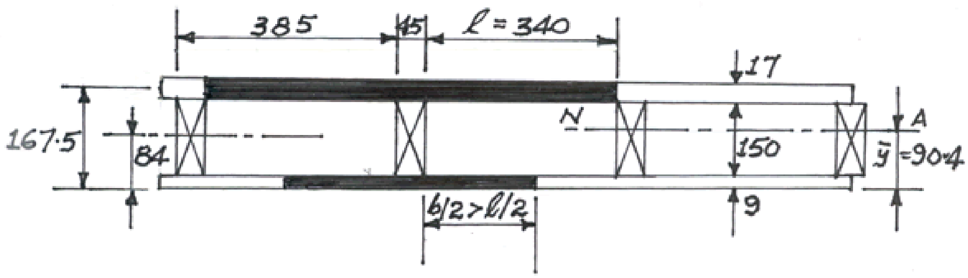


FIGURE 11.7: Shows the neutral axis and "b" relative to "ℓ"

Element	$A_{II}E$	I_o	EI_o	d	$A_{II}Ed^2$	$EI_o + A_{II}Ed^2$
Top Skin	121×10^6	285 x 1200 = 0.34×10^6	3.57×10^9	77.1	72.5×10^{10}	72.9×10^{10}
Stringer	449×10^6	12.66×10^6 = 50.6×10^6	667.9×10^9	6.4	1.84×10^{10}	68.6×10^{10}
Bottom Skin	75.6×10^6	60 x 1200 = 0.07×10^6	0.74×10^9	85.9	55.8×10^{10}	55.9×10^{10}
					ΣEI_g	197.4×10^{10}

TABLE 11.4: Gives procedure for determining (EI_g)

Hence EI_g is:

$$EI_g = 197.4 \times 10^{10} \text{N-mm}^2/1200 \text{ width}$$

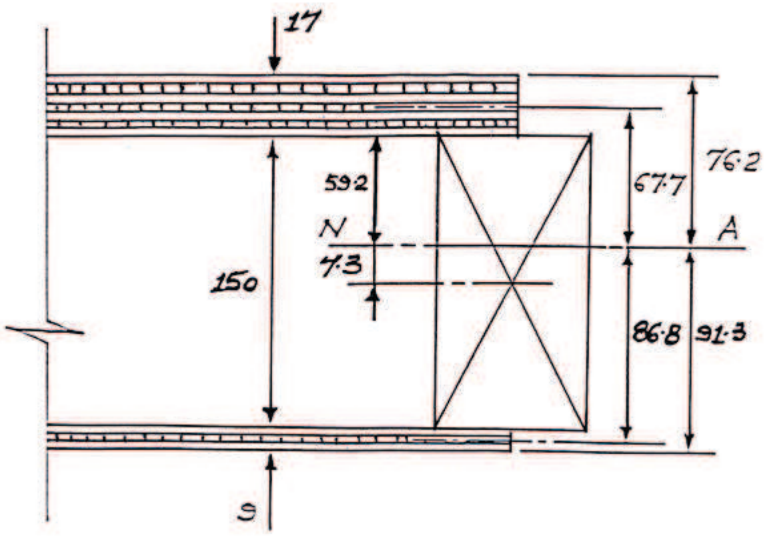


FIGURE 11.8: Shows relevant panel cross-section dimensions

Flexural Deflection Long Term Serviceability Requirements

Flexural Deflection:

$G + \Psi_1 Q = G + 0.4Q$

$G + 0.4Q = (0.5 + 0.4 \times 2)\text{kPa}$

$\mathbf{G + 0.4Q = 1.3kPa}$

$$\Delta_b = \frac{J_2\, 5\, wL^4}{384\, EIg}, \text{where } J_2 = 2$$

$$= \frac{2 \times 5 \times 1.3 \times 1.2 \times (5)^4 \times 10^{12}}{384 \times 197.4 \times 10^{10}}$$

$$\Delta_b = 12.9\,\text{mm}$$

Shear Deflection:

$$\Delta_s = \frac{J_2\, 1.8\, PL}{AG}$$

$$= \frac{21.8 \times 1.3 \times 1.2 \times 5 \times 5 \times 10^6}{4 \times 6750 \times 880}$$

$$\Delta_s = 6\,\text{mm}$$

$\mathbf{\Delta_b + \Delta_s = 12.9 + 6 = 18.9mm}$

$\mathbf{\Delta_b + \Delta_s}$ **= 18.9mm < 20.8mm, i.e. span/240 so OK**

Top Skin Deflection:

For two-sided panels the skin will function as a **fixed-ended beam** for which the equation is:

$$\Delta_{ts} = \frac{J_2 wL^4}{384\, EI}$$

$$= \frac{2 \times 0.31 \times (340)^4 \times 1}{384 \times 10500 \times 120 \times 240}$$

$$\Delta_{ts} = 0.28\,\text{mm} < 1.4\,\text{mm, i.e. span}/240$$

where:

L = clear span between stringers (mm);
E = top skin modulus of elasticity (MPa)
I = $I_\perp$ for top skin of width 240mm;
w = load in kN/m

Bending Moment

Member Design Capacity: Strength Limit State

$$\phi M = \phi \times k_1 \times k_4 \times k_6 \times k_9 \times k_{11} \times k_{12}\, (f'_b \times Z)$$

Note:
Because ℓ<b, i.e. 405 and 765 >340, for both the top and bottom skins a **re-evaluation of** the panel **section properties** is **not required**.

For **long term**, i.e. **5 month loading**, for dead and live load:

k_1 = 0.8 ; f'_b = 35MPa for F11
k_4 = 1.0 ; k_6 = 1.0
k_{11} = 1.0 ; k_{12} = 1.0
ϕ = 0.8
ϕM = [0.9 x 0.8 x 1.0 x 1.0 x 1.0 x 1.0 (35 x Z)]N-m

Determination of Z is most conveniently done through TABLE 11.5 :

Element	I_o	A_{II}	d^2	$I_o + A_{II}d^2$
Top skin	0.34×10^6	11520	77.1^2	68.5×10^6
Stringer	50.6×10^6	34020	6.4^2	52.0×10^6
Bottom skin	0.07×10^6	7200	85.9^2	53.2×10^6
			$\Sigma I_g =$	173.7×10^6

TABLE 11.5: Layout to determine the gross second moment of area

Hence I_g is:

$$I_g = 173.7 \times 10^6 \text{mm}^4 \text{ for 1200 wide panel}$$

For **the top skin**:

$$Z_t = \frac{I}{y} = \frac{I_g}{76.2} = \frac{173.7 \times 10^6}{85.6}$$

$$Z_t = 2.03 \times 10^6 \text{mm}^3$$

$$\phi M = (0.72 \times 35 \times 2.03 \times 10^6)\text{N-mm}$$

$$\boldsymbol{\phi M = 51.1 \text{ kN-m}}$$

For **bottom skin:**

$$Z_b = \frac{I_g}{91.3} = \frac{173.7 \times 10^6}{91.3}$$

$$Z_b = 2.02 \times 10^6 \text{mm}^3$$

$$\phi M = (0.72 \times 35 \times 2.02 \times 10^6)\text{N-mm}$$

$$\boldsymbol{\phi M = 50.9 \text{ kN-m}}$$

Design Action Effect

TABLE 11.6 gives the relevant loading combinations and the associated duration of load parameter D_L which shows the critical load case.

Load Combinations	Calculation	Load Effect (kN/m)	k_1	$M^+ = \frac{wL^2}{8}$ (kNm)	$D_L = \frac{M^*}{k_1}$
Permanent $1.25G + \Psi_c Q$	(1.25 x 0.5) + (0.4 x 2) = 1.43kPa	1.43 x 1.2 -= 1.72	0.57	5.38	9.43
Long term 1.25G + 1.5Q	(1.25 x 0.5) + (1.25 x 2) = 3.63kPa	3.63 x 1.2 -= 4.36	0.8	13.63	17

TABLE 11.6: Design action effect

For the **strength limit state**:

D_L from TABLE 11.6 shows the **worst loading case** to result in a **moment of 13.63kNm** which is **much less** than **30kNm moment capacity** for the bottom skin.

Splice Plate Check

Tension Splice

The relationship for a splice plate stress check given in Section 11.5 of Design Methodology is:

$$f_{st} = \left[\frac{wL^2}{8} x \frac{Es}{(EIg)} x\, y\, x \frac{Ws}{Wp} x 10^6\right] \text{MPa}$$

$$= \left[\frac{2 \times 1.2 \times 5^2}{8} x \frac{10500}{197.4 \times 10^{10}} x\, 90.4\, x \frac{984}{1200} x 10^6\right]$$

$$f_{st} = 2.95 \text{MPa}$$

If the **splice plate** was **17mm F11 structural plywood** with its **face grain parallel to** the **direction of spanning** then:

f'_t = 20MPa which, without further consideration would therefore obviously be satisfactory.

Compression Splice
Using 17mm F11 structural plywood the compression splice **will be satisfactory by inspection**, i.e. because of the **smaller y**.

Rolling Shear

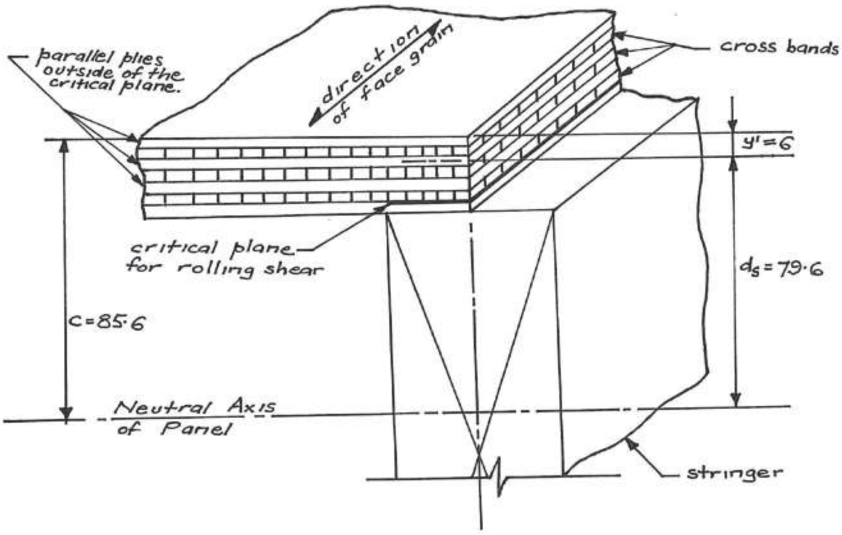


FIGURE 11.9: Shows dimensions for obtaining Q_s

$$\tau_r = \frac{S x Q s}{I x b}$$

$$= \left[\left(\frac{w x L x B}{2} \right) x\ Qs\ x\ \frac{E}{(EI_g)}\ x\ \frac{1}{b} x 10^3 \right] \text{MPa}$$

The statical moment Qs is obtained thus:

Q_s = A_{II} x d_s
= 3(2.4 x 1200)79.6
Q_s = 6877448mm^3

From FIGURE 11.6:

b_s = 3 x stringer width + 0.5 x stringer width
= (3 x 45) + (0.5 x 45)
b_s = 157.5mm

Writing (τ_r x b_{sf}) = q, a shear flow,
where:
b_{sf} = 45 + (0.5 x 45)
= **67½mm for edge stringers**
b_{sf} = **(2 x 45)**
= 90mm for internal stringers.

The **strength limit states values for rolling shear (τ_r)** can be obtained from the relationship:

τ_r = ϕ x k_1 x k_{19} x g_{19} x f'_s

For **edge stringers:**

τ_{re} = (0.8 x 0.8 x 1.0 x 0.2 x 5.3)N/mm^2
τ_{re} = 0.68MPa

For **internal stringers:**

τ_{ri} = (0.8 x 0.8 x 1.0 x 0.4 x 5.3)N/mm^2
τ_{ri} = 1.36MPa

Shear Flow:

$$q = (0.66 \times 6.75) + (1.36 \times 90)$$
$$\mathbf{q = 168.2N/mm}$$

Rewriting the rolling shear equation in terms of w (kN/m):

$$1.2w = \left[\frac{q \times 2 \times (EI_g) \times 1}{L \times E \times Q_s}\right]$$

$$1.2w = \left[\frac{168.2 \times (197.4 \times 10^{10}) \times 1}{1 \times 5 \times 10500 \times 687744} \times \frac{1}{10^3}\right] kN/m$$

$$1.2w = 9.2 kN/m$$

or

$$\mathbf{w = 7.7kPa, >2kPa, \therefore O.K.}$$

Horizontal Shear

From Equations 11.8 and 11.9 and FIGURE 11.8:

$$Q_H = Q_{stringer} + \frac{E_{skin}}{E_{stringer}} \times Q_{skin}$$

$$= 4(45 \times 68.6 \times \frac{68.6}{2}) + (\frac{10500}{13200} \times 1.2 \times 9600 \times 77.1)$$

$$Q_H = 1130053mm^3$$

$$\tau_H = \frac{S \times Q_H \times E_{ST}}{(EI_g)b}$$

$$= \left(\frac{10.9 \times 10^3 \times 1130053 \times 13200}{197.4 \times 10^{10} \times 4 \times 45}\right) MPa$$

$$\mathbf{\tau_H = 0.46MPa < 1.7MPa. \ O.K.}$$

DISCUSSION

The stressed skin panel with the stringer and sheathing dimensions and properties chosen easily satisfies all of the strength criteria.

However, with a floor panel, satisfying deflection (stiffness) criteria is of equal importance if a habitable floor is to result. A check on panel stiffness (k) obtained by evaluating the relationship $48(EI_g)/L^3$ shows k = 0.7kN/mm, which in a normal bearer/joist floor system, would be more than adequate to ensure a sufficiently vibration insensitive floor.

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7. **Theoretical & Experimental Studies on Nailed & Glued Plywood Stressed Skin Components: Part I. Theoretical Study**, E.J. Amana & L.G. Booth, Institute of Wood Science Journal, Vol. 4(1), 1967.

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Part Four

Exotic Structures & Connection Design

Exotic Structural Forms

Connection Design – Plywood and LVL

12 Exotic Structural Forms

12.1 Introduction

The purpose of this chapter is to **demonstrate** the **flexibility of plywood and LVL as a building** construction **medium**. It provides **basic design information** to allow the designer the opportunity to investigate the **feasibility** of the **chosen structural form** as a viable solution at the **preliminary design stage.**

If this **preliminary investigation** proves the structural form to be a viable design solution a **rigorous analysis** may be required. The availability of sophisticated **finite element computer programs** will facilitate this need.

An appealing feature of **plywood** and **LVL** when used as constructional materials is their **ability to be easily worked** into a **multiplicity of simple or complex shapes**. By taking advantage of this ease of working and inherent manoeuvrability it is possible to **produce highly efficient** and **aesthetically pleasing structural systems capable** of **spanning large column free spaces.**

Many exotic timber structures have been designed and built throughout the world, in particular, in North America and the United Kingdom. The **Tacoma Dome,** completed in 1983, and having a **clear spanning diameter of 162 m**, is worthy of mention.

The **structural forms** considered in this chapter are:

- **folded plates**;
- **arches**;
- **hyperbolic paraboloids (hypars)**;
- **domes**

It is hoped by including these more exotic structural forms in the Manual will provide **architects and designers** with the incentive to expand their **creative skills** beyond the pedestrian into the exciting.

12.2 Folded Plates

Introduction

Folded plate plywood and LVL structural systems, using **stressed skin** construction, offer the designer a wide range of aesthetically pleasing solutions.

FIGURE 12.1 illustrates some interesting structural forms capable of being produced through the interconnection of folded plates.

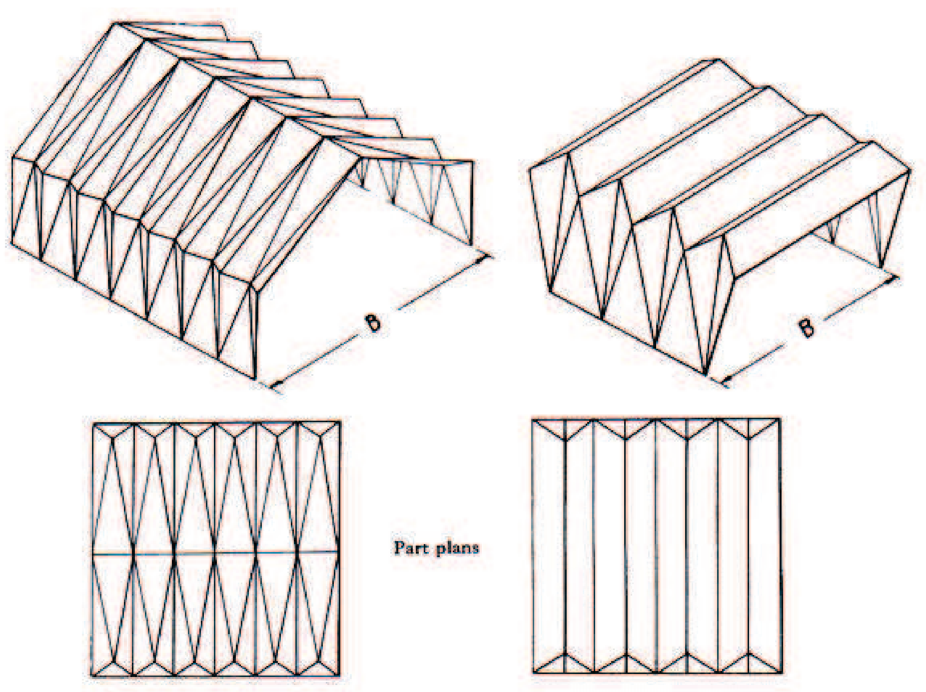


FIGURE 12.1: Various types folded plate structures

12.3 Folded Plate Design - Structural Action

A flat sheet of paper placed over supports **cannot sustain its own weight** and will collapse.

However, by placing **folds** in the flat sheet of paper, as shown in FIGURE 12.2, dramatically **increases its flexural stiffness** and hence its spanning capability.

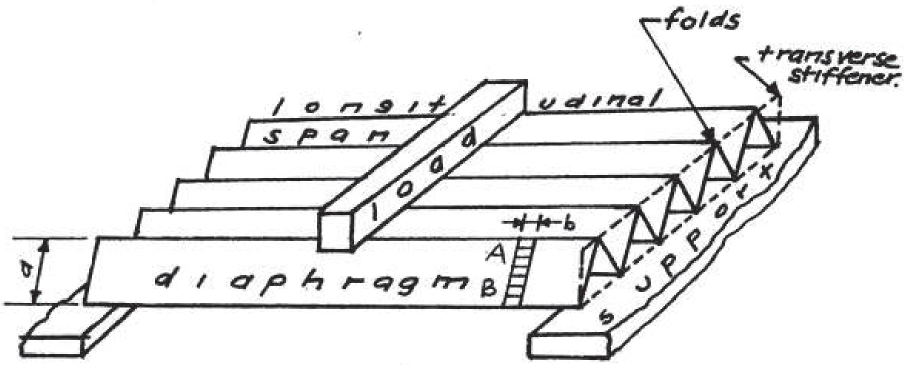


FIGURE 12.2: Sheet of paper with folds supporting a load

The load carrying capacity of the folded plate will be further enhanced by fixing **transverse stiffeners** along the ends as shown dotted in FIGURE 12.2.

Transverse action is a consequence of loads being applied **normal** to areas defined by **AB** on the **diaphragm surface.** These loads cause **one way bending** along the width AB.

Longitudinal action results in the **in-plane components** of load being **transferred to the folds** and then via **beam action** to the supports.

12.4 Folded Plate Design - Methodology

The two actions mentioned above, i.e. **transverse and longitudinal** will now be considered in some detail. **Transverse action** due to **uniform vertical loading** resulting in components acting **perpendicular** to, and **in the plane** of the diaphragm will result in **each diaphragm deforming identically** if extreme edges are fully constrained. Because there is **no relative**

displacement between diaphragms each strip of **width (b)** will behave as a **fixed ended beam** under the normal component of load as shown in FIGURE 12.3.

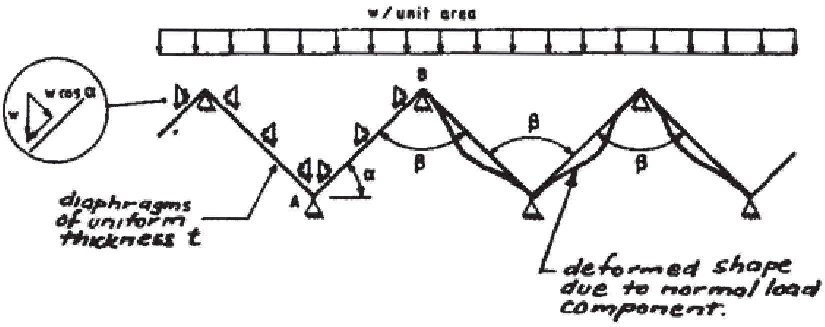


FIGURE 12.3: Load components on a transverse strip

For diaphragms arranged symmetrically the **folds will not rotate** and angle β will be maintained due to continuity. This will result in the **moments at the folds** being equal to the **fixed-end moments** for a beam of **unit width (b = 1) and length (a).** Isolating unit width of the **diaphragm AB** as shown in FIGURE 12.4 results in:

$$\mathbf{M_A} = -\frac{\mathbf{(w\cos\propto)\,a^2}}{\mathbf{12}}$$

$$\mathbf{M_B} = \frac{\mathbf{(w\cos\propto)\,a^2}}{\mathbf{12}}$$

The **mid-span moment** will be:

$$\mathbf{M_{ms}} = \frac{\mathbf{(w\cos\propto)\,a^2}}{\mathbf{24}}$$

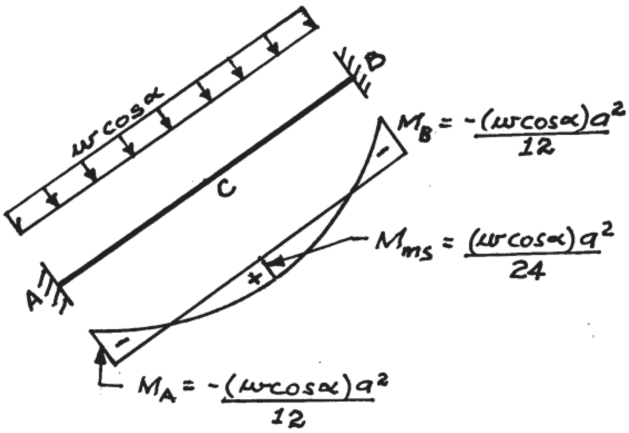


FIGURE 12.4: Isolated transverse element under load

The corresponding **stresses at A, B** or **C** in a homogenous diaphragm having a **section modulus:**

$$\mathbf{Z = 1.\,t^2/\,6 \text{ will be :}}$$

$$\sigma_{A/B} = \pm\frac{M_{A/B}}{Z}$$

$$= \pm\frac{(w\cos\alpha)\,a^2}{12(t^2/\,6)}$$

$$\sigma_{A/B} = \pm\frac{w\cos\alpha}{2}\left(\frac{a}{t}\right)^2 \qquad (12.1)$$

$$\sigma_c = \frac{w\cos\alpha}{4}\left(\frac{a}{t}\right)^2$$

Longitudinal action results from the **bending action** of the **diaphragms** transferring **reactions R** due to the **normal component** $p_n = w \times a \times \cos \propto$ to the **folds.** Simultaneously, the **tangential component** $p_t = w \times a \times \sin\propto$ is transferred to the **folds** by direct stress **along the diaphragm** as shown in FIGURE 12.5 (a).

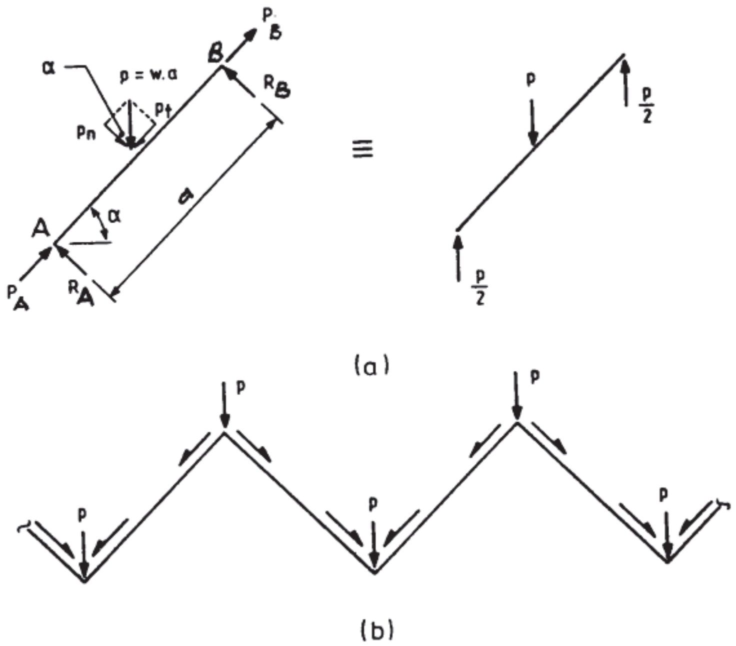


FIGURE 12.5: Determination of longitudinal load on the system

The **total load p** divides into two components at the folds, these components being **in the plane of the diaphragms** as shown in FIGURE 12.5(b). These loads are then transferred, by **longitudinal beam action of each diaphragm** to the end supports.

Hence, each **sloping diaphragm** of FIGURE 12.5 (b) **spans longitudinally as** a **beam** of:

- **length L;**
- **depth h;**
- **width b = t/sin∝**

FIGURE 12.6 shows such a **sloping section.**

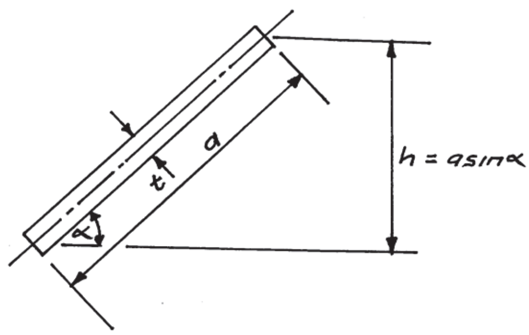


FIGURE 12.6: Inclined diaphragm

The second moment of area of the sloping section is given by:

$$I = \frac{bh^3}{12}$$

$$= \frac{1}{12}\left(\frac{t}{\sin\propto}\right)(a\sin\propto)^3$$

$$I = \frac{1}{12}.ta^3.\sin^2\propto \qquad (12.2)$$

The section modulus (Z) is:

$$Z = \frac{I}{y}$$

$$= \frac{1}{12}\left(\frac{t}{\sin\propto}\right)(a\sin\propto)^3 . \frac{1}{h/2}$$

$$Z = \frac{1}{6}.ta^2.\sin\propto \qquad (12.3)$$

For a **uniform load p = (wa) kN/m**, the **maximum bending stress** in an isotropic diaphragm is given by:

$$\sigma_{max} = \frac{pL^2/8}{Z}$$

$$= \frac{p.L^2/8}{t.a^2\sin\propto/6}$$

$$= \frac{6.w.a.L^2}{8.t.a^2.\sin\propto} \qquad (12.4)$$

$$\sigma_{max} = \frac{3}{4}.\frac{wL^2}{th}$$

12.5 Arches

Introduction

The **arch** and the **portal frame** are closely related and as such the arch can be **rigid, two** or **three** hinged as shown in FIGURE 12.7 (a), (b) and (c). FIGURE 12.7 (d), (e), (f) show some variations of the portal frame.

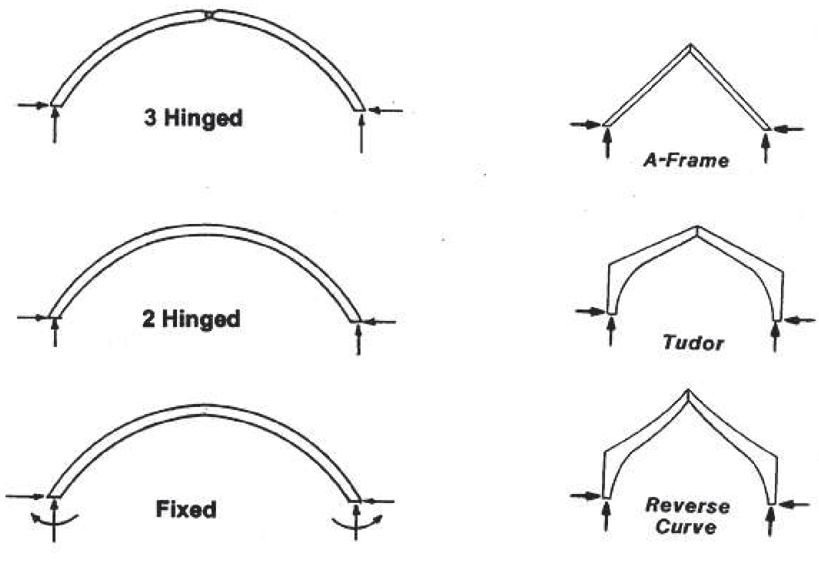
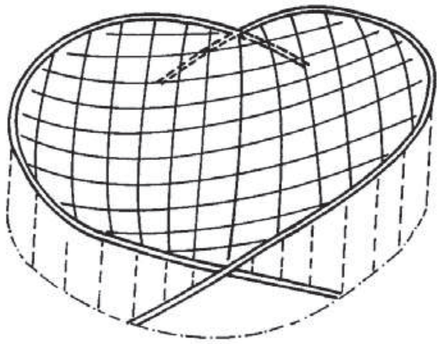


FIGURE 12.7: Basic arches and some portal derivatives

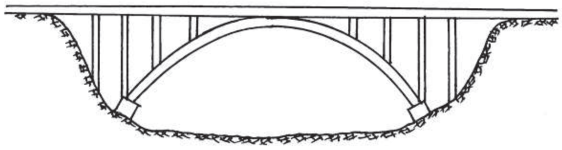
The arch provides a **very versatile** structural form fulfilling many structural roles in both **two and three dimensional** configurations, e.g. as a:

- **two dimensional** idealisation of the **singly curved cylindrical shell** or **barrel vault;**
- **two dimensional** idealisation of the **doubly curved shell** or **dome;**
- **two dimensional** idealisation of **saddle shells (hypars)** in **one direction;**
- a support for **roofs** of structures;
- a support for **bridge decks** and in **dam walls**.

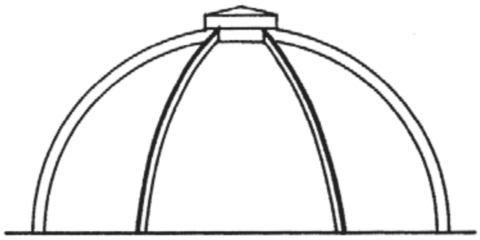
FIGURE 12.8 shows examples of arches being utilised in a range of construction situations.



Arches to suspend a roof
(a)



Arches supporting a bridge deck
(b)



Converging arches
(c)

FIGURE 12.8: Uses of arches

12.6 Arch Design - Arch Action

The arch can assume a range of **geometric shapes.** However, for various reasons it is usual for the designer to choose one of the following forms, i.e.:

- **parabola**;
- arc of a **circle**;
- **ellipse**

A **parabolic arch**, **uniformly loaded** along its length will result in its **cross-section** being subjected to **uniaxial compression only** (no bending or shear) at all sections along its length. This is because the **thrust line** follows the **parabolic profile** of cross-section **centroids**.

Because of the **reduced** influence of **bending** the **structural efficiency** of the **arch exceeds** that of the **beam** for certain load cases.

Should the arch profile **not conform** to a **parabola bending action** will still be **much less than** that of an **equivalent beam**. However, this increased structural efficiency does not come without cost, i.e. large **thrusts** are developed at the **supports**. These can be accommodated by **buttresses** or a **tie** between the supports.

The **three hinged arch** offers certain advantages both **analytically** and **structurally**. The three hinges render the structure **statically determinate** simplifying any preliminary design calculations. The three hinges also provide the **structural advantage** of being highly tolerant to any **support settlement.**

12.7 Arch Design - Methodology

To be able to determine the **internal forces** at an **interior point** of an arch, other than at the hinge, requires the **arch geometry** to be specified.

In the case of the **parabolic arch** shown in FIGURE 12.9 the profile is defined by:

$$y = h\left[1 - \left(\frac{x}{L}\right)^2\right]$$

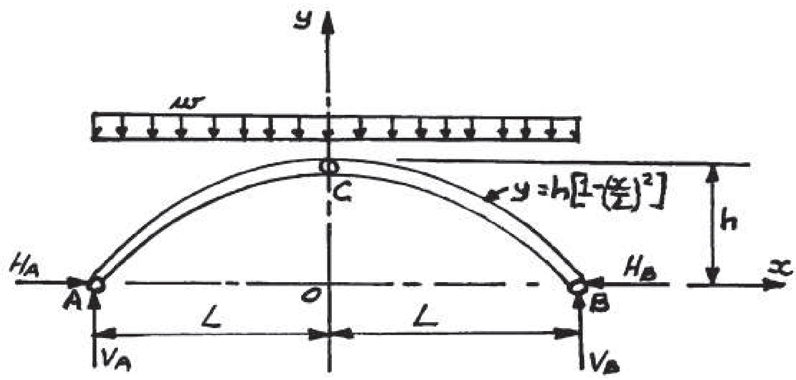


FIGURE 12.9: A parabolic arch (not to scale)

The support reactions can be determined through application of the **equilibrium equations:**

$$\Sigma F_y = 0 : V_A + V_B = 2wL$$
$$\Sigma F_x = 0 : H_A = H_B$$
$$\Sigma M_{CR} = 0 : H_B \text{ x } h - V_B \text{ x } L + \frac{wL^2}{2} = 0$$
$$\Sigma M_{CL} = 0 : V_A \text{ x} L - H_A \text{ x} h - \frac{wL^2}{2} = 0$$

(12.5)

Had the **supports** been at **different levels** the procedure would still be the same except two values of (h) would be required.

12.8 Arches – Design Example

FIGURE 12.10 shows a **three hinged parabolic arch** for which w = 10kN/m, L = 30m and h = 10m.

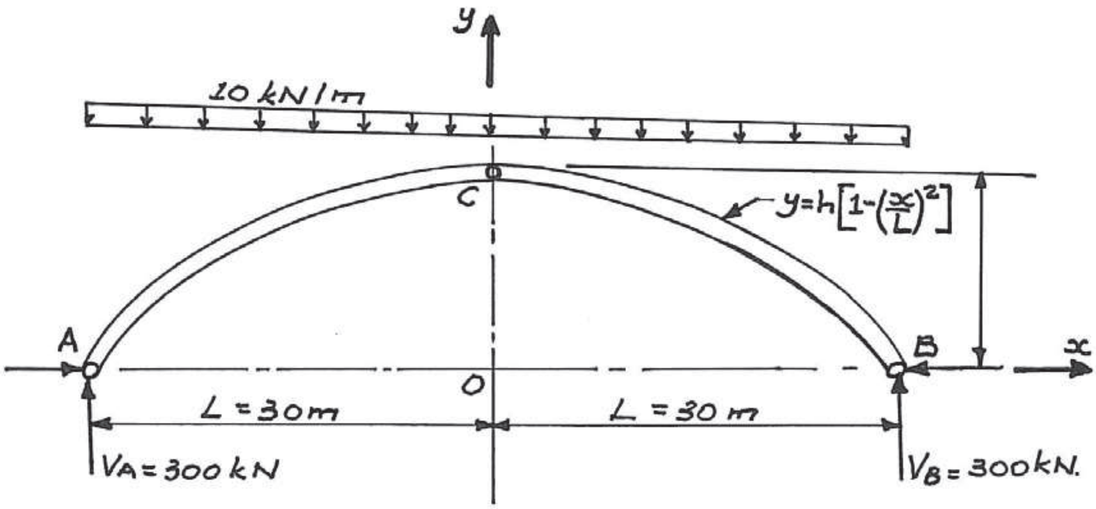


FIGURE 12.10: Symmetrical parabolic arch symmetrically loaded

12.9 Arches - Worked Example

The main objective of this worked example is to show the arch is subjected to **zero bending and shear forces** when subjected to **uniform loading.**

The **axial force** is directed along the **tangents** to the arch profile whilst the **shear force** is **perpendicular** to the **centroidal axis** of the arch. Hence, to find the components of H_B and V_B in these directions requires determining the **slope** of the arch **at the base.** From:

$$\mathbf{y} = \mathbf{h}\left[\mathbf{1}-\left(\frac{\mathbf{x}}{\mathbf{L}}\right)^{\mathbf{2}}\right]$$

$$\frac{\mathbf{dy}}{\mathbf{dx}} = -\mathbf{2}\left(\frac{\mathbf{x}}{\mathbf{L^2}}\right)\mathbf{h} \qquad \textbf{(12.6)}$$

Arch reactions are determined by application of the **equilibrium equations** of Equations 12.5. From symmetry:

$$\mathbf{V_A} = \mathbf{V_B} = \mathbf{wL}$$

$$\mathbf{V_A} = \mathbf{V_B} = \mathbf{300\,kN}$$

$$\text{From: } \sum M_{CR} = 0$$

$$0 = \mathbf{10H_B \quad 300\,x\,30 + \frac{10\,x\,30^2}{2}}$$

$$\mathbf{H_B} = \mathbf{\frac{9000 \quad 4500}{10}\ kN}$$

Hence:

$$\mathbf{H_B} = \mathbf{450\,kN}$$

$$\boldsymbol{H_A} = \boldsymbol{450kN}$$

From Equation 12.6 :

$$\frac{\mathbf{dy}}{\mathbf{dx}} = -\mathbf{2}\left(\frac{\mathbf{x}}{\mathbf{L^2}}\right)\mathbf{h}$$

when:

$$x = L = 30 \text{ gives:}$$

$$\frac{\mathbf{dy}}{\mathbf{dx}} = -\mathbf{2}\left(\frac{\mathbf{30}}{\mathbf{30^2}}\right)\mathbf{10}$$

$$\frac{\mathbf{dy}}{\mathbf{dx}} = -\frac{\mathbf{2}}{\mathbf{3}}$$

FIGURE 12.11 shows the **normal (n)** and **tangential (t) co-ordinates** at the base of the arch.

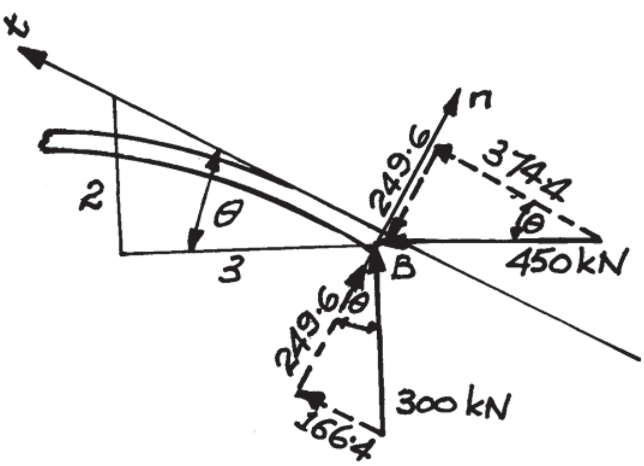


FIGURE 12.11: Components of shear and axial force

Summing the **axial (F_A) and shear (F_s)** components at **B** in FIGURE 12.11 gives:

F_A = (374.4 + 166.4)kN
F_A = 540.8kN (compression)
F_S = (+ 249.6 – 249.6)kN
F_S = 0 kN

Choosing a point mid-way between C and B on the arch as shown in FIGURE 12.12 (a) results in the **slope** being:

$$\frac{dy}{dx} = 2\,\frac{x}{L^2}\,h$$

$$= \frac{2 \times 15 \times 10}{30^2}$$

$$\frac{dy}{dx} = 1/3$$

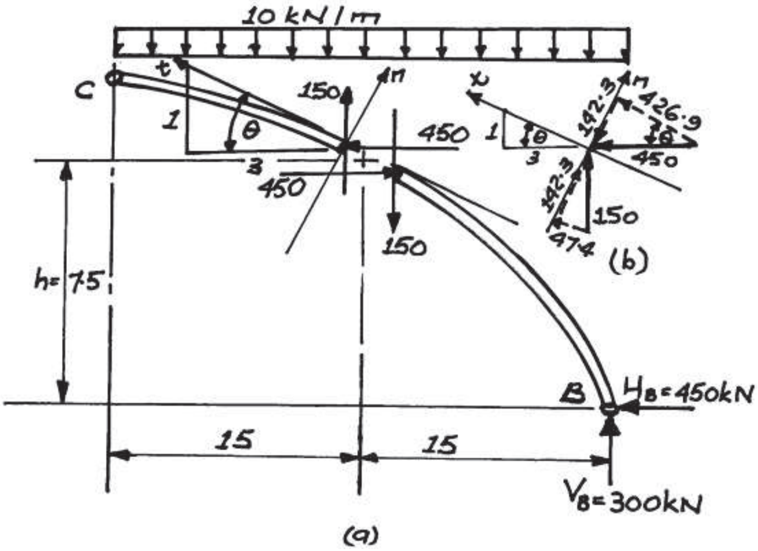


FIGURE 12.12: Exposed cross-section at mid-length and axial and shear force components

From the equilibrium relationships the **vertical (F_{VMS})** and **horizontal (F_{HMS}) forces** on the cross-sections of the **free body diagrams** are from:

$\sum F$ = 0
= 300 – F_{VMS} – 10x15
F_{VMS} = 150kN
$\sum F_{HR}$ = 0
= -450 + F_{HMS}
F_{HMS} = 450kN

Resolving **F_{vms}** and **F_{HMS}** in the **(t) and (n) directions** results in the **axial (F_A) and shear (F_s)** components being:

$$\mathbf{F_A = (426.9 + 47.4)\,kN}$$
$$\mathbf{F_A = 474.3\,kN(compression)}$$
$$\mathbf{F_S = (142.3 - 142.3)\,kN}$$
$$\mathbf{F_S = 0kN}$$

To find the **moment** at **any cross-section x** from the arch centre, as shown in the free body diagram in FIGURE 12.13.

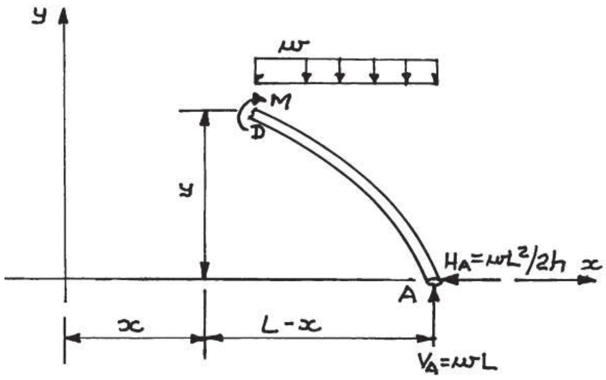


FIGURE 12.13: Free body diagram of part of arch

Taking moments about D:

$$
\begin{aligned}
\Sigma M_D &= 0 \\
&= \frac{wL^2}{2h} x\,y + \frac{w}{2}(L-x)^2 - wL(L-x) + M \\
-M &= \frac{wL^2}{2h} x h\left[1-\left(\frac{x}{L}\right)^2\right] + \frac{w}{2}\left(L^2 - 2xL + x^2\right) - wL^2 + wxL \\
\frac{-2M}{w} &= L^2 - x^2 + L^2 - 2xL + x^2 - 2L^2 + 2xL \\
\frac{-2M}{w} &= 2L^2 - 2L^2 + x^2 - x^2 + 2xL - 2xL \\
0 &= M
\end{aligned}
$$

Hence, the **parabolic profile** for the arch is the most efficient obtainable arch wise, but only for the **uniformly distributed load. Bending** presents itself for other load cases.

12.10 Hyperbolic Paraboloids (Hypar) Shells

Introduction

Besides offering a roofing solution with many interesting alternatives the **hyperbolic paraboloid (hypar)** also makes efficient use of the timber through its shape. The hypar, which is a popular member of the **saddle shell family**, can be formed into roof shapes to cover **square, rectangular** or **circular** plans.

FIGURE 12.14 shows how hypars can be used in configurations having **straight boundaries (a,b,c)** or as **saddles (d** and **e).**

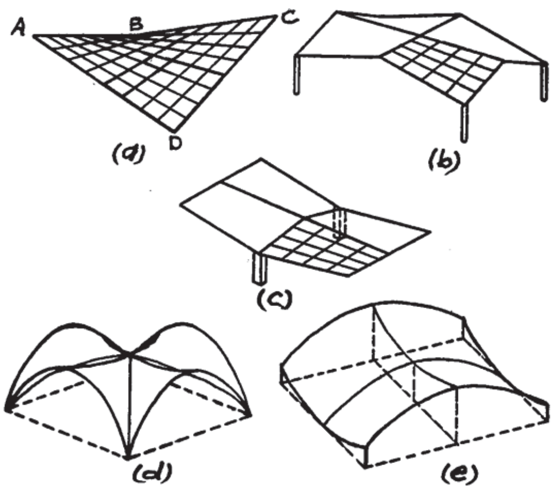


FIGURE 12.14: Various hypar configurations

12.11 Hypar Design - Geometry

To develop a hypar simply requires fixing the **two opposite corners (a** and **c)** of a rectangular or square plate and **raising the other two corners (b** and **e)** as shown in FIGURE 12.14(a). An interesting phenomena concerning the geometry of the hypar is that it is formed by a **straight line** moving over **two other straight lines inclined to one another.**

A **vertical plane** penetrating the hypar **parallel** to the direction of the **convex parabola** will result in the roof shape shown in FIGURE 12.14 (d).

Vertical planes penetrating the hypar **perpendicular** to the directions of the **diagonals AC and BD** will expose **convex and concave parabolas** resulting in the saddle shape of FIGURE 12.14(e).

Horizontal planes, parallel to the dotted outline of FIGURE 12.15(a) penetrating the hypar, will expose **hyperbolas.**

With reference to the **co-ordinate system (x,y,z)** shown in FIGURE 12.15 (a), **mathematically:**

$$z = kxy \qquad (12.7)$$

When k = o the hypar degenerates to a **plane surface.**

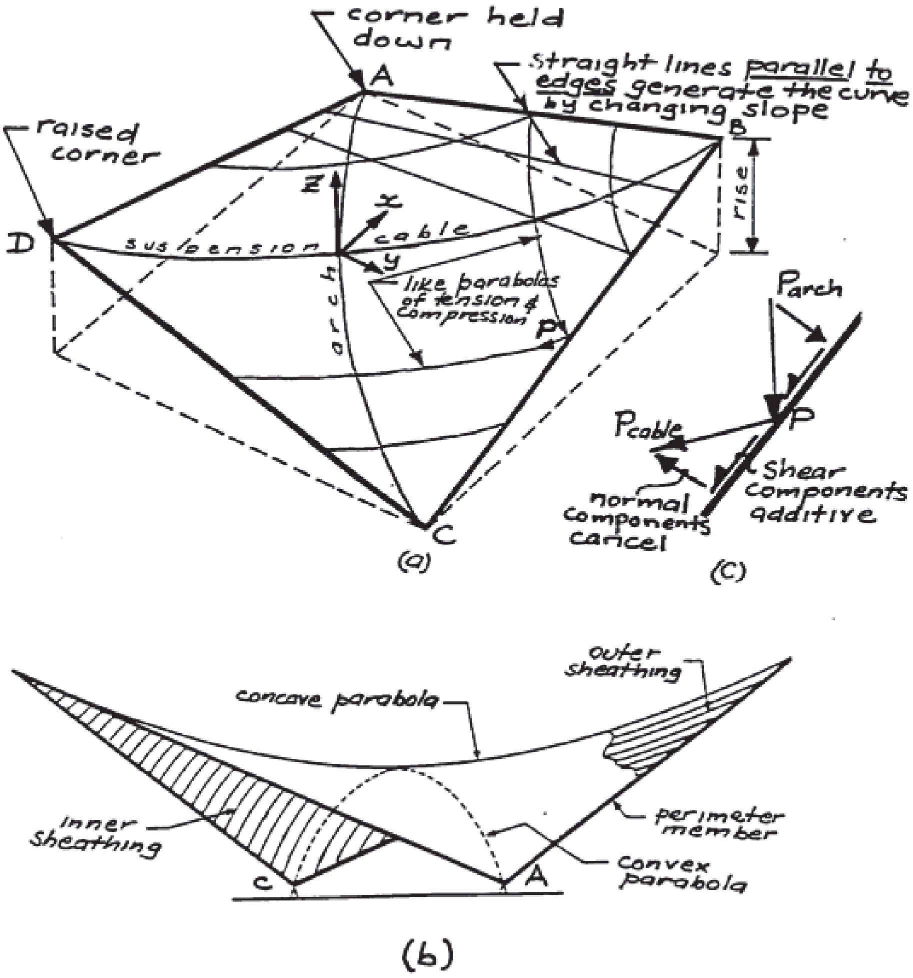


FIGURE 12.15: Two views of a single hyperbolic-paraboloid shell

12.12 Hypar Design - Structural Action

Structurally the hypar consists of a system of **intersecting arches and suspension cables,** half the load being carried in **tension** by the **suspension cables and half in compression** by the **arches.** Since sections taken **parallel to both diagonals** lead to the **same parabola,** the force at **some point P** (see FIGURE 12.15 (a)) on the edge, due to **arch action,** will be the same as the force applied by the **cable** at that point. Also, because they act at **equal angles** to the edge, but in **opposite senses,** there is **no force component perpendicular** to the edge member. Therefore, this **double system** of forces can be resolved into a series of **shear forces** along the edge requiring a **perimeter beam** to carry them as shown in FIGURE 12.15 (c).

Since arching action is associated with **compression forces,** which in turn relates to **buckling,** a limit must be placed on the ratio of **the rise of the diagonal / span of the diagonal.**

Single shell support can be effected by providing suitable restraint at **two support points,** e.g. A & C in FIGURE 12.15 (a) being the most common. Accumulation of the **membranal shears** into the intersecting perimeter members at A & C results in larger **thrusts** having to be resisted at these two locations. This can be done by suitably designed **buttresses** or a **tie** across AC which, although it is the most economical, detracts from appearance and reduces headroom. Alternatively, the **two high points** (D & B) can be supported resulting in the **perimeter members** being in **tension** and the resultant force being **inwards** rather than outwards.

12.13 Hypar Design - Methodology

There are several methods available for determining the forces in a hypar shell the one followed herein is that presented in the Western Wood Products Technical Guide; Hyperbolic Paraboloid Shells.

For **symmetrical loading** of the hypar shown in FIGURE 12.16 the **vertical reactions (R)** are **half** the sum of the **vertical load (W)**. The **horizontal thrust (H)** can be determined by considering the **triangle** of **base (ℓ/2), height (h) and hypotenuse (k).** Since the **total load (W)** can be assumed to **act vertically at (O)** along the line of (h), and if the **resultant** of **(H) and (R)** is assumed to have its **line of action (k)**, then **summation of the moments** of the **forces to the left about (O)** results in:

$$\sum \boldsymbol{M}_{0L} = 0$$

$$= \boldsymbol{R}.\ell/2 - \boldsymbol{Hh}$$

$$\frac{\boldsymbol{R}}{\boldsymbol{h}} = \frac{\boldsymbol{H}}{\ell/2} \qquad (12.8)$$

Hence:

$$\boldsymbol{H} = \frac{\boldsymbol{R}\ell}{2\boldsymbol{h}} \qquad (12.9)$$

Taking moments of the resultant force (F) and the vertical reaction (R) about (D) results in:

$$\frac{\mathbf{R}}{\mathbf{h}} = \frac{\mathbf{F}}{\mathbf{k}} \qquad (12.10)$$

Giving :

$$\mathrm{F} = \mathrm{R}\frac{\mathrm{k}}{\mathrm{h}}$$

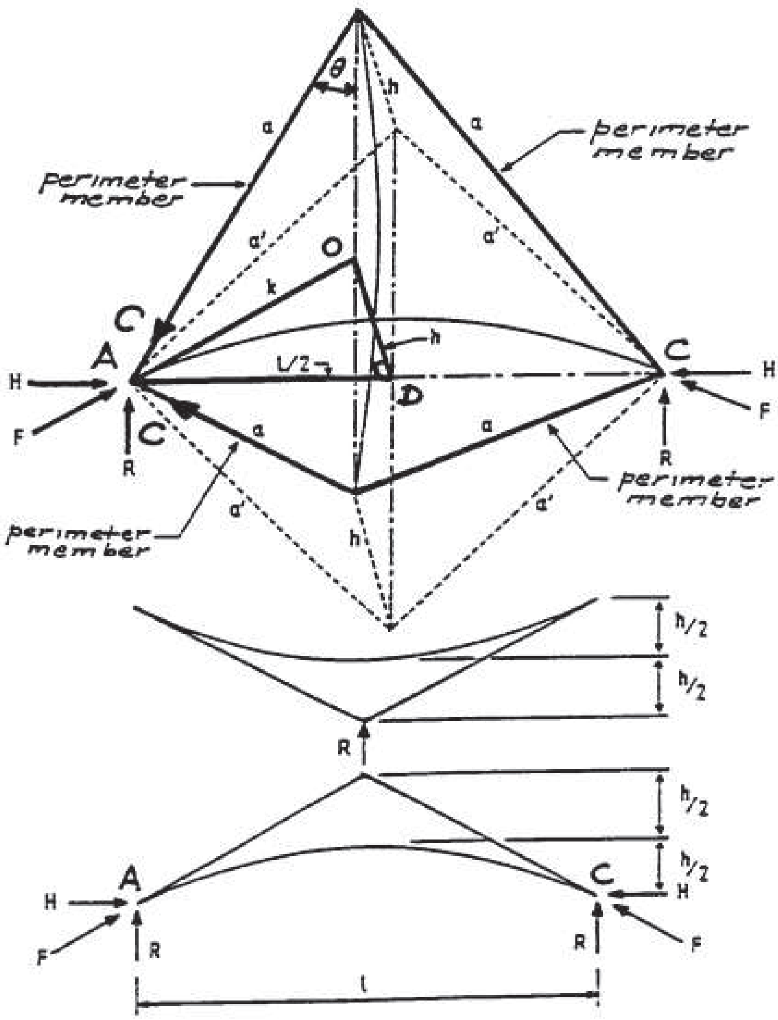


FIGURE 12.16: Reactive force components and resultant

From the plane containing the **two perimeter members (a), line (k), force (F),** the line joining the **two high points and angle (θ)** in the plan view of FIGURE 12.16, the **compression force C** in the perimeter member is:

$$2C\sin\theta = F$$

$$\sin\theta = k/a$$

Giving:

$$C = \frac{Fa}{2k}$$

But:

$$F = \frac{Rk}{h}$$

Hence:

$$C = \frac{Ra}{2h} \qquad (12.11)$$

NOTE:
This **compressive force varies uniformly** from **zero** at the **peak** to a **maximum** at the **support**

The **perimeter members** are **very important** components of the hypar shell since they:

- transfer all of the **accumulated membrane shears** to the bearing points;

- resist any **bending** induced by the sheathing being connected to the top or bottom of these members.

Hence, perimeter members can be subjected to **combined bending and direct axial compressive forces** and must be designed accordingly.

By **sandwiching the sheathing into the perimeter members** with half of the perimeter member above and half below the sheathing, eccentricity will be eliminated and the perimeter members will be subjected to **axial compression only.**

Since membranal stresses result in **boundary shears** along the perimeter member these shears can be resolved to determine **sheathing stresses.** The **principal forces** in the shell are **compressive forces c, parallel to** the direction of the **convex parabola** and **tension forces t, parallel to** the direction of the concave parabola shown in FIGURE 12.17.

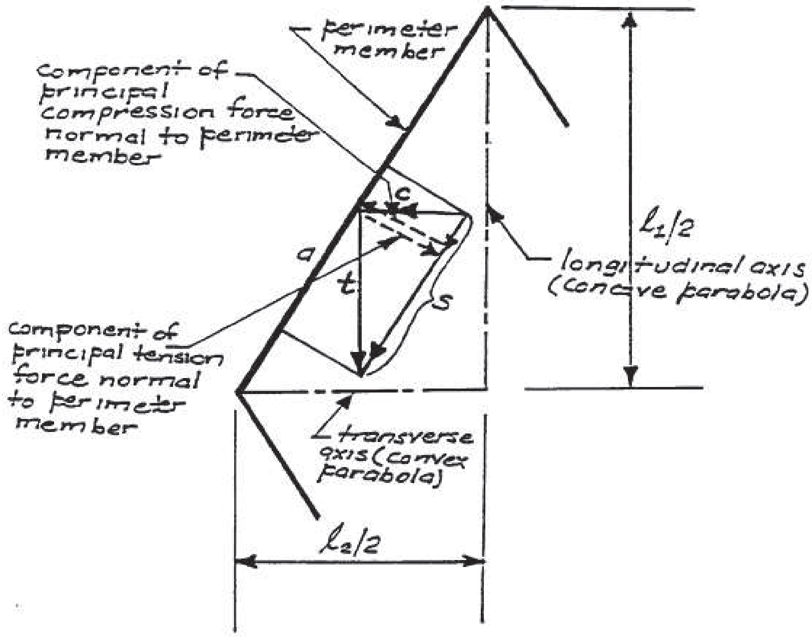


FIGURE 12.17: Resolved components of the tension and compression forces

The following lists the nomenclature applicable to FIGURE 12.16 and FIGURE 12.17

a	= length of side
a'	= length of the horizontal projection of a
C	= total compression force in perimeter member
c	= principal compressive force in sheathing / metre
F	= resultant of the vertical reaction R and the horizontal thrust H
H	= horizontal thrust
h	= vertical distance from a support to the highest point of the shell
k	= inclined distance from a support to the mid-point of the length ℓ
ℓ_1	= length along longitudinal axis
ℓ_2	= length along transverse axis
R	= vertical action
t	= principal tension force in sheathing per metre
s	= boundary shear force per metre

12.14 Methodology - Principal Membrane Forces

When the **projected plan** of the hypar is a **diamond shape** the **tension (t)** and **compression (c) forces** shown in FIGURE 12.17 can be resolved by proportion. The **principal tensile force (t) / metre width** is:

$$\frac{t}{\ell_1/2} = \frac{s}{a'}$$

$$t = \frac{\ell_1 \, x \, s}{2a'} \qquad \textbf{(12.12)}$$

The **principal compressive force (c) / metre width** is:

$$\frac{\mathbf{c}}{\ell_2/\mathbf{2}} = \frac{\mathbf{s}}{\mathbf{a'}}$$

Hence:

$$c = \frac{\ell_2 \, x \, s}{2a'} \qquad \textbf{(12.13)}$$

When the projected plan of the hypar is **square** in shape **(t)** and **(c)/metre width** will be **equal** in **magnitude** to the **boundary shears/metre length** of **perimeter member.**

12.15 Methodology - Twist in Perimeter Members

Since the hypar is a **doubly curved shell** the **sheathing slope constantly and uniformly changes** along the **length** of the **perimeter member** hence its contacting surface needs to be **appropriately shaped.** This necessitates in the determination of the **total angle of twist** shown in FIGURE 12.18(b) which applies to hypars having plan projections which are either **diamond** or **square** in **shape.** For the **diamond shaped** projection:

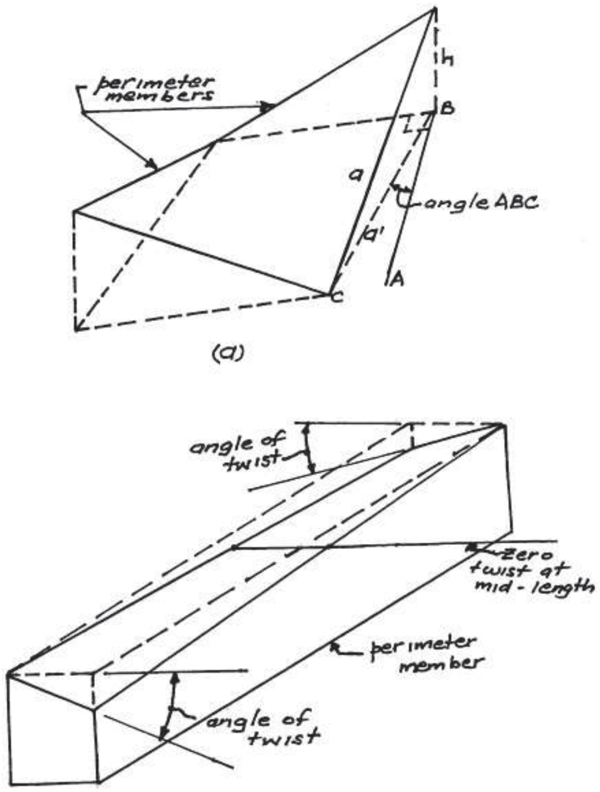
$$\textbf{tan of angle of twist} = \frac{\mathbf{ha}}{(\mathbf{a'})^2 \cos \angle \mathrm{ABC}}$$

where:

$\angle \mathrm{ABC}$ is that shown in FIGURE 12.18 (a)

For the **square shaped** projection **angle ABC** becomes **zero** and Equation 12.14 becomes:

The **total angle of twist** between the **ends** of a **perimeter member** is **twice** that determined by Equations 12.14 or 12.15.



(b)

FIGURE 12.18: Angle of twist

12.16 Hypar Design - Design Considerations

Sheathing parallel to the **longitudinal and transverse** axes of **hypar act independently.** Hence, **interconnection** is **not required for strength** but is required to **prevent buckling.**

Sheathing parallel to the **hypar sides** results in the layer **resisting** part of the **tension** and part of the **compression forces.** Hence, at the layer interfaces the forces have to be **transferred across** the **interfaces.** This results in **shear** being developed between the two layers which has to be **resisted by** the **fasteners.**

Perimeter members transfer **all loads** to the **supports** must have sufficient cross-section to **resist** the **cumulative axial compressive forces. Sheathing** provides **lateral restraint** to the perimeter members within the plane of the sheathing. In the **perpendicular direction** the perimeter members receive **no lateral support** so the possibility of **buckling** must be considered.

As the **hypar** becomes **flatter** it becomes more **flexible** increasing the tendency to **buckle.** It is therefore desirable, to **limit flatness**, which can be expressed as a ratio of **rise (h)/length of side (a)**, to 1/5.

12.17 Domes

Introduction

Domes consist of **doubly curved surfaces** which, unlike the **hypar**, cannot be formed by a series of straight lines. Hence, domes constitutes a **non-developable surface**, i.e. they **cannot be flattened** without cutting the surface at a number of sections, e.g. half of a soccer ball. Theoretically the dome offers one of the **most efficient structural forms** for covering **large column free areas** and encloses **maximum space** with **minimum** surface. **Braced domes,** which are suitable for spans of 15 to 400m, can be categorised as follows:

- **frame** or **skeleton** – single layer;
- **truss type** – double layer, very rigid and suitable for large spans;
- **stressed skin** – covering forms an integral part of the structural system;
- **formed surface** – sheets of material are bent and interconnected along their edges.

Many braced dome geometries exist but only **three** will be **mentioned herein**. These are the:

Schwedler dome which consists of polygonal rings interconnected by meridional members as shown in Figure FIGURE 12.19(a). A feature of this dome is that it can be analysed as a **statically determinate** structure.

Lamella dome developed by Dr Kiewitt and shown in FIGURE 12.19 (b). A feature of this dome is that it results in an **even stress distribution** throughout and handles large concentrated loads efficiently.

Geodesic dome developed by Buckminster Fuller and shown in FIGURE 12.19 (c). A feature of this dome is its suitability to construction situations requiring **point supports**. This is opposed to the previously mentioned domes, both of which require **continuous edge supports.**

Ribbed domes consist of **arches** or **ribs** constituting the **meridians** intersecting at the crown and either **pinned** at the base or connected to a horizontal base ring. **Horizontal rings** (hoops) are also required in conjunction with **bracing elements** as shown in FIGURE 12.19 (a).

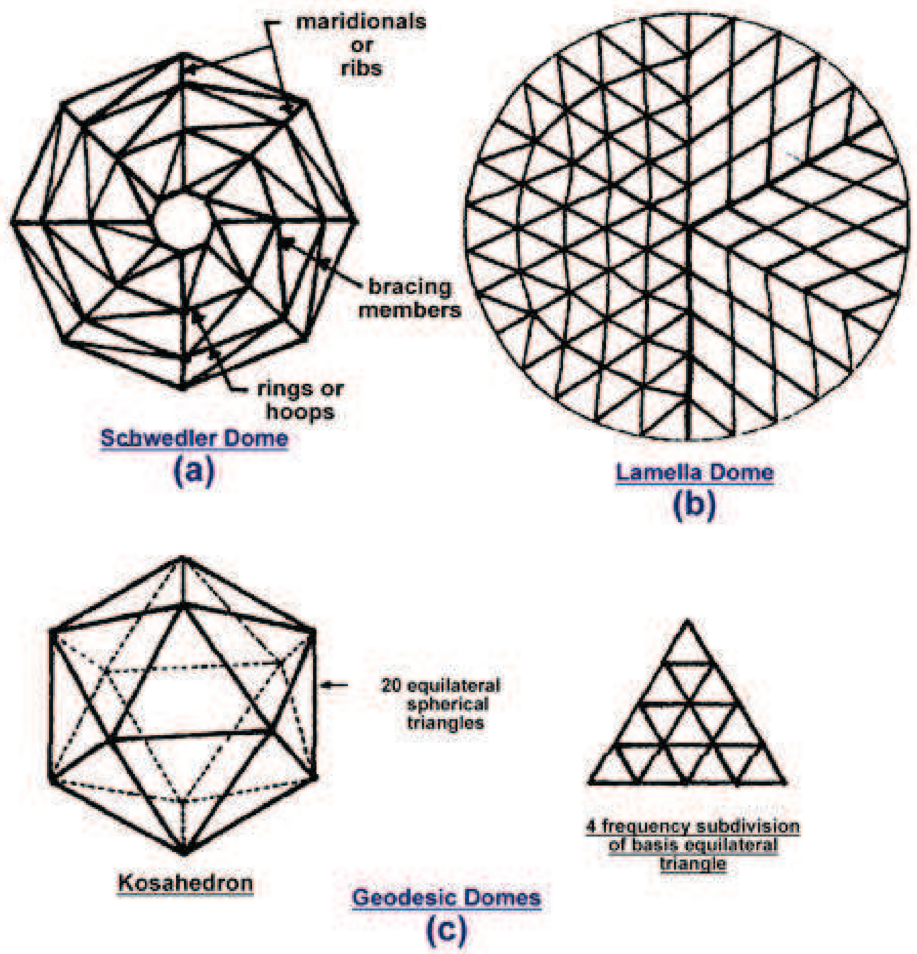


FIGURE 12.19: Shows some different dome geometries

12.18 Dome Design - Structural Action

The **ribbed dome** develops its load carrying capacity for **symmetrical loads**, through the **meridionals** acting as **funicular arches**, i.e with no bending **only compression** and the **rings** restraining the arches by developing **hoop stresses.** The hoop stresses may be **compressive only** for **shallow domes** and **compressive** and **tensile** for **high rise** domes.

Load transfer in **thin shell domes** is almost entirely due to **membrane action**, i.e. by **in-plane direct** and **shear forces**. Hence, the three active forces on a thin shell element are $\mathbf{N_x}$, $\mathbf{N_y}$ and $\mathbf{N_{xy}}$ as shown in FIGURE 12.20 (a). The **term thin** is **relative** since there is no doubt an **eggshell** fits this category but equally, an **89mm thick** shell spanning 75.6m in Germany, does so as well.

Many of the modern **braced domes** are constructed incorporating a **reticulated spatial system** of members which form the basis of the dome. These members are then covered by a sheet material, e.g. **plywood** which may **act integrally** with the spatial members to produce a **composite structure** thus performing the **bracing function.** An efficient means of attaining these spatial systems is through the interconnection of **triangular elements** to produce the **reticulated patterns** shown in FIGURE 12.20(b), (c) and (d).

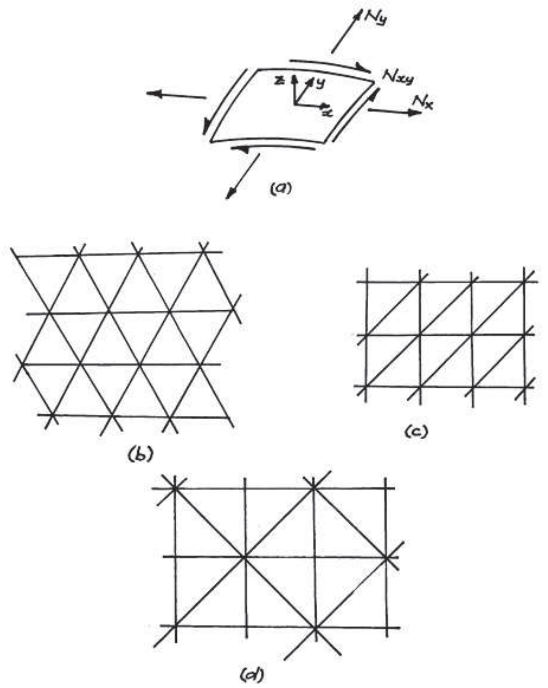


FIGURE 12.20: Membrane forces and reticulated spatial systems

Based on the premise a **reticulated shell**, having a **spatial member configuration** capable of carrying the **membrane forces N_x, N_y and N_{xy},** will function as a continuum shell allows **simple relationships** between the **forces** of the **two systems** to be developed.

Two such systems will be considered herein.

Because of the large number of members and their associated **degrees of freedom** (up to 6/node) a **membrane type analogy**, closed form solution is essential at the **preliminary design stage.**

12.19 Dome Design - Methodology

Membrane stresses in a thin spherical dome are given by:

$$\mathbf{N_{xy} = 0}$$

The hoop force:

$$N_X = wR\left(\frac{1}{1+\cos\theta} - \cos\theta\right) \qquad \textbf{(12.16)}$$

and the meridional force:

$$N_Y = -wR\frac{1}{1+\cos\theta} \qquad \textbf{(12.17)}$$

where:

W = load acting on the shell per unit area measured on the shell surface;
R = radius of curvature of the dome which is constant for

a

sphere

θ = is the angle subtended by the element under consideration with the crown

FIGURE 12.21 defines the above parameters.

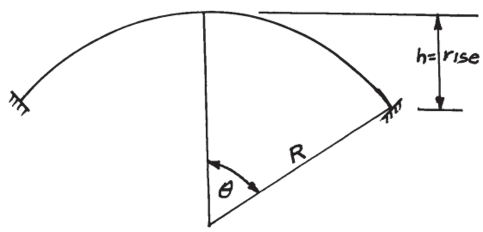


FIGURE 12.21: Shows θ = angle to crown and R = radius of curvature

when:

$$\frac{1}{1+\cos\theta} \quad \cos\theta = 0$$ from Equation 12.16

Then:

$$\theta \approx 52°$$

and

$$N_x = 0$$

With further **increase** in **θ**, $\mathbf{N_x}$ becomes **positive**, i.e. from **θ>52°** there are **tensile stresses** in the hoops. Hence, domes having a **low rise** will result in the **hoops** being in **compression.**

Braced dome member forces as stated previously, can for analytical purposes, be conveniently **related** to the **membrane forces** of a **spherical dome** subjected to **symmetrical loading.**

The **axes** of the **membrane force field** can be transformed to align with **one of the lines** of the **grid system** as can be seen from observing FIGURE 12.22.

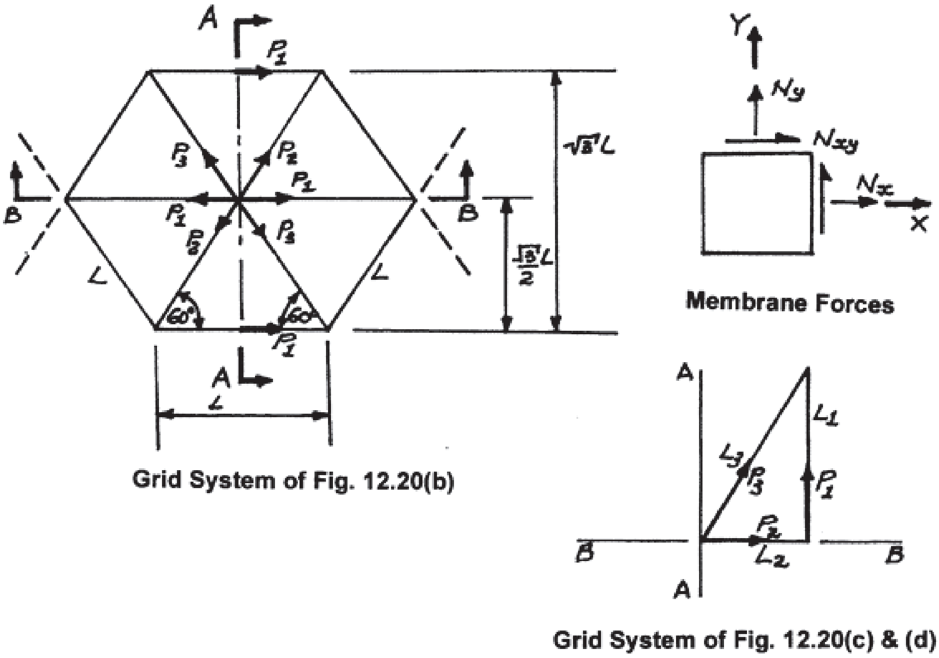


FIGURE 12.22: Grid systems and membrane forces

Satisfying equilibrium of the forces at the **section A-A,** of **length** $\sqrt{3}/2\ x\ L\ x\ 2 = \sqrt{3}\ x\ L$, in the **x-direction** for the **grid system** of FIGURE 12.20**(b)** gives:

$$\Sigma Fx = 0$$

$$\frac{P_1}{2} + \frac{P_1}{2} + P_1 + P_2 \cos 60^\circ + P_3 \cos 60^\circ = 2\sqrt{3L \, x N_x}$$

Hence :

$$4P_1 + P_2 + P_3 \quad = 2\sqrt{3\, x\ L\, x N_x} \tag{12.18}$$

Doing likewise for **section B-B** of **length** 2xL/2 = L

In the Y-direction:

$$\boldsymbol{\Sigma F_y = 0:}$$

$$\mathbf{P_2 + P_3 = \frac{2}{\sqrt{3}} x\, L\, x\, N_y}$$

Satisfying equilibrium along the B-B plane in the x-direction:

$$\begin{aligned} \mathbf{\Sigma F_X} &\mathbf{= 0} \\ \mathbf{P_2 - P_3} &\mathbf{= 2LN_{xy}} \end{aligned} \tag{12.20}$$

Re-arranging the above equations:

$$\left.\begin{aligned} \mathbf{N_x} &= \mathbf{\frac{4P_1 + P_2 + P_3}{2\sqrt{3}\ L}} \\ \mathbf{N_y} &= \mathbf{\frac{\sqrt{3}(P_2 + P_3)}{2L}} \\ \mathbf{N_{xy}} &= \mathbf{\frac{(P_2 - P_3)}{2L}} \end{aligned}\right] \tag{12.21}$$

Inverting Equations 12.21 gives:

$$\left.\begin{aligned} \mathbf{P_1} &= \mathbf{\frac{L}{2\sqrt{3}}(3N_x - N_y)} \\ \mathbf{P_2} &= \mathbf{\frac{L}{\sqrt{3}}(N_y + \sqrt{3}N_{xy})} \\ \mathbf{P_3} &= \mathbf{\frac{L}{\sqrt{3}}(N_y - \sqrt{3}N_{xy})} \end{aligned}\right] \tag{12.22}$$

At the **crown** of the dome where **Nxy = 033, Nx = Ny,** all three members forces are equal to:

$$\boldsymbol{P_1 = P_2 = P_3} \quad = \boldsymbol{-L.R.w.}/2\sqrt{3}$$

NOTE:
*The above relationships were derived from **static equilibrium requirements** and are therefore i**ndependent** of **member cross-sections**.*

In the case of the **space grids** shown in FIGURE 12.20 (c) and FIGURE 12.20 (d) which are isolated as the triangle in FIGURE 12.22, a similar process results in:

$$
\left.\begin{aligned}
N_x &= \frac{P_2}{L_1} + \frac{L_2 P_3}{L_1 L_3} \\
N_y &= \frac{P_1}{L_2} + \frac{L_1 P_3}{L_2 L_3} \\
N_{xy} &= \frac{P_3}{L_3}
\end{aligned}\right] \qquad (12.23)
$$

And

$$
\left.\begin{aligned}
P_1 &= L_2 N_y - L_1\ N_{xy} \\
P_2 &= L_1 N_x - L_2\ N_{xy} \\
P_3 &= L_3\ N_{xy}
\end{aligned}\right] \qquad (12.24)
$$

12.20 Spherical Domes - Design Example

A spherical dome has a radius of curvature of 20m, w = 1.5kPa due to self weight, fixings, finishings and the uniformly distributed live load and $\theta = 60°$ as shown in FIGURE 12.23. Assume a grid pattern identical to that of FIGURE 12.20(b) with grid member lengths = 2m. Determine a preliminary size of proposed LVL grid members.

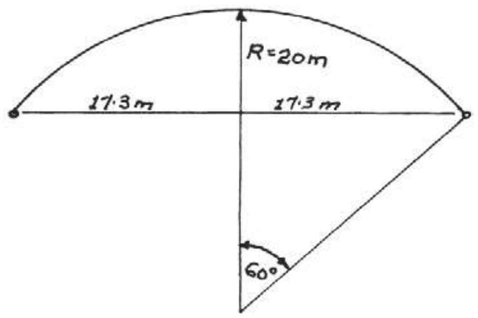


FIGURE 12.23: Dome dimensions

12.21 Domes – Worked Example

Span = 2.R.sin60°
= **34.6m**

Because of the **symmetrical loading, changes** of **slope and twist** across the membrane surface will be negligible, hence the **shears (N_{xy})** will be **zero.**

The membrane forces will be:

Angle from dome crown	N_x(hoop)* kN/m	N_Y(meridonal)~ kN/m
0	-15.0	-30.0
30	-9.9	-34.6
45	-3.6	-42.4
60	+5.0	-60.0

$* N_X = \mathbf{wR}\left(\frac{1}{1+\cos\theta} - \cos\theta\right) \qquad \sim N_Y = (-\mathbf{wR}x1/\cos\theta)$

TABLE 12.1: (-) is compression, (+) is tension

From Equations 12.22 :

$$\mathbf{P_1} = \frac{L}{2\sqrt{3}}(3N_x - N_y)$$
$$= \frac{2}{2\sqrt{3}}(3.5 + 60)$$
$$= 43.3\,kN$$

$$\mathbf{P_2} = \frac{L}{\sqrt{3}}\left(N_y + \sqrt{3}\text{ x } N_{xy}\right)$$
$$= \frac{2}{\sqrt{3}}. - 60$$
$$= -69.3\,kN$$

$$\mathbf{P_3} = \frac{L}{\sqrt{3}}(N_y - \sqrt{3}\,N_{xy})$$
$$= \frac{2}{\sqrt{3}}\text{x} - 60$$
$$= -69.3\,kN$$

Since this is a **preliminary assessment** of the structural capabilities of a spherical dome manufactured using LVL members, a value of $f_c^{'}$ (unfactored) of 20MPa will be assumed to determine a member size.

$$f_c^{'} = \frac{P}{A}$$
$$A = \frac{69.3 \times 10^3}{20}$$
$$= 3465\,mm^2$$

Assuming an LVL thickness of 45 mm:

$$d = \frac{3465}{45}$$
$$d = 77\,mm$$

Try a LVL cross-section = 75 x 45 mm

The foregoing calculation tells us **nothing** about the **deformation** of the **structure.** However, it does imply **member sizes** should be **reasonable** for a clear span of 34.6 m.

12.22 Other Design Considerations

Although the preliminary calculations indicate a **braced dome** could be a **viable solution** there are a number of other **design considerations** to be addressed. These include:

- producing a **node connection** capable of accommodating **(6) member ends** at the same time provide the **necessary stiffness;**
- having a **rigorous analysis done** to include:
  - **individual member buckling** due to direct forces on the shell;
  - **snap – through buckling** due to local load concentrations;
  - **general shell** buckling over a fairly large area;
  - effects of **unsymmetrical loads** resulting in membrane shears;
  - any **moment** effects near edges and at supports.

- whether or not to force the **plywood** to act **compositely** with the LVL;

- **construction techniques.**

Listing the above design considerations are **not meant to deter the designer**, but rather to make him/her aware of some of the vagaries, particularly those associated with buckling.

Many large diameter domes have been built over a long time span, and without the aid of computers. A **Schwedler dome**, built in Vienna in **1874** had a clear span of **64m.**

However, with the **computer power** available in the 21st century, in conjunction with **sophisticated finite element programs,** capable of **three dimensional second order analysis**, provide the structural mechanist with the necessary analytical tools to handle the **most complex of shell structures.** Additionally, the advent of **Formex Algebra** which facilitates the generation of shell topology, further enhances the use of computers where necessary.

12.23 Photographs

To demonstrate the versatility of plywood and LVL in the production of complex structural forms a collections of photographs of actual structures is presented in Appendix A12.1.

12.24 Design Aids

Appendix A12.2 contains some design aids to assist designers at the preliminary design stage.

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13. **Structural Form**, H. Bier, New Zealand Timber Industry Federation (Section A-1).

A12 Chapter 12 Appendix

EXAMPLES OF EXOTIC STRUCTURES

Folded Plates

Arches

Hypars

Domes

EXOTIC STRUCTURAL FORMS DESIGN AIDS

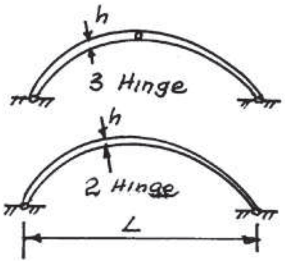
STRUCTURE	SPACING (m)	SPAN (m)	DEPTH
ARCHES			
	5-15	20-60	h = L/50
	5-20	20-100	h = L/55
FOLDED PLATE		**Max. Likely (m x m)**	**COMMENTS**
		20 x 10	Stiffening ribs may be required
HYPAR			
		25 x 25	LVL edge beams with boarded membrane
DOMES			
		30 Φ	Ribbed dome with plywood membrane
		100 + Φ	Braced dome with LVL members

TABLE A12.1: Preliminary design information

13 Connection Design – Plywood & LVL

13.1 Introduction

In no way is this chapter meant to replace Section 4 of AS 1720.1-1997 on Connection Design, but rather it is meant to supplement it. It is hoped it will make the designer, new to timber, aware of the pitfalls associated with the detailing of timber connections. A further aim is to provide some guidance in the design process to ensure a functional and aesthetically pleasing timber structure is produced at competitive cost. Hence, it is imperative that AS 1720.1-1997 is used in conjunction with the contents of this chapter.

The Crews & Boughton publication also proved to be a very useful reference during the compilation of this Chapter.

The saying **the devil is in the detail** was never truer than in its **application to connection design.** Irrespective of how much refinement is directed towards **member and/or component design** of structural systems the effort is **doomed to failure** if **connection design is neglected.** Unfortunately, all the **glamour** of structural design is associated **with** the **member design aspect,** resulting in the **connection design** not being afforded the attention it deserves. This anomaly appears to be particularly prevalent in timber design where **gross looking steel connections** are "**designed**" to, in particular, interconnect a number of timber elements meeting at a common joint. There are however, occasions where the fabricated steel connection does offer an economical and aesthetically satisfactory solution.

The **scope** of this chapter will be confined to the **dowel type connectors, i.e.:**

- **nails** and their associated connections;
- **screws**;
- **bolts** and their associated connections.

Selectivity has not been without purpose, for several reasons:

- **dowel type** connectors are the **most widely used**, by far, in timber structure construction;
- **other type connectors**, e.g. split ring, shear plate and the multiplicity of proprietary steel connectors and their **capabilities** are **well documented**;
- this is **not** an exercise designed to **subsidise** the **steel industry.**

Hence, good connection design must not only ensure **efficient load transfer** through the joint but must also ensure **serviceability and durability** have been carefully assessed and catered for. Also, **aesthetics and costs** must be given due consideration.

Simplicity of connection form should always be uppermost in the designer's mind with care being taken **not to create monsters.** Such a situation arises when **steel boots** are fixed to **exposed ends of beams** to supposedly **protect them** from the environment. These "**protectors**" can in fact create the ideal conditions for **moisture retention** followed by the **propagation of rot.**

13.2 Terms and Definitions

For **consistency of terminology** the following **definitions** apply.

Connector refers to an **individual fastener,** e.g. a **nail, screw** or **bolt.**

Connection refers to the **connector group**, also called a **joint**, constituting the **mechanism** by which **load is transferred** between members **at a discontinuity**. FIGURE 13.1(a) and (b) show simple examples of connections.

A **spliced joint** develops **continuity of load transfer**, in uniaxial tension or compression, between two members by overlapping and fixing or by butting the ends and fixing with a cover plate each side of the discontinuity. FIGURE 13.1 (c) illustrates a spliced joint.

A **dowelled connector** herein refers to a fastener which is **circular** in **cross-section**, e.g. nail, screw or bolt,

Type 1 joints referred to in **AS 1720.1-1997** result in the **fastener** being **subjected to shear.** All of the joints shown in FIGURE 13.1 are **Type 1 connections.**

Type 2 joints referred to in **AS 1720.1-1997** result in the **fastener** being subject to **tension** and/or **withdrawal.** The joints shown in FIGURE 13.2 are **Type 2 connections.**

Moment Joints (discussed in Chapter 11) interconnect structural elements, e.g. beam/columns of a portal frame with the **capability** of **transferring** the **induced moment, shear and axial force across** the **discontinuity**. The **medium** of moment transfer being a **gusset plate** (plywood or steel) **nailed, screwed** or **bolted** to the primary elements as shown in FIGURE 13.1(g).

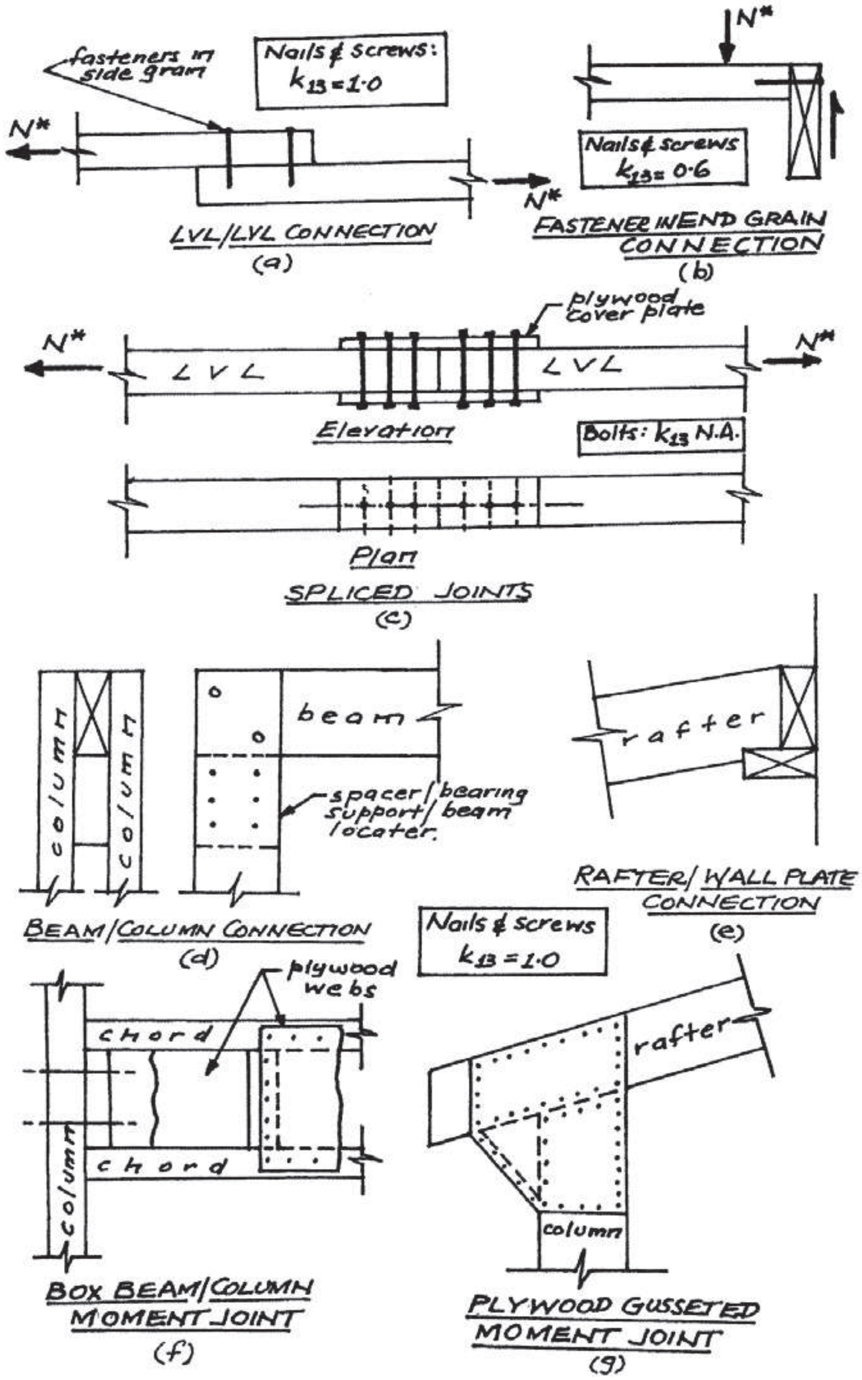


FIGURE 13.1: Example of Type 1 Connections

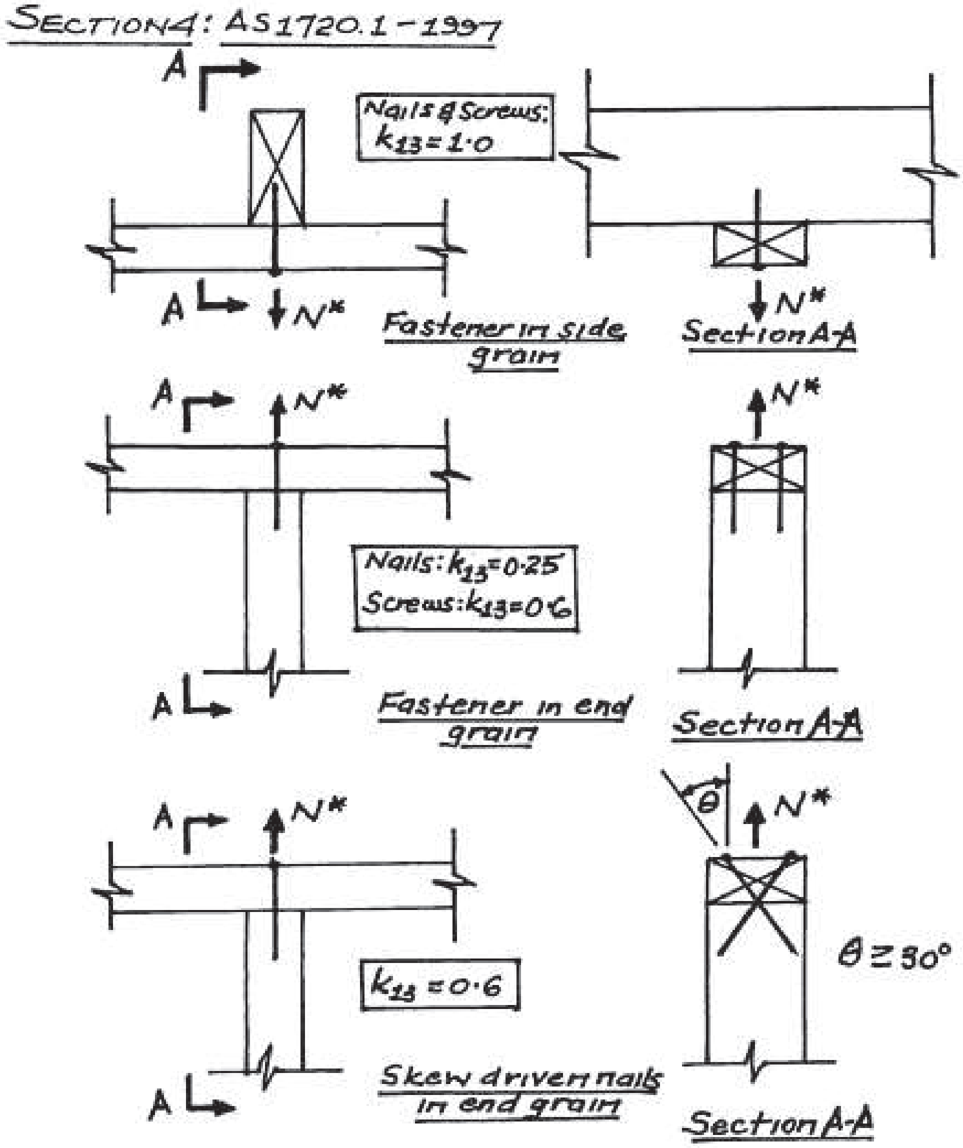


FIGURE 13.2: Examples of Type 2 Connections for nails and screws

Geometrical aspects relate to **spacing and location of fasteners** within the connection to **prevent splitting** of the timber.

FIGURE 13.3 defines these **critical dimensions** for **nails** and **screws** and TABLE 13.1 **quantifies** them in terms of the **fastener diameter D.** Adherence to these dimensions will ensure the connection modelled by AS 1720.1-1997 will attain the required capacity.

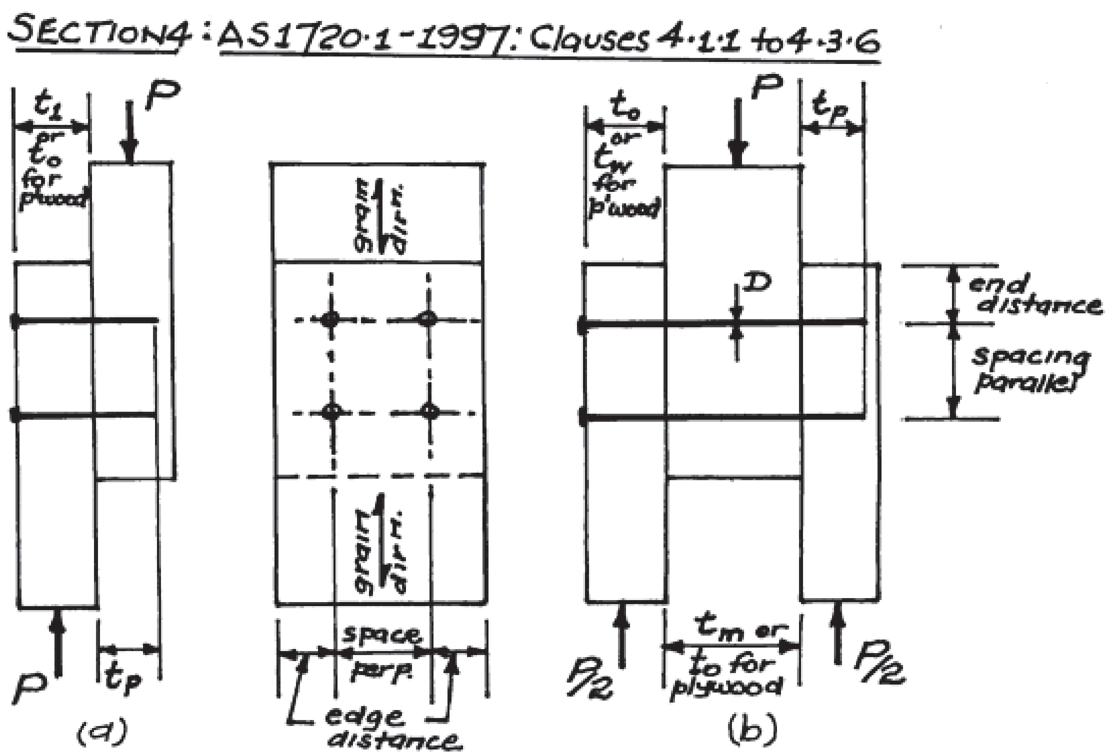


FIGURE 13.3: Two and Three member Type 1 nailed and screwed connections

Note:
Where fastener loads are at an ***angle (θ)*** *to the grain the* ***minimum spacing*** *between the fasteners can be found by application of* ***Hankinsons's Formula*** *as follows:*

$$S_\theta = \frac{S_\ell , S_p}{S_\ell \sin^2\theta + S_p \cos^2\theta}$$

where:

S_θ = **spacing** of fasteners in the **direction θ** to the grain;
S_p = **spacing perpendicular** to grain;
S_ℓ = **spacing parallel** to grain;
θ = **angle** between the line joining adjacent connectors and the general grain direction. See FIGURE 13.4.

Spacing Type	**Minimum Distance**		
	Nails		**Screws**
	Holes not pre-bored	**Holes pre-bored to 80% of nail diam.**	
End distance	20D	10D	10D
Edge distance	5D	5D	5D
Between connectors - along grain - across grain	20D 10D	10D 3D	10D 3D

TABLE 13.1: Minimum distances for nails and screws

Other requirements to attain AS 1720.1 – 1997 load capacities

Nails : Two Member Joint	t_1 > 10D; t_p > 10D For: t_1 and t_p<10D load is reduced in proportion to t_1 and t_p decrease. For: t_1 or t_p<5D, P = O.
Nails : Three member joint	t_m<10D; t_o >7.5D; t_p>7.5D For lesser values of t_m, t_o and t_p reduce load proportionally. For: t_p<5D; P = O.
Screws	t_1>10D, t_p>7D For: Lesser of t_1 and t_p reduce proportionally until, t_1 or t_p ≤4D when P = O
Plywood	Fastener capacity 10%>timber to timber joints provided t_o>1.5D; t_p>10D; t_w>10D For: t_p/D or t_w/D<5, P = O

FIGURE 13.4 defines the critical dimensions for bolts and TABLE 13.2 quantifies them in terms of the bolt diameter D.

NOTE:
Bolt characteristic capacities given in Tables 4.9 and 4.10 of AS 1720.1-1997 are for the **effective timber thicknesses** b_{eff} for **single bolts loaded parallel and perpendicular to the grain.** The **b** referred to in TABLE 13.2 is **defined therein**.

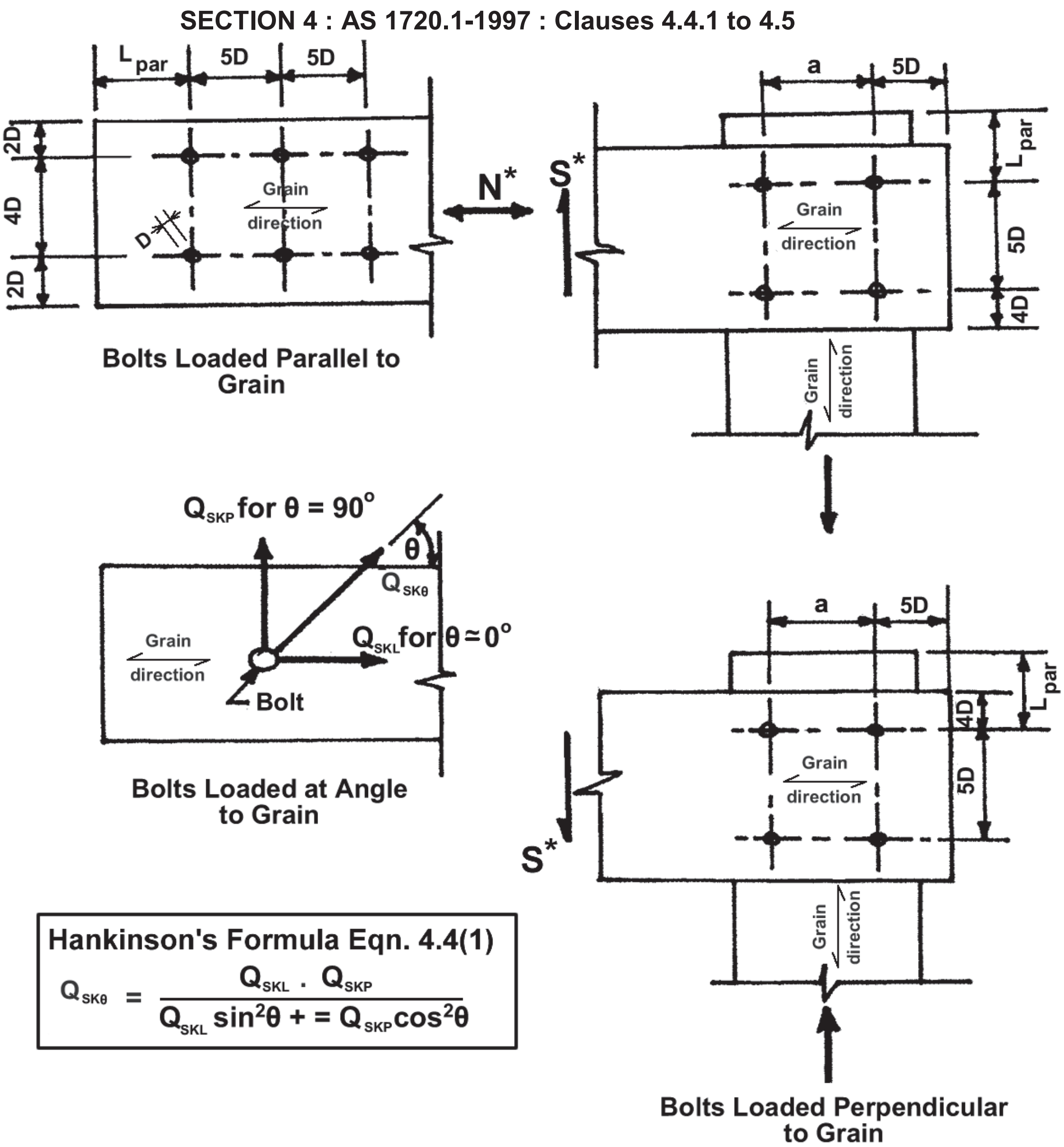


FIGURE 13.4: Critical bolt spacings and distances

Spacing Type	Distance – bolts loaded parallel to grain (θ=0° to 30°)	Distance – bolts loaded perp. to grain (θ=30° to 90°)
End distance (ℓ_{par})	8D unseasoned tension member 7D seasoned tension member 5D others	5D
Edge distance	2D	4D
Between Connectors, Along grain (a)	5D	≤ 2.5D for b/D = 2 and increase proportionally to ≤ 5D for b/D ≥ 6.*
Between Connectors, Across grain	4D	5D

*b = effective thickness of member loaded perpendicular to grain

TABLE 13.2: Other requirements for bolts

13.3 Modification Factors – Nailed and Screwed Connectors

The modification factors discussed herein allow **adjustments** to be made to the **Code Characteristic Strength values (Q_k,N)** to account for the various **influencing design parameters.**

Capacity Factor(ϕ)
Capacity factor (ϕ) given in Table 2.6 of AS 1720.1-1997 differs in magnitude to those for **members** and is **generally less.** This reduction is due to their being **more contributing factors,** each of which is **more difficult to quantify.**

Duration of load factor (k_1)
Duration of load factor (k_1) for connections also **differs** from those values given for **solid members.** TABLE 13.3 lists the duration of load factors for connections.

Load Type	Source	Duration	k_1
Dead loads	gravity	permanent	0.57
Long term live loads	furniture and partitions	permanent	0.57
Frequent live load	occupancy or vehicle	5 months	0.69
Infrequent live loads	crowds, construction	5 days	0.77
Ultimate wind gust	from AS/NZS 1170.2	gust	1.30
Earthquake loads	from AS 1170.4	5 second	1.14
Regular snow loads	alpine regions	5 month	0.69
Rare snow loads	Sub-alpine regions	5 days	0.77

TABLE 13.3: Connection duration of load factors

In connection design a **critical load combination**, i.e. the one giving the **highest D_L**, can be found from the relationship:

$$D_L = \frac{N^*}{k_1}$$

where

DL = **duration of load parameter** for the **strengt**h limit state;

N* = **Design action** for the connection due to the **applied loads;**

k_1 = duration of load factor for the **shortest duration load** in the combination

D_L performs **no other function** in the design process other than to **identify worst loading case** for the **strength limit state.**

Grain orientation factor (k_{13})
Grain orientation factor (k_{13}) for Type 1 nailed and screwed joints, **irrespective of load direction,** is k_{13} = 1.0. For nails and screws into end grain k_{13} = 0.6. FIGURE 13.1 (a) and (b) show examples.

Shear plane factor (k_{14})
Shear plane factor (k_{14}) accounts for the **number** of **shear planes penetrated** by a connector. FIGURE 13.3 (a) and (b) show examples of k_{14} for Type 1 connections. k_{14} = 1 and 2 for FIGURE 13.3 (a) and (b) respectively.

Head fixity factor (k_{16})
Head fixity factor (k_{16}) relates to the amount of **nails and screw head fixity** offered by the **member containing** the **connector head.** FIGURE 13.5 (a) shows a **fully restrained nail head** by virtue of its being driven through an **interference hole** in the **steel side plate.** This arrangement forces the nail to deform in **double curvature** under load which **increases** the

connection load carrying capacity compared to the **single curvature response** of the nail driven through a **clearance hole** illustrated in FIGURE 13.5 (b).

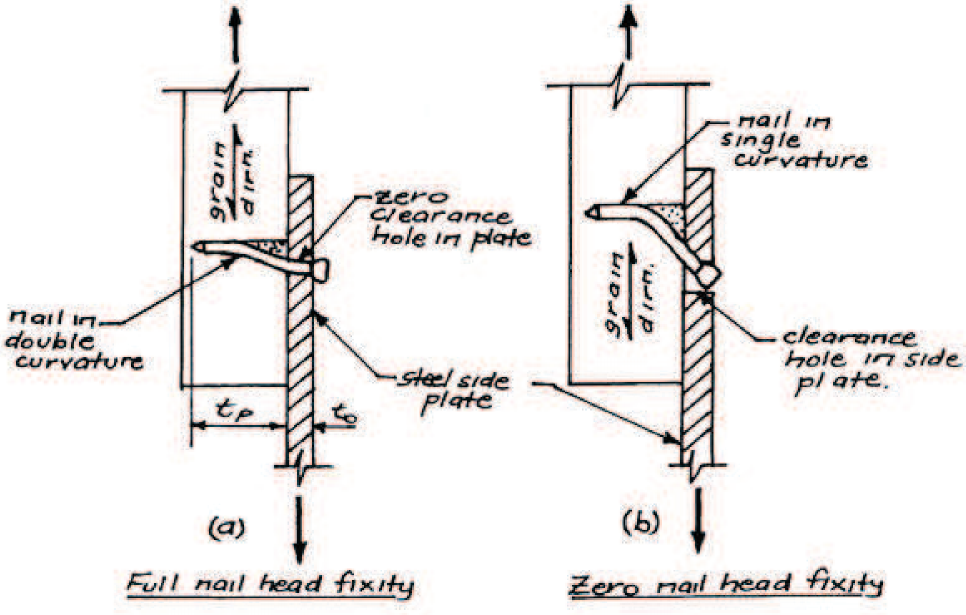


FIGURE 13.5: Nail head fixity

TABLE 13.4 gives values of $\mathbf{k_{16}}$ for nailed and screwed joints fixing side plates of **various materials**.

Side Plate Material	Plate Thickness Guide	Hole diameter	k_{16}
Steel	to >1.5D	tight fitting	1.2
Plywood	to >1.5D		1.1
Others			1.0

TABLE 13.4: Values for $\mathbf{k_{16}}$

Multiple nail factor ($\mathbf{k_{17}}$)

Multiple nail factor ($\mathbf{k_{17}}$) takes into account the fact **multiple nail and screw connections** result in the **failure load** of a **connection** being **less than** the **sum** of the **failure loads** of **all** of the **connectors.** The **number of rows ($\mathbf{n_a}$)** of fasteners in a connection is defined as **those fasteners along a line closest** to **normal** to the direction of the **applied load** as shown in FIGURE 13.6.

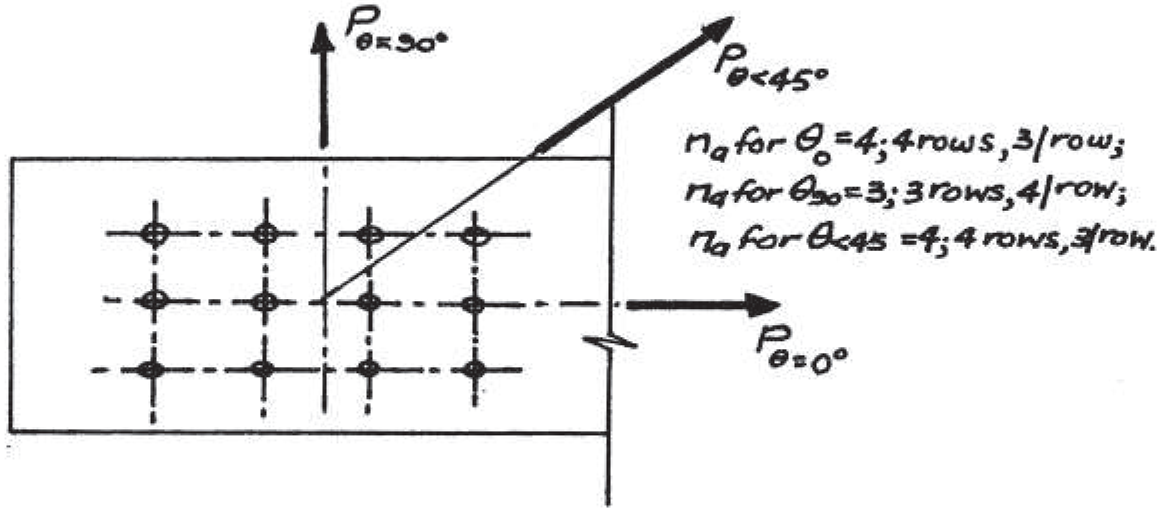


FIGURE 13.6 :Shows how rows are defined relative to applied load

TABLE 13.5 gives values for the factor $\mathbf{k_{17}}$ for use in the design of **multiple nail and screw connections.**

Condition of Timber	Values of k_{17}			
	Number of Rows of Fasteners			
	na<4	na=5	na=10	na>20
Unseasoned	1.00	0.90	0.80	0.75
Seasoned	1.00	0.94	0.90	0.85

TABLE 13.5: Values of k_{17}

13.4 Nailed and Screwed Connection Design – Methodology

Typically, a preliminary design will result in settling on a structural system that satisfies the design criteria defined by the client.

Analysis of the structure then defines the magnitude of the member forces to be transferred across the joints. **Member sizing** to satisfy the **strength limit state** requirements for the **critical load combination** provides the designer with an indication of the area of timber available to facilitate the connection design.

The following steps may then be used as a guide in the **connection design process** for **Type 1**, **nailed and screwed LVL joints**.

Steps:

1. Identify the **connection type** as Type 1 or 2 which may best be done by **sketching** or in some cases doing a **scaled drawing.**

2. Select **connector type** and diameter based on experience, availability or calculations.

3. Determine the **length of connector** to develop full load carrying capacity. This may require **adjusting member sizes or** reducing connector capacity.

4. Determine the **number of connectors** required per row. This is where the drawing will be invaluable in aiding **establishing force directions** for finding edge and end distances.

5. Obtain the **characteristic strength** of the connector from Tables in AS 1720.1-1997.

6. Apply **modification factors** to the relationship:

$$\varphi N_j = \varphi k_1 . k_{13} . k_{14} . k_{16} . k_{17} . nQ_k$$

7. Determine the **number of rows.** Check this with the chosen value of k_{17}. If incorrect **recalculate n.**

8. **Detail** the connection.

13.5 Design of Type 1 Nailed Connections (Cℓ.4.2.3)

Equation 13.2 gives the **design capacity (ΦN_j)** for a **Type 1 joint** (containing (**n nails**)) required to **resist direct loads.**

For the **strength limit state** to be satisfied:

$$\Phi Nj \geq N^* \tag{13.2}$$

where:

$$\Phi Nj = \Phi k_1 k_{13} k_{14} k_{16} k_{17} nQk \tag{13.3}$$

and

N^* = design **action** due to the **applied factored** loads on the connection

Φ = **capacity factor**;

k_1 = the **duration of loads factor** for joints;

k_{13} = 1.0 for nails in **side** grain;
= 0.6 for nails in **end** grain.

k_{14} = 1.0 for nails in **single** shear;
= 2.0 for nails in **double** shear.

k_{16} = 1.2 for nails driven through close fitting holes in **metal** side plates;
= 1.1 for nails driven through **plywood** gussets;
= 1.0 otherwise.

k_{17} = factor for **multiple nailed joints** for Type 1 connections designed to resist **direct loads** in either **tension** or compression.

N = **total number of nails** in the connection resisting the design action effect in **shear.**

Qk = **nail characteristic capacity** given in Tables 4.1(A) and 4.1(B) in AS1720.1–1997.

13.6 Design of Type 2 Nailed Connections

Equation 13.4 gives the **design capacity** (**ΦN_j**) for a **Type 2 joint** containing **(n) nails.** As shown in FIGURE 13.2 a **Type 2** connection results in the **nails** being in **tensions.**

For the **strength limit state** to be satisfied:

$$\Phi N_j \geq \mathbf{N^*}$$

where:

$$\Phi N_j = \mathbf{\Phi} k_{13} . \ell_p . nQ_k \tag{13.4}$$

and

N^* = **design action** on a **Type 2** nailed connection, along connector axis due to factored loads applied to the joints;

Φ = **capacity factor.**

k_{13} = reduction factor due to **embedment** into end **grain**.

ℓ_p = depth of **penetration** (mm) into innermost timber elements.

n = **total number of nails** in the joint.

Q_k = characteristic **nail capacity** in **withdrawal** from the specified joint strength group.

Design of Moment Resisting Nailed Connections

This topic was treated in detail in Chapter 10 of this Manual.

Serviceability Requirements for Type 1 Nailed and Screwed Joints (Cℓ.C3.2)

Section C3 of Appendix C of AS 1720.1-1997 gives some explanation regarding the **deformation of joints.**

The displacement of **nailed or screwed joints** in **single shear** for **solid wood/solid wood connections** may be estimated as follows:

$$\Delta = \left[\frac{44\, x\, j_{12}}{D^{3.5}}\right]\left[\frac{Q^*}{h_{32}}\right]^2 \textit{for}\Delta < 0.5mm \tag{13.5}$$

To determine **Q* at Δ = 0.5mm:**

$$Q^*_{0.5} = \frac{0.107D^{1.75}h_{32}}{j_{12}^{0.5}} \tag{13.6}$$

For a displacement of **Δ = 2.5mm:**

$$Q^*_{2.5} = 0.165D^{1.75}j_{13}h_{32} \tag{13.7}$$

For a displacement **0.5mm < Δ < 2.5mm** the corresponding applied load effect Q* should be obtained by **linear interpolation** between the values to give:

where:

Δ	**= 0.5mm and Δ = 2.5mm**
Δ	= Deformation of a single nail in a Type 1 connection;
D	= Nail diameter (mm);
j_{12}	= Special duration of load factor for serviceability of nailed connections;
j_{13}	= Special duration of load factor for serviceability of nailed connections;
j_{32}	= Stiffness factor for serviceability of nailed connections;
Q*	= Serviceability load effect on a single nail (N);
Q*0.5	= Serviceability load effect on a single nail when the deformation is 0.5mm;
Q*2.5	= Serviceability load effect on a single nail when the deformation is 2.5mm;

Note:
*For **plywood side plates** Equations 13.5 and 13.6 result in **conservative over-estimates** of **connector slip.***

13.7 Nailed Connections – Design Example

A spliced connection is to be designed for a LVL tension member to be used in a roof system for a commercial building in Brisbane. The member has been designed and is 150mm deep x 35mm thick. The splice plates are to be of 12mm thick F11 structural plywood fixed using 2.8mnm diameter gun driven nails.

The following unfactored loads are to be transferred by the spliced joint.

- 20kN (tension) Dead load
- 5.5kN (tension) Live load (construction)
- 26.6kN (compression) Ultimate wind load
- 6.6kN (tension) Ultimate wind load

Critical Load Combinations
The load combinations normalised for long term application are:

Load Combinations	Factors	Factored Loads (kN)	k_1	D_L=N*/k_1
Dead (permanent)	1.25G	1.25.20 = 25kN	0.57	43.9
Dead + Live (construction)	1.25G + 1.5Q	1.25.20 + 1.5.5.5 = 33.3kN	0.77	43.2
Dead + Ultimate Wind Load (compression)	0.9G – 1.5W_u	0.9.20 – 1.5.26.6 = 21.9kN	1.15	19.00
Dead + Ultimate Wind Load (tension)	1.25G + W_u	1.25.20 + 6.6	1.15	31.6

When the wind action is opposite to the gravity loads 0.9 x G is taken as resisting, not 1.25.G.

The **critical load** is the dead load with **D_L=43.9kN**. The **connection** will be **designed for N*** = **25kN** with **k_1 = 0.57**

Connection Type
The spliced joint will result in the nails being in **single shear** in a **Type 1 joint.**

Connector
The type of connector and its diameter, i.e. **2.8mm diameter gun driven nails**, has been defined. Hence, t_p>10D>28mm and t_o >1.5D = 4.2mm<12mm.

Connector length = 12 + 28 = **40mm (minimum).**

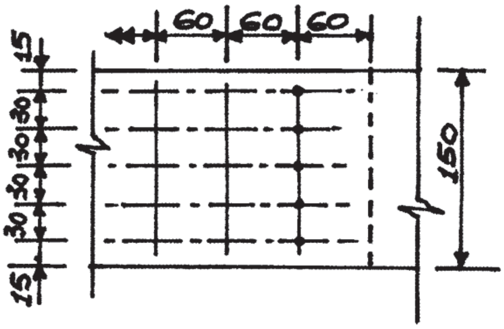
Number of Connectors/Row
The following distances have to be satisfied for nails driven into timber which has **not** been **pre-bored**.

Distance	Dimension	Minimum	Actual
End distance	20D	56	60
Edge distance	5D	14	15
Along grain spacing	20D	56	60
Across grain spacing	10D	28	30

Sketch of Joint
Knowing the cross-sectional dimensions of the member and the nail diameter allows:

Maximum number of **nails/row** to be determined. In this case:

$$\mathbf{n_r = 5}$$



Connector Capacity Factors

- **Capacity factor (Φ)** will be chosen based on the member being a part of a primary structural system.

$$\Phi = 0.8$$ **Table 2.6**

- Critical load is the **dead load:**

$$k_1 = 0.57$$

- **Grain orientation factor** for Type 1 nailed joints:

$$k_{13} = 1.0$$

- Because the gusset plates are plywood the **nails** will be in **single shear.**

$$k_{14} = 1.0$$

- For nails driven through **plywood:**

$$k_{16} = 1.1$$

- An assumed value for k_{17} is:

$$k_{17} = 0.9$$

- Since the LVL is to be of joint strength group JD4 the **characteristic capacity** of a **single 2.8mm diameter nail** driven into JD4 timber is:

$$Q_k = 665\text{N}$$

Number of Nails
From Equation 13.3:

$$n = \frac{N^*}{\phi k_1 k_{13} k_{14} k_{16} k_{17} Q_k}$$

$$= \frac{25 \times 10^3}{0.8 \times 0.57 \times 1.0 \times 1.0 \times 1.1 \times 0.9 \times 665}$$

$$n = 42 \text{ nails each side}$$

Number of Rows
The **number of rows** of nails can be found from:

$$n_a = \frac{n}{n_r} = \frac{42}{5}$$

$$n_a = 9 \text{ rows}$$

The assumed value of **0.9** for **k_{17} is satisfactory** since it applies for up to 10 rows.

Figure 13.7 shows the connection with the nailing pattern chosen.

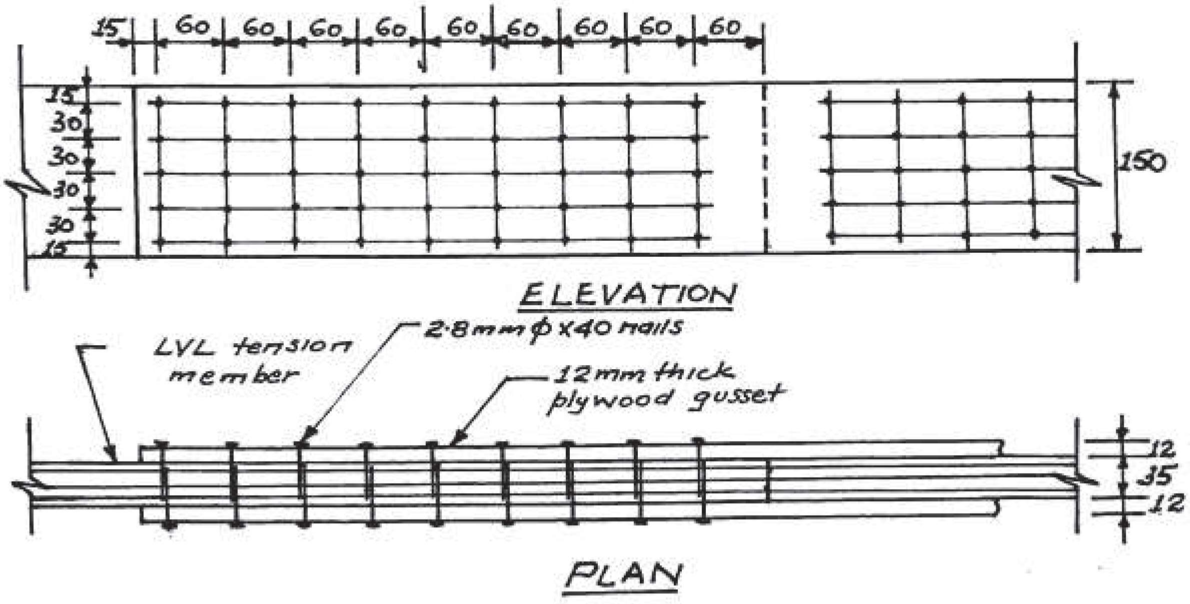


FIGURE 13.7: LVL / Plywood spliced joint

13.8 Design of Screwed Connections

Wood screws behave similarly to **nails,** the main **difference** being in **withdrawal.** The interlocking of wood fibre between the threads results in screws having higher withdrawal resistances than plane shanked nails whose capacities are given in Table 4.2 of AS 1720.1-1997.

Characteristic capacities for **single screws** are given in Tables 4.5(A) and (B), 4.6(A) and (B) and 4.7 of AS 1720.1-1997 for the **various loading conditions** and whether the timber is **unseasoned** or **seasoned.** These values are based on the **shank diameter and not** the diameter at the **root of the thread** which would result in a decided **decrease** in **section modulus**. In certain applications this may need to be taken into consideration.

The previously mentioned **characteristic capacities** also **apply** to **Type 17 self-drilling steel wood screws** manufactured to AS 3566.

The **spacing** of **screws** conforms to the provisions given for **nails** driven into **pre-bored holes.**

13.9 Screwed Connector Design – Methodology

The design process regarding **screw connection design** follows the same steps outlined in Section 13.4.

13.10 Design of Type 1 Screwed Connection (Cℓ.4.3.3)

Equation 13.8 gives the **design capacity (ΦN_j)** for a **Type 1 Joint** containing **(n) screws** required to resist the load.

For the **strength limit state** to be satisfied:

$$\Phi N_j \geq N^* \quad \textbf{(13.8)}$$

where:

$$\Phi N_j = \Phi k_1 k_{13} k_{14} k_{16} k_{17} n Q k \quad \textbf{(13.9)}$$

and

ΦN_j = **design capacity** of the connection
Φ = **capacity factor**;
k_1 = the **duration of load factor** for joints;

k_{13} = 1.0 for screws in **side grain**;
= 0.6 for screws in **end grain**.

k_{14} = 1.0 for screws in **single shear**;
= 2.0 for screws in **double shear.**

k_{16} = 1.2 for close fitting screws **metal side plates** of adequate strength to transfer the load
= 1.1 for screws through **plywood**
= 1.0 otherwise.

k_{17} = factor for **multiple screwed joints** for Type 1 connections designed to resist **direct loads** in either **tension** or **compression**

n = **total number of screws** in the connection resisting the design action effect in **shear**.

Q_k = **screw characteristic capacity** given in Tables 4.5(A) and 4.5(B) in AS1720.1–1997.

N* = **design action effect** due to application of factored loads.

13.11 Design of Type 2 Screwed Connections

As mentioned previously **Type 2 screwed** connections **differ** from **nailed** connections in **one major aspect,** i.e.:

- **Nails depend** on **friction** between the shank and the wood fibres to **resist withdrawal;**

- **Screws** depend upon a **mechanical interlocking** of wood fibre between threads thus **enhancing** the **withdrawal capabilities** of the screw over the nail.

Equation 13.10 gives the **design capacity (ΦN_j)** for a Type 2 joint containing (**n**) **screws.**

For the **strength limit state** to be satisfied:

$$\Phi N_j \geq N^* \quad \textbf{(13.10)}$$

where ΦN_j is the lesser of:

$$\Phi N_j = \Phi k_{13} . \ell_p .nQk \quad \textbf{(13.11)}$$

OR

$$\Phi N_j = n(\Phi N_{ts}) \quad \textbf{(13.12)}$$

where:

ΦN_j = joint capacity of **Type 2 screwed connection**, i.e. along connector axis.

and:

N^* = design action on a Type 2 nailed connection, along connector axis. due to factored loads applied to the joints;

Φ = capacity factor.

k_{13} = 1.0 for **withdrawal** from **side grain**;
= 0.6 for **withdrawal** from **end grain**;

ℓp = depth of **screw penetration** (mm) into primary member.

N = **total number** of screws in joint.

Qk = characteristic **screw capacity** in withdrawal given in Tables 4.6(A) and 4.6(B).strength group.

ΦN_{ts} = design **tensile capacity** of screw as per screw manufacturers specification

NOTE:
k_1 does not apply to screws subject to withdrawal

Design of Screwed Moment Joints

Screwed moment connections are **not common**, the **nailed option** being preferred because of their **lower installation cost**, and similar lateral load capabilities.

In the event a screwed joint provides the desired solution to the connection problem the procedure presented in **Error! Reference source not found.** of this Manual should be followed.

Serviceability Requirements for Type 1 Screwed Joints

AS 1720.1-1997 **does not differentiate** between nails and screws regarding joint deformations even though intuitively one may feel a screwed and nailed joint of identical construction would result in the screwed joint being stiffer.

13.12 Design of Bolted Connections

Although the **basic philosophy** for the design of **nailed and bolted joints** is similar, there are **some differences** that need to be recognised, particularly with regard to the **modification factors.**

In FIGURE 13.4 the importance of **direction of load application** relative to **nominal grain direction** has already been highlighted. In a **nailed** (or screwed) **connection timber thickness** is aligned with the **depth of nail penetration** required to develop the **full strength** of the connector.

In a **bolted connection** the **bolt capacity** is presented as a **function of timber thickness,** which in the tabulated data of AS 1720.1-1997, is referred to as the **effective thickness (b_{eff}).**

FIGURE 13.8 **defines (b_{eff})** for loads **parallel and perpendicular** to the grain in **seasoned and unseasoned timber.**

For **Type 1** bolted connections the contents of FIGURE 13.8 can be summarized thus:

- for **loads parallel to grain (b_{eff})** is the **smallest aggregate cross-section** of members **loaded parallel** to **grain:**

- for **loads perpendicular to grain (b_{eff})** is the **aggregate cross-section** of the **elements in the member** with **loads perpendicular to grain.**

The **characteristic strength** of a **single bolt** in a Type 1 timber connection is a function of a number of variables:

- **bolt diameter** – M6 to M36.
  In **seasoned timber** the **bolt hole** is the **nominal diameter** of the bolt.
  In **unseasoned timber** the **bolt hole** is **10 to 15% oversize.**

- **timber joint strength group** – J1 to J6 and JD1 to JD6;

- **timber effective thickness** – in AS 1720.1-1997 – 25 to 200mm **unseasoned** and 25 to 120mm **seasoned;**

- **moisture content**;

- **angle** between **force application** and the **grain direction;**

- **bolt spacings** – **edge, end, along and across grain** to prevent splitting and allow development of the full bolt capacity.

Type 2 bolted connections **do not depend** on **timber embedment** of the bolt for load transfer and are therefore largely **independent** of **timber thickness**. Type 2 joints depend upon:

- **bolt tensile strength;**

- **crushing strength** of the **timber** under the **washers** at each end of the bolt.

13.13 Modification Factors – Bolted Joints

Modification factors applied to bolted connection design perform a **similar function,** and take the same form, as those used in **nailed connection design.** However, a number of the factors **relevant to nailed connections** are **not relevant to bolted connection** design, e.g. the factors **k_{13} and k_{14}.**

The reasons $\mathbf{k_{13}}$ and $\mathbf{k_{14}}$ are not considered to influence bolted joint response is because:

- $\mathbf{k_{13}}$, the **grain orientation factor** for **nails and screws,** account for **frictional forces** due to the **way they are installed.** For **similar reasons** these forces are **not present** in **bolted connections.**

- $\mathbf{k_{14}}$**,** the **shear planes factor** for **nails and screws,** is accounted for in bolted connections by the **system capacity quantity** $\mathbf{Q_{skl}}$ **or** $\mathbf{Q_{skp}}$ of FIGURE 13.8.

Other factors which are **common** to both **nailed and bolted connections** are:

- **Capacity factor (Φ)** which performs the same function it did for nailed and screwed joints. However, **(Φ)** is **lower** for **bolted** connections for a number of reasons. Not the least of these is due to the **high local forces** produced in the timber by the bolt which **makes maintaining** its **load carrying capabilities** in the vicinity of local defects **more suspect** than for a **group of nails.**

- **Duration of load factor (**$\mathbf{k_1}$**)** is the **same** as defined in Section 13.3 for **nailed connections.**

- **Head fixity factor (**$\mathbf{k_{16}}$**)** applied to bolts is similar to that described for nails. **No increase** is allowed for bolts through **plywood** side plates, **only steel.** This increase is with the **proviso** that $\mathbf{b_{eff}}$ for **loads parallel** to the **grain** is $\mathbf{b_{eff}}$ >**5D** and **perpendicular** to **grain** is $\mathbf{b_{eff}}$**>10D.**

Multiple bolt factor ($\mathbf{k_{17}}$**)** differs from that applied to nail connections due to the **huge penalty** imposed on **bolted joints** in **unseasoned timber** with **transverse restraint.** TABLE 13.5 lists values of $\mathbf{k_{17}}$ for varying number of **rows of bolts** (n_a).

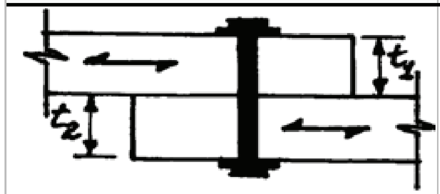
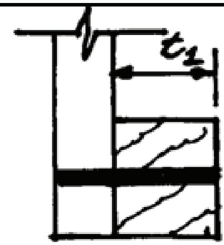
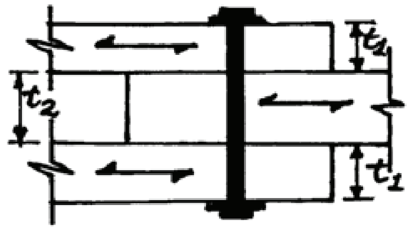
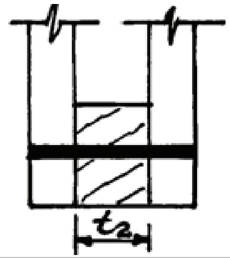
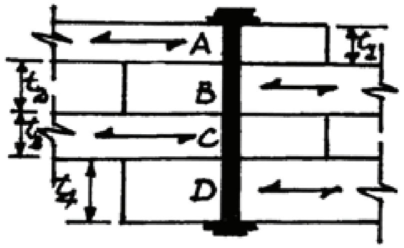
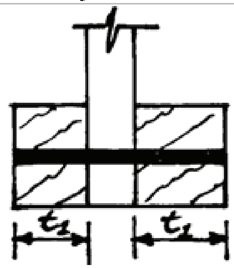
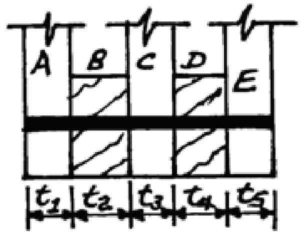
CHARACTERISTIC CAPACITIES & EFFECTIVE TIMBER THICKNESS FOR SINGLE BOLTS					
Parallel to Grain			Perpendicular to Grain		
Type of Joint	Effective thickness (b_{eff})	System Capacities $Q_{sk\ell}$	Type of Joint	Effective thickness (b_{eff})	System Capacities Q_{skp}
	b_{eff} = smaller of t_1 and t_2	$Q_{k\ell}$		$b_{eff} = t_1$	Q_{kp}
	b_{eff} = smaller of t_2 and $2t_1$	$2Q_{k\ell}$		$b_{eff} = t_2$	$2Q_{kp}$
	• Between A and B: b_{eff} = smaller of t_1 and t_2 • B to C: b_{eff} = smaller of t_2 and t_3; • etc.	• $Q_{k\ell}$ • etc • Qskℓ = sum of basic loads		$b_{eff} = 2t_1$	$2Q_{kp}$
				• A to B :$b_{eff} = t_2$; • B to C: $b_{eff} = t_2$ • C to D: $b_{eff} = t_4$	• Q_{kp} • etc sum of basic loads

FIGURE 13.8: Gives system capacities and effective timber thicknesses

Type of Joint	Values of k_{17}				
	$n_a \leq 4$	$n_a=5$	$n_a=10$	$n_a=15$	$n_a \geq 16$
Seasoned Timber	1.0	1.0	1.0	1.0	1.0
Unseasoned Timber (no transverse restraint)	1.0	0.95	0.80	0.55	0.5
Unseasoned Timber (transverse restraint)	0.5	0.5	0.5	0.5	0.5

TABLE 13.6: Values of k_{17} for bolts and coach screws

The **designer** must **closely examine** the joint configuration to assess the likelihood of some of the **timber elements drying out** (if unseasoned) during their design life.

If **all** of the **timber elements** of the system are **seasoned**, and **remain so**, there should be **no problems** with **restraint stresses**.

When **one member** of a system **can shrink** and the **lateral movement** of that member is **restrained** through **connection** to other members which are **stable**, as shown in FIGURE 13.9, **extraneous stresses** will be induced into the system.

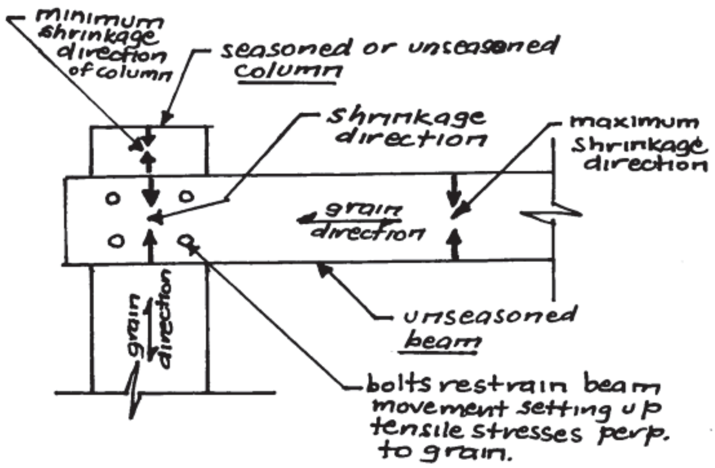


FIGURE 13.9: Lateral restraint stresses

13.14 Bolted connection Design – Methodology

- **Sketch** or **draw to scale** a typical connection which will allow the **angle of** the **force to** the **grain**, **acting on the bolt** for **each member** of the joint, to be determined. This will allow the **joint type** to be **identified.**

- Select $\mathbf{k_{16}}$ based on whether the bolts pass through **tight fitting holes** in **steel side plates** (if necessary) or otherwise.

- For **each force component parallel** or **perpendicular** to the grain **match** the **configuration** to a diagram of FIGURE 13.8. This allows $\mathbf{b_{eff}}$ to be determined, hence $Q_{sk\ell}$ and/or $\mathbf{Q_{skp}}$ the **sum** of the **individual characteristic** loads for the system, can be found.

- Select a **connector diameter** based on analysis, availability, etc.;

- Determine the **characteristic strength** for:

    - **angle** of **bolt reactive force to** the **grain,** i.e. parallel or perpendicular
    - $\mathbf{b_{eff}}$ for each **member/bolt interface**;

- **Evaluate modification factors** except for $\mathbf{k_{17}}$ which has to be **assumed conservatively** initially for inclusion in the relationship:

$$\Phi N_j = \Phi k_1 k_{16} k_{17} n Q_{sk}$$

- Find the **number of connectors/row** that can be accommodated without violating spacing requirements. The **sketch/scale drawing** will again prove very **useful.**

- Calculate the **number of rows (n_a)** of bolts required.

- Check **k_{17}** is satisfactory through reference to TABLE 13.5. If not re-calculate.

- **Detail** the connection which should be very close to being completed.

13.15 Design of a Type 1 Bolted Connection (Cℓ.4.4.3)

Equation 13.13 gives the **design capacity (ΦN_j)** for a Type 1 joint containing **(n) bolts** to resist the **applied lateral loads.**

For the **strength limit state** to be **satisfied:**

$$\Phi N_j \geq N^* \qquad \textbf{(13.13)}$$

where:

$$\Phi N_j = \Phi k_1 k_{16} k_{17} n Q_{sk} \qquad \textbf{(13.14)}$$

and:

N^* = **design action** effect due to **application of factored loads**
Φ = **capacity factor;**
k_1 = **Duration of load factor** for joints;
k_{16} = **Head fixity factor**
= **1.2** for bolts through **tight fitting holes** in **thick steel plat**es;
= **1.0** for **other cases**

k_{17} = **Multiple bolt factor**
n = **total number of bolts** resisting applied loads shear;
Q_{sk} = bolt **characteristic capacities** as determined by reference FIGURE 13.8**.**

13.16 Design of Type 2 Bolted Connections

Equation 13.15 gives the **design capacity (ΦN_j)** for a **type 2** joint containing (**n**) **bolts** which are **loaded** in **direct tension.**

For the **strength limit state** to be satisfied:

$$\Phi N_j \geq N^* \quad \textbf{(13.15)}$$

where ΦNj is the lesser of:

$$\Phi N_j = n(\Phi_{Ntb}) \quad \textbf{(13.16)}$$

OR where **crushing under** the **washer** results in a limit on strength :

$$\Phi N_j = \Phi.k_1.k_7.nf'pj.Aw \quad (13.17)$$

and:

N^*	= **design action** due to **factored tensile loads**	
N	= **number of bolts** in the joint	
ΦN_{tb}	= **design tensile capacity** of bolts	**(Table 4.12)**
Φ	= **capacity factor**	
k_1	= **duration of load factor**	
K_7	= **length** of **bearing factor** of **washer**	**(Table 4.12)**
f'_{pj}	= **characteristic bearing strength** of timber in joints	(**Table C6)**
A_w	= **effective area** of **washer** for **bearing**.	

Moment Resisting and Eccentric Bolted Joints

AS 17201.1-1997 only **real concern** regarding **moment joints** for bolted connections is that associated with **joint eccentricity. No guidance** is given concerning the design of **bolted moment joints** required to sustain **large applied moments** as can occur, for example, in **portal frame knee joints.**

The design of **bolted moment joints** incorporating **rigid steel side plates** can be effected by application of the classical mechanics formula $\tau = T\rho/J$. However, the **objective** of this Manual is to **provide guidance to designers** using **plywood and LVL** and moment joints with these materials are best done using **nails** as the connector as described in Chapter 10.

Eccentric joints arise when the **centre lines of** action of their member forces, for example, those of a **truss joint do not intersect at a common point** as shown in FIGURE 13.10. This indiscretion can cause **fairly high shear and moments** to develop and **tensile stresses perpendicular** to the grain may also be high.

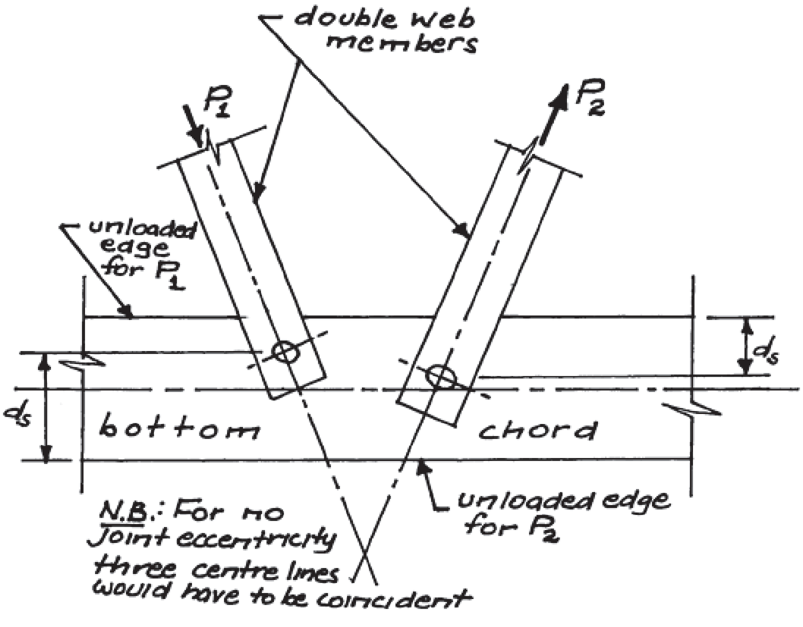


FIGURE 13.10: Eccentric joint

Because of the **lines of actions** of the (**3**) **forces** are **not concurrent** the **connection** is **now treated as two separate connections**. The **bending shear** is then **determined for each joint through** the **application** of the **classic bending shear equation**:

$$\mathbf{f_s = VQ / It}$$

This results in the A_{sj} term containing the 2/3rds of shear area, b x d_s.

To accommodate this type of situation AS 1720.1-1997 recommends the **secondary stresses** due to **bending moment** be checked to ensure **no member** or **fastener** is **overstressed.**

Further, the **design capacity** in **transverse shear** at an eccentric joint (ΦV_{sj}) satisfies Equations 13.18 and 13.19.

$$\Phi V_{sj} \geq V^*_{sj} \qquad (13.18)$$

where:

$$\Phi V_{sj} = \Phi k_1 k_4 k_6 f''_{sj} A_{sj} \qquad (13.19)$$

and:

V^*_{sj} = de**sign action** on the joint due to the **factored applied loads**, i.e. **transverse shear** at **joint**
Φ = **capacity factor**
k_1 = **duration of load** factor
k_4 = **partial seasoning** factor
k_6 = **temperature effects** factor
f'_{sj} =**characteristic strength** in shear at joint details appropriate to **species strength group**
A_{sj} = **transverse shear plane** area at joint;
= 2.b.ds/3 where **b** is **thickness** of the member. See FIGURE 13.10. for **d_s.**

Washers

AS 1720.1-1997 states **all timber-to-timber bolted structural joints** shall be fitted with a **washer each end.**

The **function** of the **washers** in a bolted structural connection is two-fold:

- having a **larger diameter** than the head and nut of the bolt, they distribute an **axial force** in the bolt over a **larger area;**

- provided the bolt is kept **tight** to combat shrinkage the **washer** can **minimise water penetrating** into the bolt hole. This reduces the possibility of **rust** of the bolt and **rotting** of the timber.

Serviceability Requirements for Type 1 Bolted Joints (Cℓ.C3.3)

AS 1720.1-1997 provides relationships to determine connection deformations of **solid timber joints** fabricated **with bolts** as the connectors. The equations provide **estimates** of displacements if **no test data** for the connection response is available.

The equations give reasonable results for the deflection of Type 1 joints **under serviceability loadings.** Joints become **less stiff** after a number of **load cycles** resulting in the **deformation predictions** become **less accurate.**

Equations 13.20 and 13.21 give the **displacement** Δ**,** taking into **account grain direction.**

$$\Delta = \Delta_i + \frac{j_{14}}{j_{33}} \frac{Q^*}{Q_{k\ell}} \quad \text{for loads parallel to grain} \qquad (13.20)$$

$$\Delta = \Delta i + \frac{j_{14}}{h_{33} x h_{35}} \quad \frac{Q^*}{Q_{kp}} \quad \text{for loads perp' to grain} \qquad \textbf{(13.21)}$$

where:

Δ = **deformation** (mm) of a **single bolt** in a Type 1 joint
j_{14} = **duration** of **load factor** for bolted joints;
h_{33} = **stiffness factor**
h_{35} = **1.5** for **first (3) joints** of FIGURE 13.8.
= **2.5** for **multiple member connections**, the **fourth** joint in FIGURE 13.8.
Δ_i = **initial displacement** of joint due to **oversize holes**
N_{con} = total **number of bolts** in the connection
Q* = **serviceability load effect (N)** parallel or perpendicular to the grain for a **single bolt**
$Q_{k\ell}$ = **characteristic strength** of **bolt** parallel to grain (**N**)
Q_{kp} = **characteristic strength** of **bolt** perpendicular to grain (**N**)

13.17 Bolted Connection - Design Example

The connections in a roof truss provide the opportunity to expose a number of important factors regarding bolted joint design. Therefore, in this example the heel joint of a truss will be designed.

A roof truss having the geometry shown in Figure 13.11 is to be featured in a commercial building to be constructed on Queensland's Gold Coast. The truss has been designed, but LVL with A faces for appearance, is being considered as an alternative. The joints of the truss are to be bolted using M12 galvanised bolts. The design load is to be taken as the load in the top chord.

The **critical load combination** for the strength limit state is to be:

nominal dead load : G = 8kN (axial **compression top chord**)
nominal live load : Q = 6kN (duration of load 5 days) axial **compression top chord**

The LVL for the single top chord is 150 x 45mm and for the double bottom chard 150 x 35mm. The joint strength group of the LVL is JD3.

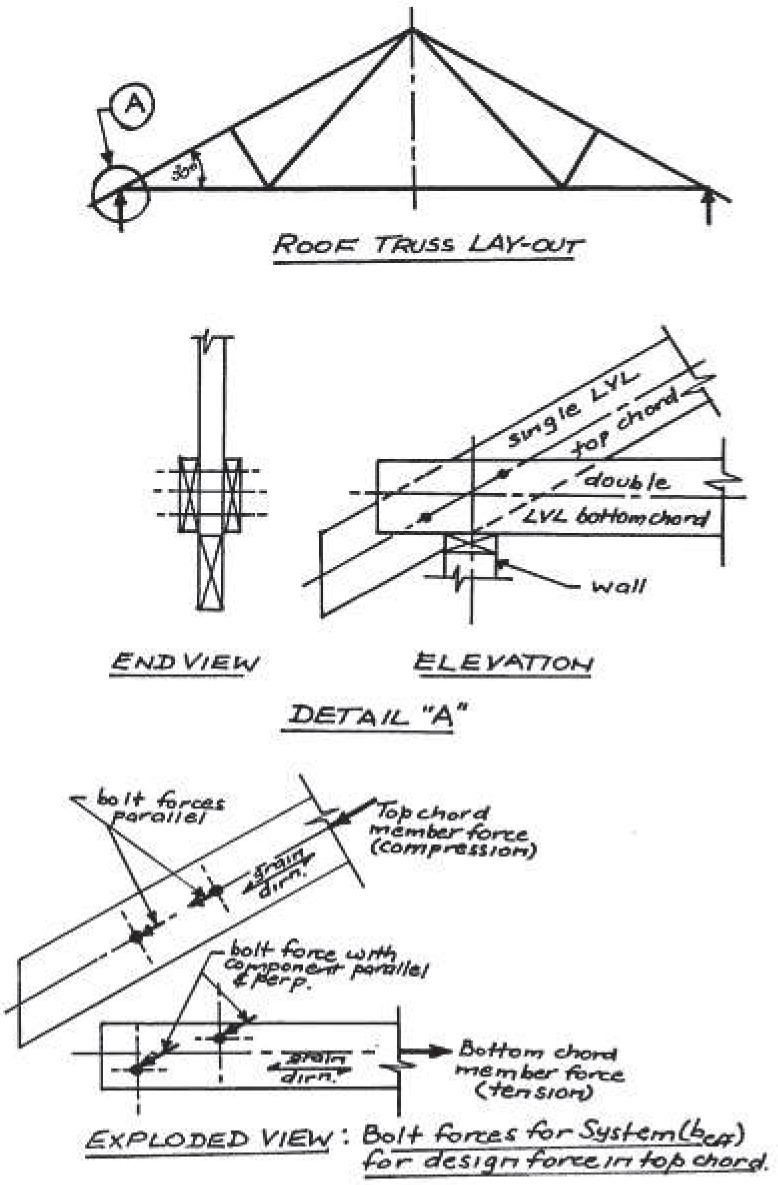


FIGURE 13.11: Bolted truss joint

Bolted Connection - Worked Example

FIGURE 13.11 shows the truss plus a detail of the heel joint, but most importantly it gives an **exploded view of the joint,** showing **bolt force directions relative** to the **grain direction.** The **design force** in the **top chord member** will be **equilibrated by a vertical force at** the **wall plate** and a **horizontal force in** the bottom **chord members.** This results in the **total design force** passing **through** the **connection.** For this loading the **force** in the **bottom chord members** due to the applied load is of **no concern for this exercise.**

Critical Limit State Design Load

The critical limit state load for strength is due to **dead and live load combination** applied to the **top chord.**

$N^* = 1.25.8 + 1.5.6.0$

$\mathbf{N^* = 19kN}$

Connector Capacity Factors:

For the **live load** being applied for **5 days:**

$\mathbf{k_1 = 0.77}$

Because there are **no rigid steel side plates:**

$\mathbf{k_{16} = 1.0}$

The **exploded view** in Figure 13.11 shows **bolt forces angle** to the **grain** to be:

θ = **0°** for **top chord;**

θ = **30°** for **bottom chord.**

The truss is a primary structural component hence its connections will assume the same status. **Capacity factor** will be:

Φ = **0.65**

Defining System Category for (b_{eff}) and Q_{sk}:

For **bolt loads parallel to grain**, it can be seen from the **exploded view** this will apply to:

- **top chord member** with full bolt load;
- **bottom chord member** with a **component.**

A **(3) member system loaded parallel to grain** has (**b_{eff}**) and **$Q_{sk\ell}$** defined by the **middle diagram** of FIGURE 13.8. **b_{eff}** is the **smaller of 45mm or 2 x 35mm = 70mm;**

b_{eff} = **45mm** for **top chord;**

$Q_{sk\ell}$ = **$2Q_{k\ell}$**

For **JD3 LVL** and **12mm Φ bolts:**

$Q_{k\ell}$ = 11900N **Table 4.9(c)**

$Q_{sk\ell}$ = 2 x 11.9

= 23.8kN

For bolt loads perpendicular to grain:

A (3) member system loaded perpendicular to the grain has (b_{eff}) and Q_{skp} defined by the third joint down in FIGURE 13.8.

b_{eff} = 2 x 35mm

= 70mm for bottom chord

Q_{kp} = 7410N **Table 4.10**

Q_{skp} = 2 x 7.41

= 14.28kN

Bolt Loads at an Angle to Grain:

The **bolt forces** in the **bottom chord** are at an angle of **30°** to the **grain direction.**

> **NOTE:**
> This shows the importance of the exploded view showing there is a component of bolt force perpendicular to the grain.

From **Hankinson's Formula:**

$$Q_{sk\theta} = \frac{Q_{sk\ell} \cdot Q_{skp}}{Q_{sk\ell} \cdot \sin^2\theta + Q_{skp}\cos^2\theta}$$

$$= \frac{23 \cdot 8 \times 14 \cdot 28}{23 \cdot 8 \sin^2 30^\circ + 14 \cdot 28 . \cos^2 30^\circ}$$

$$Q_{sk\theta} = 20 \cdot 4\,\text{kN}$$

Number of Bolts:

The **joint capacity** is **determined** by the **lower bolt capacity** in the **bottom chord.** Hence, the **critical connection load** will be:

$Q_{sk\theta}$ = **20.4kN**

The **number (n)** of bolts required:

$$n = \frac{N_j^*}{\varphi k_1 k_{16} k_{17} Q_{sk}}$$

Assume k_{17} = 1.0

$$n = \frac{19}{0.65 \times 0.77 \times 1.0 \times 1.0 \times 20.4}$$

$$n = 1.86$$

$$= \text{say 2 bolts}$$

Number of Rows:

Number of rows of bolts (n_a):

$$\mathbf{n_a} = \frac{\mathbf{n}}{\mathbf{n_r}}$$

For 2 rows of bolts:

$$\mathbf{n_a} = \mathbf{2}$$

Number of bolts / row (n_r):

$$\mathbf{n_r} = \frac{\mathbf{2}}{\mathbf{2}} = \mathbf{1}$$

i.e. $\mathbf{n_r}$ = **1**, 2 rows, with 1 bolt / row

Joint Capacity Check

The joint capacity, in this instance, is controlled by the **bolt capacity perpendicular** to the **grain** in the **bottom chord**, i.e. $\mathbf{Q_{sk}}$ **= 20.4kN**

Design capacity for joint $= \mathbf{\Phi N_j}$
$= \mathbf{\Phi k_1 k_{16} k_{17} n Q_{sk}}$
= 0.65 x 0.77 x 1.0. x 1.0 x 2 x 20.4

$\mathbf{\Phi N_j}$ = **20.4kN ≥ N_j* so OK**

Joint Geometry

To develop **full joint capacity** the **bolts must be located** such that **end, edge and bolt spacings** satisfy the requirements set by AS 1720.1-1997. These are:

end distance (**tension** member)	7D	84
end distance (**compression** member)	5D	60
edge distance	2D	24
spacing (**parallel** to grain)	5D	60
Spacing (perpendicular to grain)	4D	48

FIGURE 13.12 shows these **distances and spacings satisfying** the necessary **requirements.** The **dashed hatched** area is within the **edge, end and spacing between bolts distances.**

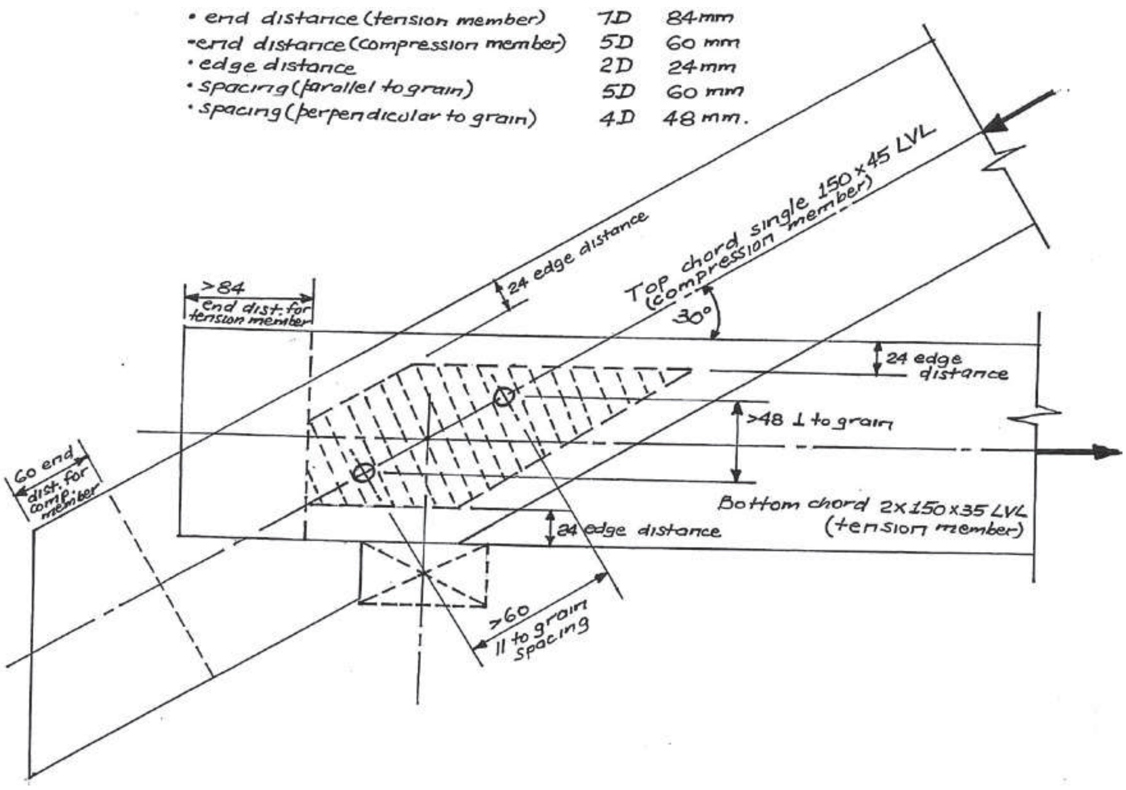


FIGURE 13.12: Edge, end and bolt spacings

13.18 Design of Coach Screwed Connections (Cℓ.4.5.2)

The **coach screw** has the **hexagon head** of a bolt (as do some wood screws) but the **shank** of a **wood screw** as shown in FIGURE 13.13. The **pitch** of the thread of the **coach screw** is much **coarser** than that for a bolt.

Although the **coach screw** has a **strong resemblance** to a **screw,** for **design purposes**, it is **categorised with bolts.**

13.19 Design of Type 1 Coach Screw Connections

Characteristic capacities for **coach screws** loaded **laterally** in **shear** in **side grain** can assume the values given for bolts (C1.4.4.2) provided:

- coach screw **diameter** is that of its **shank not** the **core diameter** (bottom of thread). In critical loading cases it may be wise to take the **core diameter** for determination of **characteristic capacities;**
- coach screw is fitted with a **washer;**
- for a **two-member joint** the **thinner member** must have a **thickness** (t_t):

$$t_t \geq 3D_s$$

where: D_s = **shank diameter** (mm);

- **hole** for **shank:** = **(D_s+1mm)** or **(D_s+0.1D_s)** whichever is **lesser;**

  hole for **threaded section:** ≤ **core diameter;**

  depth of hole: ≥ **length of screw**

- depth of **coach screw penetration(t_p)** into the **second member** for various **species** groups is given in FIGURE 13.13.

For **lesser** values of t_p **reduce load proportionally** to decrease in t_p, **until $t_p = 4D_s$, after** which **coach screw** is **non-load bearing.**

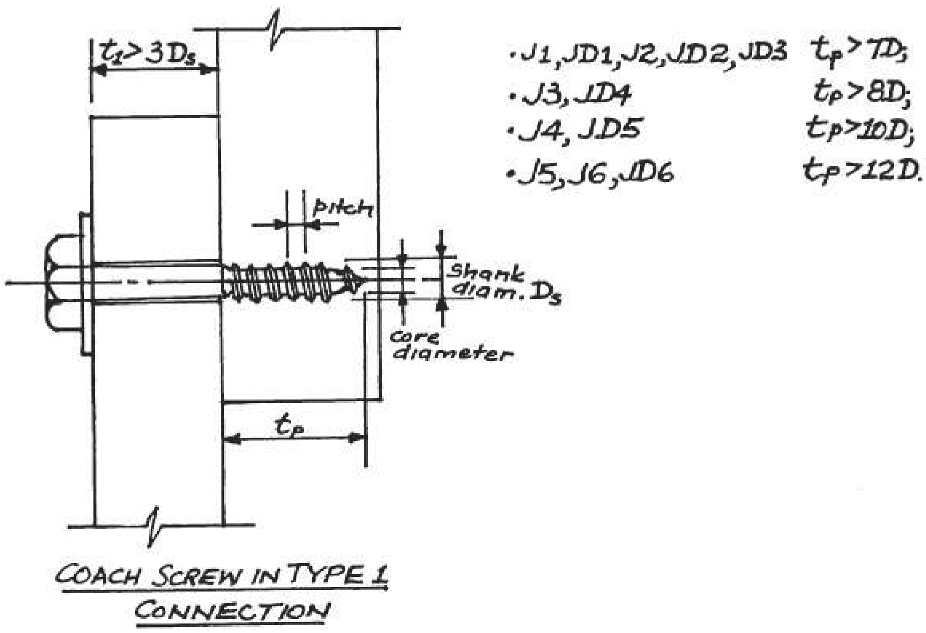


FIGURE 13.13: Coach screw depth of penetration and timber thickness

For **lateral loads** in **end grain** as shown in FIGURE 13.1(b):

- **characteristic capacities** must **not exceed 60%** of values obtained for **lateral loads** in **side grain.**

13.20 Design Capacity of Type 1 Coach Screwed Joints (Cℓ.4.5.3)

Equation 13.22 gives the **design capacity (ΦN_j)** for a Type 1 joint containing **(n) coach screws** to resist the applied load.

For the **strength limit state** to be satisfied:

$$\Phi N_j \geq N^* \quad \textbf{(13.22)}$$

where:

$$\Phi N_j = \Phi k_1 k_{13} k_{16} k_{17} n Q_{sk} \quad \textbf{(13.23)}$$

and:

N^* = **Design act**ion due to **applied factored loads** on the connection
Φ = **capacity factor**
k_{13} = 1.0 **withdrawal** from **side grain**;
= 0.6 **withdrawal** from **end grain**;

k_{16} = **head fixity factor**
= 1.2 for coach screws through rigid side plates;
= 1.0 others;

k_{17} = **multiple screw factor**
n = **total number** of **coach scre**ws resisting **applied load**
Qsk = **characteristic capacity** for a **single screw by Cl.4.4.2.4** and whose **innermost member thickness** is taken as **t_p**.

13.21 Design of Type 2 Coach Screwed Connections

Typically **Type 2 joints** result in the **connector** being subjected to **uniaxial tension.**

Load response characteristics of the **coach screw** closely **resembles** that of **screws** except for the **extra** possible **failure mode**, i.e.:

- the need for the **timber** to **resist crushing** under the **washer**

Equation 13.24 gives the design capacity (ΦN_j) for a Type 2 joint containing (**n**) **coach screws.** For the **strength limit state** to be satisfied:

$$\Phi N_j \geq N^* \quad (13.24)$$

where: ΦN_j is the **lesser** of

$$\Phi N_j = n(\Phi N_{tc}) \quad (13.25)$$

Or

$$\Phi N_j = \Phi k_{13} \ell_p . n Q_k \quad (13.26)$$

or where **crushing** under the **washer** may occur:

$$\Phi N_j = \Phi k_1 k_7 n f'_{pj} Q_k \quad (13.27)$$

where:

N^* = **design action effect** due to the application of **factored loads** causing tension **in the jo**int;
Φ = **capacity factor**;
N = **total number** of **coach screws** in joint;
ΦN_{tc} = tensile capacity of a single coach screw **(Table 4.14)**

k_{13} = grain orientation factor:
= **1.0** for **withdrawal** from **side grain**;
= **0.6** for withdrawal from **end grain**;

ℓ_p = depth of **screw penetration** into the **primary member**;

Q_k = **characteristic capacity** **(Table 4.13)**

k_1 = **duration of load factor** for **fasteners;**

k_7 = **length** of **bearing factor**, which for a washer is its **diameter** or **side length**
f'_{pj} = **characteristic bearing capacity** of **timber in joints**;
A_w = **effective area** of **washer** for **bearing.**

Note:
***k_1* *does* not apply *to coach screw* withdrawal capacity *as was the case for* screwed and nailed connections.**

Serviceability Requirements for Type 1 Coach Screwed Joints

AS 1720.1-1997 provides **no direct guidance** regarding **coach screw joint deformation.** Since the **structural response** of the **coach screw** is **closely allied** to that of the **bolt** it is not unreasonable to assume the **contents** of **C3.3** should also apply.

Coach Screwed Joint – Design Example

Because the **design methodology** described for **bolted joints applies** to **coach screwed joints no** design **example** is considered **necessary.**

13.22 Dowelled Connections

As mentioned in the **introduction** to **this chapter only dowelled connectors** have been **discussed. Dowelled connectors** were **defined** as those with a **circular cross-section,** e.g. nails, screws and bolts.

There is, however, **another connector** which is called a **dowel.** Its **structural response** is **similar** to that of a **bolt** and is best described as a **bolt** with **no head** and **no thread** for a **nut.**

The **main use** of the dowel is in the incorporation of **steel fin plates** in **truss joint design** where the **end** of the **timber member** is **slotted** to **fit over** the **steel projection.** The dowels are driven into tight fitting holes drilled through the timber and steel. Since the **design methodology** applied to bolts can be applied to dowels no further discussion is considered warranted.

13.23 Photographs

Appendix A13 gives some examples of joints designed to interconnect timber members. It should be noted not all of these connections display the ideal means of member jointing. In fact it is hoped they convey a range of images, i.e. from a virtual total lack of connectivity, through aesthetically displeasing, to interesting, functional and challenging. Since “beauty is said to be in the eye of the beholder” it is left to the reader to do their own categorization of the connections. However, in so doing it is further hoped something is gleaned from the exercise.

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A13 Chapter 13 Appendix

CONNECTIONS

Plate 1

Plate 2

Plate 3

Plate 4

Plate 5

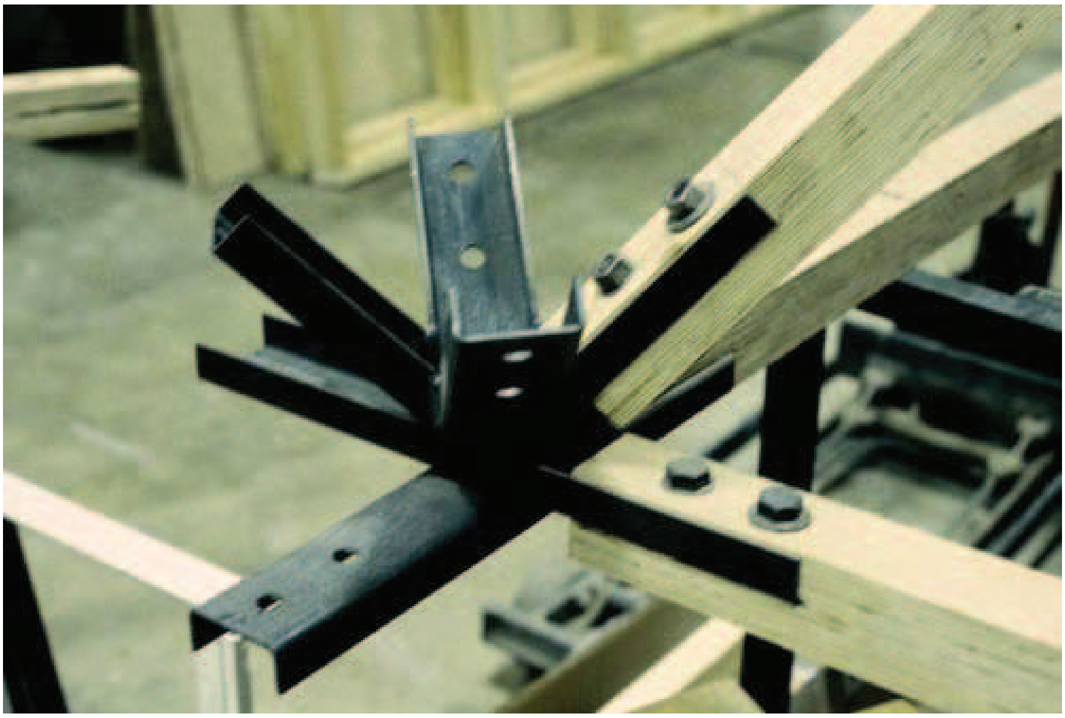

Plate 6

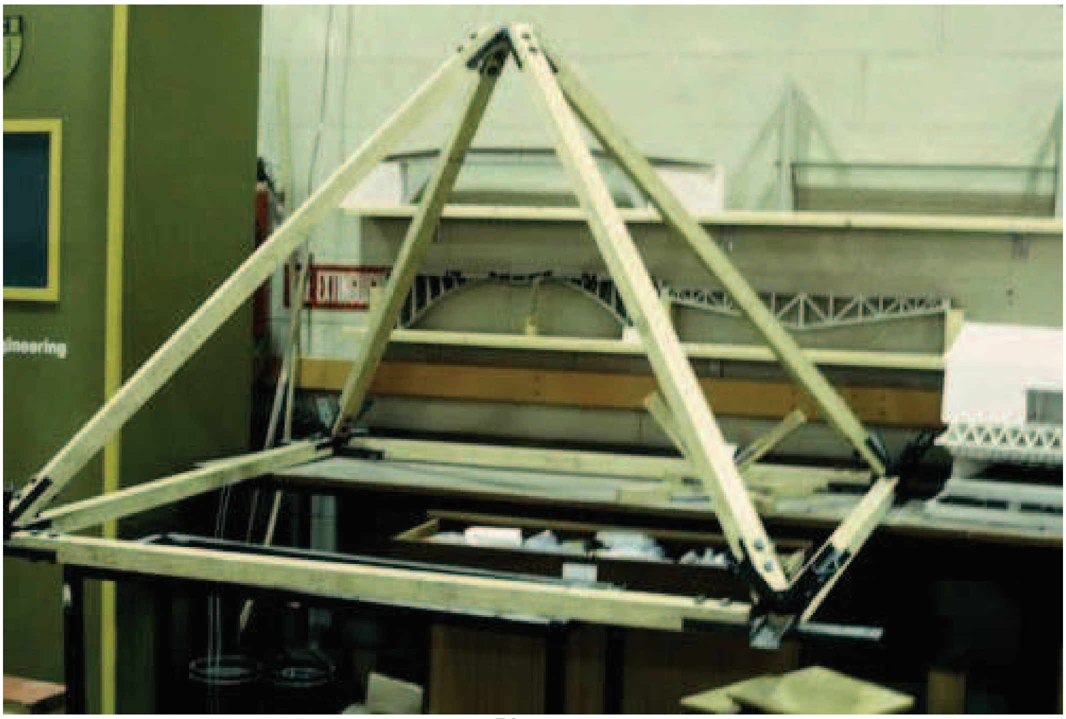

Plate 7

Plate 8

Part Five

Miscellaneous Topics

Noise Control

Condensation and Thermal Transmission

Resistance to Fire Decay and Bugs

Finishing

14 Noise Control

14.1 Introduction

The purpose of this chapter is to provide designers, conveniently located under one cover, with some **fundamental information on sound** and its unwelcome **by-product noise.** This is considered important because noise (or its control) has become a major issue as we are confronted, on a daily basis, with **closer living conditions** which alone poses many problems and **traffic noise**, to mention just two of the main contributors.

In this chapter **noise** assumes the **non-technical definition,** i.e. of being **those sounds found to be obnoxious to the ear of the recipient.** Such sounds may take many forms including the **children's** choice of **music,** those from being located in the **flight path** of aeroplanes or from **traffic** on a busy road to **squeaky floors** and other structure borne noises due to **impact** or **vibration.**

This chapter **does not** pretend to **offer designers a solution to all** of their **noise problems** but rather to make them more aware of their existence and a little better equipped to deal with them.

14.2 Nature of Sound

A **sound source transmits** the associated **noise** in **wave form**, analogous to the way in which a **pebble** dropped into a **still pond** of water, **propagates waves.**

Hence, **sound** presents itself as a **pressure wave** i.e. as a form of **mechanical energy.** To **reduce** the effect of the **noise source** it is **necessary to convert** the **energy** of the wave to **another form**, e.g. **heat energy** by making it work.

The **human ear** detects sound as **variations** in **air pressure** which are **measured** in units of **micro Newtons/metre (μN/m^2)** or **micro Pascals (μPa).**

The **amplitude (loudness)** of sound pressures **registered** by the **human ear** vary from **20 to 200 million μPa** which is within the **frequency range** of **20 to 20,000 Hz.** Because of this **wide pressure range** it is measured on a **logarithmic scale** known as the **decibel (dB) scale.**

FIGURE 14.1 gives a **scale** of **sounds** commonly encountered, together with approximate **dB values.**

	(dB)	
Threshold of pain —	140	
	90	— Jet aeroplane at 300m altitude
Highway traffic at 30m —	75	
	50	—Quiet restaurant
Residential area at night —	40	
	20	—Rustling of leaves
	0	—Threshold of hearing

FIGURE 14.1: Decibel scale

14.3 The "A – Weighted" Decibel (dBA)

Because the **human ear** is **not equally sensitive to all frequencies highway traffic noise** is measured using an **"A-Weighted"** approach. A-weighting **emphasizes** sound within the **frequency range 1000 to 6300Hz** and de-emphasizes sounds above and below these values.

14.4 Sound Pressure Level (SPL)

The **SPL** converts the **sound pressure (energy)** presented on a **logarithmic scale to decibels (dB)** given by Equations 14.1 and 14.2.

In terms of **pressure:**

$$SPL = 10.\log_{10}\left(\frac{p}{p_{ref}}\right)^2 \qquad \textbf{(14.1)}$$

where:

p = **sound pressure;**
p_{ref} = **reference sound pressure of 20μPa**

Sound energy is related to SPL thus:

$$\left(\frac{p}{p_{ref}}\right)^2 = 10^{(spl-10)} \qquad \textbf{(14.2)}$$

Because **decibels** are represented on a **logarithmic scale** they **cannot** be **added algebraically.** For example, say a **source produces 50dB** at a **receiver** and an **additional 50dB** was added to the source their **combined SPL** would **not be 100dB** at the receiver, but **53dB.** How this result was arrived at will be discussed in some detail in **Section 14.8**.

14.5 Transmission Loss (TL)

Transmission Loss is the ability of a **material** to **reduce** or **resist** the **transmission** of sound by absorption as shown in FIGURE 14.2.

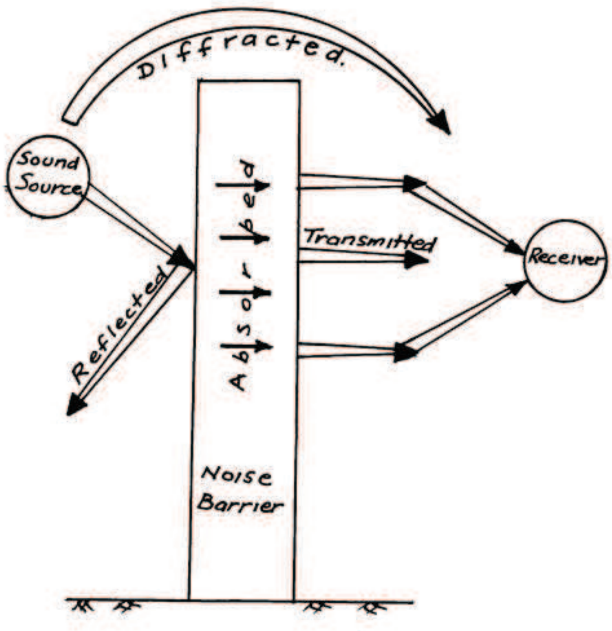

FIGURE 14.2: Sound response on meeting a barrier

The **absorption process** of a **single leaf panel** is a function of:

- panel **mass/m^2;**
- panel **bending stiffness** (or better its **lack thereof**);
- **frequency** of the **sound source;**

For a **sound source** of a **given frequency,** randomly incident to the panel, the **transmission loss** is:

$$TL = 20 \log (M.f) - 47.2dB \quad (14.3)$$

where:

M **=** mass of panel in **kg/m^2**;
f = **frequency** of source;

The relationship of **Equation 14.3** is **known** as the **Mass Law**. From Equation 14.3:

20 log 2 = 6 i.e.

- **doubling** the **mass increases TL** by **6dB;**
- **doubling** the **frequency increases TL** by **6dB.**

The consequence of the influence of **frequency response** on **TL** for a **single leaf partition** is shown in FIGURE 14.3.

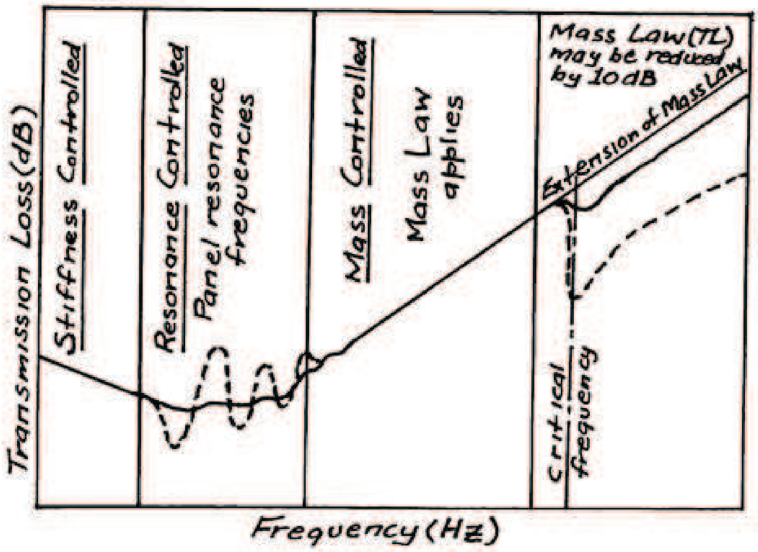

FIGURE 14.3: Shows various regions of performance for single leaf partition

14.6 Sound Transmission Reduction – Airborne & Impact

It can be seen from FIGURE 14.3, because of the **variation** in **magnitude** of **Transmission Loss (TL)** with **frequency,** difficulties are presented in the **assigning** of a **single number rating** to characterise the TL of the partition. However, a **single number rating** is desirable and the **Sound Transmission Class (STC)** used in Multi-Residential Timber Framed Construction 2, was to be such a number. The **STC** was a type of **average value** of **TL** over the **range of frequencies.**

The **STC** was **limited** in that it only **applied** to **walls insulating** against **speech**, i.e. **airborne sound,** or similar sound sources. **STC** was **not** really **suited** for **external wall** systems and even for **some** internal sound sources.

The **STC concept** was replaced by the **Weighted Sound Reduction Index** (**R_w**) for sound insulation against **airborne and impact noise** on **walls and floors** separating sole occupancy units. **R_w** better **accounted for** the **low frequency regime** of the **sound frequency distribution** than did the **STC.**

With **structure borne sounds,** i.e. **impact and vibration** the **Impact Isolation Class (IIC)** applies to **floor construction,** and is a **single number** rating the **effectiveness** of a **floor system** in providing **insulation against impact noise** such as **footsteps.**

The **IIC** system of impact rating has now been **replaced** by the **Weighted Standardised Impact Sound Pressure Level** ($L_{nt.w}$).

Because **building product** information from some sources (includes Multi-Residential Timber Framed Construction MRTFC 2) is still quoted in **IIC** the following **relationship** has been devised by the Association of Australian Acoustical Consultants to **allow conversion.**

$$L_{nt.w} = 110 - IIC$$

The effect of **holes**, **openings, and gaps** will significantly **downgrade** the acoustic **performance** a **wall.** Even **small air gaps** between panels affect performance. Doors and windows (both closed) incorporated in a wall system **change** its **insulation rating** quite dramatically.

14.7 Subtraction and Addition of Decibels

Subtraction:

When noise passes through a barrier, e.g. a plywood sound barrier, a **transmission loss** results. Assume the sound source to be a **truck** producing **70dBA** and the **plywood barrier** results in a **transmission loss** of **21dBA** then the **noise** received **through** the **barrier** is the **algebraic difference:**

70 – 21 = 46 dBA

Addition:

Decibels cannot be added algebraically. Addition of **decibels requires** the use of TABLE 14.1**.**

For combining two decibel levels of sound with random frequency characteristics	
Difference between levels (dB)	**Amount to be added to higher level (dB)**
0 or <1	3.0
1	2.5
2	2.1
3	1.8
4	1.5
5	1.2
6	1.0
7	0.8
8	0.6
9	0.5
10	0.4
>10	0.0

TABLE 14.1 : Addition of (dB's) to be added for various (dB) differences

As an example consider a **person** being **exposed** to a **sound pressure level** of **90dB** from **one source and 88dB** from **another source.**

The **resultant** total sound pressure is **not** the **algebraic sum**, i.e. **(90 + 88 = 178dB).**

To find the **combined sources intensity subtract** the **smaller** value **from** the **larger** to give:

90 – 88 = 2dB

From TABLE 14.1**,** the **difference** of **2dB** (left column) **results** in **2.1dB** being **added** to the **higher value,** i.e.

90 + 2.1 = 92.1dB

Rounded to the **nearest whole dB →92dB**

When it is required to **add more than two sound sources** they must be **arranged** in **numerically increasing order.**

For example, to add: 88dB**,** 89dB, 84dB and 86dB.

Arranging in **numerically increasing order**: 84, 86, 88, 89

For: 84 + 86
= **difference of 2dB.**

From TABLE 14.1**, 2.1 dB** is to be added to 86 to give 88.1dB.

i.e. 88.1 **rounded to 88dB.**

Add 88 to 88dB giving a **difference of 0.**

From TABLE 14.1**, 3 dB** is to be added to 88 to give 91dB.

Add 89 to 91dB giving a **difference of 2**

From TABLE 14.1**, 2.1 dB** is to be added to 91 to give **93.1dB**.

i.e. 93.1 **rounded to 93dB**

14.8 Sound Barriers (from Ref. 1) - Design Example

There are **two designs** to **reduce traffic noise** into a **home.** FIGURE 14.4 shows the **sound paths** for **diffraction** and **transmission.**

One consists of a **solid filled concrete block wall** giving a **sound reduction** of **35dBA**.

The **other** is **25mm thick timber** giving a **TL** of **21dBA. Intuitively** this may suggest the **block structure** would give the best result.

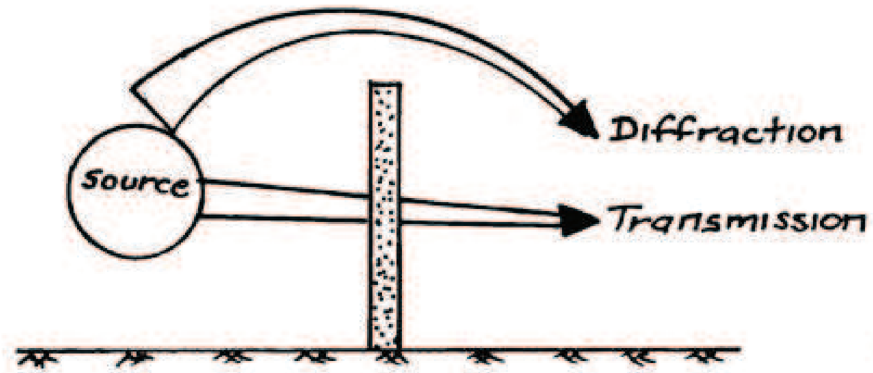

FIGURE 14.4: Sound paths

Sound Barriers – Worked Example

Noise is received at the **house,** mainly by **two paths.**

- **diffracted** rays **over** the **walls;**
- **transmitted through** the **wall**.

Diffraction can be attributed to a **reduction** of **10-12dBA,** maximum. In **this case** take **12dBA.**

For a truck noise of 70dBA - Solid Block Wall

Due to **transmission**: 70 – 35 = **35dba**

By **diffraction**: **70 – 12 = 58dBA**

Adding: 58 – 35 = 23dBA> 10 so +0.

Noise received **= 58dBA**

Timber Wall

Due to **transmission** 70 – 21 = **49dBA**

By **diffraction**: 70 – 12 = **58dBA**

Adding 58 – 49 = 9dBA so + 0.5
= 58.5dBA

Rounding could go either way. Going down:

Noise received **= 58dBA**

Hence, **no** additional **benefits** are gained by **using** a **material** having a **higher acoustic performance** than **25mm thick timber.**

14.9 Noise in Buildings

Identifying potential noise sources at the **design stage** of a building is imperative since **remedial work** can be **very costly** and inconvenient to the client.

Noise in **buildings** can be categorised into **two types:**

airborne	**from within** from voices, TV's and radios, **from outside** from traffic, weather, etc.
structure borne	from **vibrating machines, impact** from footsteps from people walking or running, **moving furniture**, etc.

Materials providing **adequate insulation** against **airborne sound** may **not** be so **effective** against **impact.** This is particularly so if the **Mass Law** is invoked to improve transmission loss.

14.10 Timber Stud Cavity Walls – Airborne Noise

The **Mass Law** shows by **doubling** the **mass/m^2** of a **single skin wall** contributes to a **6dB increase in TL.** That is, a **10mm** thick panel increased to **20mm** thick gives an **additional 6dB** noise **reduction.** However, if further 6dB increases are required it can be seen taking the **Mass Law approach** soon becomes **impractical.**

If **instead** of **doubling** the **thickness** of the single skin, another **identical single skin wall panel** is located **beside** the **first** one, but sufficiently separated to render them acoustically independent. This system would **not** result in **just** a **6dB gain** but rather it would **double** the **TL** of the first panel.

Again, the **practicalities** of the cavity stud wall construction shown in FIGURE 14.5(a) **dictates** the **sheathing material** must be **relatively close together.** This results in the **gain** in **TL not** even **approaching** that of the **idealised case** due to **resonances** within the cavity.

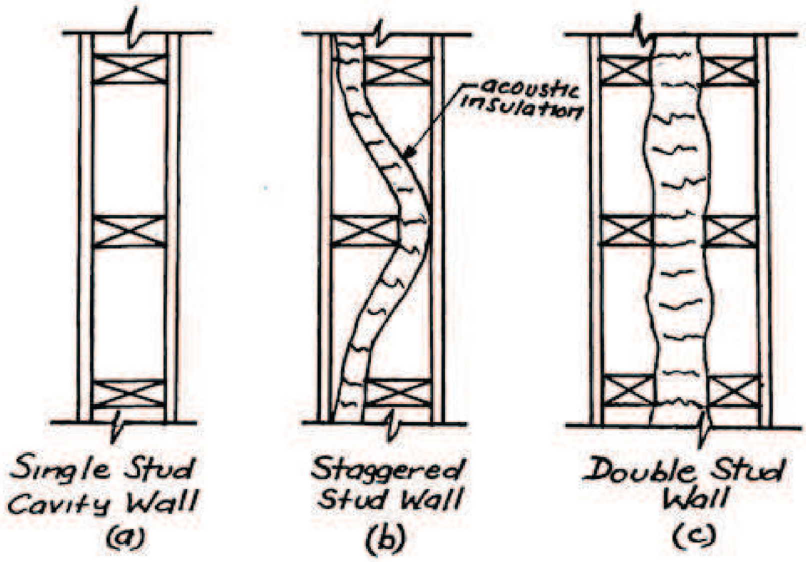

FIGURE 14.5: Types of cavity walls

To **maximise** the **insulations** contribution within the cavity requires **not having** a stud wall of the type shown in **FIGURE 14.5 (a).** This can be achieved by either **staggering** or **doubling** the **studs** as shown in FIGURE 14.5 (b) and (c).

Resilient steel channels, which are thin steel sections arranged such that when **attached** to the **timber studs** will provide a **flexible connection between** sheathing and studs**, can also be** used to **enhance TL.**

14.11 Floor Insulation

Currently **polished timber floors** are popular in floor finishes in single dwelling **houses and apartments.** However, because of their **lack** of **resilience** they pose definite **challenges** to the designer, particularly with regard to **control** of the **transmission** of **footfall noise.**

To attain a suitable **impact insulation rating** for a **timber floor,** although presenting a considerable **challenge** to the designer, should still be **attainable** with a suitable combination of:

- **carpet and underfelt** (although not so well performed at low frequencies) **over plywood** flooring;
- **LVL joists;**
- **suspended ceiling** with **fibreglass absorber;**
- **suitable thickness** of **plasterboard** ceiling.

To attain the desired outcome may require the application of new technologies or better use of old ones.

14.12 Conclusion

There is little doubt the **control** of **noise** to acceptable levels within the **habitable environment** of places of **residence, work, entertainment,** etc. should be given the same **careful consideration** as structural aspects. Whilst this chapter does not pretend to convert the designer into an instant acoustics expert it is hoped it provides sufficient background to raise the awareness of a very important parameter within the overall design process.

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15 Condensation & Thermal Transmission

15.1 Introduction

The main objective of this chapter is to also provide, under the one cover, some basic information pertaining to **condensation and heat flow** in habitable type buildings be they **domestic, commercial or industrial.**

It is imperative the **designer** gives **due consideration** to the question of **heat flow and ventilation** at an **early stage** of the **design process.** Early attention to such detail will eliminate the need for later, costly repairs and inconveniences.

Again, the purpose of this chapter is not to attempt to convert readers into being thermo-fluid experts, but rather to make them more aware of the problems that exist and to assist in their identification and solution.

15.2 Condensation – Causes

Condensation causes **mould growth** in houses and **rot** in the **timber framing** of the house thus **threatening** its **structural integrity. Thermal insulation,** whose function it is to **prevent surface condensation**, if not installed correctly, can cause it.

Terms and Definitions

Only those terms and definitions considered relevant to the topic are presented here.

- **Dry-bulb temperature** – The temperature of the air as registered by an ordinary thermometer **(t)**.

- **Wet-bulb temperature** – The temperature registered by a thermometer when its **bulb** is covered by a **wetted** wick and is **exposed** to a **current** of rapidly **moving air (t').**

- **Relative humidity** – Ratio of the **partial pressure** of the **water vapour** in the mixture to the **saturated partial pressure** at the **dry-bulb temperature,** expressed as a **percentage**.

$$R_h = \frac{p_w}{p_s} x100 \quad \textbf{(15.1)}$$

  Note:
  *If the air is **completely saturated**, the **partial pressure** will be the **vapour pressure of water** at the **dry–bulb temperature,** i.e. at **saturation t=t'.***

- **Dew-point temperature (DP)** – Temperature to which **air must be reduced** in order to **cause condensation** of any of its **water vapour.**

The above **terms** will be **required** if the **moisture content** of timber was to be determined **using FIGURE 15.1** Such a **situation** may arise where, **during a wet period, water** has **ponded** under a house without a vapour barrier. This can cause the **underside** of the timber floor to **take up moisture** which in turn can result in **buckling** of the floor if the **underside is unprotected** and the **top** surface **has** been **coated** with say a **polyurethane finish.**

15.3 Condensation – An Explanation

Air can **retain water** as **vapour** provided the **temperature** of the **air** and the **amount** of **water** are **compatible.** The **ratio** of the **water** in the **air relative** to the **amount** which the **air can hold** is by definition the **relative humidity.**

Warm air can **hold more moisture** than **cool air.** This means if **air** at a **certain temperature** is **saturated,** this corresponds to **100% humidity.** If this **air** is then **cooled water must condense out.** This will occur as a **fog** of **liquid droplets** if the air is **cooled en mass** or as a **condensate** if **cooled** in **contact** with a **surface.** The temperature at which some of the **moisture condenses** as **dew** is the **dew-point temperature.**

Problem Areas:

In general the problem areas can be classified as:

- **high humidity areas,** most likely to give problems during the **cooling season;**
- **cold wet climates** which would most likely present problems during the **heating season.**

Vapour Retarders:

Vapour retarders are used extensively **under concrete slabs** and **sheet metal roofs** and take the form of **aluminium foil or polyethylene** sheet and have **high resistance** to the **flow** of **water vapour.** These type retarders are placed on the **warm side** of the **building elements**, whilst **membranes** that breathe should be placed on the **cold side.**

When these **membranes** are **incorrectly specified** and used as **insulation** or **sarking** they may **contribute** to **condensation** by stopping water vapour from escaping from **high humidity areas.**

Condensation Control:

Factors to be taken into account to **control condensation** are:

- **reduce moisture inside** the **home.** This can be done by **controlling** the **output** from various sources, e.g. **clothes driers, bathrooms, kitchens,** etc. by **venting** to the outdoors if necessary.

- by using a **vapour retarder** ground cover **under low set houses** to prevent moisture reaching the underside of the floor. **Suitable drainage** should also be ensured.

- doing **regular checks** looking for any sign of **moisture accumulation.**

- noting in general, **timber floors** do **not cool** sufficiently, to **cause condensation** from within the house.

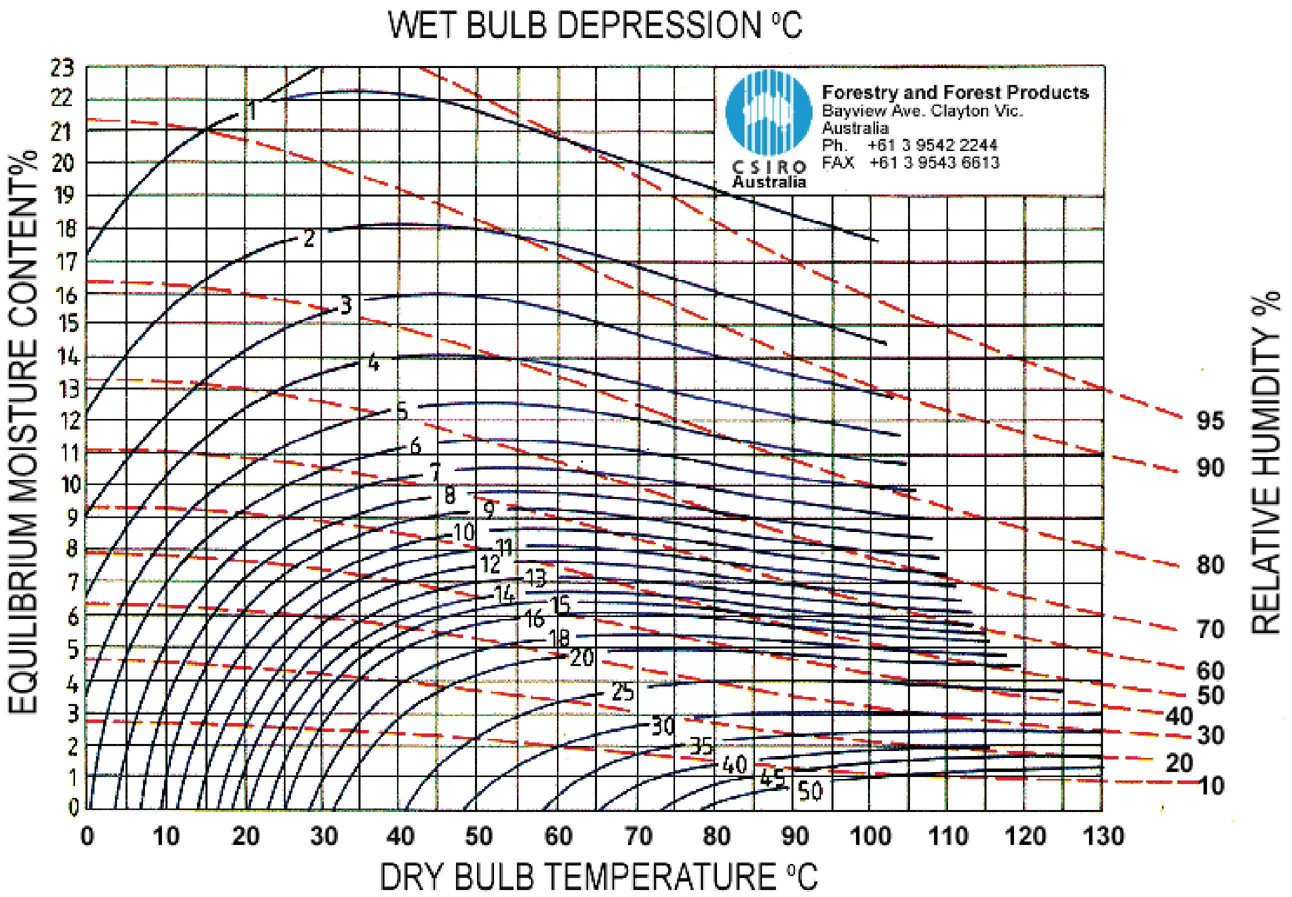

FIGURE 15.1: Equilibrium Moisture Content of Wood as a function of Dry Bulb Temperature, Wet Bulb Depression and Relative Humidity

15.4 Thermal Transmission

Thermal transmission, or more specifically for this section, **heat flow through building materials** is of prime importance in this day and age where **efficient energy usage** is so important. Therefore, it is imperative architects, engineers and building designers are at least conversant with the topic.

Terms and Definitions

Unit thermal conductivity (k), a fundamental heat transmission property, is a measure of the **rate of heat flow** through **unit area** of a material of **unit thickness** subjected to a **unit temperature gradient.**

Thermal conductivity of wood is affected by:

- **density** – increases with increasing density;
- **moisture content** – increases with increasing moisture content;
  (density and moisture content have the greatest influence)
- **extractive content** – increases with increasing extractive content;
- **grain direction** – about the same in the radial and tangential directions but can be about twice this along the grain;
- **natural characteristics** – increases with the amount of knots, checks, etc;
- **temperature** – increases marginally with temperature

The **unit** of **thermal conductivity (k)** is the **Watt/m°C** where:

1 watt = 1 Joule/second or **1 Newton metre/second**

Unit thermal conductivity (k) of **softwood timbers** at **12% moisture content** is in the range **0.11 to 0.18 W/m°C** compared with **216 for aluminium, 45 for steel** and **0.9 for concrete.**

Thermal resistivity (R) is the **reciprocal of unit thermal conductivity**, i.e.

$$\mathbf{R = \frac{1}{k} m^{\circ}C/W} \qquad \textbf{(15.2)}$$

Thermal conductivity and **thermal resistivity** refer to **thermal properties of homogeneous materials** of uniform composition and **specifically relate** to a **thickness** of **1m** of the material.

Thermal resistance (r) refers to the **individual resistances** of the **barriers** encountered **during** the **transmission** from **one side to the other** of the system of barriers.

The **thermal resistance** of an **individual barrier**, i.e. **plywood**, etc. is obtained thus:

$$\mathbf{r_i = \frac{material\ thickness\ (metres)}{unit\ thermal\ conductivity} = \frac{T_i}{k_i}}$$

where:

$\mathbf{T_i}$ = **thickness** of the **barrier (m)**
$\mathbf{k_i}$ = **unit** (1m thick) **thermal conductivity (W/m°C)**

TABLE 15.1 gives a range of **thermal resistances** for **various thicknesses** of **softwood plywood** having an average density of **550kg/m^3.**

Thickness (mm)	Density (kg/m^3)	Unit Thermal Conductivity k(W/m°C)	Thermal Resistivity R=1/k (m°C/W)	Thermal Resistance (r)* (m^2°C/W)
3	550	0.13	7.7	0.02
6				0.05
9				0.07
12				0.09
18				0.14
25				0.19

$$*\,r = \frac{T(m)}{k} = RT(m)$$

TABLE 15.1: Thermal resistances for different plywood thicknesses

The **total thermal resistance (R_t)** is determined by summation of the individual thermal resistances of the successive thermal barriers.

Hence:

$$R_t = r_1 + r_2 + r_3 + \ldots \qquad (15.3)$$

The **overall co-efficient of heat transfer (U)** is given by:

$$U = \frac{1}{R_t}\ \mathrm{W/m^{2\circ}C} \qquad (15.4)$$

TABLE 15.2 gives **thermal resistivities** for a number of common building materials.

Material	Density (kg/m^3)	Thermal Resistivity (m°C/W)
Brickwork		0.87
Concrete 1:2:4	2400	0.69
Concrete (aerated)	480	9.25
Plasterboard	880	5.88
Weatherboard		7.17
Softwood Timber	520	9.06
Hardwood Timber	870	5.30

TABLE 15.2: Thermal resistivities of some common construction
(reproduced from Victoria Appendix, Part F6, Guide to the Insulation Regulations for Residential Buildings, 1992 Edition)

TABLE 15.3 gives indoor and outdoor **surface air resistances** for walls, roofs and floors.

Location	Surface	Surface Resistance (m^2°C/W)
Walls	Outdoor surface air;	0.04
	Indoor surface air;	0.12
Roofs	Outdoor surface, moving air;	0.03
	Outdoor surface, still air;	0.11
	Indoor surface.	0.11
Floors	Outdoor surface; moving air 3m/s;	0.03
	Outdoor surface, air movement 0.5m/s	0.08
	Indoor surface.	0.16

TABLE 15.3: Surface resistances for walls, roofs and floors.
(Reproduced from Victoria Appendix, Part F6, Guide to the Insulation Regulations for Residential Buildings, 1992 Edition)

15.5 Thermal Transmission – Design Example

FIGURE 15.2 shows a wall configuration consisting of **90 x 45mm timber framing** and **EWPAA branded 12mm thick plywood cladding** over **rigid foam sheathing.** Including:

- **batt insulation** fitted between the studs;
- a **vapour barrier** nearest the winter warm side to prevent vapour reaching any part of the construction resulting from a temperature below the dew point;
- **plasterboard internal lining** which results in an effective method of reducing annual heating or cooling costs. In this case it is 12mm thick.

It should be **noted** the **vapour barrier** provides **no** significant thermal **resistance to** the **heat flow.** Also, **vapour retarders** may be **omitted** from walls in **hot humid climates.**

A number of **foam sheathing** types are available, e.g. **polystyrene, polyurethane** and **isocyanurate foams** being the most common. These are available in thicknesses of 19 or 25mm with **R values** ranging from **0.5** to **1.27 m°C/W.**

Reflective Foil Liners can be either **single** or **double sided** and result in a **reflective air gap**, which for **walls** has a **thermal resistance (r = T / k) of :**

20mm reflective air gap = 0.58m^2°C/W
> 20mm reflective air gap
= 0.61m^2°C/W **not required – for information only.**

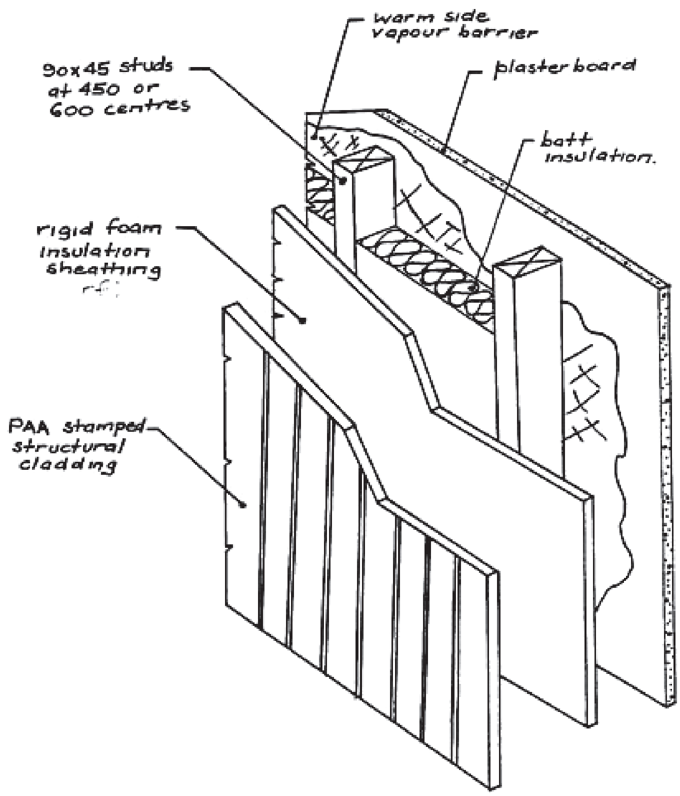

FIGURE 15.2: Sample Wall

$$R_t = r_{oa} + \frac{T_{ply}}{k_{ply}} + r_{rf} + \frac{T_{batt}}{r_{batt}} + r_{vb} + \frac{T_{pb}}{r_{pb}} + r_{ia}$$

where:

r_{oa} = **outdoor** surface **air resistance;**

T_{ply} = **thickness** of **plywood sheathing (m);**

r_{rf} = **rigid foam** resistance;

T_{batt} = **batt insulation thickness (m);**

r_{vb} = **vapour barrier resistance = 0**;

T_{pb} = **thickness** of **plasterboard (m);**

r_{ia} = **inside surface air resistance**

$$R_t = 0.04 + \frac{0.012}{0.13} + 0.5 + \frac{0.095}{0.05} + 0 + \frac{0.012}{0.22} + 0.12$$

$$= 0.04 + 0.092 + 0.5 + 1.9 + 0 + 0.05 + 0.12$$

$$R_t = 2.25\,m^2\,^\circ C/W$$

$$U = \frac{1}{R_t} \qquad (15.5)$$

$$= \frac{1}{2.25}$$

$$U = 0.44\,W/m^2\,^\circ C \text{ (between studs)}$$

If the heat flow **through** the **studs** was being **considered** $r_{stud} = T_{stud}/k_{timber}$, which in this case would be **0.09/0.13 = 0.69**, would have to be **included in** the **calculation** for R_t.

The **Building Code of Australia (BCA) – Part J1, Building Fabric** provides **guidelines** for **minimum R – values** for the various climate zones for:

- various **roof and ceiling types;**
- **walls** of various construction;
- **floors** of **timber** and concrete construction.

For a **comprehensive treatment** of **floor insulation** the reader is referred to the publication:

"Insulation Solutions to enhance the Thermal resistance of Suspended Timber Floor System in Australia"

and for more details on availability see **Reference 7.**

15.6 Conclusion

The information contained in this chapter is not meant to be exhaustive, but rather, informative. Where the designer has any doubt as to the likely outcome of the choice of the insulation components constituting a barrier professional help should be sought. Recognition of potential problems and implementing the correct steps towards their solution is a fundamental part of the design process. Hence, the age old truism **A LITTLE KNOWLEDGE CAN BE DANGEROUS** should forever be uppermost in ones mind.

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16 Resistance to Fire, Decay and Bugs

16.1 Fire & Wood

Three components are required for a **fire,** i.e. **fuel, heat and oxygen**. This knowledge is essential when considering containment which requires eliminating one of these three components from the other two. That is, to extinguish the fire requires **removing:**

- **heat** by wetting;
- **fuel** by eliminating the source;
- **oxygen** by smothering the fire.

Wood is composed of a mixture of cellulose, hemicellulose, and lignin bound together in a complex network. Heating wood **above 280°C causes** decomposition or **pyrolysis** converting it to gases, tar and charcoal. At temperatures **above 280°C** the **gases** will **flame vigorously** but the **charcoal** requires temperatures of about **500°C** for **its consumption.** A build-up of char tends to protect the unburnt wood from rapid pyrolysis. The unburnt timber, being a good insulator, results in the timber close to the char edge being unaffected by the fire. FIGURE 16.1 shows a schematic representation of burning wood.

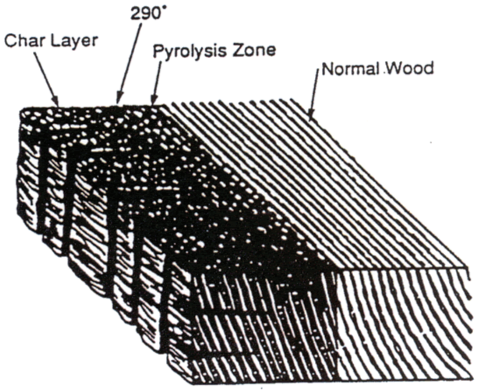

FIGURE 16.1: Zones of burning wood

16.2 Fire Hazard Properties – Test Methods

The most dangerous period with regards to human safety is usually at the **initial stages** of the fire **prior to flashover. Hence** the need for provisions in the BCA which **limit** the **spread of fire** and the **development of smoke** until the building occupants have time to evacuate. **Tests** have been developed which **simulate a fire in a building or are done on test specimen** to facilitate the **generation** of **relevant design data.**

AS 1530 Part 3

Early fire hazard tests to AS 1530 Part 3 (which has now been superseded by ISO 9239.1, ISO 9705 and AS/NZS 3837 see FIGURE 16.2) were **performed to assess** the **surface burning characteristics** of materials. The **data generated** through these tests is **still valid for sarking type materials,** i.e. **reflective foil or other flexible membranes** for waterproofing, vapour proofing or thermal reflectance in Class (2) to (9) buildings. However, it **does not apply to**:

- **floor materials and floor coverings;**
- **wall and ceiling linings fire hazard properties.**

The test sample **parameters quantified** in the AS 1530 Part 3 test are:

- **tendency to ignite** through assigning an **ignitability index;**
- **tendency to propagate flame** through assigning a **spread of flame index;**
- **ability to release heat once ignited** through assigning a **heat evolved index;**
- **tendency to produce smoke** while burning through assigning a **smoke developed index**

The **early fire hazard test indices** are scaled according to their performance from **best 0 to worst 10.** The **two most important parameters,** i.e. **spread of flame index and smoke developed index** are given in **Table 16.6** for a number of **plywood species.**

In **Class 2 to 9** buildings **sarking** type materials must have **fire hazard properties** thus:

- **flammability index ≤5;**

In the case of **other materials:**

- **spread of flame index ≤9;**
- **smoke developed index ≤8** if **spread of flame index** is **≤5.**

ISO 9239.1

This test applies specifically to **floor materials and floor coverings** and is summarised in FIGURE 16.2. The test results in the material being assigned a **number (in kW/m^2)** based on its **critical radiant flux.** The test also allows the **smoke development rate** to be determined and must **not exceed 750 percent – minutes** when a **sprinkler system** has **not** been **installed**.

The **critical radiant flux** is an indication of the **amount** of **heat flux** required to be applied to a material to **cause a small flame to ignite it.**

TABLE 16.1 of BCA Specification C1.10(a) sets out **critical radiant flux values** for **buildings with and without** sprinkler systems.

Class of building	General		Fire-Isolated Exits
	Building not fitted with a sprinkler system complying with Specification E1.5	Building fitted with a sprinkler system complying with Specification E1.5	
Class 2,3,5,6,7,8 or 9b, Excluding accommodation for the aged	2.2	1.2	2.2
Class 3, Accommodation for the aged	4.5	2.2	4.5
Class 9a,Patient care areas	4.5	2.2	4.5
Class 9a, Areas other than patient care areas	2.2	1.2	4.5
Class 9c, Residential use areas	-	2.2	4.5
Class 9c, Areas other than residential use areas	-	1.2	4.5

TABLE 16.1: Critical Radiant Flux (CRF in kW/M^2) of Floor Materials and Floor Coverings

FLOOR AND WALL AND CEILING COVERING AND LINING TESTS

AS/NZS 1530.3 were the **tests performed to determine fire hazard properties** for **floor materials and coverings** and also for **wall and ceiling linings.**

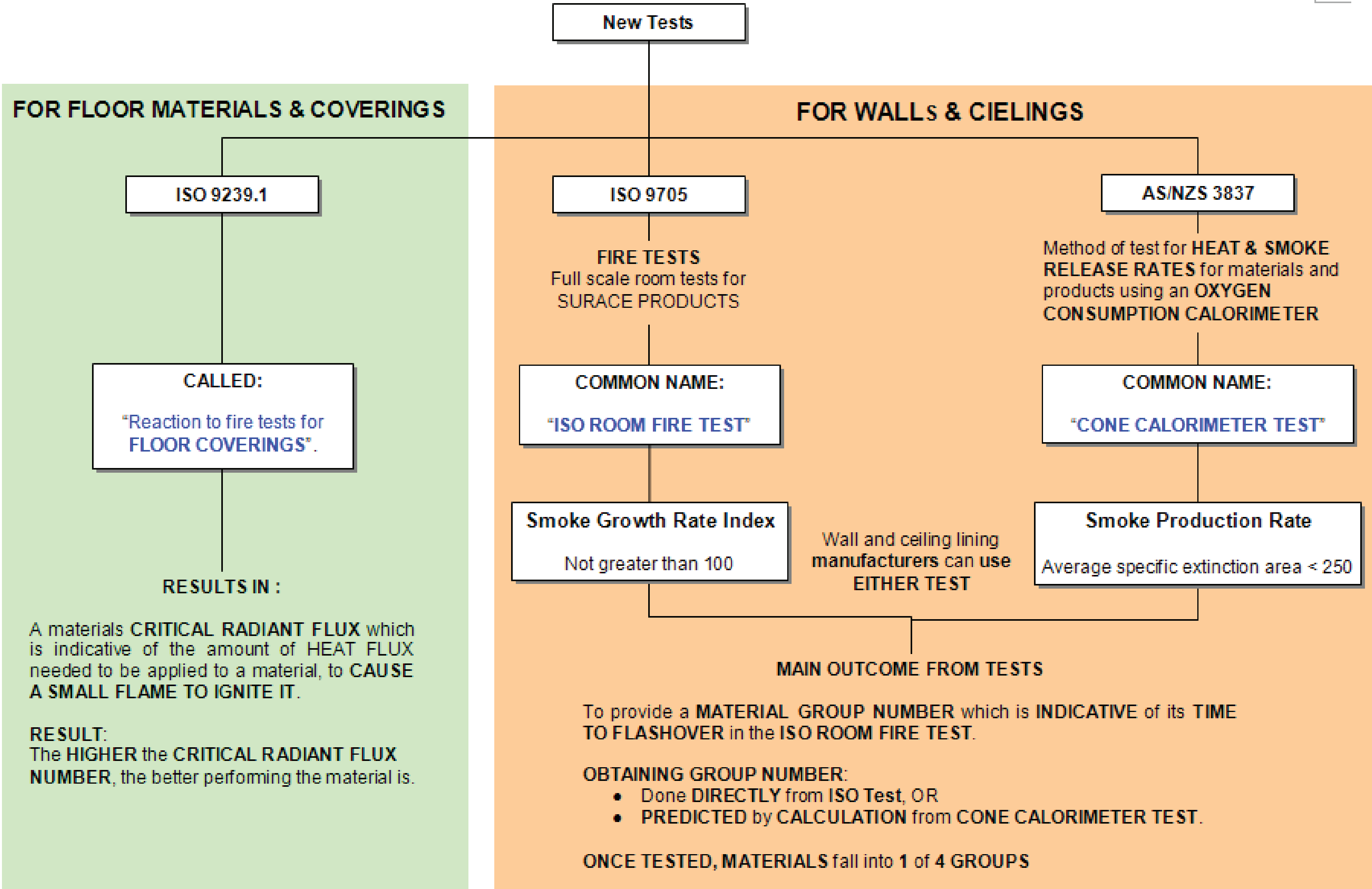

FIGURE 16.2: Summary of Floor, Wall and Ceiling Fire Tests

ISO 9705 AND AS/NZS 3837

These tests apply specifically to **wall and ceiling** linings and are summarised in FIGURE 16.2. The **main outcome** of the tests is to assign a **Material Group Number.** Once tested the material will fall into **1 of 4 groups** as listed in TABLE 16.2. The tests also assign **smoke production numbers** which are of consequence when **no sprinkler system** has been **installed.**

Material Group Number	Description
Group 1	Materials that do not reach flashover following exposure to 300kW for 600 seconds, after not reaching flashover when exposed to 100kW for 600 seconds.
Group 2	Materials that do reach flashover after exposure to 300kW for 600 seconds, after not reaching flashover when exposed to 100kW for 600 seconds.
Group 3	Materials that reach flashover in more than 120 seconds but less than 600 seconds after exposure to 100kW.
Group 4	Materials that reach flashover in less than 120 seconds after exposure to 100kW.

TABLE 16.2: Material Group Numbers

Group 1 materials are **suitable** for the **most stringent fire hazard requirements** whilst **Group 4 do not meet** the **requirements for lining materials** for walls and ceilings.

TABLE 16.3 of BCA Specification C1.10(a) gives Deemed-to-Satisfy Provision for wall and ceiling lining materials, in terms of **Material Group Numbers,** for **sprinklered and unsprinklered buildings.**

Deemed-to-Satisfy

Class of building	Fire-isolated exits Wall/ceiling	Public corridors		Specific areas		Other areas Wall/ceiling
		Wall	Ceiling	Wall	Ceiling	
Class 2 or 3, Excluding accommodation for the aged, people with disabilities, and children						
Unsprinklered	1	1,2	1,2	1,2,3	1,2,3	1,2,3
Sprinklered	1	1,2,3	1,2,3	1,2,3	1,2,3	1,2,3
Class 3 or 9a, Accommodation for the aged, people with disabilities, children and health-care buildings						
Unsprinklered	1	1	1	1,2	1,2	1,2,3
Sprinklered	1	1,2	1,2	1,2,3	1,2,3	1,2,3
Class 5,6,7,8 or 9b schools						
Unsprinklered	1	1,2	1,2	1,2,3	1,2	1,2,3
Sprinklered	1	1,2,3	1,2,3	1,2,3	1,2,3	1,2,3
Class 9b other than schools						
Unsprinklered	1	1	1	1,2	1,2	1,2,3
Sprinklered	1	1,2	1,2	1,2,3	1,2,3	1,2,3
Class 9c						
Sprinklered	1	1,2	1,2	1,2,3	1,2,3	1,2,3

For the purpose of this Table:
1. "Sprinklered" means a building fitted with a sprinkler system complying with Specification E1.5.
2. "Specific areas" means within:
(a) for Class 2 and 3 buildings, a *sole-occupancy unit.*
(b) for Class 5 buildings, open plan offices with a minimum floor dimension/floor to ceiling height ratio >5.
(c) for Class 6 buildings, shops or other building with a minimum floor dimension/floor to ceiling height ratio >5.
(d) for Class 9a *health-care buildings, patient care areas.*
(e) for Class 9b theatres and halls, etc. an auditorium.
(f) for Class 9b schools, a classroom
(g) for Class 9c *aged care buildings, resident use areas*.

TABLE 16.3: Wall and Ceiling Lining Materials (Material Groups Permitted)

Specific Areas – Example

Consider an **open plan office** area having **floor dimensions** of **18m x 20m** and a floor to **ceiling height** of **3m**. Since the **minimum floor dimension** (18m) **divided** by the **ceiling height** (3m) is **greater than 5** (6 in fact), the **area is** a **specific area**. Hence, the ceiling linings in this area would need to comply with the requirements for Class 5 buildings.

BCA Building Classes

The various classes of buildings are described in the Part A3 of the BCA 2007 document, and are copied here for convenience :

Class	Definition
Class 1a	A single dwelling being – (i) A detached house; or (ii) One of a group of 2 or more attached dwellings, each being a building, separated by a fire resisting wall, including a row house, terrace house, town house or villa unit.
Class 1b	A boarding house, guest house, hostel or the like – (i) with a total area of all floors not exceeding 300 m^2 measured over the enclosing walls of the Class 1b; and (ii) in which not more than 12 persons would ordinarily be resident.
Class 2	A building containing 2 or more sole-occupancy units each being a separate dwelling.
Class 3	A residential building, other than a building of Class 1 or 2, which is a common place of long term or transient living for a number of unrelated persons, including – (a) a boarding house, guest house, hostel, lodging house or backpackers accommodation; or (b) a residential part of a hotel or motel; or (c) a residential part of a school; or (d) accommodation for the aged, children or people with disabilities; or (e) a residential part of a health-care building with accommodates members of staff; or (f) a residential part of a detention centred.
Class 4	A dwelling or building that is class 5, 6, 7,8, or 9 if it is the only dwelling in the building
Class 5	An office building used for professional or commercial purposes, excluding buildings of class 6, 7, 8, or 9.
Class 6	A shop or other public building for the sale of goods by retail or the supply of services direct to the public, including- (a) an eating room, café, restaurant, milk or soft drink bar; or (b) a dining room, bar, shop or kiosk part of a hotel or motel; or (c) market or sale room, show room or service station
Class 7a	A building which is a car park
Class 7b	A building which is for storage, or display of gods or produce for sale by wholesale
Class 8	A laboratory or a building in which a handcraft or process for the production, assembling, altering, repairing, packing, finishing, or cleaning of goods or produce is carried on for trade sale or gain
Class 9a	A building of a public nature that is a health care building, including those parts of a building set aside as a laboratory
Class 9b	An assembly building, including a trade workshop, laboratory or the like in a primary or secondary school, but excluding any other parts of the building that are of another class
Class 9c	An aged care building
Class 10a	A non-habitable building being a private garage, carport, shed, or the like
Class10b	A structure being a fence, mast, antenna, retaining or free standing wall, swimming pool, or the like

General Information

Design of structures for **fire resistance** can pose **difficulties** for the uninitiated, and with the new approach contained in the BCA, initially for the initiated.

It is necessary for the Designer to be familiar with the section on Fire Resistance given in the BCA. It is hoped the contents of this chapter will aid in the application of the BCA requirements, which in order to satisfy them it is necessary to:

- **comply** with the **Deemed to Satisfy provisions**, i.e. sections in the BCA **listing ways** to **satisfy performance criteria;**
- formulate an alternative solution that:
  - complies with the performance criteria, or
  - is shown to be **at least equivalent to** the **Deemed to Satisfy** provision.

The following lists the **relevant sections** of the BCA **regarding fire resistance.**

Reference	Description
Part A3	BCA Building Classes
Section C	**Deemed to Satisfy** provisions regarding fire resistance and stability
Section C1.10	**Deemed to Satisfy** provisions for Fire Hazard Properties of Materials
Specification C1.10	**Deemed to Satisfy** requirements for materials other than floors, walls and ceilings
Specification C1.10a	**Deemed to Satisfy** requirements for floors, walls and ceilings

The **first time designers in** the area of **fire resistance should** also **refer to** the document:

EWPAA : Fire Resistance

Class 2 and 9 Buildings

Because of the **way** in which **buildings** have been **categorised, Class 2 to 9** structures **demand** the **most stringent fire resisting characteristics** of **materials used in their construction**.

Such buildings may **have** a **wide variety of** unrelated **occupants** or provide **storage for flammable materials** of various types.

Therefore, materials used in the construction of floors, walls and ceiling linings for these buildings must have a certain minimum capability regarding resistance to the **spread of fire** and the **development of smoke.**

Class 1 and 10 Buildings

Class 1 buildings can generally be classed as **detached residential dwellings** or **two or more attached buildings separated** by a **fire resisting wall.** This is a **very different occupancy** to that described for Class 2 to 9 buildings and as such also **rates** a **separate volume** (Volume 2) by the **BCA** which cannot be covered in any detail herein.

An **important aspect** of **fire safety** with regards to **Class 1 buildilngs** is the **incorporation** of a **separating wall** (party wall). This must have a **Fire Resistance Level** (**FRL**) of **not less than 60/60/60 for** its **separation** of **Class 1 from Class 1** or **Class 1 from Class 10** buildings.

P2.3.1 from Volume 2 of the BCA **regarding Protection from Spread of Fire** states:

A **Class 1 building** must be **protected from** the **spread of fire** from:

- **another building by not less than 1.8m other than** an **associated Class 10** building or a **detached part** of the **same Class 1 buidling**;
- the **allotment boundary** by **not less than 900mm**, **other than** a **boundary adjoinng** a **road or public space.**

3.7.1.5 regarding **Construction of External Walls** states:

- The **intent** of construction is to **ensure combustible materials** (external and internal) are **not directly exposed to fire** at the **junction of** the **wall and non-combustible roof, eaves lining, guttering** and the like.

Bushfire Areas

A **Class 1 building** constructed in a designated **bushfire prone area must be designed and constructed to reduce** the **risk of ignition** from a bushfire **while** the **fire front passes**.

Performance Requirements, P2.3.4 of Volume 2 **is satisfied for a Class 1 building** located in a designated bushfire prone area **if** it is **constructed in accordance** with:

AS 3959 – Construction of Buildings in Bushfire-Prone Areas.

AS 3959 contains a **methodology for assessment of** the **category of bushfire attack** for a **site**. The **categories** are **determined by considering**:

- **predominant vegetation type**;
- **distance between** the **site** and the **predominant vegetation**;
- **slope of** the **land between** the **site** and the **predominant vegetation**

The **categories** of **bushfire attack** are **low, medium, high** and **extreme.**

States and Territories replace clauses in the BCA **stating Acceptable Construction Practice** with their **own compliance requirements** and are **detailed** in **Part 3.7.4 Bushfire Areas** in Volume 2 of the BCA.

16.3 Plywood and LVL Performance

This section provides the **most recent data** obtained from test results **for plywood and LVL** (solid wood) regarding the Building Code of Australia specifications for:

- **wall and ceiling lining materials**;
- **flooring materials**

Plywood – Wall and Ceiling Linings

Testing done **by Warrington Fire Research (Aust) Pty Ltd** during 2007 was to **AS/NZS 3837:1998, in conjunction** with the **prediction method.** This resulted in the determination of a **material group number and average specific extinction area (m^3/kg)** for **veneer timber species** given in **Table 16.4.**

Plywoods constructed from the **species** of **Table 16.4** are **suitable for wall and ceiling linings** provided they:

- have a **minimum thickness** of **6mm;**
- a **tongue and groove** or **square edge profile**;
- a **smooth sanded finish.**

Species	Species
Ash, Alpine – *Eucalyptus delegatensis*	Gum, Spotted – *Corymbia maculata*
Ash, Mountain – *Eucalyptus regnans*	Gum, Sugar – *Eucalyptus Cladocalyx*
Ash, Silvertop – *Eucalyptus sieberi*	Gum, Yellow – *Eucalyptus leucoxylon*
Beech Myrtle – *Nothofagus cunninghamii*	Ironbark, Grey – *Eucalyptus drepanophylla*
Blackbutt – *Eucalyptus pilularis*	Ironbark, Red – *Eucalyptus sideroxylon*
Blackbutt, New England – *Eucalyptus andrewsii*	Jarrah – *Eucalyptus marginata*
Blackbutt, WA – *Eucalyptus pantens*	Karri – *Eucalyptus diversicolor*
Blackwood – *Acacia melanoxylon*	Mahogany, Red – *Eucalyptus resinifera*
Bloodwood Red – *Corymbia gummifera*	Marri – *Eucalyptus calophylla*
Box, Brush – *Lopehostman confertus*	Merbau – *Instia bijuga*
Box, Grey – *Eucalyptus microcarpa*	Messmate – *Eucalyptus oblique*
Box, Grey, Coast – *Eucalyptus bosistoana*	Pine, Baltic – *Picea abies*
Brownbarrel – *Eucalyptus fastigata*	Pine, Radiata – *Pinus Radiata*
Gum, Blue, Sydney – *Eucalyptus saligna*	Pine, White Cypress – *Callitris glaucophylla*
Gum, Blue, Southern (TAS) – *Eucalyptus globulus*	Sheoak, WA – *Allocosuarina fraserana*
Gum, Blue, Southern (VIC) – *Eucalyptus globulus*	Stringybark, Yellow – *Eucalyptus muellerana*
Gum, Manna – *Eucalyptus viminalis*	Tallowwood – *Eucalyptus microcorys*
Gum, Red, River – *Eucalyptus camaldulensis*	Turpentine – *Syncarpa glomulifera*
Gum, Rose – *Eucalyptus grandis*	Wattle, Silver – *Acacia dealbata*
Gum, Shining – *Eucalyptus nitens*	

TABLE 16.4 : Suitable veneer species for plywood construction

The timber species listed in Table 16.4 have been tested and achieve the following performance when tested in accordance with AS/NZS 3837:1998 and using the Method of Kokkala, Thomas and Karisson to calculate Material group number.

Material Group Number	3
Average Extinction area	Less than 250m^{2}kg

Floor Materials

Tests done **by WFRA** in accordance with **AS ISO 9239.1-2003 resulted** in the **plywood constructions** given in **Table 16.5.**

Species	Thickness (mm)	Performance	
		Critical Radiant Heat Flux	
Pine, Hoop – *Araucaria cunninghamii* Pine, Radiata – *Pinus Radiata* Pine, Slash – *Pinus elliotti* (plywoods with no substrate)	15 or > 17 or > 17 or >	Between 2.2 (kW/m^2) and 4.5 (kW/m^2)	Less than 750(%-min)

TABLE 16.5 : Plywood Flooring

Early Fire Indices – To AS 1530.3

Table 16.6 gives the **relevant data** regarding **early fire hazard** tests for both **untreated** and **fire retardant treated plywood** which is **no longer acceptable.** If **fire retardant plywood** is **desired treatment** must be **by impregnation** with fire retardant salts and must be certified to show compliance with the building regulations.

PLYWOOD (UNTREATED)				
Face veneer's Common Name	**Botanical Name**	**Spread of Flame Index (0-10)**	**Smoke Developed Index (0-10)**	**Report Reference**
Australian Red Cedar	*Toona australis*	9	9	E.B.S. 5/10/76 E.4248
Australian Red Cedar (grooved)	*Toona australis*	8	2	E.B.S 5/10/78 E.4250
Blackbean	*Castanospermum australe*	9	3	E.B.S 5/10/78 E.4238
Coachwood	*Ceratopetalum apetalum*	8	2	E.B.S. 5/10/78 E.4235
Hickory Ash (grooved)	*Flindersia ifflaiana*	8	3	E.B.S. 5/10/78 E.4249
Klinkii pine	*Aurancaria hunsteinii*	8	4	E.B.S. 5/10/78 E.4245
Lauan	*Parashorea Spp.* *Shorea Spp.*	8	3	E.B.S. 5/10/78 E.4244
Meranti	*Shorea Spp.*	8	2	E.B.S. 5/10/78 E.4240
Pacific Maple	*Shorea Spp.*	8	2	E.B.S. 5/10/78 E.4240
Queensland Maple	*Flindersia brayleyana*	8	2	E.B.S. 5/10/78 E.4239
Queensland Walnut	*Endiandra palmerstoni*	8	3	E.B.S. 5/10/78 E.4241
Radiata Pine	*Pinus radiata*	8	2	E.B.S. 5/10/78 E.4237
Radiata Pine (scorched and brushed surface)	*Pinus radiata*	7	2	E.B.S. 5/10/78 E.4246
Sapele	*Entandrophragma cylindricum*	8	2	E.B.S. 5/10/78 E.4243
Silver Ash	*Flindersia bourjotiana*	8	3	E.B.S. 5/10/78 E.4242
Tasmanian Oak	Mixture of: *Euc. obliqua* *Euc. delegatensis* *Euc. regnans*	8	2	E.B.S. 5/10/78 E.4236
Teak	*Tectona grandis*	8	3	E.B.S. 5/10/78 E.4247
Victorian Ash	Mixture of: *Euc. regnans* *Euc. delegatensis*	8	2	E.B.S. 5/10/78

TIMBERS AND PLYWOODS TREATED WITH FIRE RETARDANTS				
Timber Species	**Treatment**	**Spread of Flame Index (0-10)**	**Smoke Developed Index (0-10)**	**Report Reference**
Hoop Pine	Retardant Impregnated	0	2	(a)
Redwood	Surface coated with 3 coats of fire retardant	0	5	E.B.S. 23/2/79 E.4362
	Surface coated with 1 coat of fire retardant	8	4	E.B.S. 23/2/79 E.4361
Western Red Cedar	Surface Coated with 3 coats of fire retardant	0	4	E.B.S. 23/2/79 E.4360
	Surface coated with 1 coat of fire retardant	8	4	E.B.S. 23/2/79
Yellow Walnut	Retardant impregnated	0	1	(a)

REFERENCES: (a) *'Early Burning Properties of Australian Building Timber', J. Beesley, J.J. Keogh, A.W. Moulen, Division of Building Research Technical Paper No. 6 24 pages published by C.S.I.R.O. 1974

TABLE 16.6: Early Fire Hazard Data for Untreated and Fire Retardant Treated Plywood

Results of **recent tests** organised by the Timber Development Association and done by CSIRO to AS 1530.3 has **found four timber species**, i.e. White Mahogany, Grey Ironbark, Mountain Grey Gum and Merbau (Kwila) to have a **spread of flame index** of **zero**. These are the **first timber species** to **have** a **zero spread of flame index**.

Included in this testing program were **four plywood species not listed** in **Table 16.6.** These were Blackbutt, Rose Gum, Spotted Gum, and Hoop Pine. To **view** the **test results** derived for the (**27**) **timber species**, **including** the **plywoods** go to **www.timber.net.au**.

16.4 LVL Performance

WFRA Short Form Report (**SFR4 1117.2**) dated September 26, 2007 **confirm timber species** tested by them have **attained** the **stated performance.** The **tests** were done to **satisfy** the **requirements** of the **Building Code of Australia specification C1.10a.**

Wall and Ceiling Linings

For **solid timber** (including **LVL**) the **assessed construction** was a **solid timber wall** and **ceiling linings** made **from** the timber **species listed** in **Table 16.4** and:

- being **not less than 12mm thick**;
- having a **tongue and groove** or **square edge profile**;
- a **smooth milled surface finish**

The **material group number** is (**3**) and the **average extinction area** is **less than 250m^2/kg** the **same as** for **plywood** of **not less than 6mm thickness.**

Floor Materials

Solid timber floors (with **no substrate**) were **assessed** for solid **flooring** made **from** the **species listed in Table 16.7.** The flooring material must:

- be **not less than 12mm** thick;
- be **fixed** to **structural framing** with an **air gap** under;
- incorporate a **tongue and groove** or **square profile**;
- have a **smooth** milled **surface finish**

Species	Species
Ash, Alpine – *Eucalyptus delegatensis*	Gum, Shining – *Eucalyptus nitens*
Ash, Mountain – *Eucalyptus regnans*	Pine, Celery-top – *Phyllocladus asplenifolius*
Gum, Blue, Sydney – *Eucalyptus saligna*	Stringybark, Yellow – *Eucalyptus muellerana*
Gum, Rose – *Eucalyptus grandis*	

TABLE 16.7 : LVLspecies suitable for 12mm thick flooring (**no substrate**)

Solid timber floors (**no substrate**) were **assessed for flooring** made **from** the **species listed** in **Table 16.8.** The flooring material must:

- be **not less than 19mm** thick;
- be **fixed** to **structural framing** with an **air gap** under;
- incorporate a **tongue and groove** or **square edge profile**;
- have a **smooth** milled **surface finish**

Species	Species
Ash, Alpine – *Eucalyptus delegatensis*	Gum, Rose – *Eucalyptus grandis*
Ash, Mountain – *Eucalyptus regnans*	Gum, Shining – *Eucalyptus nitens*
Ash, Silvertop – *Eucalyptus sieberi*	Messmate – *Eucalyptus oblique*
Blackbutt – *Eucalyptus pilularis*	Pine, Celery-top – *Phyllocladus asplenifolius*
Brownbarrel – *Eucalyptus fastigata*	Pine, Radiata – *Pinus Radiata*
Gum, Blue, Sydney – *Eucalyptus saligna*	Stringybark, Yellow – *Eucalyptus muellerana*
Gum, Manna – *Eucalyptus viminalis*	

TABLE 16.8(a)

Species	Species
Beech Myrtle – *Nothofagus cunninghamii*	Ironbark, Grey – *Eucalyptus drepanophylla*
Blackbutt, New England – *Eucalyptus andrewsii*	Ironbark, Red – *Eucalyptus sideroxylon*
Blackwood – *Acacia melanoxylon*	Jarrah – *Eucalyptus marginata*
Bloodwood Red – *Corymbia gummifera*	Karri – *Eucalyptus diversicolor*
Box, Brush – *Lopehostman confertus*	Mahogany, Red – *Eucalyptus resinifera*
Box, Grey – *Eucalyptus microcarpa*	Merbau – *Instia bijuga*
Gum, Blue, Southern (TAS) – *Eucalyptus globulus*	Pine, Baltic – *Picea abies*
Gum, Blue, Southern (VIC) – *Eucalyptus globulus*	Pine, White Cypress – *Callitris glaucophylla*
Gum, Red, River – *Eucalyptus camaldulensis*	Tallowwood – *Eucalyptus microcorys*
Gum, Spotted – *Corymbia maculata*	Turpentine – *Syncarpa glomulifera*
Gum, Sugar – *Eucalyptus Cladocalyx*	Wattle, Silver – *Acacia dealbata*
Gum, Yellow – *Eucalyptus leucoxylon*	

TABLE 16.8(b) LVL species suitable for 19mm thick flooring (no substrate)

Solid timber floors (**with substrate**) were assessed from flooring made **from** the **species listed** in **Table 16.8(a) and (b).** The flooring material must:

- be **not less than 12mm** thick;
- be **fixed** to a **substrate** listed in **Table 16.9** with **PVA** adhesive;
- incorporate a **tongue and groove** or **square edge** profile;
- have a **smooth** milled **surface finish**

Substrate Specification	Thickness (mm)
Particleboard 716kg/m^3	**not less than** 19
Fibre cement	Not less than 15mm
Normal weight concrete floor	Not less than 75mm
Lightweight concrete floor	Not less than 75mm

TABLE 16.9 : Substrates for LVL flooring

Critical Radiant Heat Flux for LVL

The timber **species** listed in **Table 16.10 achieve** the stated **performance when tested in accordance** with **AS ISO 9239.1-2003.**

Flooring Construction	Minimum Thickness	Applicable Species	Performance	
			Critical Radiant Heat Flux	Smoke Development Rate
LVL (Substrates in Table 16.9)	12mm	Table 16.6(a) and Table 16.6(b)	Between 2.2(kW/m^2) and 4.5 (kW/m^2)	
LVL (No substrate)	12mm	Table 16.5	Between 2.2(kW/m^2) and 4.5(kW/m^2)	
	19mm	Table 16.5	Between 2.2(kW/m^2) and 4.5(kW/m^2)	Less than 750 (%-min)
	19mm	Table 16.6	More than 4.5(kW/m^2)	

TABLE 16.10 : critical Radiant Heat Flux for LVL Flooring

NOTE: ***Many** of the **timber species listed** in **Tables 16.4 through 16.8** are **not used** to **manufacture plywood or LVL.** Therefore, the **Designer** must **chec**k with the **manufacturer before specifying** a particular **species** as a **wall and ceiling lining** or as **flooring.***

16.5 Resistance to Fire

Fire Resistance is the **ability** of a **building component** to **resist** a **fully developed fire**, while **still performing its structural function**. **Fire resistance levels (FRL) are assigned as performance criteria, in minutes**, for **structural adequacy**, **integrity** and **insulation**. This important parameter is defined by **three numbers,** e.g. 30/30/30 for which the:

- **first number** relates to **structural stability,** i.e. the **time** to elapse **before collapse;**
- **second number** is an **integrity requirement,** i.e. **flames must not pass through the component** for this number of minutes;
- **third number** is an **insulation** value, i.e. **limits heat transfer** through the component.

Plywood is quite **acceptable as a material used in fire resistant** components **provided** it is **combined with other materials** so as **to meet** the **fire resistant requirements**. This can be **achieved by combining** plywood **with non-combustible materials** such as **fibrous cement or fire grade plasterboard**. The FRL rating is evaluated in a Standard Fire Test as specified in AS 1530.4.

LVL beam or column components can be **assessed** for fire resistance levels as per the requirements of **AS 1720.4 Timber Structures** – **Fire-resistance of structural timber members**. To ascertain the **retained load carrying capabilities** of a structural element is done through a **fire resistance test. This assesses how long** a **component can continue to perform when exposed to a fire.** This ability is measured in terms of the **elapsed time to failure.**

When establishing the **Fire Resistance Level (FRL)** of structural **untreated** wood and wood based products the **charring rate** of the surface is very important. As previously mentioned charring produces a **protective layer** which slows down the charring process. The unburnt timber can then be used in calculations to determine the **structural integrity** of the load bearing member.

16.6 Steps in Establishing an FRL

After a protective layer of char has developed the **char rate slows considerably**. The charring rate of dry wood has been shown to continue for several hours at a reasonably constant rate given in AS1720.4—2006 by:

$$c = dh/dt = 0.4 + (280/\rho)^2 \quad \textbf{(16.1)}$$

where:

c = dh/dt = **notional charring rate** (mm/minute);

ρ = **timber density** (kg/m^3) at a moisture content of 12%.

The **charring rate** of a typical **softwood** having a density of 500kg/m^3 is **0.76mm/minute.** During a fire a realistic assessment of structural response can be made by **neglecting 10mm of unburnt wood** and assuming the **remainder retains its full strength and stiffness.**

- The **effective depth of charring (d_c)** for each exposed surface after a period of time (**t**) is given by:

$$\mathbf{d_c = ct + 7.5} \quad (16.2)$$

where:

d_c = **calculated effective depth of charring** (mm);
c = **notional charring rate**;
t = **period of time** (minutes)

NOTE:
t can be taken as either the:
(a) **time** taken for the **FRL** to be **achieved;**
(b) **fire resistance period determined by** a series of **successive iterations.**

- The **effective residual section** is determined by subtracting $\mathbf{d_c}$ from all fire-exposed surfaces of the timber member as shown in FIGURE 16.3

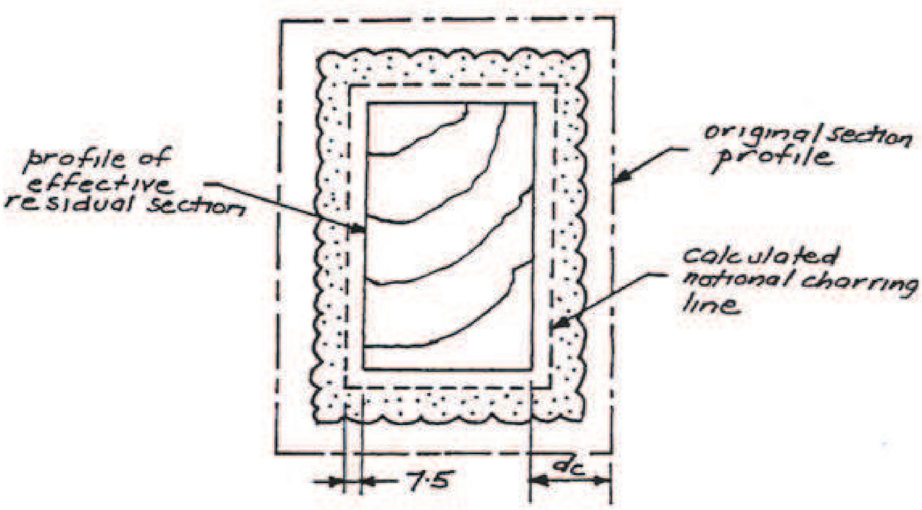

FIGURE 16.3: Shows loss of section due to charring

- the **design loads** to be resisted by the structural elements/components are **determined from** the application of **Clause 4.2.4 and Section 6.1** of **AS 1170.0.**

- a check of the **strength** of the **residual section** is done in accordance with the requirements of AS 1720.1-1997. The **deflection limits** can be:

  - **set by** the **design engineer**
  - a **maximum** of **span / 300.**

TABLE 16. provides a **guide to selecting** a **minimum beam width for a FRL** of **60/-/-**,

as **expressed in the BCA.**

Species	Average Density (kg/m^3)	Typical Minimum Width (mm)
Hardwood	800	100
Softwood	550	140

TABLE 16.11: Minimum beam thicknesses

When **determining** the **strength** of the **effective residual section** take k_1 = **5 hours.**

16.7 Other Factors

There are a number of **other factors to be considered** when **assessing** the **structural adequacy** of a member **designed** to **achieve** a **desired FPL** in accordance with AS 1720.4 – 2006. These are:

Determination of Fire Resistance Period (FRP).

The **FRP** may be required to:

- **determine** a **member size** to satisfy **Building Regulations**;
- **check** the **effective residual section** of an **existing member against** the **FRP**, i.e. against for example, 60/-/-.

The **FRP** is **determined** by doing **a series** of **successive iterations** of **time (t)**. **FRP** is **reached** when the **effective residual section** can **no longer support** the **design load**.

Barrier Junctions

When **included** in a **fire-resisting barrier** a **timber member** has to have allowance **made** for the **effect** the **barrier junction** has on the **effective residual section**. This effect is shown in FIGURE 16.4.

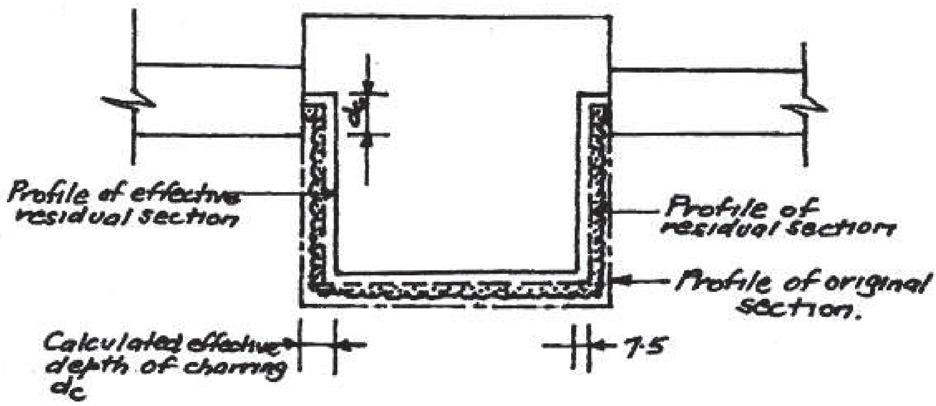

FIGURE 16.4: Charring at junction with fire proof barrier

Protected Timber

Timber members with **fire exposed surfaces protected** by a **fire-resistant insulation** results in the **fire resistance for structural adequacy** of the timber member being **increased.** To **quantify** this increase **AS1720.4 modifies** the **fire resistance period** thus:

$$T_p = t_i + t_m \quad (16.3)$$

where:

T_p = **fire resistance period** of a **timber member protected** with **resistant insulation, in minutes;**

t_i = **fire resistance period** appropriate to the **protective insulating systems, in minutes;**

t_m = **fire resistance period** of the **structural timber member**

Note:

- *For **protected timber c** of **Equation 16.1** is **multiplied** by **1.1**;*
- ***T_p** of **Equation 16.3** is a **conservative estimate** of the **FRP** and can be **modified** if **acceptable test data** is **available**, through reference to **manufacturers' product catalogues, technical reports** and **reports on tests performed** in accordance with **AS 1530.4.***

16.8 Fire Protection of Joints with Metal Connectors

There are **two** possible **scenarios** in which **joints** having **metal connectors** can **occur in a fire** within a structure. These are as:

1. **Unprotected connectors** whereby **structural adequacy** can be established **by test** or is **negligible** if test data does not exist.

2. **Protected connectors** which can be achieved by:

   - **embedding,** which results in the **connectors being embedded** into the member to a **depth equal** to the **calculated effective depth** of **charring** as shown in FIGURE 16.5. The **resulting holes** must be **plugged,** using **timber, glued** into place;

   - **cladding** which is effected by **covering the joint**, e.g. a nailed plywood gusseted moment joint, covered with **fire-resistant claddings.**

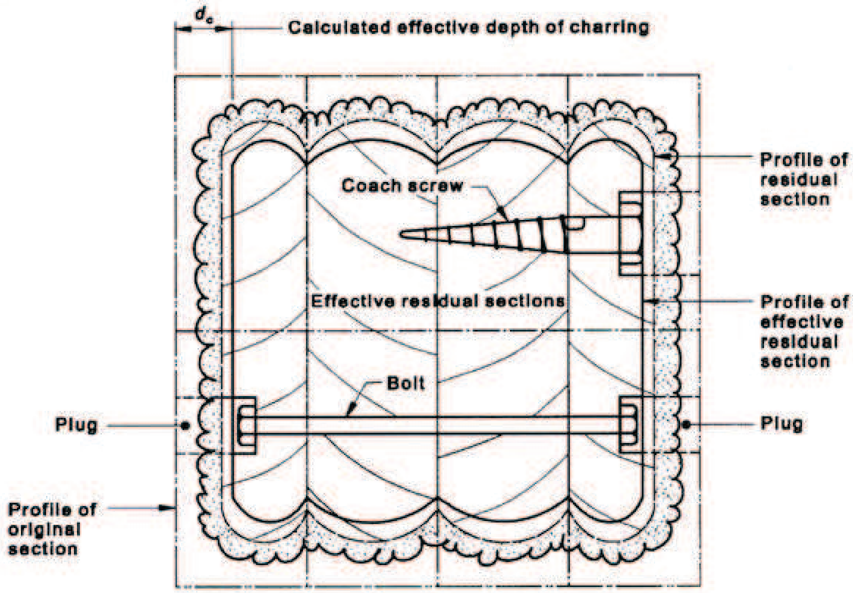

FIGURE 16.5: Fire protected connectors

TABLE 16.1 provides comparative data for fire resistance levels for **structural stability** between **Douglas fir plywood** as published in the Fire Protection Handbook published by the National Fire Protection Association, USA and **radiata pine plywood** as published by Carter Holt Harvey in their Technical Note 95/3/14, March, 1995.

Rating (Minutes)	Plywood Thickness for:		
	Douglas fir (USA) (mm)	Comparable CHH Radiata Pine (mm)	Recommended CHH Radiata Pine (mm)
10	6.4	7	12
15	9.5	12	12
20	12.7	15	17
25	15.9	17	21
30	19	19	25

TABLE 16.12: Fire Resistance Level for Structural Stability for Non-load Bearing Plywood

Closure

The **main aim** of this revision is to **provide** the **first time designer** of fire resistant structures **with some background** information and **pointers to aid in** the **plotting** of a **path through** the **process**.

Also, it is **hoped it provides** the **experienced practitioner with** relevant **updated** fire resistant **design data regarding plywood and LVL**. This should allow their use, with confidence, in a wide range of applications.

The ramifications of a fire, be it localised in a kitchen, bedroom, etc. or from a bushfire, can be horrendous. It is, therefore imperative, the designer provides the occupants with every chance of survival.

16.9 Resistance to Decay

The **durability** of structural **laminated wood veneer products** is **dependent** on the **durability** of the **adhesive** used to bond the veneers and the **durability** of the **timber veneers** themselves.

1 ' XUDELOLW\ RI WKH $GKHVLYH

The **Type A phenolic bond**, used in structural plywood manufactured to AS/NZS 2269 and structural LVL manufactured to AS/NZS 4357, **will not creep or break-down in applications involving long-term structural performance and/or extreme long-term exposure to weather, wet or damp conditions. It is a durable, permanent bond.**

> *The **EWPAA tests bond quality** of samples obtained **from every production shift of EWPAA manufacturing members**. The **bond quality test** for a Type A **bond involves a 72 hour boil** of the plywood or LVL sample (**or 6 hours steaming at 200 kPa pressure**). The **specimen** is then **chiselled apart along each glueline** and the amount of **wood fibre failure evaluated**. The quality of the bond is determined from the amount of wood fibre failure present. **More than 50% wood fibre retention by the adhesive after chiselling indicates** the **bond** is **stronger than** the **surrounding wood fibre**, i.e. a good bond has been achieved. **Less than 50%** wood fibre retention would indicate a **failed bond**.*

' XUDELOLW\ RI WKH 7LPEHU 9HQHHUV

Structural **plywood** and **LVL** are predominantly wood products and **in addition to** the **adhesive durability,** the **durability of** the **timber veneers must be considered** for each specified application. The **majority** of structural **plywood** and **LVL** manufactured in Australia and New Zealand is **made from radiata, slash** or **hoop pine** timber species. These **pine species** have an **expected service life** of **less than 5 years** when used **in exposed applications in contact with** the **ground**, **if** they are **not preservative treated** or otherwise protected, (based on CSIRO durability classifications). Their expected service life when **not in ground** contact but fully exposed to the weather would be much longer.

As a general rule, structural plywood and LVL **used in exposed application** will **need to be preservative treated and surface finished** to meet the exposure hazard and required service life. Generally, the **main hazards for** which structural **plywood** and structural **LVL durability** needs to be considered are:

- **decay**

- **surface moulds**
- **poor detailing**

Decay: Decay or **rot** is **caused by fungi**. Decay fungi can cause a significant loss in strength of timber. Decay or rot of timber **will not occur unless conditions** are **favourable** for the fungi. The **four required conditions are**: a **suitable temperature range (5 to 500C), moisture content** of the timber approximately **19% or higher**, the **presence of oxygen**, and a **food source** (eg. starches and sugars in the timber).

Wood which is **kept dry** with a **moisture content below 19%** will **not** be **subject to fungal attack**. **Occasional wetting** during the construction phase or while in service, for example due to wind blown rain, **will not usually require preservative treatment**. However, if the **plywood** or **LVL** is **frequently wetted or cannot dry out** or be kept dry, then the **plywood or LVL should be preservative treated** to an appropriate level for the decay hazard and required service life. Note that in applications or **locations where high relative humidity is experienced** for extended periods of time, **moisture content** of the timber **may be high** and **preservative treatment required**. FIGURE 15.1 **(**Error! Reference source not found.**) shows timber moisture content relative to temperature and humidity.**

Surface Moulds: Moulds are a type of **fungi** whose activities are mainly **confined to** the **wood surface**. When exposed to moisture, untreated or unprotected timber surfaces may develop surface moulds. These **surface moulds require** the **moisture content of** the timber to be **about 20 percent or greater** and are **more prevalent in warm, humid conditions. Moulds** are limited to the surface and can be **cleaned off with bleaches or wood cleaners** commercially available. Surface **moulds** have **no** significant **effect on structural performance.**

The surface mould becomes inactive when the timber dries out (below 20% moisture content), but will reactivate if the timber is not protected and becomes wet again. **Surface moulds** can be **avoided** by **keeping** the **plywood or LVL dry or** alternatively **surface finishing** the **plywood with** a **coating containing mouldicides or fungicides.**

Typical example of surface mould

Detailing: If it **allows moisture to saturate** or **become trapped in** or **on timber will cause untreated timber to decay quickly** and will considerably shorten the service life of the timber product. **Good detailing** includes details that **reduce or prevent** the **timber** from and reduce moisture ingress through end grain. Where **timber** will **get wet**, **good detailing** should **ensure moisture is shed rapidly** and that the **timber** is **able to dry** out quickly. If moisture traps exist, preservative treatment to meet the intended service life will usually be required.

16.10 Resistance to Insect Attack

The **main insect destroyers** of timber are **termites** and **borers**.

Termites are **not usually** a **problem with plywood** and **LVL provided** the application **does not involve ground contact and good building practices have been implemented** in the design and construction stages. Ongoing inspection and maintenance is essential. **Where** a **termite hazard exists**, for example, in applications involving ground contact structural **plywood** or **LVL should be preservative treated to an appropriate level** for the required service life.

Borers are **rarely a problem with** structural **plywood** or **LVL** except in the **marine environment**. The **main land borers** which **attack seasoned timbers** are the **lyctid borers**, which **only attack** the **sapwood of some hardwoods**, and the **anobium borer** which **attacks both softwoods** and **hardwoods** and is most commonly a problem in old furniture. In **New South Wales** and **Queensland**, **lyctid susceptible hardwood products**, from which a purchaser might reasonably expect a long life, **must by law be treated against lyctid borers**.

Marine borers found in marine waters, **can be highly destructive of timber products.** It is advisable to check with local marine authorities to determine the hazard level in any particular area. **Some marine borers bore holes** in the wood **for shelter** rather than food and do not digest the wood, making it difficult to protect the wood through chemical treatment. **Other marine borers** such as the **Teredo borers, digest the wood** through which they tunnel and **chemical preservative treatments are effective** in protecting the timber.

Marine borer damage to a hardwood pylon

Preservative Treatments

Preservative treatment types and preservative **retention levels** for treatment of structural plywood and structural LVL are **specified in Australian Standard AS/NZS 1604.3** Specification for preservative treatment, Part 3: Plywood and **AS/NZS 1604.4** Specification for preservative treatment Part 4: Laminated Veneer Lumber (LVL).

AS/NZS 1604.3 and AS/NZS 1604.4 describe **six hazard level classifications**, denoted by a hazard number form **H1 to H6** as shown in TABLE 16.13. Each hazard level is defined in terms of the expected service exposure. **H6** is the **most severe hazard level**. Where preservative treatment is required for plywood or LVL, the appropriate standard and hazard level should be specified. It should be noted that there are several different methods of incorporating preservative treatment into plywood and LVL products. Preservative treatment methods for plywood and LVL include:

- **impregnation of veneers prior to manufacture**,
- a **glueline preservative additive** during manufacture,
- **pressure treating** of the **finished product**,
- **preservative treating surfaces after manufacture**.

Veneer preservative treatments preservative **treat** each **individual veneer prior to manufacture** and **no further treatment** will be required **if** the **plywood or LVL is cut.**

A glueline additive is a preservative **added to** the **adhesive prior to bonding** of the **individual veneers.** The **flow** of **moisture** from the glueline into the individual veneers **during** the **hot press phase** of manufacture, **carries** the **preservative into** the **individual veneers** ensuring each individual veneer is preservative treated. Face veneers have only one associated glueline and thicker face veneers may require additional preservative treatment, which is typically achieved in the manufacturing process by spraying face veneers as the product exits the hot press.

Pressure treatment of the finished plywood or LVL results in an "**envelope**" type **treatment**. The **outer veneers** and ends of the sheet or beam will have been **preservative treated** but the **preservative may not have penetrated** through the gluelines **to** the **inner veneers**. If the **plywood or LVL** is **cut** after preservative treating, a **paint** or **preservative treatment should be applied to the cut edge.** Where possible, pressure preservative treatment of the finished product should be done after any machining, sawing and boring.

Fasteners: Hot dipped galvanised or stainless steel fasteners are **recommended** for use with preservative treated plywood.

Hazard Class	Exposure	Specific service conditions	Biological hazard	Typical uses	Preservative Treatments
H1	Inside, above ground	Completely protected from the weather and well ventilated, and protected from termites	Lyctid Borers	Flooring, furniture, interior joinery, wall bracing, interior beams, staircases, stringers	CCA, ACQ Synthetic pyrethroids
H2	Inside, above ground	Protected fro wetting. Nil leaching	Borers and termites	Flooring, wall bracing, interior beams, joists, trusses, staircases	CCA, ACQ Synthetic pyrethroids
H3	Outside, above ground	Subject to periodic moderate wetting and leaching	Moderate decay, borers and termites	Exterior decking, Claddings Exterior beams	CCA, ACQ, LOSP, Copper azole, synthetic pyrethroids
H4	Outside In-ground	Subject to severe wetting and leaching	Severe decay, borers and termites	Noise barriers at ground level, bridges foundation structures	CCA, ACQ, Copper Azole, Creosote
H5	Outside, in-ground contact with or in fresh water	Subject to extreme wetting and leaching and/or where the critical use requires a higher degree of protection	Very sever decay, borers and termites	Cooling tower structure Retaining wall structures, boat hulls	CCA, ACQ, Creosote
H6	Marine Waters	Subject to prolonged immersion in sea water	Marine wood borers and decay	Pontoons, landing steps, boat hulls	CCA, Creosote

TABLE 16.13: Hazard Class Selection Guide for Preservative Treatments
(from AS1604 Specification for preservative treatment, Part 3: Glued wood veneer-based products)

17 Finishing

17.1 Dry Interior Applications:

Structural **plywood** and **LVL** used **in dry interior applications** can be **finished in** any finishing **products suitable for wood surfaces**. For **plywood**, **A** or **B** quality **faces** should be **specified as** a suitable **substrate for high quality interior finishes**, **stains** or **paints**. An A quality face grade is suitable for clear finishing.

17.2 Exterior Applications

As a **general rule** all structural **plywood** and **LVL exposed to** the **weather should be preservative treated** against decay and surface finished to prevent surface breakdown due to weathering.

weathering of unprotected wood surfaces **is caused by exposure to sunlight and rain or other moisture sources** and is **characterised by** a **change in colour** of the exposed wood surface **followed by** a **gradual surface degradation**. **Rain** and **sunlight cause** wetting and drying of the timber surface resulting in **swelling and shrinkage**, stressing the wood surface and **causing cracks** and **checks**. The leaching and bleaching of the timber surface from weathering eventually results in the timber surface turning grey. In the case of **plywood** and **LVL** the **small peeler checks** produced in the back of the veneer during manufacture **become enlarged** and **break through to the face** of the plywood when exposed to continuous wetting and during cycles. This results in **surface checking** which allows more moisture to penetrate and **can** eventually **cause** the **surface veneers to breakup**. **All plywood** and **LVL surfaces should be protected from weathering** to achieve a long service life.

In **exterior applications** the **plywood** or **LVL** surface can be **finished** by:

- **painting**
- **coating with water repellents**
- **overlaying with medium density phenolic impregnated papers** (plywood only)

Plywoods with an A or B grade face veneer quality are suitable for a high quality paint or stain finish. **Plywood** with **C** or **D** quality **face veneer** is **not designed to provide** a **high quality paint substrate**. Plywood cladding products with machined or textured faces are also very suitable for paint or stain finishes.

Where paint systems are **required** in exterior applications, **full acrylic latex paint systems are recommended** for structural plywood and LVL. **Acrylic latex paint systems** are **more flexible than oil based or alkyd enamel paint systems** and better tolerate any expansion and contraction of the timber substrate due to moisture movement.

Rigid paint systems, including oil based and alkyd enamel paint systems are **not recommended for** use on **plywood or LVL in** weather **exposed applications**. However, they **can be used on medium density overlaid plywood** because the overlay acts to prevent surface checking of the plywood face veneer.

Edge sealing of plywood and end sealing of LVL is considered **good practice** to **minimise moisture uptake** through the end grain and reduce localised swelling and surface checking at the plywood panel edges or LVL ends.

The back or unexposed face of plywood should be **left unsealed** if possible **to prevent moisture being trapped** within the panel.

Orientation of the plywood or LVL **needs to be considered** when finishing requirements are being determined. **Horizontal surfaces** are **more exposed to sunlight and moisture ponding than vertical surfaces**, and consequently present a greater hazard to paint breakdown and surface checking. The hazard will be increased if the horizontal surface is also subject to traffic.

17.3 Durability and Finishing Applications

Dry interior environments

Structural **plywood and LVL** used in dry interior environments where the plywood and LVL are installed and **kept** in the **dry** condition (moisture content below 15%) **will not be subject to** the **moisture related issues of weathering, surface mould, or decay**. No particular finish or treatment will be required for durability provided that in termite susceptible areas, good building practices have been implemented including regular inspection and maintenance.

Exterior exposed above ground

Structural **plywood and LVL** used in applications **exposed to high moisture conditions should be preservative treated** to resist decay and insect attack and surface finished to minimise weathering. **Good detailing** should include sealing of the end grain **to minimise moisture ingress**. Construction details and installation should allow sufficient space for expansion and contraction of the plywood or LVL due to moisture content changes.

In ground contact with water

Applications in which **plywood or LVL** are **in contact with ground water** for extended periods of time provide **conditions highly conducive to fungal or insect attack**. **Preservative treatment** appropriate to the hazard level **must be specified**. Typical applications might include tanks, cooling towers, retaining walls, foundations etc.

Contact with sea water

Salt from sea water will have **no adverse effect on plywood or LVL**. The water will cause the wood to swell as would exposure to moisture. The **main durability issue for plywood or LVL in contact with sea water** is **marine borers**. Preservative treatment to H6 preservative levels will be required where marine borers are present.

18 Revision History

Revision	Changes	Date	Who
2	• Added k_1 to the calculation of the bending strength limit state in the table. • Updated stamps and logos to the current versions. • Corrected Δ_{max} equations for Load Case 1 in the Critical Load Action Effects in Section 7.5 • Corrected equations in the table "Design Action Effects on Member due to factored loads" and in the "Deflection Criteria – determine minimum required EI" in Section 7.6. • Corrected equations in the table "Design Action Effects on Member due to factored loads" and corrected results in Section 7.14. • Corrected "Check Panel Shear Capacity" equation and corrected the "Plywood Webbed Beam Dimensions" drawing in Section 8.3. • Corrcted nail slip equations in Section 8.4 • Added Section 8.5 – "Box Beam Portal Joints". • Corrected the equation for "A" in Section A8. • Corrected the case of some symbols and corrected equations in the "Diaphragm Deflection" sub-section of Section 9.49.5. • Changed case of some symbols in Section 9.8. • Corrcted an equation in Section 9.9. • Replaced the following images with revisions to clarify various points : Figures 10.4, 10.5, 10.6, 10.7, 11.9, 12.10 and 13.12. • Made Photos of Section 10.5 into an Appendix, and clarified some headings of this section. • Corrected equations and results of the "Bending Moment", "Shear Flow" and "Horizontal Shear" sub-sections in Section 11.6. • Section 16 rewritten and updated. • Added information to section 7.10 regarding 17.4 kN/m plywood bracing sheathed both sides and the restrictions to panel lengths in these cases. • Various other equation changes / clarifications and spelling correctons.	April 2010	MM
1	• Initial Release	June 07	MM, JM, LP

EWPAA Members

Plywood and Laminated Veneer Lumber (LVL)

Member Name	Location	Phone	Fax	Web
Ausply Pty Ltd.	Australia / NSW	+612 6922 7274	+612 6922 7824	www.ausply.com
Austral Plywoods Pty Ltd.	Australia / QLD	+617 3426 8600	+617 3848 0646	www.australply.com.au
Big River Group Pty Ltd.	Australia / NSW	+612 6644 0900	+612 6643 3328	www.bigrivertimbers.com.au
Boral Hancock Plywood	Australia / QLD	+617 3432 6500	+617 3281 5293	www.boral.com.au
Brown Wood Panels	Australia / SA	+618 8294 3877	+618 8294 6871	www.bwp.com.au,
Carter Holt Harvey Woodproducts Australia (Plywood) – Myrtleford	Australia / Vic	+613 5751 9201	+613 5751 9296	www.chhwoodlogic.com.au
Carter Holt Harvey Woodproducts Australia – Nangwarry LVL	Australia / SA	+618 8721 2709		www.chhfuturebuild.com
Wesbeam	Australia / WA	+618 9306 0400	+618 9306 0444	www.wesbeam.com,
Carter Holt Harvey Woodproducts - Marsden Point LVL	New Zealand	+649 432 8800	+649 432 8830	www.chhfuturebuild.com
Carter Holt Harvey Woodproducts (Plywood) - Tokoroa	New Zealand	+647 886 2100	+647 886 0068	www.shadowclad.co.nz www.ecoply.co.nz
Fiji Forest Industries	Fiji	+679 8811 088	+679 8813 088	
IPL (West Coast) Ltd	New Zealand	+643 762 6759	+643 762 6789	
Juken New Zealand Ltd. (Gisborne)	New Zealand	+646 869 1100	+646 869 1130	
Juken New Zealand Ltd. (Wairarapa)	New Zealand	+646 377 4944	+646 377 1166	
Nelson Pine Industries Ltd	New Zealand	+643 543 8800	+643 543 8890	www.nelsonpine.co.nz
PNG Forest Products Ltd	PNG	+675 472 4944	+675 472 6017	
RH Group (PNG) Ltd	PNG	+675 325 7677	+675 323 0522	www.rhpng.com.pg
Valebasoga Tropikboards Ltd.	Fiji	+679 8814 286	+679 8813 848	
Wesbeam	Australia / WA	+618 9306 0400	+618 9306 0444	www.wesbeam.com,

Particleboard and MDF

Member Name	Location	Phone	Fax	Web
Alpine MDF Industries Pty Ltd	Australia / Vic	+613 5721 3522	+613 5721 3588	www.alpinemdf.com.au
Carter Holt Harvey Woodproducts Australia	Australia / NSW	1300 658 828	+612 9468 5793	www.chhwoodlogic.com.au
D & R Henderson Pty Ltd	Australia / NSW	+612 4577 4033	+612 4577 4759	www.drhenderson.com.au
The Laminex Group	Australia / Vic	+613 9848 4811	+613 9848 8158	www.thelaminexgroup.com.au
Tasmanian Wood Panels (Aust)	Australia / TAS	+613 9460 7766	+613 9460 7268	

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