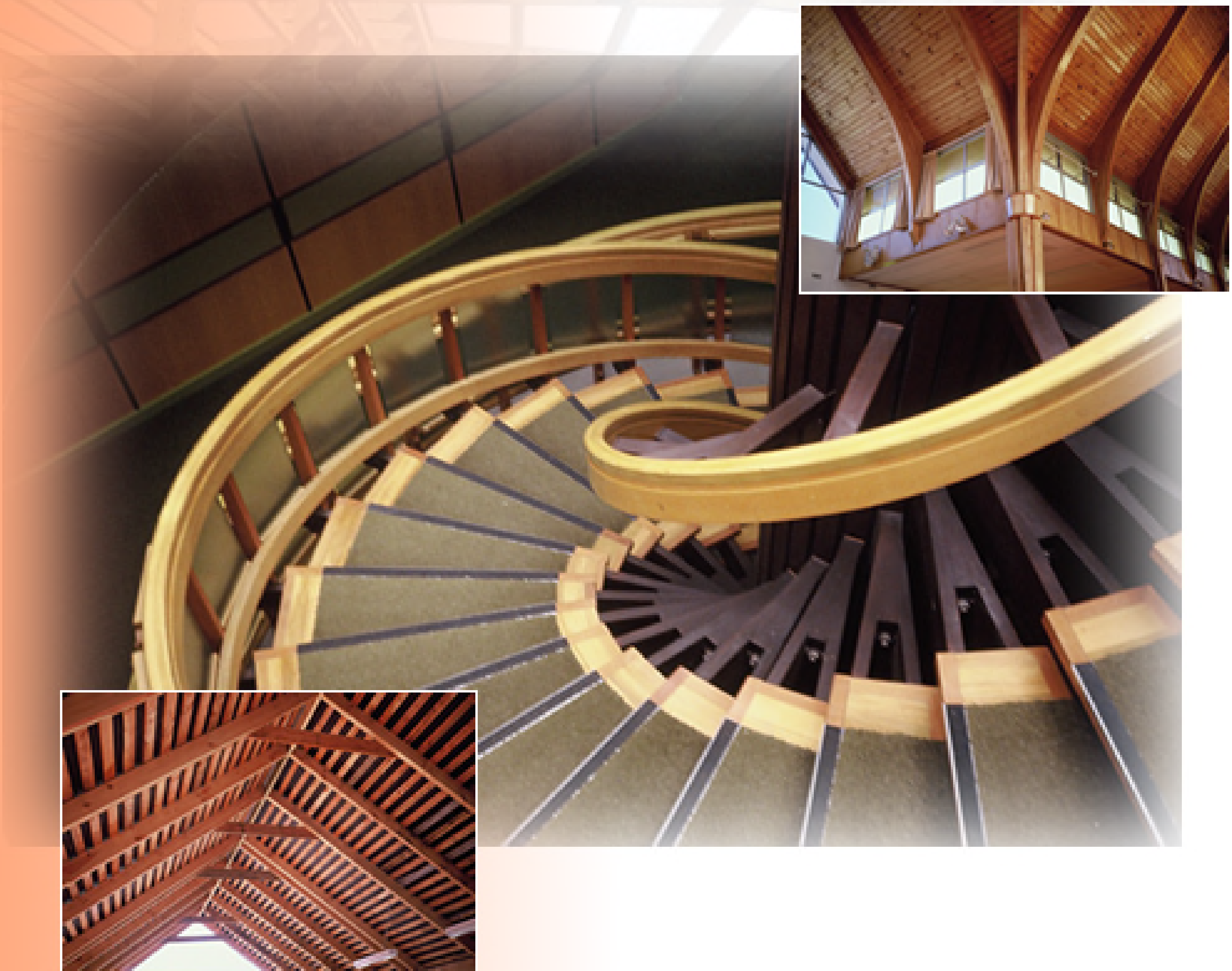
ENGINEERED WOOD PRODUCTS ASSOCIATION OF AUSTRALASIA

Structural Plywood & LVL Design Manual



with Worked Examples

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Forest and Wood Products Research and Development Corporation

PREFACE

This Manual has been compiled for those practitioners inexperienced in the use of plywood and LVL as a structural material, but on occasions find they offer an optimum solution to their structural problem. It is also hoped the Manual will prove useful as a reference for students of architecture, building and engineering enrolled at TAFE Colleges and Universities.

The main objective of the worked examples is to provide guidance in the solution of practitioner's immediate problems and encourage further, more innovative use of these fully engineered, 'fit for purpose' materials.

The main objective of the Manual is to provide the user with, as nearly as practicable, all the design information required for the solution of a range of problems, under the one cover. Australian Standards, e.g. AS 1720.1-1997, Timber Structures and other references will still be required.

Design methodology for the solution of a range of structural problems is presented in a step-by-step format. A worked example is then done which includes Code references. The methodology presented will provide an adequate solution. However, there is no doubt, through the availability of modern technology other more efficient and economical solutions may be implemented. Until complete familiarity with the idiosyncrasies of the material and the design concepts have been fully digested the contents of the Manual will provide a more than adequate solution.

Not every structural component has been considered. For example, trusses have not rated a mention. The thought behind this omission was **'a truss is a truss is a truss"** and the major concern with truss design is to ensure the adequacy of the tension members. LVL ensures this requirement can easily be satisfied. On the other hand, however, it may be questioned why structures not considered to be the norm, e.g., folded plates, arches, hypars and domes rated a chapter. The reason behind this inclusion, be it right or wrong, is to provide the reader with some **'motivational fodder'** to encourage **'thinking outside the square'** during the preliminary design stage.

The chapter dealing with connections is considered to be of prime importance, and therefore, the **'centre of gravity'** of the Manual. If the designer cannot get member connectivity right, irrespective of how well individual elements and components are designed, the structure will be **'doomed to failure'**.

In the writing of such a technical document there will invariably be mistakes, even though subjected to independent checks. Therefore, the EWPAA welcomes correspondence regarding these, together with suggestions relating to improvements and additions. The EWPAA contact details are on the back cover of this manual and are also available from the EWPAA web site.

Happy and fruitful designing,
Mick McDowall
January 2007

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- Those writers of technical papers and text books who have made significant contributions to the compilation of this Manual. These contributions have been acknowledged in the list of references at the end of the chapter or at the relevant point within the text. If an author has been inadvertently missed in the references, sincerest apologies are offered, with the assurance such an omission was completely unintended.

- Photograph contributors, whose contributions confirm the age-old adage “a picture is worth a thousand words”. Also, irrespective of whether the photographs were considered to be “good” or “bad” is totally irrelevant, their contribution to the overall picture is equally significant. Unfortunately, the sources were so many it is impossible to thank individual contributors.

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Edited by: James MacGregor, BE, MIE(Aust), CPEng. some time Production Manager, Merchandising Manager, Fabrication Manager, Market Manager – Tubemakers, Market Development Engineer and Consulting Timber Engineer.

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Part One

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CHAPTER 1

1 PLYWOOD & LVL – the manufacturing process

1.1 Introduction

Structural plywood and **structural Laminated Veneer Lumber** are **engineered, timber veneer products**, made **by bonding** thin timber **veneers** together **under heat and pressure.**

1.2 Manufacturing Standards

Structural plywood is manufactured to the Australian / New Zealand Standard AS/NZS 2269 Plywood – Structural. **Structural Laminated Veneer Lumber** is manufactured in accordance with the requirements of AS/NZS 4357 Structural Laminated Veneer Lumber.

1.3 Process Control

Structural **plywood** and **LVL** products certified by the EWPAA are **branded with** the **EWPAA product certification stamp as well as** the **JAS-ANZ** (Joint Accreditation Scheme of Australia and New Zealand) **mark**. The **EWPAA / JAS-ANZ brand** on a plywood or LVL product **certifies** the **product** has been **manufactured to** the **relevant Australian / New Zealand Standard, under a quality control** and **product certification scheme** accredited by the peak government quality control accreditation body in Australia and New Zealand. **Purchasers** of products stamped with the EWPAA / JAS-ANZ brand will be **purchasing** a **product**, **manufactured under** an **accredited third party audited**, **process based quality control program** that **ensures** the **product** will have uniform, predictable, reliable properties and **will be fit for purpose**. A list of EWPAA plywood and LVL manufacturing members whose products carry the EWPAA / JAS-ANZ brand is given on the back cover of this Manual.

1.4 Manufacturing Processes

The **manufacturing process for** both **plywood** and **Laminated Veneer Lumber** are **similar**. Materials used in their manufacture are thin timber veneers bonded with an adhesive. However as the intended end application is different (panel product versus framing member) the essential differences in the products is in how the veneers are orientated. In essence, **LVL** could be considered as **plywood without cross-bands**, or, **alternatively plywood** could be defined as **cross-laminated LVL**. Hence the **main differences** in the manufacturing process occur **at** the **lay-up and pressing** stages. Prior to manufacture, logs from suitable timber species are selected for peeling based on size, straightness and nature and quantity of defects. The **majority** of **EWPAA branded plywood** is **manufactured from plantation sourced radiata, hoop or slash pine.**

Manufacturing processes may vary from manufacturer to manufacturer, however the **stages of production** are **essentially as follows:**

Conditioning

Logs are **conditioned by immersion in a heated water bath or** alternatively **by steam treating**. Conditioning facilitates the peeling process by assisting in producing a smooth and even veneer. Roughly peeled veneer is undesirable as it is more difficult to bond, requires more adhesive and the veneer is more difficult to handle without damage.

Peeling

Veneer ribbon exiting lathe after peeling

After conditioning, the **logs** are **debarked** and **cut into** suitable **lengths**, **ready for peeling**. These lengths are referred to as peeler billets or peeler blocks. **Peeling** of the billets is **usually done in** a **rotary lathe**. The peeler **billets** are centred in the lathe and **rotated for** their **full length against** the **lathe knife**. The lathe knife is fed toward the centre of the log at a constant rate producing a continuous ribbon of veneer of uniform thickness. Typical **veneer thicknesses** peeled for commercial plywoods range from **1 mm to 3.2 mm**.

Drying

After peeling, the **continuous ribbon** of veneer is either **clipped to size** and **dried**, **or continuously dried in ribbon form and clipped after drying**. The drying process ensures the veneer moisture content is uniform and an appropriate value is achieved for bonding. The **target moisture content is dependent on** a number of factors including the **adhesive used, prevailing ambient conditions** and the **veneer species**. Common veneer moisture content limits after drying are in the range 6 to 12 %.

Grading

Plywood

The clipped and dried **veneer sheets** are **sorted into veneer grades**. **Five veneer grades, A, B, C, D** and **S** are permitted for structural plywood.

LVL

The clipped and **dried veneer** is **sorted** for **acceptable veneer quality**. **Some veneers** are then **passed through** a **scarfing machine** which **creates a bevel** each end. This **allows** the **sheets to overlap**, be effectively glued and remain a uniform thickness. Structural **LVL veneer** is **graded in accordance with** a **predetermined manufacturer's specification** that **ensures** the minimum defined and **published structural properties** of the LVL will be **obtained.**

Lay Up & Bonding

Plywood

Adhesive is **applied** to the **cross-band veneers** and **veneers** are **laid up with alternating long bands and cross-bands, ready for pressing**. The normal plywood assembly is laid up such that each veneer in a finished sheet of plywood has its grain direction at right angles to each adjacent veneer. **Face grade** veneers and **long band core veneers** have the **timber grain direction running in** the **long direction** of the veneer. **Cross-band veneers** have the **timber grain direction** running **in** the **short direction.** The plywood laid up in this manner has a "balanced" construction. That is, veneer orientation and thickness is equal either side of the centre of the plywood thickness.

LVL

Glue is **spread on veneers** by passing them **through** the **rollers of** a **glue spreader or** through a **curtain coater**. The **veneers** are then **usually laid up**, with the **grain direction of all veneers** running **in** the **long direction**. When required, **LVL can be manufactured with cross-banded veneers** to improve dimensional stability and/or increase resistance to splitting when nailed. Typically, where cross-bands are included, the veneer immediately below the face veneers is cross-banded.

After drying, the veneer is sorted into grades ready for lay-up

Veneer passing through the rollers of a glue spreader

Pressing

The assembled **veneer lay ups** are then **cold pressed** to facilitate the bonding process and **ensure good adhesive transfer** from the spread **to** the **unspread veneers.**

After cold pressing, the **plywood** or **LVL** is **hot pressed for a set time between heated platens at a set temperature and time** to achieve proper bonding. Typically plywood hot presses are suitable for maximum plywood sheet sizes of 2700 x 1200 mm and have multiple layers of platens so that 8 to 45 sheets of plywood are pressed in each press load.

Plywood panels exiting 15 daylight (15 layers of platens) hot press

Structural **LVL fabricated in** a **dedicated LVL hot press**, is **laid up on a moveable conveyor belt and progressively hot pressed** in a single layer press, such that very long, continuous lengths are achieved. Typically, LVL hot presses are 600 to 1200 mm in width, permitting production of beam or column elements of 1200 mm depths by lengths in excess of 24 metres and in thicknesses ranging from 35 to 75 mm.

Structural LVL manufactured in a plywood hot press will be 2700 mm in length maximum.

LVL production line

Sanding, Trimming and Branding

After pressing, the **plywood panels** are **cooled** and then **trimmed to precise dimensions**. Plywood **panels** are then **sanded if required** and inspected for face quality.

LVL slabs are ripped into increments of the LVL slab width, allowing for saw cuts. For example, a 1200 mm wide LVL slab may be trimmed to a 1200 mm deep beam/column element or into smaller elements that are divisors of the maximum slab width. Typically maximum LVL slab widths are approximately 1200 mm.

LVL beams ripped from the slab have depths, of for example, 95, 130, 150, 170, 200, 240, 300, 360, 400, 450, 600 mm. Structural LVL face veneers are not usually sanded, but can be if required.

Prior to packing, the LVL or plywood is individually branded to identify the product type and structural properties.

After trimming, sanding and branding, plywood panels undergo a final inspection for face quality

CHAPTER 2

2 STRUCTURAL PLYWOOD

2.1 Introduction

Structural plywood is an **engineered wood panel with defined** and **codified physical and mechanical properties.** Structural plywood in Australia and New Zealand is **manufactured to** Australian/New Zealand Standard **AS/NZS 2269 Plywood - Structural**. This **Standard sets** out the **minimum performance requirements for** the **manufacture** of structural plywood **acceptable to users**, **specifiers, manufacturers and building authorities in Australia and New Zealand**. **Plywood** manufactured **to AS/NZS 2269** is **suitable for use in all permanent structures** and is the plywood type intended for use in structural applications discussed in this manual.

Structural plywood branded with the EWPAA / JAS-ANZ mark certifies **the product has been manufactured** fit-for-purpose **to the structural plywood Standard AS/NZS 2269.**

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Structural plywood manufactured **to AS/NZS 2269** is available with **one bond type** and in a **range of timber species, stress grades, veneer qualities, veneer arrangements** (constructions) **and thicknesses**.

2.2 Bond Type

All structural plywood manufactured **to AS/NZS 2269 has a permanent Type A phenolic resin bonding** the **individual timber veneers**. The Type A bond is produced from phenol or resorcinol formaldehyde and is readily recognisable by its dark colour. The **type A bond is durable and permanent under conditions of full weather exposure, long term stress**, and **combinations of exposure and stress.**

> Note:
> Even though the structural plywood **phenolic bond is durable**, the **plywood will only be as durable as the timber species from which it is made**. If the plywood is going to be **used in weather exposed applications** or under other exposure conditions of severe hazard, the **durability of the timber veneers must be considered** and the plywood preservative treated if required to meet the hazard requirement.

2.3 Timber Species Used

Structural **plywood** is **manufactured from** either **hardwood or softwood** timber **veneers or** a **combination of both**. The **dominant timber species** used in structural plywood in Australia and New Zealand is plantation pine (**radiata, hoop or slash**) however **other** timber **species**, including **eucalypt hardwoods**, are available.

2.4 Stress Grades

A **stress grade defines** a **codified suite of strength and stiffness properties**. There are eight possible stress grades for structural plywood listed in AS 1720.1 Timber Structures Code. The stress grades are: F7, F8, F11, F14, F17, F22, F27 and F34. The **characteristic strength and stiffness properties** for each stress grade are **tabulated in the Timber Structures Code AS1720.1-1997** and **reproduced** in **Table 5.1A,** CHAPTER 5 **of this manual**. The **most commonly available stress grades are F8, F11** and **F14,** higher stress grades F17, F22, F27 and F34 are also available. However availability should be checked before specifying.

2.5 Veneer Quality

There are **five veneer qualities** permitted for structural plywood **in AS/NZS 2269**. The **standard veneer qualities** are **A, S, B, C**, and **D.** The **five** veneer **grades allow** structural **plywood** to be **specified with face and back veneer qualities** to suit the intended application. These **include decorative structural uses** through to applications where **aesthetics** is **not a consideration** and **structural performance alone** is **the requirement**. Other non-standard face veneer qualities are permitted under AS/NZS 2269.

> Note:
> Panels with **A, S, B**, and **C** faces are **sanded smooth**, **D** grade **faces** may be **unsanded** as they are typically **used in structural, non-aesthetic applications**. Hence, there **will be knot holes, splits, gum pockets, etc.**

2.6 Specifying Structural Plywood Grades

Structural plywood face veneer qualities can be **specified to suit** the **appropriate application,** for example, where one face is required to meet a specific requirement and the back will not be visible. This is typical for **plywood flooring** which **may require** a quality **C solid face**, but in most applications, a quality **D back** veneer **will suffice**. The **structural plywood** is **specified** with the required **face veneer quality first followed by** the **back veneer** quality **e.g. CD**. A guide for selecting suitable grades for various uses is shown in TABLE 2.1. Availability of the higher face grades should be checked before specifying.

Grade	Description and Suggested Uses	Face	Back
AA	Used where the appearance of both faces is important. Boats, signs, cabinets	A	A
AB	For uses similar to AA panels, but where the appearance of one side is less important	A	B
AC, AD	Use where the appearance of only one side is important. Feature walls, soffits, furniture	A	C or D
BB	Uses where high quality paint finish is required both sides. Hoardings, furniture	B	B
BC, BD	Used where a high quality paint finish is required one side and the appearance of the other side is not important. Hoardings, internal walls, soffits	B	C or D
CC	A utility grade panel with two sanded, solid faces. Flooring, gussets, containers	C	C
CD	A utility grade panel with one solid face. Flooring, containers, pallets, gussets	C	D
DD	A utility grade intended for structural applications where appearance is not important. Bracing, gussets, webs in beams	D D	D D

TABLE 2.1: Grade Use Guide

Veneer Quality A

Veneer quality A describes a high quality appearance grade veneer suitable for clear finishing. This appearance grade quality should be specified as a **face veneer** for plywood **where surface decorative appearance is** a **primary consideration** in addition to structural performance and reliability.

Veneer Quality S

Veneer quality S defines an appearance grade veneer which **permits natural characteristics as a decorative feature, subject to agreement**. The type and frequency of the natural characteristics that are acceptable is to be based on a written specification, acceptable to both the manufacturer and the purchaser.

Veneer Quality B

Veneer quality B is an **appearance grade veneer with limited** permitted **amounts of sound** inter-grown **knots and filled splits and holes**. Plywood with a quality B face is suitable for high quality paint finishing.

Veneer Quality C

Veneer quality C is defined as a **non-appearance grade with a solid surface**. All **open defects such as holes or splits are filled.** Plywood with a quality C face is intended specifically for applications requiring a solid non decorative surface such as in plywood flooring which is to be covered with carpet or other flooring overlays.

Veneer Quality D

Veneer quality D is defined as a **non-appearance grade with permitted open imperfections**. **Unfilled holes** up to **75 mm wide are permitted** in Veneer Quality D. Plywood manufactured with a quality D face has the lowest appearance grade of structural plywood under the Standard. It is designed specifically for applications where decorative appearance is not a requirement and structural performance is the prime consideration. **Structural plywood bracing is such an application.**

2.7 Identification Code

The plywood **identification code provides information on** the **veneer arrangement** within the structural plywood and is required to establish section properties of a particular plywood. The **I.D code gives** the **following information**: the **nominal plywood thickness**, the **face veneer thickness multiplied by 10**, and the **number of plies in the assembly**. For example, the **ID code 21-30-7** describes a **21 mm thick** plywood with **3.0 mm face veneer** thicknesses and **7 veneer layers**. **Standard constructions** are given in Chapter 5, TABLE 5.2.

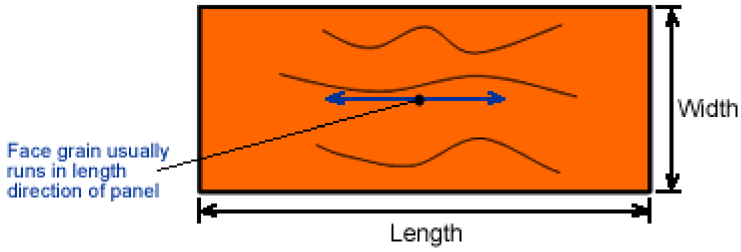
2.8 Panel Dimensions

Length and Width

EWPAA / JAS-ANZ branded structural plywood is commonly available in two standard sizes.

(**Length** x **Width**)

2400 mm x **1200 mm**
2700 mm x **1200 mm**



Other panel lengths are available including **1800, 2100, and 2250**. **Panel widths** of **900 mm** are also available from some manufacturers. Panel **lengths may** be intended to **suit** a particular **end application**. For example, **2250 mm length** plywood is manufactured as flooring to **suit** the **standard floor joist spacing of 450 mm**. Flooring plywood is usually supplied with plastic tongue and grooved (T&G) edges. Plywood **bracing** is available in panel **lengths** of **2440** and **2745 mm** to **allow for top** and **bottom plate coverage**.

Non standard panel sizes and larger panel sizes **in scarf jointed form** are **also available** from some manufacturers.

Thickness

A range of standard plywood **panel thicknesses** are **available** including **3, 4, 4.5, 6, 7, 9, 12, 15, 16, 17, 19, 20, 21, 25 and 28 mm and thicker**. Thickness availability will vary between different manufacturers and it is best to **check** the **thickness, stress grade** and **panel sizes** locally **available before specifying** the plywood.

Standard Tolerances

Standard dimensional tolerances, as specified in AS/NZS 2269 for structural plywood, and measured in accordance with AS/NZS 2098 - Method of test for veneer and plywood, are:

Thickness:-

Sanded sheets up to and **including 7.5 mm thick**	±7%
Sanded sheets over 7.5 mm thick and **up to 17.5 mm thick**	±4%
Sanded sheets over 17.5 mm thick	±3%
Unsanded sheets – **as per sanded sheet tolerances plus** an additional tolerance of **+0.3 mm**	

Length and Width: ±1.5 mm

Squareness:

Difference in length of the diagonals within 0.2 % of the length of the longer diagonal

Straightness of edges:

Not to deviate from a straight line by more than 0.05% of length of edge.

Flatness:

Maximum distance between the underside of the sheet and a flat horizontal surface:

Unloaded sheets:	Sheets up to 7.5 mm thick	**50 mm**
	Sheets over 7.5 mm thick	**30 mm**

Sheets up to 7.5 mm thick loaded with a 10kg weight 0 mm
Sheets over 7.5 mm thick loaded with a 15kg weight 0 mm

Moisture Content

Sheets up to 7.5 mm thick	**10 – 15 %**
Sheets exceeding 7.5 mm thick	**8 – 15 %**

Specification

Specifications for structural plywood should include the following information:

Specify	Example
Number of panels x length (mm) x width (mm) x thickness (mm)	30 sheets of 2400 x 1200 x 19 mm
Plywood type and Standard	Structural plywood to AS/NZS 2269
Stress grade and ID code	F14 (19-30-7)
Face and back grades and glue bond type	CD - A BOND
EWPAA / JAS-ANZ product certification stamp	EWPAA / JAS-ANZ Product Certified

2.9 Other Plywood Types

Non-Standard Structural Plywoods

A number of non-standard structural plywoods manufactured for specific applications are also available. Typical **non-standard structural plywoods have**, **in addition to** their **structural characteristics**, **features that provide aesthetic or finishing characteristics**. **Typical examples** are structural plywoods with **textured and/or grooved face veneers**, or structural plywood with an **overlay on both faces** for protection against weather, wear or abrasion. **Overlays** used include the **high density overlay used on** the **faces of formply** to give face veneer protection and smoothness to the finished concrete surface, and the **medium density overlay** used to **provide** a **substrate suitable** for **high quality paint finishes**. It should be noted that **formply** is a **specialised form** of **structural plywood with thinner face veneers and veneer arrangements suited to** the **intended application**. Section properties for formply are not usually the same as those for standard structural plywood.

Marine Plywood

Marine plywood is manufactured to AS/NZS 2272 : 2006 Plywood – Marine. Unless otherwise branded Marine plywood has a **minimum stress grade of F14** and therefore **has** an **associated suite of structural properties**. However, it should be **noted** the **veneer arrangement and** veneer **thicknesses** used **in marine plywood** commonly result in **different section properties to** those for **structural plywood** of the **same thickness**. Therefore **these two** plywoods are **not usually directly substitutable** for each other for the same structural application. **Marine plywood** is manufactured with higher quality veneers and usually has more veneer layers and **thinner face veneers**. This **provides** more **uniform section properties** in **both directions** but lower stiffness and strength in the face grain direction than an equivalent thickness structural plywood. **Structural plywood** with **thicker face grain veneers** will be **stiffer and stronger in the face grain direction** and is the plywood type intended for use in structural applications described in this manual.

2.10 Non Structural Plywoods

Interior Plywood (manufactured **to AS/NZS 2270 : 2006** Plywood and Blockboard for Interior Use and **Exterior Plywood** (manufactured **to AS/NZS 2271: 2004** Plywood and Blockboard for Exterior Use) are non structural plywoods **used** in applications **where a high quality aesthetic finish is required**. **Even when bonded with phenolic adhesive**, they are **not suitable for** use in **structural applications** and **must not be used i**n conjunction with **structural applications** given in this manual.

CHAPTER 3

3 STRUCTURAL LAMINATED VENEER LUMBER (LVL)

3.1 Introduction

Structural Laminated Veneer Lumber (**LVL**) **is an engineered structural element with published engineering properties**. All **EWPAA / JAS-ANZ branded** structural **LVL** is **manufactured to comply with** Australian and New Zealand Standard **AS/NZS 4357** Structural Laminated Veneer Lumber. This **Standard sets out** the **minimum requirements for** the **manufacture**, **mechanical property characterisation** and **verification of** the **structural properties of LVL** intended **for structural applications, and for** which, structural **design** is performed **in accordance with AS 1720.1** Timber Structures Code, Part 1 - Design Methods or NZS 3603 Timber Structures Standard - Code of Practice for Timber Design.

LVL branded with the EWPAA / JAS-ANZ mark certifies the product **has been** manufactured to AS/NZS4357 **and** is suitable for use in all permanent structural applications.

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3.2 Bond Type

Structural **LVL manufactured to AS/NZS 4357 has a Type A phenolic bond**. The **Type A bond** is produced from phenol or resorcinol formaldehyde and is **recognisable by its dark colour**. The Type A bond is durable and permanent under conditions of full weather exposure, long term stress, and combinations of exposure and stress.

Note:
Even though the structural LVL **phenolic bond is durable**, the **LVL** will **only** be **as durable as** the **timber species from which it is made**. If the **LVL** is going to be **used in weather exposed applications** or under other exposure conditions of severe hazard, the **durability of** the timber **veneers must be considered and the LVL preservative treated to meet the hazard requirement.**

3.3 Timber Species Used

The Structural LVL Standard **AS/NZS 4357** permits the use of **any hardwood or softwood** timber **veneers or** a **combination of both**, **in the manufacture of Structural LVL**. The **dominant timber** species used in the manufacture of structural LVL in Australia and New Zealand is **plantation pine** (radiata and Maritime).

3.4 Stress Grade or Structural Properties

Structural LVL is manufactured to a manufacturing specification that defines and limits all variables that affect structural performance of that manufacturer's LVL product. **AS/NZS 4357 requires** the **manufacturer to publish** the **design properties for their LVL or adopt** a **stress grade classification** as given for structural timber **in AS 1720.1 or NZS 3603**. Alternatively the manufacturer may determine the properties pertaining to a specific application, e.g. scaffold planks. **Current practice for EWPAA / JAS-ANZ branded LVL**, is **for manufacturers to publish** the **design properties** of their LVL **as** a **suite of engineering properties and/or a set of span tables**. The manufacturer's brand name in conjunction with their published literature. The manufacturer's brand name or mark should therefore be included in any specification.

3.5 Veneer Quality

Veneer quality used in structural LVL is **specified by** the **manufacturer to ensure minimum structural properties** are maintained. **Aesthetics** are **not usually a consideration** when manufacturer's veneer quality specifications are set.

3.6 Standard LVL Dimensions

3.6.1 Length

Structural **LVL** fabricated in a dedicated continuous LVL press is **available in very long lengths.** However, **lengths** are **usually restricted by transportation requirements** from the manufacturer's factory and are typically **supplied in lengths up to 12 meters**. Longer lengths are available as special orders if required.

Structural LVL fabricated in a plywood press is available from some manufacturers, in 2.4 or 2.7 metre lengths, which can be supplied nail plated together into continuous lengths.

3.6.2 Cross-section

Structural LVL is available in a range of thicknesses and depths. **Common thicknesses are 35, 36, 45**, **63**, and **75 mm**. Standard thicknesses relate to the veneer thickness (typically 3.2mm) x the number of veneers in the cross-section. Thicker beams are available from some manufacturers. **Beam depths** will **relate to an increment of** the maximum **billet width of 1200 mm**. Typical beam **depths** are **95, 130**, **150, 170, 200, 240, 300, 360, 400, 450, 600 and 1200 mm**. Thickness and depth availability will vary between different manufacturers and it is best to check sizes locally available before specifying the structural LVL.

3.7 Standard Tolerances

Standard dimensional tolerances for structural LVL measured in accordance with AS/NZS 2098, are:

Dimension	**Tolerance**
Thickness	+4 mm, -0 mm.
Width	
up to 400mm	+2 mm, -0 mm.
over 400 mm	+5 mm, -0 mm.
Length	-0 mm
Straightness	
Spring	1 mm in 1000 mm
Bow	1 mm in 1000 mm
Twist	Length (mm) x Width (mm)
	3500 x Thickness (mm)
Squareness of Section	1 mm in 100 mm
Moisture Content	8 – 15 %

3.8 Specification

Specifications for structural LVL should include the following information:

Specification	**Example**
Beam depth (mm) x thickness (mm), number of beams/ length (m)	400 x 35, 30/6.4m
LVL type and Standard	Structural LVL to AS/NZS 4357
Manufacturers' identification mark	Manufacturers brand name
Glue bond type	A Bond
EWPAA / JAS-ANZ product certification stamp	EWPAA / JAS-ANZ Product

CHAPTER 4

4 PLYWOOD & LVL PHYSICAL AND MECHANICAL PROPERTIES

4.1 Introduction

Structural plywood **and structural** LVL **are** composed of individual timber veneers **which can be** selected**,** positioned **and** orientated to optimise **the** finished product properties **for the intended end application.**

Structural **LVL** is typically manufactured with all **veneer grain** directions **parallel with** the **member** l**ength**. This **maximises strength** and **stiffness in** the **spanned direction**.

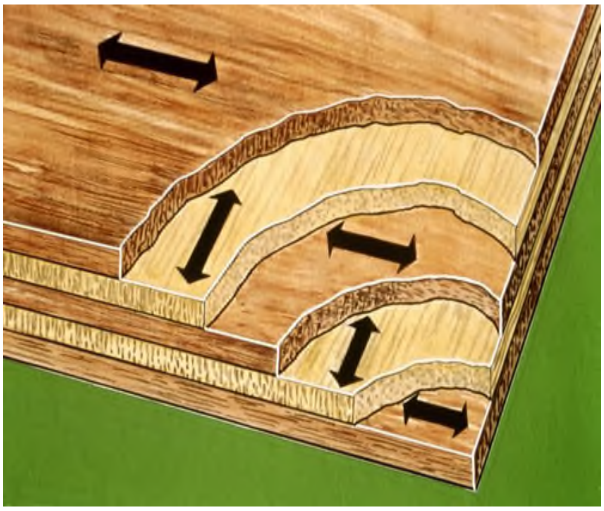
Structural **plywood,** being a panel product, is **manufactured** with **veneer grain** orientation **alternating in** the panel **length and width directions** to **give engineered strength and stiffness properties in both panel directions.** Veneers can be selected and orientated to either maximize strength and stiffness in one panel direction or alternatively provide more equal properties in both directions.

4.2 Cross-Lamination

The **alternating** change in **grain direction** of the veneers **in plywood is** referred to as **cross-lamination,** and in addition to enhanced strength and stiffness properties, a **number of other useful characteristics** are **imparted,** as discussed below. Where required, these characteristics can also be incorporated into LVL, by the inclusion of cross-laminated veneers in the LVL member.

Resistance to Splitting

Cross-lamination of the veneers means there is **no natural cleavage plane** and therefore **plywood will not readily split** either lengthwise or crosswise. This **allows plywood to be nailed at closer spacings** and with reduced distances to the panel edges, **than could be achieved with sawn timber** and some **other** engineered **wood based panel products.**



Cross-lamination in plywood as a result of alternating the veneer grain direction of adjacent veneers

Impact Resistance and Resistance to Puncture

Plywood performs well under heavy concentrated loads and impact loads as the cross-laminations in plywood distribute the stresses over a wide area of the panel. This can be **important in** many structural applications including **structural flooring in commercial** or **industrial situations**, **wall claddings**, **materials handling applications** and **barriers against airborne missiles in cyclones.**

Panel Shear Strength

The **cross-lamination** of veneers in plywood **results in high shear strength within the plane of the panel. This** is one of the **characteristics** that **results in plywoods superior performance in** a number of critical structural applications including **plywood webs in beams, plywood gussets** in portal frames and **as a bracing material**.

4.3 Dimensional Stability under Changes in Moisture Content

Plywood's cross-laminated construction **improves** its **dimensional stability in** the **plane of** the **panel in comparison to solid wood**. **Solid wood** undergoes **little expansion or contraction along** the wood **grain under moisture content changes**, however, **across** the **grain,** it **may undergo considerable movement** due to changes in moisture content. In **plywood**, the **veneer movement due** to **moisture changes** is **restricted across the grain** relative to that along the grain **due to the cross-laminations**. As a result, structural plywood has superior dimensional stability to other timber and wood based panels. TABLE 4.1 details the hygroscopic movement of structural plywood along and across the grain. The **dimensional stability** of plywood is beneficial in many structural applications and is **particularly important in concrete formply** applications where large areas of structural plywood formply are subjected to high temperatures and moisture contents at the time of the concrete pour.

Plywood Thickness (mm)	Number of Plies	Direction* of Movement	Moisture Content Range %			
			5% - 12%	12% - 17%	17% - Saturation	Average. 5% to Saturation
12	5	II ⊥	0.016 0.021	0.009 0.008	0.006 0.005	0.011 0.011
15	5	II ⊥	0.016 0.022	0.008 0.010	0.004 0.009	0.010 0.013
17	7	II ⊥	0.017 0.022	0.009 0.010	0.005 0.010	0.011 0.014
22	9	II ⊥	0.017 0.018	0.012 0.010	0.004 0.008	0.012 0.014

- Direction II is along the face grain
- Direction ⊥ is across the face grain

Example

Determine the hygroscopic expansion in mm across the grain of a 1200mm wide, 17mm thick structural plywood panel, when installed at 10% moisture content and used in a fully exposed application in which the plywood could become fully saturated with water. Assume fibre saturation is 28%.

1. *As the range is 10% - 28% the correct selection from Table 4.1 is from the 'average' column, and is 0.014% per % change of moisture content.*
2. *Total change in moisture content = 28% - 10% = 18%*
3. *Movement in mm of 1200mm panel width = (0.014/100) x 1200 x 18 = 3.0 m*

TABLE 4.1: Percent Movement of Structural Plywood Per Percent Change of Moisture Content

4.4 Thermal Properties

Fire Resistance is the **ability** of a building component **to resist** a **fully developed fire**, while **still performing** its **structural function**. Fire resistance in the form of a **fire rating**, can only be **applied to** a **total building element incorporating plywood**. **For example**, a **fire door** or **wall** or **roof system**. A product cannot be fire rated.

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.

<u>Early Fire Hazard Indicies</u> provide a measure of the **plywood's surface characteristics relating to spread of flame**, **heat evolved**, **smoke emission** and **ignition**. A **low index value indicates better early fire hazard properties.** The early fire hazard indices as defined in AS 1530 Part 3, for untreated pine plywood are given below. The possible index range is given in brackets.

Ignitability index (0 - 20)	14
Spread of Flame index (0 - 10)	8
Heat Evolved index (0 - 10)	9
Smoke Developed index (0 - 10)	2

The early fire hazard indicies of plywood permit it to be used untreated in most typical building applications. Plywood is suitable for use in most building linings, walls, ceiling partitions and floors. Building codes may restrict its use in areas of severe hazard such as flues, hearths, public exits, public corridors, lift wells and certain public areas and buildings.

The use of **intumescent finishes and paints** to reduce the early fire hazard indicies is **not acceptable** under current building regulations.

For **further information** concerning **fire** see CHAPTER 16**.**

Thermal Expansion: Wood, including LVL and plywood expand upon heating as do practically all solids. The **thermal expansion of plywood is quite small**. **The average co-efficient of thermal expansion** of plywood **is in the range 4.5 x 10^{-6} to 7 x 10^{-6} mm/mm/^{0}C.**

Thermal Conductivity: The ability of a material to conduct heat is measured by its thermal conductivity, k. The higher the k value, the greater the ability of the material to conduct heat; the lower the k, the higher the thermal insulation value. k varies with timber species, moisture content, the presence of knots and other natural characteristics, and temperature. However an average value of k=0.1154 W.m/(m^{2}.^{0}C) for softwood timbers is sufficiently accurate for determining the overall co-efficient of heat transmission (U value) of a construction assembly.

Thermal Resistance: The **thermal resistance** or **insulating effectiveness of LVL** and **plywood panels based on k=0.1154 W.m/(m^{2}.^{0}C) is** its **reciprocal, i.e., R=8.67 (m^{2}.^{0}C)/(W.m).** The higher the R value, the more effective the insulation. For example, the R value for 12mm pine plywood = (12/1000) x 8.67 = 0.10 m^{2}.^{0}C/W. Similarly, the R value for 25mm thick pine plywood is (25/1000) x 8.67 = 0.22 m^{2}.^{0}C/W.

Vapour Resistance

Condensation occurs when **warm moisture laden air comes in contact with a cooler surface.** In cold climates, vapour barriers should be used on or near the warm side of exterior walls clad with plywood. Plywood also provides good resistance to vapour transmission. Where an additional vapour barrier is required on the warm side, internal plywood linings may be considered to act as a secondary vapour barrier. For **further information** on the topic of thermal transmissions **see** CHAPTER 15.

4.5 Acoustical Properties

Plywood has unique properties which **allow it to be** effectively **used in sound control and reduction for residential and industrial applications**. Audible sound is a propagation of energy and is usually measured in terms of decibels (dB). 1 dB is the lower threshold of human hearing while 130 dB is considered the threshold of pain.

Sound waves in air is energy in motion and may be absorbed or reflected by a surface. Plywood, like other materials will absorb some of the sound energy and reflect the remainder. A material which exhibits **perfect absorptivity** is rated as **1.0**; a **perfect reflector** of sound would have a **co-efficient of sound absorption of 0.0**. The acoustic properties of plywood will vary with density, moisture content and surface coatings, however for most practical purposes plywood can be considered a reflector of sound. Relative co-efficients of sound absorption are given in TABLE 4.2. For **further information see** CHAPTER 14.

Material	Coefficient
Open Window	1.0
Brick	0.03
Window glass	0.03
Plywood	0.04

TABLE 4.2: Sound Absorption Co-efficients of Various Building Materials

4.6 Electrical Properties

Plywood and **LVL** are **excellent electrical insulators**, **provided** they are **in the dry condition**. Resistance falls off considerably with an increase of moisture content. The **glueline** in plywood and LVL is **not as**

effective an **insulator as** the **wood** itself. This will **not** be **of significance** in applications **in electric fields in** the range of **household voltages**, but it **may be** important **on certain test benches supporting sensitive electrical instruments.**

4.7 Chemical Resistance

Plywood and **LVL** are **highly resistant to many chemicals** and are **effectively used in** many industrial **applications involving** contact with chemicals including **dilute acids**, **alkalies, organic chemicals**, **neutral** and **acid salts**, **both hot and cold**. Provided the chemical reagent has a **pH above 2** and **below 10**, any **weakening** effect will be **minimal at room temperature**.

4.8 Workability and Bending Radii

Structural **plywood** and structural **LVL can** be **sawn**, **drilled**, **shaped**, **nailed, screwed** and **glued similarly to solid wood**. In addition structural **plywood can be moulded** and **curved**. TABLE 4.3 gives bending radii for various thicknesses of structural plywood. These **radii** can be **further reduced** by **soaking** or **steaming** the **panel prior to bending**.

Nominal Thickness (mm)	Along face (m)	Across face (m)
4.5	1.1	0.6
7	1.8	1.0
9	2.3	1.3
12	3.6	2.4
15	4.6	3.0

TABLE 4.3: Recommended Minimum Bending Radii for Plywood Linings

Notes

1. These **radii** are **theoretical values only** and have not been verified experimentally
2. **Thicker panels require considerable force and increased fixings** to pull and hold the panel in a tight radius.

FIGURE 4.1 shows the **orientation** of the **bent plywood sheet with respect to** the **face of** the **sheet.**

(a) **(b)**

FIGURE 4.1: Plywood sheet bent in the easy direction (a) and hard direction (b)

4.9 Plywood Density

The **density** of **plywood** and **LVL** is **approximately equivalent to** the **density of** the **timber species from which they were manufactured**. The **density** of **pine plywood** is typically in the **range 500 to 650 kg/m^3**. **Eucalypt hardwood plywood density can exceed 900 kg/m^3** depending on the timber species used.

Part Two

Plywood & LVL Design Principles, Procedures and Application

Structural Plywood – Design Principles and Procedures

Structural LVL – Design Principles and Procedures

Basic Structural Plywood & LVL Building Components

CHAPTER 5

5 STRUCTURAL PLYWOOD - DESIGN PRINCIPLES & PROCEDURES

5.1 Introduction – Principles

The design strength capacity and stiffness of structural plywood, whether loaded normal to the face of the sheet or in the plane of the panel, is calculated using standard principles of engineering mechanics**.** **Structural plywood characteristic properties** are **allocated via** the **F-Grade system**. Design capacities are then determined by multiplying the characteristic property by a section property and capacity and in-service factors. The essential **differences in** the **design process for structural plywood** when **compared** with **solid** (sawn) **timber**, arise as a **result** of the **cross-lamination** of the **plywood veneers**. In **plywood**, those **veneers** with grain direction **orientated in the direction** of the **principal stress** are considered to **transfer all the loads to the supports**. **Shear stresses** are the **exception**, being **resisted by all veneers**. The **contribution** of **each veneer** to the structural plywood capacity, with respect to veneer thickness and orientation, is **allowed for** by using **parallel ply theory** in the **derivation** of the **plywood section properties**.

5.2 Characteristic Strengths and Stiffness

Characteristic strengths and **stiffness values** are derived from test and are an **estimate of** the **5th percentile strength** and **average stiffness of** the **population** from which the reference sample is taken. Structural plywood characteristic strength and stiffness values are typically allocated via the F-grade classification system, as displayed in TABLE 5.1. This Table is a reprint of TABLE 5.1 from AS1720.1-1997 Timber Structures Code. These values must be modified in accordance with the in service factors in AS1720.1-1997.

Stress Grade	Characteristic Strength, MPa				Short duration average modulus of elasticity MPa	Short duration average modulus of rigidity MPa
	Bending (f'_b)	Tension (f'_t)	Panel Shear (f'_s)	Compression in the plane of the sheet (f'_c)	(E)	(G)
F34	100	60	6.8	75	21 500	1 075
F27	80	50	6.8	60	18 500	925
F22	65	40	6.8	50	16 000	800
F17	50	30	6.8	40	14 000	700
F14	40	25	6.1	30	12 000	625
F11	35	20	5.3	25	10 500	525
F8	25	15	4.7	20	9 100	455
F7	20	12	4.2	15	7 900	345

TABLE 5.1: Structural Plywood – Characteristic Properties for F-Grades (Moisture Content not more than 15%)

5.3 Section Properties

Parallel Ply Theory

Parallel Ply theory is used **to calculate** the **structural plywood section properties**, e.g. **Second Moment of Area, (I)** and **Section Modulus, (Z)**. Parallel Ply theory accounts for the differing strength and stiffness properties in the length and width directions of the plywood panel which results from the alternating grain direction of individual veneers in a plywood sheet. **Parallel Ply theory assumes veneers** with **grain direction parallel to** the **span**, **carry all** of the **bending from** the **applied load**, **to** the **supports**, as shown In

FIGURE 5.1. Veneers with grain direction perpendicular to the span are assumed to contribute nothing to strength and only a minor amount (3%) to stiffness.

Methods for **determination** of **I** are given in **Appendix 0.**

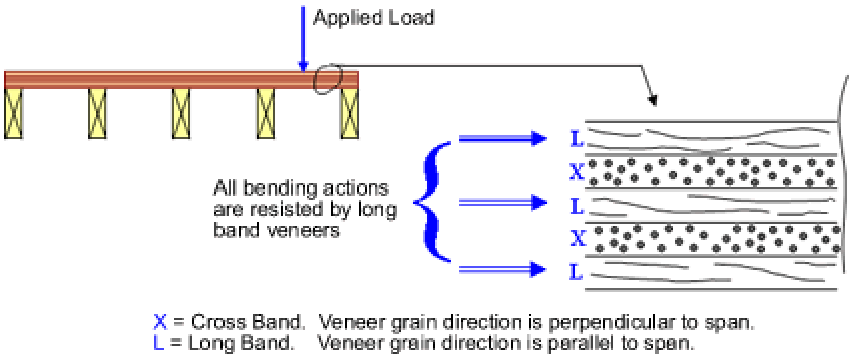


FIGURE 5.1: Parallel Ply Theory

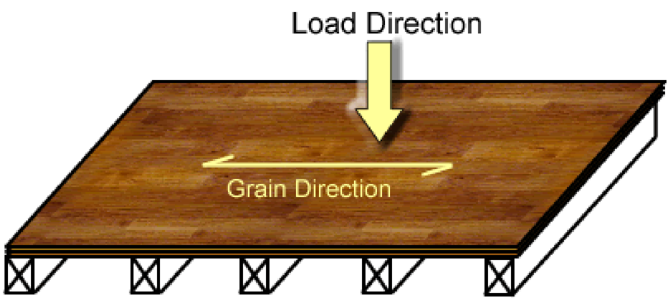
Identification Code

The plywood Identification Code provides information on the veneer arrangement within the structural plywood. This information is required to establish the section properties of a particular plywood. The **Identification Code gives** the following information: the **nominal plywood thickness**, the **face veneer thicknesses multiplied by 10**, and **the number of plies** in the assembly. For example, the **ID code 21-30-7 describes a 21 mm thick plywood** with **3.0 mm face veneer thicknesses and 7 veneer layers.**

5.4 Structural Plywood - Loaded Normal to the Face

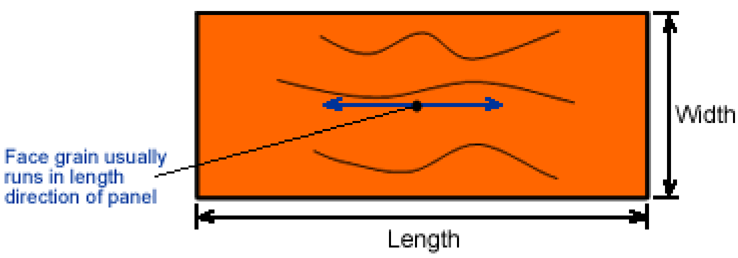
Typical applications in which structural plywood is **loaded normal** to the face include **flooring**, **cladding**, **bridge decking, trafficable roofs, and signboards.**

<u>**Section Properties – Standard Plywood Layups**</u>



Section properties for standard plywood constructions loaded normal to the plane of the plywood panel, are given with respect to the orientation of the plywood face grain direction relative to the span direction. The face veneer grain direction of structural plywood panels usually runs in the panel length direction. **Thicker veneers**, further from the panel neutral axis and with **grain direction parallel to span**, will be the **major contributors to I**, and therefore to both **bending strength and stiffness**.

TABLE 5.3 gives **section properties** for plywood **loaded normal to the plane** of the plywood panel. These are for standard thicknesses and constructions of structural plywood specified in AS/NZS 2269 together with some additional thicknesses made by some manufacturers. A **method for calculating the section modulus (Z)** and **second moment of area (I)**, for **structural plywood loaded normal to the face**, is **detailed** in **Appendix J of AS1720.1–1997** Timber Structures Code. This Appendix is reprinted in **Appendix 0** of this manual.



<u>**Load Distribution Width**</u>

When **calculating strength** and **stiffness capacities** for **concentrated loads** applied normal to the plywood face, it is **necessary** to **determine** the **distribution width** of the concentrated load **across** the **plywood**

sheet width. **Load distribution widths** established from testing conducted by the EWPAA*, and **used in** calculating EWPAA **span/deflection tables** for **structural plywood flooring**, are **reproduced in** TABLE 5.2.

Plywood Thickness (mm)	Load Distribution Width (mm)
12 – 13	400
15 – 19	450
20 – 25	520
26+	600

TABLE 5.2: Load Distribution Widths

*Other methods for establishing load distribution width are used, including formula based on the ratio of the I values for veneers parallel to and perpendicular to the direction of the principal stress.

Bending Strength and Bending Stiffness for Loading Normal to the Face

When **loaded normal to the face** of the plywood sheet, **parallel ply theory assumes veneers with face grain direction parallel to the span are** the **sole contributors to bending strength** and the **major contributors to bending stiffness**. Veneers with **grain direction perpendicular** to the span direction **contribute** nothing to bending strength and only **3% to bending stiffness**. The **outermost veneers** furtherest from the panel neutral axis and orientated in the span direction **carry** the **maximum tension and compression flexure forces** and are the **major contributors to** the second moment of area (**I**) and section modulus (**Z**) and therefore bending capacity.

In typical applications where the plywood is loaded normal to the face, such as flooring, bending stiffness will often be the governing criteria that determines the plywood specification. When setting **deflection limits** for applications in which **clearance limits are critical**, **allowance** should be made **for the modulus of elasticity** given in AS1720.1-1997 **(and reprinted in TABLE 5.1), being an average modulus of elasticity**. However, it should also be noted that the **process control applied to EWPAA/JAS-ANZ** branded products **minimises** the **variability of** the **E** value from the published average value.

For evaluation of bending strength TABLE 5.4 provides comparative bending strength ($\mathbf{f'_b.Z}$) values for a range of standard plywood constructions and stress grades.

For evaluation of bending stiffness TABLE 5.5 gives comparative values of (**EI**) for **structural plywood loaded normal to the face**, for a range of stress grades and standard plywood constructions. The **table provides indicative stiffness values** for both **plywood** supported with **face grain** orientated **parallel** to the span and for **plywood** supported with **face grain** orientated **perpendicular to span.**

Shear Strength (interlamina shear) for Loading Normal to the Face

The **interlamina shear strength** of structural plywood loaded normal to the panel face is calculated **based on** a **shear area of**:

A_s = **2/3 bt** (derived from the basic beam shear equation)
where: **b** = **load distribution width** (refer TABLE 5.2);
and: **t** = **full thickness** of the plywood sheet.

For applications where **high concentrated loads** are present, the plywood capacity for **punching** or local **shear** may also need to be checked. The relevant shear area is then:

A_s = **perimeter of loaded area x full thickness of the panel**

It should be noted that the **shear capacity** of structural **plywood loaded normal to** the **face** is **governed by** the **"rolling" shear tendency** of the plywood cross-bands. Rolling shear is a term used to describe shearing forces which tend to roll the wood fibres across the grain. The **reduced shear capacity** of plywood loaded normal to the face, **due to rolling shear**, is **accounted for** in AS1720.1-1997, **by** the use of an **assembly factor** $\mathbf{g_{19}}$ in the calculation of **both interlamina and punching shear capacity**.

TABLE 5.4 provides **interlamina shear strengths (0.4 x f'_sA_s)** for a range of standard plywood constructions and stress grades.

Bearing Strength for Loading Normal to the Face

Plywood (and all timber) have **less compressive capacity** when **load** is applied **perpendicular to the grain**, **compared** to when load is applied **parallel to grain**. The bearing or crushing strength of the plywood may govern design where high localised point loads are applied to the plywood surface. For example, **small diameter metal castor wheels supporting high loads**, **on** structural **plywood flooring**. Where bearing strength is critical, the simplest solution is often to increase the bearing area. In the example of the small diameter metal wheels, the **use of larger diameter wheels** and/**or softer compound wheels** will spread the load.

Characteristic bearing strengths are not incorporated in the F rating system. Characteristic bearing strength can be obtained from plywood manufacturers.

5.5 Structural Plywood Loaded In the Plane of the Panel

Some applications in which **structural plywood** is **loaded in** its **plane** are shown in FIGURE 5.2 and include **bracing walls**, **structural diaphragms** such as floors and ceilings loaded in their plane and the **webs of composite** beams. Typically the plywood acts as part of a composite member in a structural system with the structural **plywood** being **utilised** for **its capacity** to **carry high in-plane shear loads**. The **tension** and **compression** actions due to **bending** are carried by the framing members in the composite system. For example, in **bracing walls** and **diaphragms** the **plywood** is designed to **carry in-plane shear loads**. The **top** and **bottom wall plate** members or **edge framing members carry** the **tension** and **compression due to bending** loads. Similarly in **composite beams**, the **flange members carry** the **compression** and **tension forces** while the structural **plywood web/s resist the in-plane shear forces**.

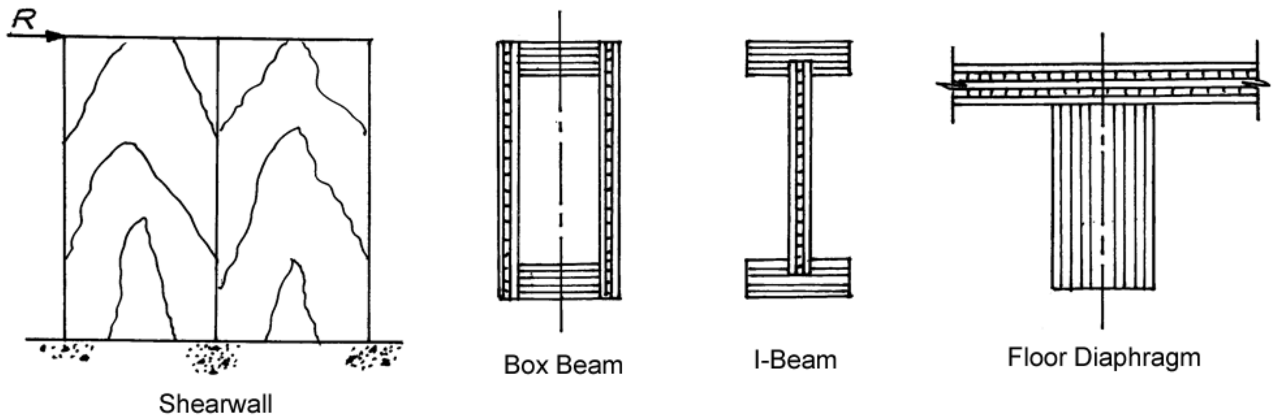


FIGURE 5.2: Structural Plywood Loaded in its plane

Section Properties for Shear Strength and Shear Deformation of Structural Plywood Loaded In-Plane

Section properties for **shear strength** and **shear deformation** are based on the **full cross-sectional thickness** of the panel. For shear capacity in bending, the area of shear A_s= 2/3td and for local shear A_s = dt, where t = full thickness of the plywood panel and d = depth of panel.

Section Properties for Bending, Tension and Compressive Strength and Bending Deflection of Structural Plywood Loaded In-Plane

Section properties for structural plywood loaded in plane, **for bending, tension**, and **compressive strength** and **bending deflection**, are based on the depth of the plywood panel and the **sum of the thicknesses of the veneers with grain direction orientated in the span or stress direction**.

TABLE 5.3: Standard Structural Plywood Constructions, Thickness of Parallel Plies (tp), Second Moment of Area (Ip) and Section Modulus (Zp)

Nominal Thickness	Nominal Mass[2]	Identification Code	Nominal Thickness of Individual Plies Through Assembly (mm)	Face grain parallel to span			Face grain perpendicular to span		
				Thickness of Parallel Plies (t_{pt})	Second Moment of Area (I_p)	Section Modulus (Z_p)	Thickness of Parallel Plies (t_{pl})	Second Moment of Area (I_s)	Section Modulus (Z_p)
mm	kg/m^2			mm	mm^4/mm	mm^3/mm	mm	mm^4/mm	mm^3/mm
4.5	2.7	4.5-15-3	1.5/1.5/1.5	3	7.3	3.3	1.5	0.5	0.4
6	3.6	6-15-3	1.5/3.0/1.5	3	16	5.3	3	2.7	1.5
7	4.2	**7-24-3**	**2.4/2.4/2.4**	4.8	30	8.3	2.4	2.1	1
7.5	4.5	7.5-25-3	2.5/2.5/2.5	5	34	9	2.5	2.3	1
9	5.4	9-15-3	1.5/1.5/3.0/1.5/1.5 or 1.5/2.4/1.5/2.4/1.5	6 or 4.5	45	10	3 or 4.8	17	5.3
9	5.4	**9-30-03**	**3.0/3.0/3.0**	6	60	13	3	4	1.5
12	7.2	12-15-5	1.5/3.0/3.0/3.0/1.5	6	85	14.5	6	60	13
12	7.2	12-24-5	2.4/2.4/2.4/2.4/2.4	7.2	115	19	4.8	33	8.3
12.5	7.5	**12.5-25-5**	**2.5/2.5/2.5/2.5/2.5**	7.5	130	20.5	5	38	9
13	7.8	13-24-5	2.4/3.0/2.4/3.0/2.4	9	145	21.5	6	55	11.5
13	7.8	13-30-5	3.0/2.4/2.4/2.4/3.0	7.8	165	24.5	4.8	35	8.3
14	8.4	14-24-5	2.4/3.0/3.0/3.0/2.4	7.8	160	23.5	6	65	13
14	8.4	14-30-5	3.0/2.4/3.0/2.4/3.0	9	185	26.5	4.8	43	9.6
15	9	15-15-7	1.5/2/4/2.4/2.4/2.4/2.4/1.5	7.8	170	22.5	7.2	120	19
15	9	15-24-7	2.4/2.4/1.5/2.4/1.5/2.4/2.4	7.8	205	27.5	7.2	85	15
15	9	**15-30-5**	**3.0/3.0/3.0/3.0/3.0**	9	225	29.5	6	65	13
17	10.2	17-15-7	1.5/3.02.4/3.0/2.4/3.0/1.5	7.8	220	25.5	9	190	26.5
17	10.2	17-24-7	2.4/2.4/2.4/2.4/2.4/2.4/2.4	9.6	285	33.5	7.2	120	19
17.5	10.5	17.5-25-7	2.5/2.5/2.5/2.5/2.5/2.5/2.5	10	320	36.5	7.5	140	20.5
18	10.8	18-15-7	1.5/3.0/3.0/3.0/3.0/3.0/1.5	9	270	29.5	9	230	29.5
18	10.8	18-30-7	3.0/2.4/2.4/2.4/2.4/2.4/3.0	10.8	375	41.5	7.2	125	19
19	11.4	19-24-7	2.4/3.0/2.4/3.0/2.4/3.0/2.4	9.6	360	38	9	190	26.5
19	11.4	19-24-9	2.4/2.4/1.5/2.4/1.5/2.4/1.5/2.4/2.4	9.3	380	39.5	9.6	200	26.5
19	11.4	19-30-7	3.0/2.4/3.0/2.4/3.0/2.4/3.0	12	450	46.5	7.2	155	21.5
21	12.6	21-24-9	2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4	12	565	51.5	9.6	300	33.5
21	12.6	21-30-7	3.0/3.0/3.0/3.0/3.0/3.0/3.0	12	555	52.5	9	240	29.5
25	15	25-30-9	3.0/2.4/3.0/2.4/3.0/2.4/3.0/2.4/3.0	15	900	70.5	9.6	380	38
25	15	25-30-9	3.0/3.0/2.4/2.4/2.4/2.4/2.4/3.0/3.0	13.2	900	70.5	12	380	38
26	15.6	26-24-11	2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4	1.4	990	74	12	590	51.5
27	16.2	27-30-9	3.0/3.0/3.0/3.0/3.0/3.0/3.0/3.0/3.0	15	1110	81	12	580	52.5
28	16.8	28-15-13	1.5/2.4/2.4/2.4/2.4/2.4/1.5/2.4/2.4/2.4/2.4/2.4/1.5	14.1	1070	73.5	14.4	920	69.5
28	16.8	28-30-11	3.0/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/3.0	15.6	1210	86.5	12	595	51.5

Notes:

1. The subscript "p" in Ip and Zp denotes plywood loaded normal to the plane of the plywood panel
2. Mass of plywood is based on a density of 600 kg/m^3. This will be appropriate for most pine species of plywood. Eucalypt hardwood plywood will usually be denser

TABLE 5.4: Limit State Bending and Shear Strength Capacity – Loading Normal to the Plane of the Plywood Panel

Nominal Thickness (mm)	I.D. Code	Bending Strength Capacity: (N/mm width)														Shear Strength Capacity in Bending $= 0.4 \times f'_s . A_s$ (N/mm width)						
		Face Grain **Parallel** to Span = $f'_b . Z_{para}$							Face Grain **Perpendicular** to Span = $f'_b . Z_{perp}$													
		F8	F11	F14	F17	F22	F27	F34	F8	F11	F14	F17	F22	F27	F34	F8	F11	F14	F17	F22	F27	F24
4.5	4.5-15-3	83	116	132	165	215	264	330	10	14	16	20	26	32	40	6	6	7	8	8	8	8
6	6-15-3	133	186	212	265	345	424	530	38	53	60	75	98	120	150	8	8	10	11	11	11	11
7	7-24-3	208	219	332	415	540	664	830	25	35	40	50	65	80	100	9	10	11	13	13	13	13
7.5	7.5-25-3	225	315	360	450	585	720	900	25	35	40	50	65	80	100	9	11	12	14	14	14	14
9	9-15-5	250	350	400	500	650	800	1000	133	186	212	265	345	424	530	11	13	15	16	16	16	16
9	9-30-3	325	455	520	650	845	1040	1300	38	53	60	75	98	120	150	11	13	15	16	16	16	16
12	12-15-5	363	508	580	725	943	1160	1450	325	455	520	650	845	1040	1300	15	17	20	22	22	22	22
12	12-24-5	475	665	760	950	1235	1520	1900	208	291	332	415	540	664	830	15	17	20	22	22	22	22
12.5	12.5-25-5	513	718	820	1025	1333	1640	2050	225	315	360	450	585	720	900	16	18	20	23	23	23	23
13	13-24-5	538	753	860	1075	1398	1720	2150	288	403	460	575	748	920	1150	16	18	21	24	24	24	24
13	13-30-5	613	858	980	1225	1593	1960	2450	208	291	332	415	540	664	830	16	18	21	24	24	24	24
14	14-24-5	588	823	940	1175	1528	1880	2350	325	455	520	650	845	1040	1300	18	20	23	25	25	25	25
14	14-30-5	663	928	1060	1325	1723	2120	2650	240	336	384	480	624	768	960	18	20	23	25	25	25	25
15	15-15-7	563	788	900	1125	1463	1800	2250	475	665	760	950	1235	1520	1900	19	21	24	27	27	27	27
15	15-24-7	688	963	1100	1375	1788	2200	2750	375	525	600	750	975	1200	1500	19	21	24	27	27	27	27
15	15-30-5	738	1033	1180	1475	1918	2360	2950	325	455	520	650	845	1040	1300	19	21	24	27	27	27	27
17	17-15-7	638	893	1020	1275	1658	2040	2550	663	928	1060	1325	1723	2120	2650	21	24	28	31	31	31	31
17	17-24-7	838	1173	1340	1675	2178	2680	3350	475	665	760	950	1235	1520	1900	21	24	28	31	31	31	31
17.5	17.5-25-7	913	1278	1460	1825	2373	2920	3650	513	718	820	1025	1333	1640	2050	22	25	28	32	32	32	32
18	18-15-7	738	1033	1180	1475	1918	2360	2950	738	1033	1180	1475	1918	2360	2950	23	25	29	33	33	33	33
18	18-30-7	1038	1453	1660	2075	2698	3320	4150	475	665	760	950	1235	1520	1900	23	25	29	33	33	33	33
19	19-24-7	950	1330	1520	1900	2470	3040	3800	663	928	1060	1325	1723	2120	2650	24	27	31	34	34	34	34
19	19-24-9	988	1383	1580	1975	2568	3160	3950	663	928	1060	1325	1723	2120	2650	24	27	31	34	34	34	34
19	19-30-7	1163	1628	1860	2325	3023	3720	4650	538	753	860	1075	1398	1720	2150	24	27	31	34	34	34	34
20	20-30-7	1240	1736	1984	2480	3224	3968	4960	635	889	1016	1270	1651	2032	2540	25	28	33	36	36	36	36
21	21-24-9	1288	1803	2060	2575	3348	4120	5150	838	1173	1340	1675	2178	2680	3350	26	30	34	38	38	38	38
21	21-30-7	1313	1838	2100	2625	3413	4200	5250	738	1033	1180	1475	1918	2360	2950	26	30	34	38	38	38	38
25	25-30-9	1763	2468	2820	3525	4583	5640	7050	950	1330	1520	1900	2470	3040	3800	31	35	41	45	45	45	45
26	26-24-11	1850	2590	2960	3700	4810	5920	7400	1288	1803	2060	2575	3348	4120	5150	33	37	42	47	47	47	47
27	27-30-9	2025	2835	3240	4050	5265	6480	8100	1313	1838	2100	2625	3413	4200	5250	34	38	44	49	49	49	49
31	31-24-13	2540	3556	4064	5080	6604	8128	10160	1935	2709	3096	3870	5031	6192	7740	39	44	50	56	56	56	56
33	33-30-11	2933	4106	4692	5865	7625	9384	11730	2128	2979	3404	4255	5532	6808	8510	41	47	54	60	60	60	60
36	36-24-15	3303	4624	5284	6605	8587	10568	13210	2605	3647	4168	5210	6773	8336	10420	45	51	59	65	65	65	65
39	39-30-13	3970	5558	6352	7940	10322	12704	15880	3023	4232	4836	6045	7859	9672	12090	49	55	63	71	71	71	71

TABLE 5.5: Indicative Stiffness Values (EI) Per MM Width – Loading Normal to the Plane of the Plywood Panel

Nominal Thickness (mm)	I.D. Code	EI x 10^3 Nmm^2/mm width													
		Face Grain **Parallel** to Span							Face Grain **Perpendicular** to Span						
		F8	F11	F14	F17	F22	F27	F34	F8	F11	F14	F17	F22	F27	F34
4.5	4.5-15-3	66	77	88	102	117	135	157	5	5	6	7	8	9	11
6	6-15-3	146	168	192	224	256	296	344	25	28	32	38	43	49	58
7	7-24-3	273	315	360	420	480	555	645	19	22	25	29	34	38	45
7.5	7.5-25-3	309	357	408	476	544	629	731	21	24	28	32	37	41	49
9	9-15-5	410	473	540	630	720	833	968	155	179	204	238	272	306	366
9	9-30-3	546	630	720	840	960	1110	1290	36	42	48	56	64	72	86
12	12-15-5	774	893	1020	1190	1360	1573	1828	546	630	720	840	960	1080	1290
12	12-24-5	1047	1208	1380	1610	1840	2128	2473	300	347	396	462	528	594	710
12.5	12.5-25-5	1183	1365	1560	1820	2080	2405	2795	346	399	456	532	608	684	817
13	13-24-5	1320	1523	1740	2030	2320	2683	3118	501	578	660	770	880	990	1183
13	13-30-5	1502	1733	1980	2310	2640	3053	3548	319	368	420	490	560	630	753
14	14-24-5	1456	1680	1920	2240	2560	2960	3440	592	683	780	910	1040	1170	1398
14	14-30-5	1684	1943	2220	2590	2960	3423	3978	391	452	516	602	688	774	925
15	15-15-7	1547	1785	2040	2380	2720	3145	3655	1092	1260	1440	1680	1920	2160	2580
15	15-24-7	1866	2153	2460	2870	3280	3793	4408	774	893	1020	1190	1360	1530	1828
15	15-30-5	2048	2363	2700	3150	3600	4163	4838	592	683	780	910	1040	1170	1398
17	17-15-7	2002	2310	2640	3080	3520	4070	4730	1729	1995	2280	2660	3040	3420	4085
17	17-24-7	2594	2993	3420	3990	4560	5273	6128	1092	1260	1440	1680	1920	2160	2580
17.5	17.5-25-7	2912	3360	3840	4480	5120	5920	6880	1274	1470	1680	1960	2240	2520	3010
18	18-15-7	2457	2835	3240	3780	4320	4995	5805	2093	2415	2760	3220	3680	4140	4945
18	18-30-7	3413	3938	4500	5250	6000	6938	8063	1138	1313	1500	1750	2000	2250	2688
19	19-24-7	3276	3780	4320	5040	5760	6660	7740	1729	1995	2280	2660	3040	3420	4085
19	19-24-9	3458	3990	4560	5320	6080	7030	8170	1820	2100	2400	2800	3200	3600	4300
19	19-30-7	4095	4725	5400	6300	7200	8325	9675	1411	1628	1860	2170	2480	2790	3333
20	20-30-7	4277	4935	5640	6580	7520	8695	10105	1775	2048	2340	2730	3120	3510	4193
21	21-30-7	5051	5828	6660	7770	8880	10268	11933	2184	2520	2880	3360	3840	4320	5160
21	21-24-9	5142	5933	6780	7910	9040	10453	12148	2730	3150	3600	4200	4800	5400	6450
24	24-32-9	7490	8643	9877	11523	13170	15227	17697	3039	3507	4008	4676	5344	6012	7181
25	25-30-9	8190	9450	10800	12600	14400	16650	19350	3458	3990	4560	5320	6080	6840	8170
26	26-24-11	9009	10395	11880	13860	15840	18315	21285	5369	6195	7080	8260	9440	10620	12685
27	27-30-9	10101	11655	13320	15540	17760	20535	23865	5278	6090	6960	8120	9280	10440	12470
31	31-24-13	14429	16649	19027	22198	25370	29334	34090	9294	10724	12256	14298	16341	18383	21958
33	33-30-11	17619	20330	23234	27107	30979	35820	41628	10450	12057	13780	16076	18373	20669	24688
36	36-24-15	21643	24972	28540	33296	38053	43999	51133	14799	17076	19516	22768	26021	29273	34965
39	39-30-13	28181	32516	37162	43355	49549	57291	66581	18151	20943	23935	27924	31914	35903	42884

5.6 Structural Plywood - Design Procedures

Limit State Design to AS 1720.1-1997

The **design capacity** of structural plywood designed in accordance with the **limit states design format** of AS 1720.1–1997, is achieved by **modifying** the **characteristic strength capacities by** a material capacity factor **Φ**, a **geometric section property**, and **in-service factors** (**k, j** and **g factors**). Structural capacity factor reliability is achieved through the use of these modified characteristic strength capacities and factored loads as detailed in AS/NZS 1170.1 - 1997.

Strength Limit State Capacity

The **strength limit state condition** is **satisfied when** the **design capacity of** the structural **plywood exceeds** the **design load effects from the factored loads**. That is:

	Φ R	**≥ S***
where	Φ R	= design capacity of the plywood member
	S*	= design action effect, eg. bending moment, M*, shear force, V*, etc.
and	**Φ R**	**= Φ k_{mod} [f_o'.X]**
where	**Φ**	= **capacity factor**
	k_{mod}	= product of relevant **modification factors**(eg. k_1, k_6, k_7, k_{12}, k_{19}, g_{19}).
	f_o'	= **appropriate characteristic strength**
	X	= **geometric section property**.

Serviceability Limit States Capacity

The **serviceability limit states** are **achieved** when **in-service displacements and vibrations are kept within acceptable limits**. Calculated bending deflections and shear deformations must be modified by in service modification factors (j_2, j_6, and g_{19}, as appropriate). **Guidance on serviceability limit states** are given **in Appendix B** of **AS 1720.1-1997.**

5.7 Strength & Stiffness limit states design capacities

5.7.1 Loading Normal to the Plane of the Plywood Panel

FIGURE 5.3 Shows Loading Normal to the Plane of the Plywood Panel.

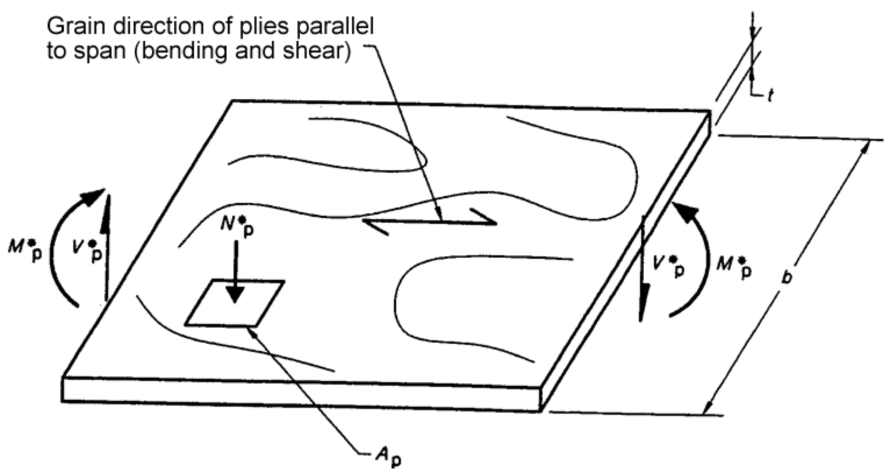


FIGURE 5.3: Notation for bearing and shear normal to the face of the plywood panel and for flatwise bending plywood

Strength Limit State

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied When	AS 1720.1-1997 Reference
Bending	M*p	$\Phi Mp = \Phi_1 k_{19} g_{19}[f'bZp]$	ΦMp > M*p	**Clause 5.4.2**
Shear	V*p	$\Phi Vp = \Phi k_1 k_{19} g_{19}[f'sAs]$	ΦVp > V*p	**Clause 5.4.3**
Bearing	N*p	$\Phi Np = \Phi k_1 k_7 k_{19} g_{19}[f'pAp]$	ΦNp > N*p	**Clause 5.4.4**

where:

M^*_p, V^*_p, N^*_p = **Design action effect** in bending, shear and bearing respectively
ΦM_p, ΦV_p, ΦN_p = **Design capacity** in bending, shear and bearing respectively
Φ = **Capacity factor** for plywood
k_1 = **Duration of load strength** modification **factor**
k_7 = **Length of bearing** modification **factor**
k_{19} = **Moisture condition strength** modification **factor**
g_{19} = Plywood **assembly modification factor**
f'_b, f'_s, f'_p = **Characteristic strengths** in bending, panel shear and bearing normal to the plane of the panel respectively.
Z_p = Plywood **section modulus** $=I_p/y_p$
A_s = **shear plane area** = 2/3 x (bt) for shear in bending
= full shear area for local (punching) shear.
A_p = **bearing area** under the design load.

Serviceability Limit State

Calculated deflection x j_2 x j_6 x g_{19} $\leq$ deflection limit **Clause 5.4.5**

where:

j_2 = Duration of load **stiffness** modification **factor**
j_6 = **Moisture condition** stiffness modification **factor**
g_{19} = Plywood **assembly** modification **factor**

5.7.2 Loading in Plane of the Plywood Panel

FIGURE 5.4 shows loading in the plane of the plywood.

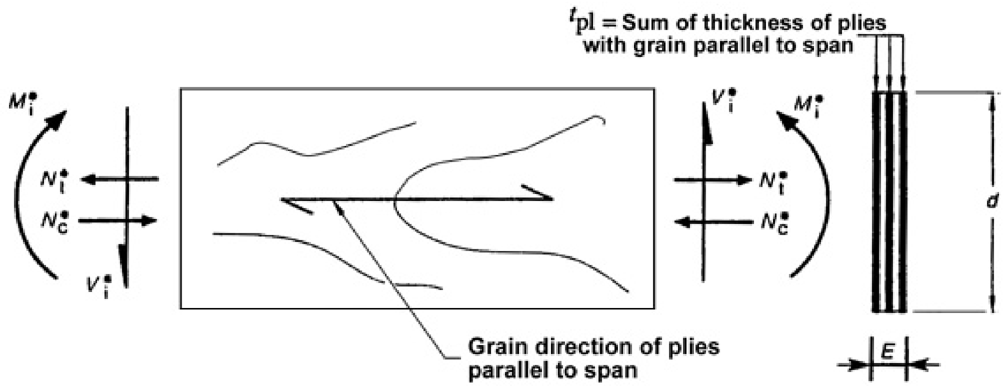


FIGURE 5.4: Notation for shear, compression and tension acting in the plane of a plywood panel and for edgewise bending

Strength Limit State

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied When:	AS 1720.1-1997 Reference
Bending	M*i	$\Phi Mi = \Phi k_1 k_{12} k_{19} g_{19}$[f'bZi]	ΦMi > M*I	**Clause 5.5.2**
Shear	V*i	$\Phi Vi = \Phi k_1 k_{12} k_{19} g_{19}$[f'sAs]	ΦVi > V*I	**Clause 5.5.3**
Tension	N*t	$\Phi Nt = \Phi k_1 k_{19} g_{19}$[f'tAt]	ΦNt > N*	**Clause 5.5.4**
Compression	N* c	$\Phi Nc = \Phi k_1 k_{12} k_{19} g_{19}$[f'cAc]	ΦNc > N*c	**Clause 5.5.5**

where:

M^*_i, V^*_i, N^*_t, N^*_c = **Design action effect** in edgewise bending, and shear, tension and compression in the plane of the plywood panel, respectively.

ΦM_i, ΦV_i, ΦN_t, ΦN_c = **Design capacity** in bending, shear, tension and compression respectively.

Φ = **Capacity factor** for plywood

k_1 = **Duration of load strength** modification **factor**

k_{12} = **Stability modification factor**

k_{19} = **Moisture condition strength** modification **factor**

g_{19} = Plywood **assembly modification factor**

f'_b, f'_s, f'_t, f'_c = **Characteristic strengths** in bending, panel shear, tension and compression respectively

Z_i = Plywood **section modulus** = $t_{pl}d^2/6$

A_s = **shear plane area** = 2/3 (dt) for shear in bending
= (dt) for localised shear

A_t, A_c = **Effective cross sectional area**
= t_{pl} x d for load applied parallel sum of the thickness of veneers with grain parallel to span
= t x d for load applied at 45^0 to plywood grain direction
= t_{pl}

Serviceability Limit State:

(Calculated bending deflection x j_2 xj_6 xg_{19})+(Calculated shear deflection x j_2 xj_6 xg_{19})≤ deflection limit

Clause 5.5.6 & 5.5.7

where: j_2 = Duration of load **stiffness** modification **factor**

j_6 = **Moisture condition** stiffness modification **factor**

g_{19} = Plywood **assembly** modification **factor**

5.8 Factors

Capacity Factor, Φ

The Φ factor given in TABLE 5.6, is a **material capacity factor** and **allows** for **variability in material strength and** the **consequence of failure**. The material **capacity factor**, **Φ**, assigned via AS1720.1-1997, to structural materials, is **based on current knowledge of product structural performance**, **intended structural application and material reliability**. The **capacity factors** applied to structural plywood manufactured to AS/NZS 2269 **reflect** the **high degree** of **manufacturing process control**, the **low material variability** and **high product reliability**.

Application of Structural member	Plywood Capacity Factor, Φ
All structural elements in houses. All secondary structural elements in structures other than houses	0.9
Primary structural elements in structures other than houses	0.8
Primary structural elements in structures intended to fulfil an essential service or post disaster function	0.75

TABLE 5.6: Capacity Factor, Φ

Factor k_1 – Load Duration

The $\mathbf{k_1}$ duration of load factor given in TABLE 5.7 **allows for** the **time dependant nature** of the **strength of timber**. A timber member subjected to a short term load without failure may fail over time if the load is sustained. The $\mathbf{k_1}$ factor allows for the **reduction in** the **strength capacity** of the plywood member **when subjected** to **long term loads**. For **load combinations** of **differing duration**, the **appropriate** $\mathbf{k_1}$ factor is **that for** the **shortest duration load**.

Type of Load	k_1
Permanent loads (50+ years duration)	0.57
Live loads on floors due to vehicles or people applied at frequent but irregular intervals (5 months total duration)	0.80
Live loads applied for periods of a few days and at infrequent intervals (5 days total duration)	0.94
Impact or wind loads (5 seconds duration)	1.00
Wind gust	1.15

TABLE 5.7: Duration of Load Strength Modification Factor

Factor k_6 – Ambient temperature factor

AS1720.1-1997

The ambient temperature factor relates **temperature effects** in buildings to **geographical locations** and is taken as k_6 = 1.0 for normal structures, except for coastal regions of Queensland north of latitude 25°S and all other regions of Australia north of 16°S. For these regions strength is modified by taking k_6 = 0.9. If floods are due to cyclonic winds then temperature modification may not be required.

Factor k_7 – Factor for length and position of bearing

AS1720.1-1997 Clause 2.4.4

The $\mathbf{k_7}$ bearing factor given in TABLE 5.8 may be used to increase the bearing capacity perpendicular to the grain for bearing lengths less than 150mm along the grain when the bearing length is 5mm or more from the end of the member.

Length of bearing of Member (mm)	12	25	50	75	125	150 or more
Value of k_7	1.85	1.60	1.30	1.15	1.05	1.00

TABLE 5.8: Factor for Length and Position of Bearing

k_{12} – Stability factor for plywood loaded in the plane of the panel

AS1720.1-1997 Appendix J

The $\mathbf{k_{12}}$ factor allows for the **reduction in strength due to buckling of plywood diaphragms loaded in-plane**. The **ratio** of the **plywood diaphragm depth to plywood thickness** is **critical in determining whether** the **diaphragm will buckle**. **TABLE 5.9** gives k_{12} factors for typical diaphragm depths and plywood and plywood thicknesses when diaphragm lateral edges are supported and subject to uniform edge forces.

TABLE 5.9: Buckling Strength of Plywood Diaphragms Loaded In-Plane – Appendix J of AS 17201.1-1997
Diaphragms with Lateral Edges Supported & Subjected to Uniform Edge Forces (from App. J2.2)

k_{12} - Bending

F14 $k_1 = 1.15$ **(Non conservative for $k_1>1.0$, F grades>F14)**

Nominal Thickness (mm)	ID Code	Face grain direction is horizontal (θ=0°)										Face grain direction is vertical (θ=90°)									
		Depth of Web (mm)										Depth of Web (mm)									
		150	200	300	400	450	600	800	900	1000	1200	150	200	300	400	450	600	800	900	1000	1200
4.5	4.5-15-3	1.00	0.84	0.49	0.36	0.33	0.27	0.24	0.23	0.23	0.22	1.00	1.00	0.78	0.53	0.46	0.34	0.29	0.27	0.26	0.24
7	7-24-3	1.00	1.00	0.89	0.59	0.51	0.37	0.30	0.28	0.26	0.24	1.00	1.00	1.00	0.99	0.82	0.55	0.43	0.38	0.34	0.30
7.5	7.5-25-3	1.00	1.00	0.99	0.65	0.55	0.40	0.31	0.29	0.27	0.25	1.00	1.00	1.00	1.00	0.91	0.60	0.46	0.41	0.37	0.32
9	9-30-3	1.00	1.00	1.00	0.84	0.71	0.49	0.36	0.33	0.30	0.27	1.00	1.00	1.00	1.00	1.00	0.78	0.57	0.50	0.44	0.37
12	12-24-5	1.00	1.00	1.00	1.00	1.00	1.00	0.70	0.59	0.52	0.42	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.87	0.74	0.58
15	15-30-5	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.82	0.70	0.55	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.79
17	17-24-7	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.84	0.65	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96

k_{12}- Compression

F14, $k_1 = 1.15$ **(Non conservative for $k_1>1.0$, F grades>F14)**

Nominal Thickness (mm)	ID Code	Face grain direction is horizontal (θ=0°)										Face grain direction is vertical (θ=90°)									
		Depth of Web (mm)										Depth of Web (mm)									
		150	200	300	400	450	600	800	900	1000	1200	150	200	300	400	450	600	800	900	1000	1200
4.5	4.5-15-3	0.38	0.30	0.25	0.23	0.22	0.21	0.21	0.21	0.20	0.20	0.56	0.40	0.29	0.25	0.24	0.22	0.21	0.21	0.21	0.21
7	7-24-3	0.64	0.45	0.31	0.26	0.25	0.23	0.22	0.21	0.21	0.21	1.00	0.69	0.42	0.32	0.30	0.25	0.23	0.22	0.22	0.21
7.5	7.5-25-3	0.70	0.48	0.33	0.27	0.26	0.23	0.22	0.21	0.21	0.21	1.00	0.76	0.45	0.34	0.31	0.26	0.24	0.23	0.22	0.22
9	9-30-3	0.92	0.61	0.38	0.30	0.28	0.25	0.23	0.22	0.22	0.21	1.00	1.00	0.46	0.40	0.36	0.29	0.25	0.24	0.23	0.22
12	12-24-5	1.00	1.00	0.72	0.49	0.43	0.33	0.27	0.26	0.25	0.23	1.00	1.00	0.97	0.63	0.54	0.39	0.31	0.29	0.27	0.25
15	15-30-5	1.00	1.00	1.00	0.66	0.56	0.40	0.31	0.29	0.27	0.25	1.00	1.00	1.00	0.88	0.74	0.50	0.37	0.33	0.31	0.28
17	17-24-7	1.00	1.00	1.00	0.79	0.67	0.46	0.35	0.32	0.29	0.27	1.00	1.00	1.00	1.00	0.89	0.59	0.42	0.37	0.34	0.30

k_{12}- Shear

F11, $k_1 = 1.15$ **(Non conservative for $k_1>1.0$)**

Nominal Thickness (mm)	ID Code	Face grain direction is horizontal (θ=0°)										Face grain direction is vertical (θ=90°)									
		Depth of Web (mm)										Depth of Web (mm)									
		150	200	300	400	450	600	800	900	1000	1200	150	200	300	400	450	600	800	900	1000	1200
4.5	4.5-15-3	0.79	0.66	0.57	0.54	0.53	0.52	0.51	0.51	0.51	0.50	1.00	1.00	0.77	0.65	0.62	0.57	0.54	0.53	0.52	0.52
7	7-24-3	1.00	0.89	0.67	0.60	0.58	0.54	0.52	0.52	0.52	0.51	1.00	1.00	1.00	0.87	0.79	0.66	0.59	0.57	0.56	0.54
7.5	7.5-25-3	1.00	0.95	0.70	0.61	0.59	0.55	0.53	0.52	0.52	0.51	1.00	1.00	1.00	0.92	0.83	0.69	0.61	0.58	0.57	0.55
9	9-30-3	1.00	1.00	0.79	0.66	0.63	0.57	0.54	0.53	0.53	0.52	1.00	1.00	1.00	1.00	0.98	0.77	0.65	0.62	0.60	0.57
12	12-24-5	1.00	1.00	1.00	1.00	1.00	0.79	0.66	0.63	0.60	0.57	1.00	1.00	1.00	1.00	1.00	1.00	0.79	0.73	0.68	0.63
15	15-30-5	1.00	1.00	1.00	1.00	1.00	0.95	0.75	0.70	0.66	0.61	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.85	0.79	0.70
17	17-24-7	1.00	1.00	1.00	1.00	1.00	1.00	0.82	0.76	0.71	0.64	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.87	0.76

FIGURE 5.5 shows an **I-beam defining** the **relevant design parameters**, with respect to the **values given** in **TABLE 5.9**, **for** the **edge axial forces, moments and shears.**

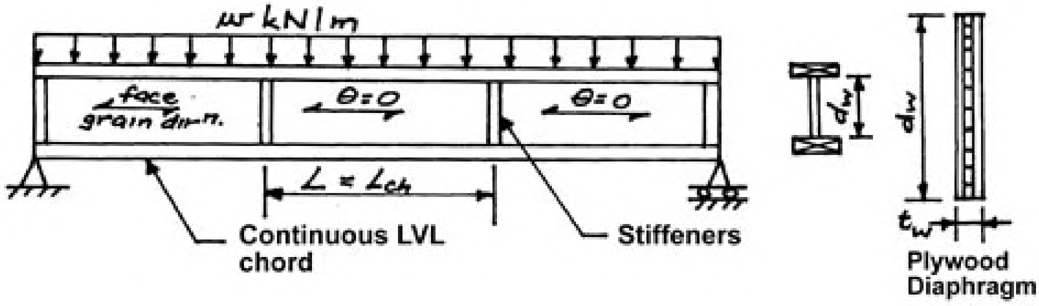


FIGURE 5.5: Defines diaphragm buckling parameters

k_{19} – Moisture content factor

AS1720.1-1997
Clause 5.3.3

The $\mathbf{k_{19}}$ moisture content factor given in TABLE 5.10, is used to modify plywood strength capacity to **allow for** the **reduction in strength** that will result **if** for a **12 month period** the average moisture content of the plywood in service remains higher than 15%. Where the average moisture content of plywood, over a 12 month period is less than or equal to 15%, k_{19} = 1.0. Some examples of where average moisture content may remain above 15% for a 12 month period are applications in continuously humid environments and also where the plywood is constantly sprayed with water.

Strength Property	Factor k_{19}	
	Moisture Content* 15% or less	Moisture Content* 25% or more
Bending	1.0	0.6
Tension in plane of sheet	1.0	0.7
Shear	1.0	0.6
Compression in plane of sheet	1.0	0.4
Compression in normal to plane of sheet	1.0	0.45

For moisture contents between 15 and 25%, use linear interpolation to obtain k_{19} factor

TABLE 5.10: Moisture Content Factor, k_{19}

g_{19} – Plywood Assembly Factor

AS1720.1-1997
Clause 5.3.5

The $\mathbf{g_{19}}$ **factor** in TABLE 5.11 and TABLE 5.12, **allows for the differing grain orientation** of the timber veneers within the plywood sheet. The $\mathbf{g_{19}}$ factor **affects both strength** and **stiffness** and varies depending on whether the plywood is loaded in plane or normal to the face.

For Plywood Loaded Normal to the Face : The $\mathbf{g_{19}}$ assembly factor given in TABLE 5.11 is used to **increase** the **bending** strength **capacity** of **three ply plywood** when **loaded** with face **grain perpendicular to span** and to **reduce** the **shear** strength **capacity**. For all other properties listed in TABLE 5.11 g_{19} = 1. The $\mathbf{g_{19}}$ factor applied to the bending strength capacity of plywood with **3 veneer layers, loaded perpendicular to span**, **compensates** for the **underestimation** in the value **of** the **section modulus** for three ply plywood with face grain perpendicular to span, calculated using parallel ply theory. The $\mathbf{g_{19}}$ factor applied to shear strength accounts for the **reduced shear strength** capacity of the plywood **due to** the **rolling shear** tendency of the plywood cross-bands.

Property	Direction of Face Plies	Assembly Factor g_{19}
Bending strength	3 ply perpendicular to span 5 ply or more perpendicular to span Parallel to span	1.20 1.0 1.0
Shear strength Bearing strength Bending deflection Shear deformation	any orientation any orientation parallel or perpendicular to span parallel or perpendicular to span	0.4 1.0 1.0 1.0

TABLE 5.11: Assembly Factors g_{19} for Plywood Loaded Normal to the Plane of the Plywood Panel

For Plywood Loaded in the Plane of the Plywood Panel:

The **g_{19} assembly factor** given in TABLE 5.12 is used to modify properties of plywood loaded in-plane, when the load direction is other than parallel or perpendicular to the face grain direction of the plywood.

In-Plane Compression/Tension Loads:

For **plywood loaded parallel or perpendicular** to the plywood face grain direction, the **effective cross-sectional area** in tension/compression is the **sum of the** thicknesses of the **plies** with **grain direction parallel to** the **force**. These plies being loaded in their strong direction, are effective at full tensile or compressive capacity. That is g_{19} = 1.0. However when the **load direction** is **inclined** at an angle **to** the plywood **face grain direction**, **all veneer layers carry some component of force** and the **effective cross-sectional area is** the **full thickness** of the plywood. Under this type of loading, components of the **load** are **carried** both parallel to the grain **in** the **stronger** direction **and** in the **weaker direction across** the **grain**. The **lower strength capacity** of the plywood veneers **across** the **grain results in** a **significant reduction in strength capacity**. Hankinson's formula is used to calculate the g_{19} = factor for the reduction in capacity. TABLE 5.12 gives values for g_{19} for compressive / bending and tensile capacity for load inclined at 45° to the face grain direction.

In-Plane Shear Loads:

Shear stresses in the plane of the plywood are **carried by all veneer layers**. To cause a shear failure, wood fibres must fail in shear both across the grain in one veneer layer and parallel with the grain in the adjacent veneer layer. This results in plywood having superior (approximately double) in-plane shear capacity compared to sawn timber products. As all veneer layers are carrying shear stresses, the effective cross-sectional shear area is based on the full plywood thickness and g_{19} = 1.0. **When** the in-plane **shear load** is **inclined** at an angle to the plywood face grain direction, **all veneer layers carry** a **component of shear force normal to** the **strong axis of** the **fibres**. Hence **shear strength** capacity is further **increased** For shear load applied at 45° to the face grain direction TABLE 5.13 gives g_{19} = 1.5.

Property	Direction of Grain of Face Plies Relative to Load Direction	Assembly Factor g_{19}
Compression and Bending Strength	parallel or perpendicular (ll plies only) ±45°	1.0 0.34
Tension Strength	parallel or perpendicular ±45°	1.0 0.17
Shear Strength	parallel or perpendicular ±45°	1.0 1.5
Shear Deformation Bending Deflection	parallel or perpendicular parallel or perpendicular (ll plies only)	1.00 1.00
Deformation in compression or tension	parallel or perpendicular (ll plies only) ±45°	1.0 1.5

TABLE 5.12: Assembly Factors g_{19} for Plywood Loaded in the Plane of the Plywood Panel

j_2 – Duration of Load Factor for Creep Deformation (bending, compression and shear members)

AS1720.1-1997 Clause 5.3.2

The j_2 load factor given in TABLE 5.13 **allows for the time dependent increase in deformation** of timber components **under constant bending**, **compression** and **shear loads**. The magnitude of the **creep deformation** in timber products **increases with longer term loads** and **higher moisture content.** Typically plywood moisture contents are less than 15% when used in dry environments.

Initial Moisture Content %	Load Duration						
	≤1 day	1 week	1 mth	3 mths	6 mths	9 mths	≥1 year
≤15	1	1.2	1.7	1.9	2.0	2.0	2.0
20	1	1.4	2.0	2.4	2.4	2.5	2.5
≥25	1	1.5	2.3	2.8	2.9	2.9	3.0

TABLE 5.13: Duration of Load Factor j_2 for Creep Deformation for Bending, Compression and Shear Members

j_3 – Duration of Load Factor for Creep Deformation (tension members)

AS1720.1-1997 Clause 5.3.2

The j_3 load factor given in TABLE 5.14, **allows for** the time dependent **increase in deformation** in timber members subjected to **tension type loads**.

Initial Moisture Content %	Load Duration	
	≤1 day	≥1 year
≤15	1	1.0
20	1	1.25
≥25	1	1.5

Use the logarithm of time for interpolation

TABLE 5.14: Duration of Load Factor j_3 for Creep Deformation for Tension Members

j_6 – Plywood in Service Moisture Content Factor for Stiffness

AS1720.1-1997 Clause 5.3.3

The j_6 factor given in TABLE 5.15 accounts for the **reduction in stiffness** of structural plywood when the **average moisture content exceeds 15% over a 12 month period**. No modification is required when the average annual moisture content is less than or equal to 15 percent.

Type of Stiffness	Factor j_6	
	Moisture Content* 15% or less	Moisture Content* 25% or more
Modulus of Elasticity	1.0	0.8
Modulus of Rigidity	1.0	0.6

For moisture contents between 15 and 25%, linear interpolation should be used to obtain j_6

TABLE 5.15: Plywood in Service Moisture Content Factor j_6 for Stiffness

Chapter 5 Appendix

Method of Calculation of Section Properties

General

The method of calculation of section properties in AS/NZS 2269, or an **equivalent alternative**, shall be used to establish the **second moment of area** (moment of inertia) and **section modulus** of structural plywood panels.

For the computation of **bending strength**, the second moment of area (**I**) shall be based only **on plies parallel** to the **direction** of **span**.

For the computation of **bending stiffness**, the second moment of area (**I**) shall be computed based on **parallel plies plus 0.03** times **plies perpendicular to the span.**

This method satisfies the requirements of AS/NZS 2269.

Definitions for use in calculation of section properties

Definitions for use in calculation of section properties are as follows:

(a) The thickness of individual veneers (d) in the plywood assembly shall be taken as the actual value given to the thickness of individual plies through the assembly in Table J5 (AS 1720.1-1997) for standard plywood constructions. In non-standard constructions are the value of (d) shall be taken as the thickness of the green veneer less 6 percent to allow for compression and sanding losses.

(b) The overall thickness of the panel (t) is the summation of the actual individual veneer thicknesses as defined in Item (a).

(c) $\bar{y}$ is the distance between the neutral axis of the panel (NA) and the neutral axis of each individual veneer as computed based upon Items (a) and (b).

Calculation Method

<u>Face Grain Parallel to the Span</u>

An illustration and section of face grain parallel to the span is shown in **FIGURE A5.1** (AS 1720.1-1997).

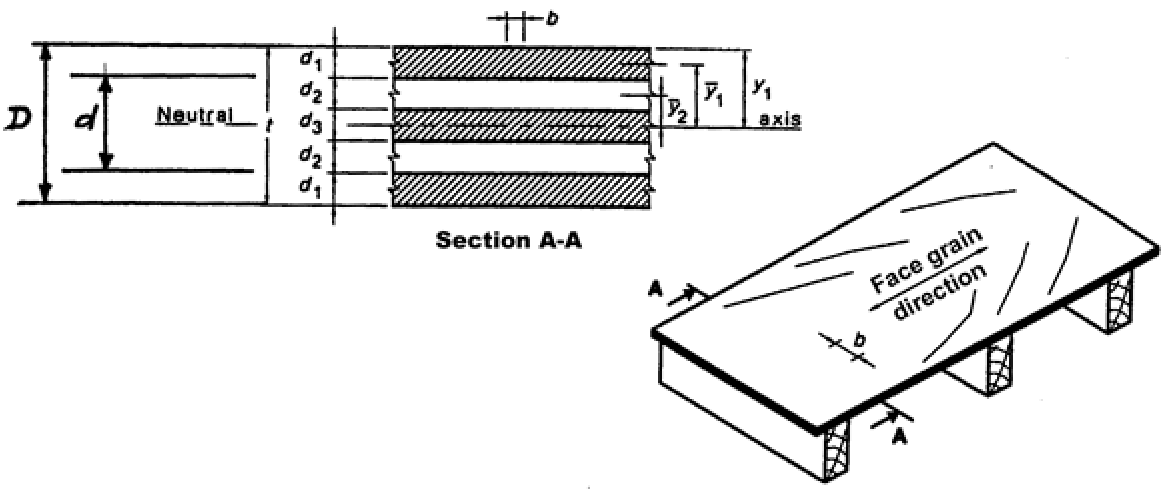


FIGURE A5.1 : Section properties – face grain parallel to the span

Using the theory of parallel axes and parallel ply theory, the calculation is as follows:

I neutral axis (NA) – stiffness, parallel to face grain per width b

$$\mathbf{I(NA)} \quad = 2\left[\frac{1}{12}bd_1^3 + A_1(\bar{y}_1)^2\right] + 2 \times 0.03\left[\frac{1}{12}bd_2^3 + A_2(\bar{y}_2)^2\right] + \frac{1}{12}bd_3^3 \qquad \textbf{A1.5.1}$$

where

A_1 = d_1b
A_2 = d_2b
0.03 = factor for plies running at right angles to span for I used in stiffness computations only.

I (NA)-strength, parallel to face grain per width b

$$= 2\left[\frac{1}{12}bd_1^3 + A_1(\bar{y}_1)^2\right] + \frac{1}{12}bd_3^3 \qquad \textbf{A1.5.2}$$

Neglecting cross-directional veneers as required by AS/NZS 2269 –

$$\text{Z(NA) parallel to face grain} = \frac{\boldsymbol{I(NA)strength\ parallel}}{\boldsymbol{y_1}}$$

where y_1 is the distance from neutral axis (NA) which is the centre-line of balanced plywood to the outside of the farthest veneer which is parallel to the span (see **FIGURE A5.1**).

<u>Face Grain Perpendicular to the Span</u>

An illustration and section of face grain perpendicular to the span is shown in **FIGURE A5.2** (AS 1720.1-1997).

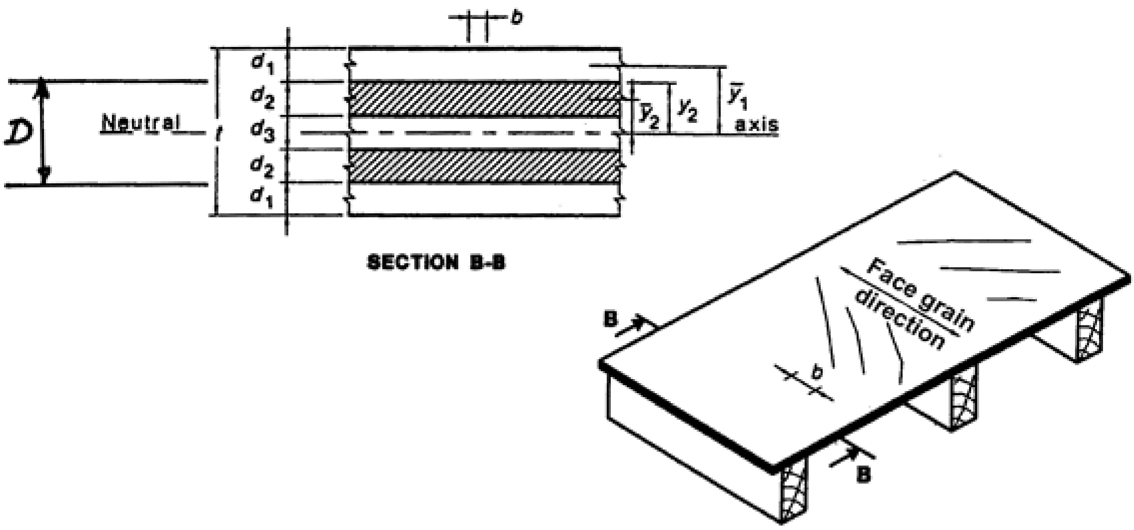


FIGURE A5.2: Face grain perpendicular to span

The calculation is as follows:

I(NA) – stiffness, perpendicular to face grain per width b

$$\mathbf{I(NA)} \quad = 2 \times 0.03\left[\frac{1}{12}bd_1^3 + A_1(\bar{y}_1)^2\right] + 2\left[\frac{1}{12}bd_2^3 + A_2(\bar{y}_2)^2\right] + 0.03\frac{1}{12}bd_3^3 \qquad \textbf{A1.5.3}$$

Again, the 0.03 factor is used for those veneers at right angles to span.

I(NA) – strength, perpendicular to face grain per width b

$$= 2\left[\frac{1}{12}bd_2^3 + A_2(\bar{y}_2)^2\right]$$

Z(NA) perpendicular to face grain $= \dfrac{I(NA)\,strength\ perpendicular}{y_2}$

where y_2 is the distance from neutral axis to the outside of the farthest veneer parallel to the span (see **FIGURE A5.2**

An Equivalent Alternative

The previously presented method of determining (**I**) is necessary if, and only if the:

- **lay-up** results in an **unbalanced section**, i.e. there are **different thicknesses either side** of the **geometrical centre** of the cross-section;

- **species either side** of the geometrical centre of the **cross-section** are **different requiring** the application of the **transformed section concept.**

For a **balance cross-section** as shown in **FIGURE A5.1** (I) can be evaluated fairly easily by **applying** the **generalised relationship**:

$$I_{NA} = \sum \frac{bD^3}{12} - \frac{bd^3}{12} \qquad \textbf{A1.5.4}$$

where:

b = **width** of section (mm);
D = **depth** of **major thickness** being considered (mm);
d = **depth** of section to be **removed** (mm).

Applying Equation 5.4 to the cross-section shown in **FIGURE A5.1** , for **face grain parallel** to the **span:**

$$I_{NA} = \frac{bD^3}{12} - \frac{bd^3}{12} + \frac{bd_3^3}{12}$$

Referring to **FIGURE A5.2** for **face grain perpendicular** to the **span:**

$$I_{NA} = \frac{bD^3}{12} - \frac{bd_3^3}{12}$$

CHAPTER 6

6 STRUCTURAL LAMINATED VENEER LUMBER (LVL) – Design Principles and Procedures

6.1 Design Principles

The design strength capacity and stiffness of structural Laminated Veneer Lumber is determined from the application of standard principles of engineering mechanics. Structural **LVL characteristic strength and stiffness properties are derived from testing and evaluation methods specified in AS/NZS 4357**. Strength and stiffness properties are based on testing at the point of manufacture to establish an estimate of the 5th percentile strength and average stiffness of the population from which the reference sample is taken. **Characteristic strength and stiffness properties** are **published by** the **manufacturer** for their particular product. Design capacities are then determined in the conventional manner by multiplying the published characteristic strength property by a section property and capacity and in-service factors as determined from AS1720.1-1997. Typically, structural LVL is used as a beam, tension or column element and therefore grain direction of all veneers is usually orientated in the longitudinal direction to maximise strength and stiffness in the spanned direction. **Section properties** for standard LVL containing **no cross-banded veneer**, is **calculated** using **actual cross-section dimensions.** However, **where cross-bands** have been **included**, for example to increase resistance to nail splitting or to improve dimensional stability, **parallel ply theory** as applied to plywood (refer CHAPTER 5) will apply to the derivation of section properties. For **LVL** used **on edge**, the contribution of the **cross-bands** is **disregarded** when calculating section properties. For **LVL** containing cross-bands used **on flat, parallel ply theory is applied** in the same manner as for plywood.

6.2 Characteristic strengths and stiffness

Current practice of manufacturers of structural **LVL** is to **publish** actual product **characteristic strength and stiffness** values **rather than** allocate properties **via** the **F-grade** system. Properties published by a manufacturer are unique to that manufacturer's product, with the manufacturer's product often identified by a brand name.

6.3 Section Properties

Structural LVL is usually manufactured with the grain direction of all veneers orientated in the longitudinal direction. **Where all veneers are orientated in the longitudinal direction**, section properties are calculated using actual cross-section dimensions. Refer FIGURE 6.1.

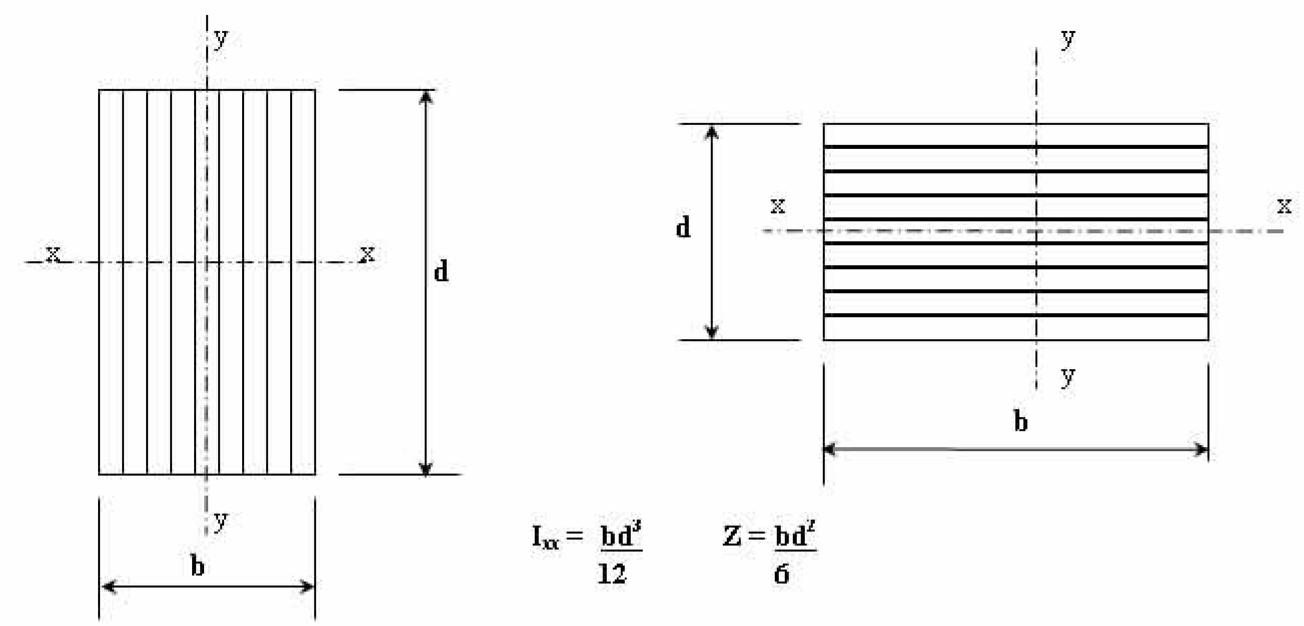


FIGURE 6.1: Section Properties for LVL with all veneers orientated in the longitudinal direction

When LVL contains cross-bands, the **section properties** are calculated based on the **parallel ply theory** used in plywood design.

Section properties for **cross-banded LVL** are **calculated as follows**:

(a) for **on edge** bending, tension, and compressive capacities and edgewise flexural rigidity, **veneers** with **grain direction at right angles to** the **direction of stress** are **ignored** in the **calculation of area**, **first moment** of area and **second moment of area**. A typical example of cross-banded LVL and section properties is shown in FIGURE 6.2.

(b) for **on flat** bending and shear applications, **section properties** are **calculated based** on **parallel ply theory** used in calculating plywood section properties.(Refer Appendix 0 of Chapter 5). An example calculation of cross-banded LVL section properties for on flat applications is shown in FIGURE 6.3.

(c) the **full cross-sectional area** is **effective** when **resisting in-plane** shear.

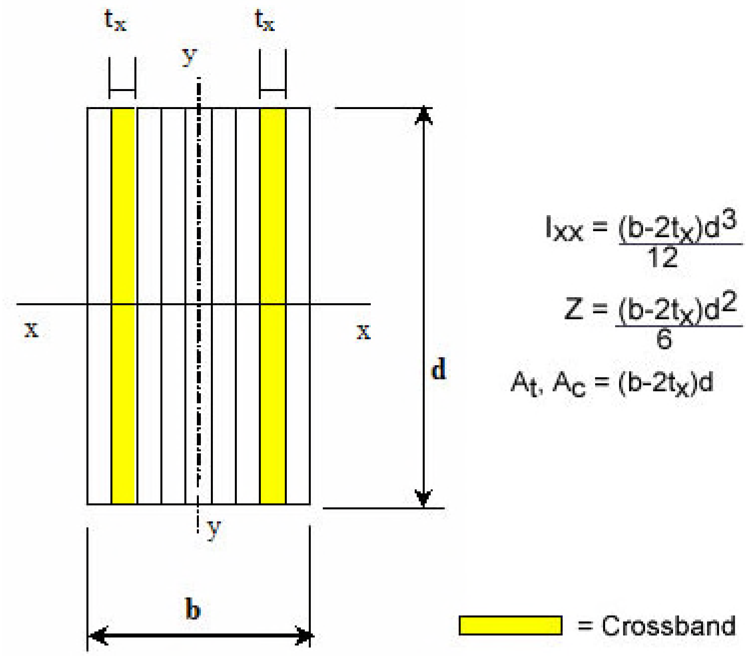
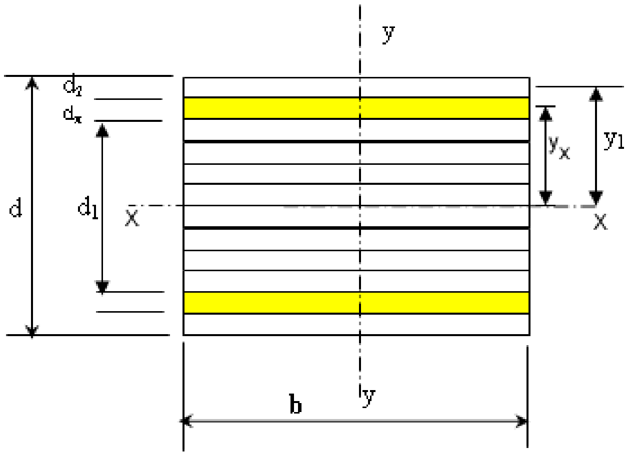


FIGURE 6.2 : Cross-banded LVL section properties for edgewise bending, tension, compression and flexural rigidity



I (neutral axis(NA)-stiffness
= $\mathbf{(bd_1^3/12)} + 2[(bd_2^3/12) + (bd_2\,\mathbf{x}\,\mathbf{(y_1)}^2] +$
$0.03 \times 2\,[(bd_x^3/12 + (2bd_x\,\mathbf{x}\,\mathbf{(y_x)}^2)$

I (neutral axis(NA)-strength
= $\mathbf{(bd_1^3/12)} + 2[\,(bd_2^3/12) + (bd_2\,\mathbf{x}\,\mathbf{(y_1)}^2]$

Z (NA)
= I (NA) strength **/** 0.5d

FIGURE 6.3: Example of cross-banded LVL section properties for on flat bending, bending deflection and shear

6.4 LVL – Design Methodology

Limit States Design to AS1720.1-1997

The **design capacity** of structural LVL, designed **in accordance with the limit states design format** of AS1720.1-1997, is **achieved** by **modifying** the **characteristic strengths by** a **geometric section property**, a material capacity factor **Φ** and **in-service factors** (k and j factors). Structural reliability is achieved through the use of these modified characteristic strength capacities and factored loads as detailed in AS/NZS 1170.0 : 2002.

Strength Limit State Capacity

The **strength limit state** condition is **satisfied** when the **design capacity** of the structural LVL **exceeds** the **design load effects from** the **factored loads**. That is:

$$\Phi R \geq S^*$$

where ΦR = design capacity of the LVL member
S* = design action effect, eg. bending moment, M*, shear force, V*, etc.
and **ΦR** = $\mathbf{k_{mod}\,[f_o'.X]}$

where Φ = **capacity factor**
k_{mod} = product of relevant **modification factors** (eg.k_1, k_6, k_7, k_{12}, k_{19}, g_{19}).
f_o' = **appropriate characteristic strength**
X = geometric **section property**.

Serviceability Limit State Capacity

The **serviceability limit state** is achieved when **in-service displacements** are **kept within acceptable limits**. Calculated bending deflections and shear deformations must be modified by in service modification factors (j_2, j_3 and j_6 as appropriate). Guidance on serviceability limit states is given in Appendix C of AS/NZS 1170.0 : 2002.

6.5 Beam Design

Figure 6.4 defines the **minor (y-y)** and **major (x-x) axes** of bending referred to in establishing strength limit states for beams.

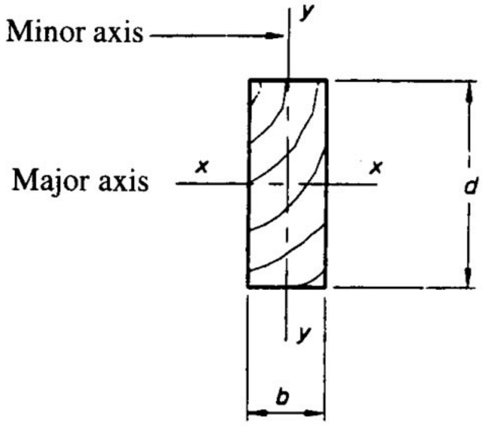


Figure 6.4: Shows major and minor axes of bending

Strength Limit State:

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied when:	AS1720.1-1997 Reference
Bending	M*	$\Phi M = \Phi k_1 k_4 k_6 k_9 k_{11} k_{12}[f'_b Z]$	ΦM > M*	clause 3.2.1.1
For beams that can bend about both the major and minor axes simultaneously:			$\frac{M_x^*}{(\phi M_x)} + \frac{M_y^*}{(\phi M_y)} \le 1.0$	clause 3.2.1.2
Shear	V*	$\Phi V = k_1 k_4 k_6 k_{11}\ [f'_s A_s]$	ΦV > V*	clause 3.2.5
Bearing perpendicular to grain	N^*_p	$\Phi N_p = \Phi\ k_1 k_4 k_6 k_7 [f'_p A_p]$	$\Phi N_p > N^*_p$	clause 3.2.6.1
parallel to grain	N^*_l	$\Phi N_l = \Phi\ k_1 k_4 k_6\ [f'_l A_l]$	$\Phi N_l > N^*_l$	clause 3.2.6.2

where:

M^*, V^*, N^*_p, N_l	= **Design action effect** in bending, shear and bearing respectively
$\Phi M, \Phi V, \Phi N_p, \Phi N_l$	= **Design capacity** in bending, shear and bearing respectively
$M^*_x,\ M^*_y$	= **Design action effect** in bending about the major principal x-axis and minor principal y-axis.
$\Phi M_x, \Phi M_y$	= **Design capacity** in bending about the major principal x-axis and minor principal y-axis.
Φ	= **Capacity factor** for LVL
k_1	= **Duration of load** strength modification **factor**
k_4	= **Moisture condition** modification **factor**
k_6	= **Temperature modification factor**
k_7	= **Length of bearing** modification **factor**
k_9	= **Strength sharing** modification **factor**
k_{11}	= **Size** modification **factor**
k_{12}	= **Stability** modification **factor**
f'_b, f'_s, f'_p	= **Characteristic strengths** in bending, shear and bearing respectively
Z	= **LVL** beam **section modulus** $= I_p/y_p$
A_s	= **shear plane area** = **2/3(bd)** for a beam loaded about its major axis in bending
A_p, A_l	= **bearing area** under the design load perpendicular and parallel to the grain as shown in FIGURE 6.5.

Serviceability Limit State:

Calculated deflection x j_2 x j_6 ≤ deflection limit clause 5.4.5

where: j_2 = Duration of load stiffness modification factor
j_6 = Moisture condition stiffness modification factor

FIGURE 6.5 defines the **design parameters** referred to when satisfying strength limit states in column design.

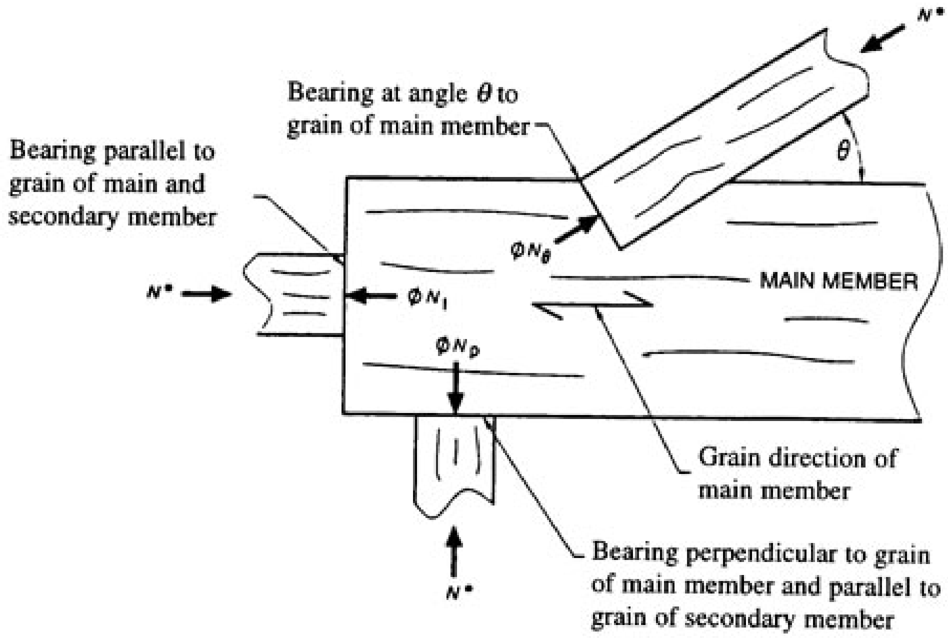


FIGURE 6.5: Notation for bearing

6.6 Column Design

Strength Limit State:

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied when:	AS 1720.1-1997 Reference
Compression	N^*_c	$\Phi N_c = \Phi k_1 k_4 k_6 k_9 k_{11} k_{12} [f'_c A_c]$	$\Phi N_c > N^*_c$	**clause 3.3**
For columns that can buckle about both axes:			$\Phi N_{cx} > N^*_c$ and $\Phi N_{cy} > N^*_c$	**clause 3.3.1.2**

where:

N^*_c = **Design action effect** in compression
ΦN_c = **Design capacity** in compression parallel to the grain
ΦN_{cx}, ΦN_{cy} = **Design capacity** in compression parallel to the grain for buckling about the major x-axis and minor y-axis respectively.
Φ = **Capacity factor** for LVL
k_1 = **Duration of load** strength modification **factor**
k_4 = **Moisture content** modification **factor**
k_6 = **Temperature** modification **factor**
k_{11} = **Size** modification **factor**
k_{12} = **Stability** modification **factor**
f'_c = **Characteristic strengths** in compression parallel to grain
A_c = **Cross-sectional area** of column

6.7 Tension Member Design

Tension member design is defined by the **direction of load application with respect to grain direction** as shown in FIGURE 6.6(a) for perpendicular to the grain.

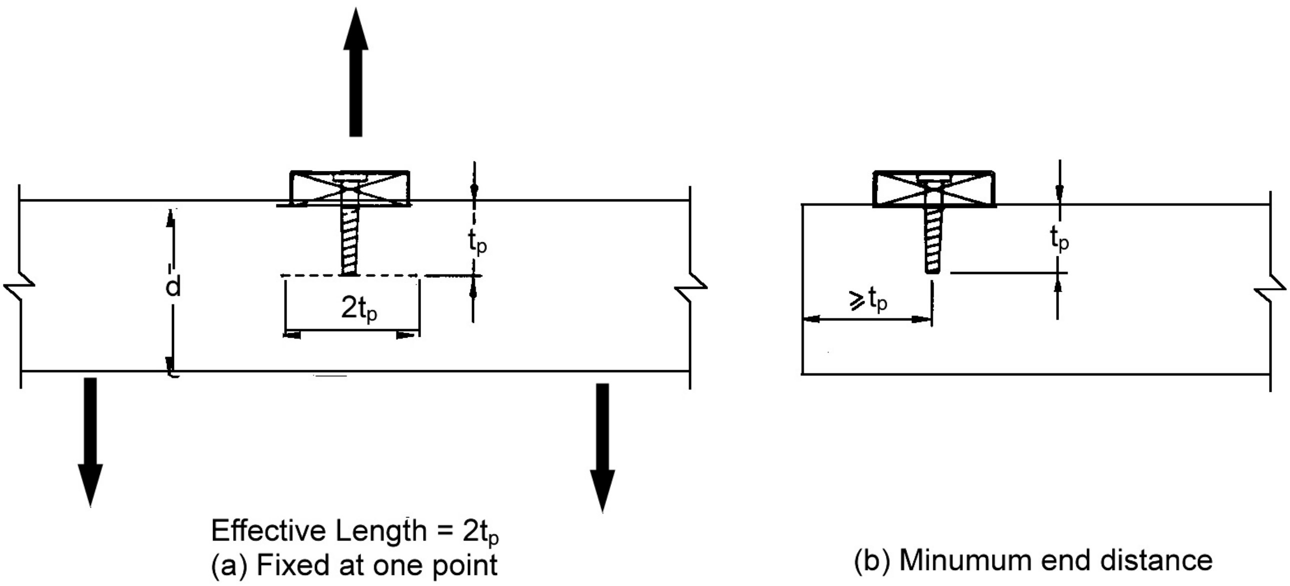


FIGURE 6.6: Effective length stressed in tension perpendicular to grain

Strength Limit State:

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied when:	AS 1720.1-1997 Reference
Tension parallel to grain	N^*_t	$\Phi N_t = \Phi\ k_1 k_4 k_6 k_{11}\ [f'_t A_t]$	$\Phi N_t > N^*_t$	**clause 3.4.1**
Tension perpendicular to grain	N^*_{tp}	$\Phi N_{tp} = \Phi k_1 k_{11} [f'_{tp} A_{tp}]$	$\Phi N_{tp} > N^*_{tp}$	**clause 3.5**

where:

N^*_t, N^*_{tp} = **Design action effect** in tension parallel and perpendicular to grain respectively
$\Phi N_t, \Phi N_{tp}$ = **Design capacity** in tension parallel and perpendicular to grain respectively
Φ = **Capacity factor** for LVL
k_1 = **Duration of load** strength modification **factor**
k_4 = **Moisture content** modification f**actor**
k_6 = **Temperature** modification **factor**
k_{11} = **Size** modification **factor**
f'_t, f'_{tp} = **Characteristic strengths** in tension parallel and perpendicular to grain respectively
A_t = **Net cross-sectional area** of tension member
A_{tp} = **Member width** (thickness) by effective length stressed in tension

6.8 Combined Bending and Axial Actions

There are many instances where structural elements and/or components are subjected to single force actions, e.g. uniaxial tension or compression, bending or torsion. Likewise there are **many** other **instances** when the **elements** and/or **components** are **subjected to combined** actions.

An **example** of **combined bending and axial actions** is the **stud** in an **external shearwall** of a building subjected to **wind loading.** The **wall wind pressure** causes **bending** of the stud and the **roof wind pressure** (provided the roof pitch is suitable) **causes** the stud to be in **compression.**

Strength Limit State:

Strength Limit State	Design Action Effect	Strength Limit State Satisfied when:	AS1720.1-1997 Reference
Combined Bending and Compression about the x axis	M^*_x N^*_c	$(M^*_x / \Phi M_x)2 + (N^*_c / \Phi N_{cy}) \leq 1$ and $(M^*_x / \Phi M_x) + (N^*_c / \Phi N_{cx}) \leq 1$	**clause 3.6.1**
Combined Bending and Tension actions	M^* M^*_x N^*_t	$(k_{12}M^* / \Phi M) + (N^*_t / \Phi N_t) \leq 1$ and $(M^*_x / \Phi M_x) - (ZN^*_t / A\Phi M_x) \leq 1$	**clause 3.6.2**

where:

- M^*_x = **Design action effect** produced by the strength limit states design loads acting in bending about a beam's major principal x-axis.
- M^* = **Design action effect** produced by the strength limit states design loads acting in bending about a beam's appropriate axis.
- N^*_c , N^*_t = **Design action effect** produced by the strength limit states design loads acting in compression and tension respectively.
- ΦM_x = **Design capacity in bending** about a beam's major principal x-axis.
- ΦM = **Design capacity in bending** about a beam's appropriate axis.
- ΦN_{cy} , ΦN_{cx} = **Design capacity in compression** for buckling about a beam's major y-axis and x-axis respectively.
- ΦN_t = **Design capacity** of a member in **tension**.
- k_{12} = **Stability factor** used to calculate bending strength.
- Z = **Section modulus** about the appropriate axis
- A = **Cross-sectional area**.

6.9 Factors

Capacity Factor

The **Φ factor** given in TABLE 6.1 is a **material capacity factor** and **allows for variability in material strength** and the **consequence of failure**. The material capacity factor, **Φ**, assigned via AS1720.1-1997, to structural materials, is **based** on **current knowledge of product structural performance**, intended structural application and material reliability. The capacity factors applied to structural LVL manufactured to AS/NZS 4357 reflects the high degree of manufacturing process control, the low material variability and high product reliability.

Application of Structural Member	LVL Capacity Factor, Ø
All structural elements in houses. All secondary structural Elements in structures other than houses	0.9
Primary structural elements in structures other than houses	0.85
Primary structural elements in structures intended to fulfil an essential services or post disaster function	0.80

TABLE 6.1: Capacity Factor

In Service Modification Factors:

The following in service modification factors are applicable to structural LVL -

Modification Factor	AS 1720.1-1997 Reference
Strength modification factors	
k_1 = Factor for **load duration**	**clause 8.4.2 & 2.4**
k_4 = Factor for in-service **moisture content**	**clause 8.4.3**
k_6 = Factor for **temperature effects**	**clause 8.4.4 & 2.4.3**
k_7 = Factor for **bearing length**	**clause 8.4.5 & 2.4.4**
k_9 = Factor for **load sharing in grid systems**	**clause 8.4.6 & 2.4.5**
k_{11} = Factor for **member size**	**clause 8.4.7**
k_{12} = Factor for **instability**	**clause 8.4.8**
Stiffness modification factors:	
j_2 = Factor for **duration of load** for bending, compression and shear	**clause 8.4.2 & 2.4.1.2**
j_3 = Factor for **duration of load** for	**clause 8.4.2 & 2.4.1.2**
k_1 = **Duration of load strength modification factor**	**clause 8.4.3**
j_6 = Factor for in **service moisture content**	**clause 2.4.1**

k_1 – Duration of Load Strength Modification Factor

AS1720.1-1997
clause 8.4.2
clause 2.4

The **k_1** duration of load factor given in TABLE 6.2 **allows for the time dependant nature** of the **strength** of **timber**. A timber member subjected to a short term load without failure may fail over time if the load is sustained. The k_1 factor allows for the reduction in the strength capacity of the LVL member when subjected to long term loads. For **load combinations** of **differing duration**, the appropriate **k_1** factor is that **for** the **shortest duration load**.

Type of Load	k_1
Permanent loads	0.57
Live loads on floors due to vehicles or people applied at frequent but irregular intervals	0.80
Live loads applied for periods of a few days and at infrequent intervals	0.94
Impact of wind loads	1.0
Wind gusts	1.15

TABLE 6.2: Duration of load strength modification factor

k_4 – Moisture Content Factor

AS17201-1997
clause 8.4.3

The **k_4** and **j_6 moisture content factors** given in TABLE 6.3, are used to **modify LVL strength** and **LVL stiffness** capacity to **allow** for the **reduction in strength** that will result **if average moisture content** of the **LVL in service** remains **higher than 15% for** a period of **12 months**. Where the average moisture content of LVL, over a 12 months period is less than or equal to 15%, k_4 = 1.0 and j_6 = 1.0. **When dispatched** by the manufacturer, **structural LVL moisture content** will **not exceed 15%.** LVL subsequently exposed to moisture for a sufficient period of time may exceed 15% moisture content. However, the LVL will dry to below 15%, in time, if the source of moisture is not constant.

Property	Equilibrium Moisture Content (EMC)		
	15% or less	15% to 25%	25% or more
Bending and Compression	$k_4 = 1.0$	$k_4 = 1.45 - 0.03$ EMC	$k_4 = 0.7$
Tension and Shear	$k_4 = 1.0$	$k_4 = 1.30 - 0.02$ EMC	$k_4 = 0.8$
Modulus of Elasticity	$j_6 = 1.0$	$j_6 = 1.30 - 0.02$ EMC	$j_6 = 0.8$

TABLE 6.3: Moisture content factor (k_4 for strength and j_6 for stiffness)

k_6 – Factor for Temperature

AS1720.1-1997 clause 2.4.3

k_6 = 1.0 except where used in structures erected in **coastal regions** of **Queensland north** of latitude **25°S** and all **other regions** of Australia **north** of latitude **16°S, k_6 = 0.9**. Refer FIGURE 6.7.

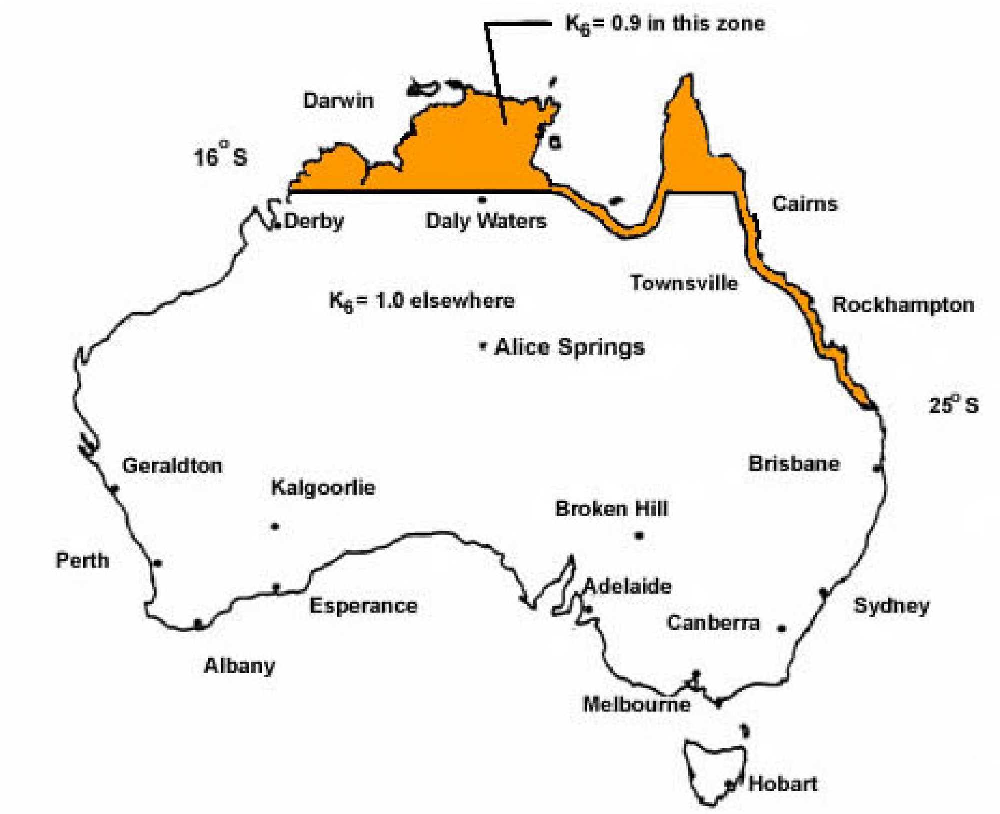


FIGURE 6.7: Shows regions k_6 applies

k_7 – Factor for Length and Position of Bearing

AS1720.1-1997 clause 2.4.4

The **k_7** bearing factor **modifies bearing strength perpendicular to grain**. The modification factor allows for bearing configurations which differ from the standard test configuration from which the bearing perpendicular to grain strength data was derived. **k_7 = 1.0 unless the bearing length is less than 150 mm** long **and is 75 mm or more from the end** of the member. In this case k_7 may be greater than 1.0. Refer TABLE 6.4. The bearing length is measured parallel to the face grain of the member.

Length of Bearing of Member (mm)	12	25	50	75	125	150 or more
Value of k_7	1.85	1.60	1.30	1.15	1.05	1.00

TABLE 6.4: Factor for length and position on bearing

k_9 – Strength sharing modification factor for grid systems

AS1720.1-1997 clause 8.4.6 clause 2.4.5

k_9 = 1.0 for LVL used in parallel systems **clause 8.4.7**

For **bending members k_9** applies in **two different scenarios,** i.e. for:

- **combined parallel systems,** does **not apply** to **LVL** because it is treated as **solid sawn timber**. However, it has all of the attributes, since it is made from **parallel elements rigidly connected forcing** them to deflect the same amount.

  AS1720.1-1997 (Clause 2.4.5) defines a parameter **n_{com}** which is the **effective number** of **parallel elements** shown in FIGURE 6.8 which **combine to** form a **single member** and **for** which **n_{com} = 4.0;**

- **discrete systems**, which applies to, e.g. **LVL joists** sheathed with plywood causing **load sharing** between joists in the system. **Effectiveness** of the **load sharing** is dependent upon the **joist spacing** and the **stiffness** of the **plywood interconnecting** the **joists** as shown in FIGURE 6.8. The **number** of **members** involved in the **load sharing** is defined in Clause 2.4.5 as **n_{mem}.** In a **normal** plywood sheathed **floor system n_{mem} = 3** would be usual for a floor of **5** or **more joists.**

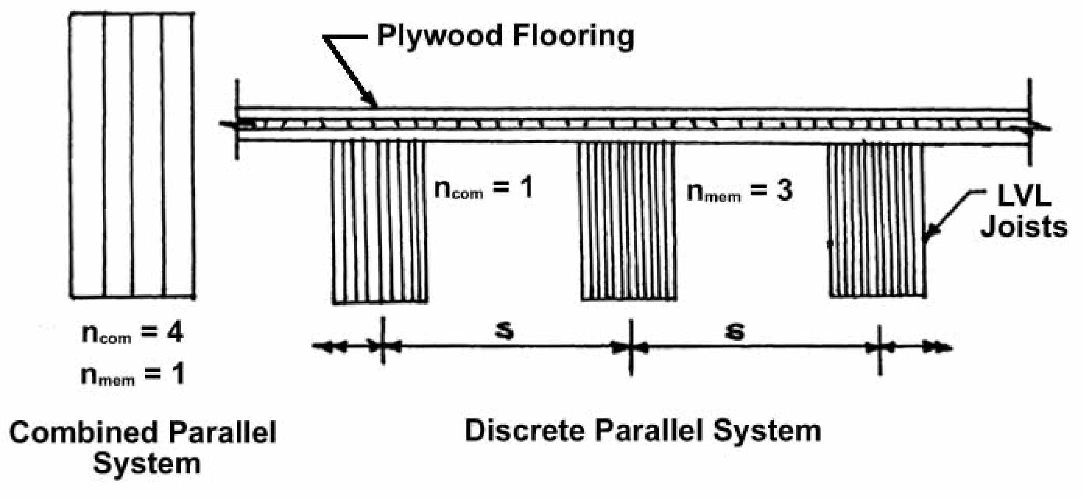


FIGURE 6.8: Parallel and grid systems

k_{11} – Size Factor

AS1720.1-1997 clause 8.4.7

The tensile behaviour of timber modelled on brittle fracture mechanics indicates a higher probability of finding a flaw (eg naturally occurring characteristic such as a knot, split, gum vein, etc.) leading to a brittle fracture in higher volume members. The **k_{11}** size factor is **used to account fo**r this **increased probability** of finding a flaw in **larger volume, tensile members**.

LVL members are modified for size effects as follows:

(a) For **bending k_{11}** = **1.0** for member **depths** up **to 300mm**. For member **depth greater than 300mm k_{11} = $(300/d)^{0.167}$**

(b) For **tension parallel** to grain k_{11} = **1.0** for member **depths** up **to 150mm**. For member **depth greater than 150mm, k_{11}** = **$(150/d)^{0.167}$**

(c) For **shear** k_{11} = **1.0**

(d) For **compression** k_{11} = **1.0**

(e) For **tension perpendicular** to grain $k_{11} = (10^7/V)^{0.2}$ where **V** is the **volume** of timber in **mm**^{3}, stressed above 80% of the maximum value in tension perpendicular to grain.

k_{12}– Stability Factor

AS1720.1-1997 clause 8.4.8

The **stability factor accounts for** the fact that in slender members the compression capacity is determined by the **buckling capacity rather than** the **material capacity**. **k_{12} for** structural **LVL** is calculated in the same manner **as for** structural **sawn timber**. The stability factor is used to modify the characteristic strength in bending and compression and is calculated based on a material constant and a slenderness co-efficient.

Stability Factor k_{12} is calculated from the following –

(a) For: $\rho S \leq 10$, $k_{12} = 1.0$

(b) For: $10 \leq \rho S \leq 2$ $k_{12} = 1.5 - 0.5\,\rho S$

(c) For: $\rho S \geq 20$, $k_{12} = 200/(\rho S)^2$

Material constants for LVL are:

For bending members: $\rho = 14.71(E/f'_b)^{-0.480}\, r^{-0.061}$

For compressions members: $\rho = 11.39(E/f'_c)^{-0.408}\, r^{-0.074}$

Where:

ρ = ratio (temporary design action effect) / (total design action effect).

NOTE:
slenderness co-efficients, S, for lateral buckling under bending and compression are given in Appendix A6.1 at the end of this chapter.

j_2– Duration of load factor for creep deformation (bending, compression and shear members)

AS1720.1-1997 clause 8.4.2

The **j_2** load factor given in TABLE 6.5 **allows for** the **time dependent increase in deformation** of **LVL** components **under constant bending**, **compression** and **shear loads**. The magnitude of the **creep deformation** in timber products **increases with longer term loads** and **higher moisture content**. Typically LVL moisture contents are less than 15% when used in dry environments.

Initial Moisture Content %	LOAD DURATION						
	≤1 day	1 week	1 mth	3 mths	6 mths	9 mths	≥1 year
≤15	1	1.2	1.7	1.9	2.0	2.0	2.0
20	1	1.4	2.0	2.4	2.4	2.5	2.5
≥25	1	1.5	2.3	2.8	2.9	2.9	3.0

TABLE 6.5: Duration of load factor j_2 for creep deformation for bending,

j_3– Duration of load factor for creep deformation (tension members)

The $\mathbf{j_3}$ load factor given in TABLE 6.6, **allows for** the **time dependent increase in deformation** in LVL members subjected to **tension type loads**.

Initial Moisture Content %	LOAD DURATION	
	≤1 day	≥1 year
≤15	1	1.0
20	1	1.25
≥25	1	1.5

Use the logarithm of time for interpolation

TABLE 6.6: Duration of load factor j_3 for creep deformation for tension members

Chapter 6 Appendix

Slenderness Co-Efficient for Lateral Buckling Under Bending

General

For the general case, and for several useful specific cases, equations for evaluating the slenderness co-efficient are given in Appendix E of AS1720.1-1997. For the special cases of solid beams of rectangular cross-section, the simple approximations given in Clause 3.2.3.2 may be used. For notation for beam restraints see.

Beams of rectangular cross-section. For beams of rectangular cross-section, the slenderness co-efficients may be taken as follows:

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

For a beam **loaded along** its **compression edge** and **has discrete lateral restraints** at points L_{ay} apart, along the compression edge of the beam as indicated in **FIGURE A6.1** then the slenderness co-efficient, denoted by S_1, may be taken to be –

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

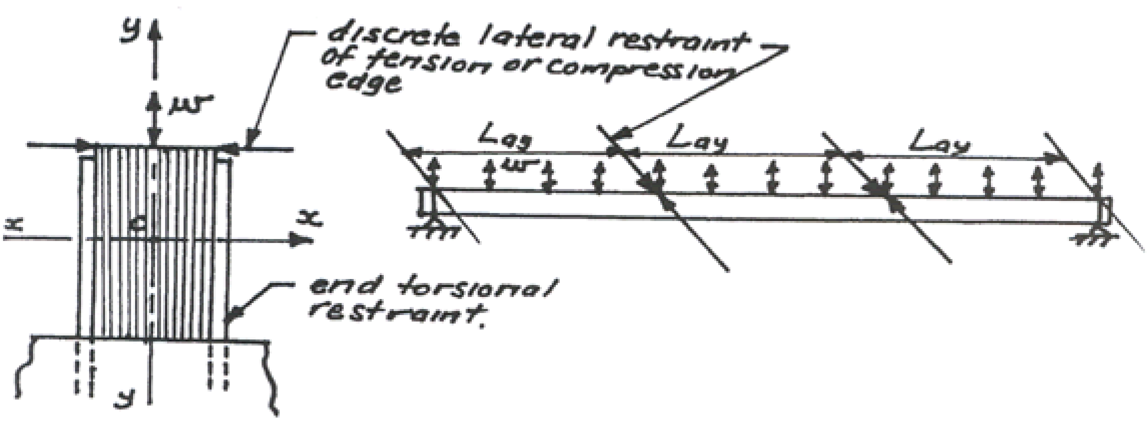


FIGURE A6.1: Discrete restraints to the compression and tension edge

For a beam **loaded along** its **tension edge** and having **discrete lateral restrains** at points L_{ay} apart along the tension edge, as indicated in **FIGURE A6.1**, then the slenderness co-efficient, denoted by S_1, may be taken to be –

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

Beams that bend about their major axis having continuous lateral restraint systems

A **continuous lateral restraint system** (see **FIGURE A6.2**) may be assumed to **exist when** –

$$\frac{L_{ay}}{d} \quad \geq 64\left(\frac{b}{\rho_b d}\right)^2$$

For a beam that is **loaded** along its **compression edge** and has a **continuous lateral restraint** system along the compression edge (see **FIGURE A6.2**), then the slenderness co-efficient, denoted by S_1, may be taken to be **equal** to **zero**.

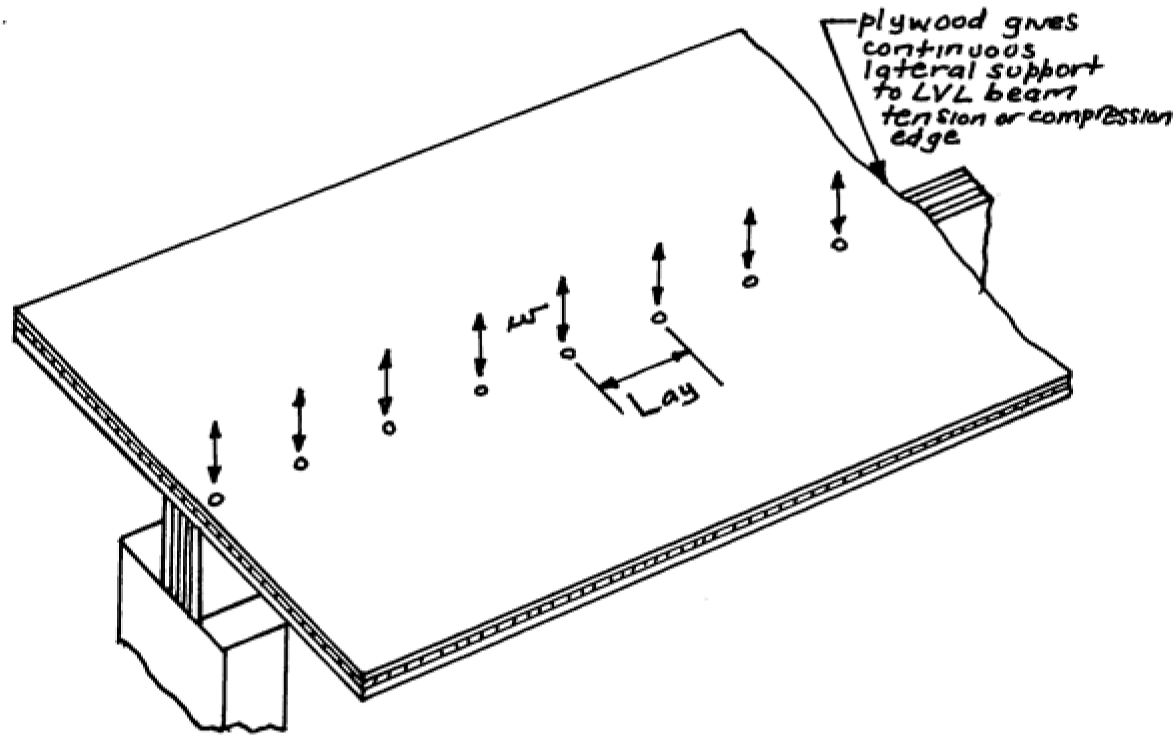


FIGURE A6.2: Continuous restraint along the compression and tension edge

For a beam **loaded** along its **tension edge** and which has a **continuous lateral restraint** system **along** the **tension edge** (see **FIGURE A6.2**), the slenderness co-efficient, denoted by S_1, may be **taken to be** –

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

For a beam **loaded** along its **tension edge**, which in addition to **having** a **continuous lateral restraint** system along its tension edge, has **equally spaced torsional restraints** at points $L_{a\Phi}$ **apart**, indicated in **FIGURE A6.3**, to **prevent rotation about** the beams **Z axis**, the slenderness co-efficient, denoted by S_1, may be **taken to be** –

$$S_1 = \frac{1.5\,d/b}{\left[\left(\frac{\pi d}{L_{a\varphi}}\right)^2 + 0.4\right]^{0.5}}$$

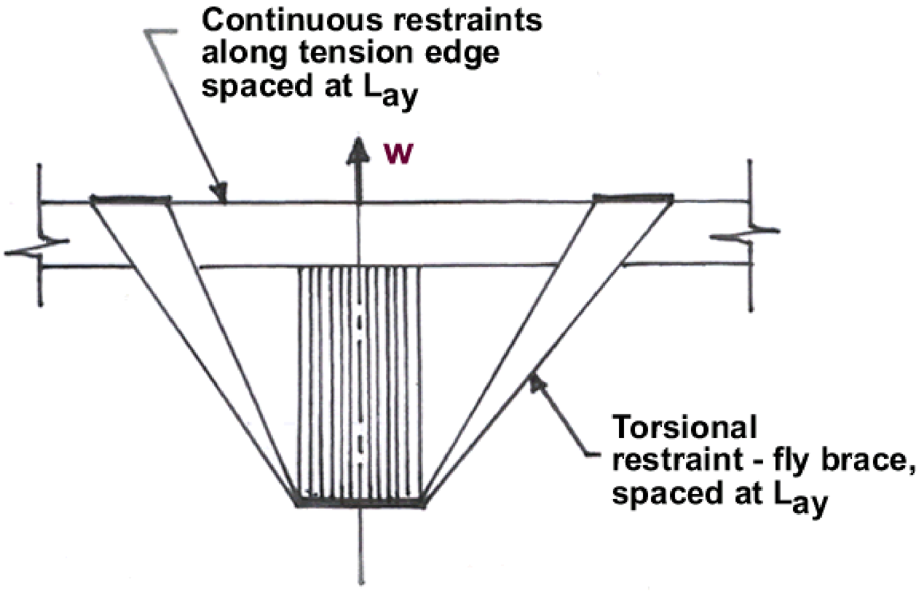


FIGURE A6.3: Continuous restraint along the tension edge combined with discrete torsional restraints

Beams that bend only about their minor axis

For **all cases**, the slenderness co-efficient, denoted by S_2, may be **taken to be** –

$$S_2 = 0.0$$

Beams that bend about both axis

The design of such beams, described in Section 6.8, is based on an **interaction of the two special cases** for **bending about single axis only**, and hence no special definition of slenderness is required for this case.

Slenderness co-efficient for lateral buckling under compression

General

For the **general case**, and for **several** useful **specific cases**, **equations** for evaluating the slenderness co-efficient are **given** in **Paragraph E4**, **Appendix E**. For the case of solid columns of rectangular cross-section as shown in **FIGURE A6.4**. The simple approximations given in Clause 3.3.2.2 may be used.

Columns of rectangular cross-section

For **columns** of **rectangular cross-section**, the slenderness co-efficients may be taken as follows:

(a) **Slenderness co-efficient for buckling about the major axis**.
For the case of **discrete restraint systems**, the slenderness co-efficient, denoted by S_3, shall be taken to be the **lesser of** the **following:**

$$S_3 = \frac{L_{ax}}{d}$$

and

$$S_3 = \frac{g_{13}L}{d}$$

where

L_{ax} = the **distance between** points of effectively **rigid restraint** between which **bending** about the **major** (**x**) **axis** would be **produced by buckling** under load. See **FIGURE A6.4**.

g_{13} = the **co-efficient** given **in Table 3.2, AS1720.1-1997**

For restrain systems that restrain movement in the direction of the y-axis, and are continuous along the length of the column, the slenderness co-efficient may be taken to be:

$S_3 \quad = 0.0$

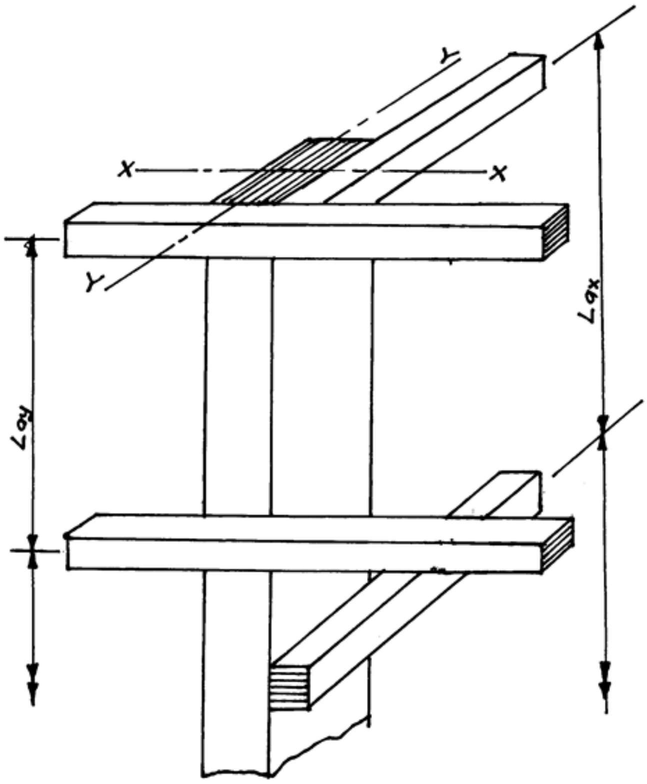


FIGURE A6.4: Notation for column restraints

where

L_{ay} = the **distance between** points of effectively **rigid restraint** between which **bending** about the **minor (y) axis** would be **produced by buckling** under load. See **FIGURE A6.4**.

g_{13} = the **co-efficient** given in **Table 3.2**, AS 1720.1 – 1997 and reproduced herein in **TABLE A6.1**.

For **restrain systems** that act **continuously along one edge only** and which **restrain movement in** the **direction of** the **x-axis**, the **slenderness co-efficient may be taken to be –**

$$S_4 = \frac{3.5\,d}{b}$$

(b) Columns that can bend about both axes.

The design of such columns, described in Clause 3.1.2 is based on an **interaction** of the **two special cases** for bending about single axes only, and hence no special definition of slenderness is required for this case.

Stability factor.

The stability factor k_{12} for **modification** of the **characteristic strength in compression** shall be given by the following:

(a) For $\rho_C S \leq 10$

$k_{12} = 1.0$

(b) For $10 \leq \rho_C S \leq 20$

$k_{12} = 1.5 - (0.05 \times \rho_C S)$

(c) For $\rho_C S \geq 20$

$$k_{12} = \frac{200}{(\rho_c S)^2}$$

where:

S = S_3 for **buckling** about the **major axis;**
= S_4 for **buckling** about the **minor axis**

Condition of End Restraint	Effective Length Factor (g_{13})
Flat ends	0.7
Restrained at both ends in position and direction	0.7
Each end held by two bolts (substantially restrained)	0.75
One end fixed in position and direction, the other restrained in position only	0.35
Studs in light framing	0.9
Restrained at both ends in position only	1.0
Restrained at one end in position and direction and at the other end partially restrained in direction but not in position	1.5
Restrained at one end in position and direction but not restrained in either position or direction at other end	2.0

NOTE: 'Flat ends' refers to **perfectly flat ends bearing on flat unyielding bases**

TABLE A6.1: Effective length factor g_{13} for columns without intermediate lateral restraint

CHAPTER 7

7 BASIC STRUCTURAL PLYWOOD & LVL BUILDING COMPONENTS

7.1 Introduction

EWPAA product certified structural **plywood** and **LVL** products are **used extensively** in **residential, commercial** and **industrial building components**. Dimensional uniformity and trueness, and reliable, consistent structural properties, make them an attractive material choice from both a design and construction perspective. Basic structural **plywood** components include **flooring** of all types (domestic, commercial, industrial, sport floors and containers), **bracing**, **combined wall cladding** and **bracing**, and **roof sheathing**. The use of **structural plywood** as residential **flooring, bracing** and non-trafficable **roofing** is detailed in AS1684 Residential Timber-Framed Construction Code which is **deemed to comply under State building ordinances** and the **Building Code of Australia**. Structural **LVL** and **plywood/LVL I-beams** are used in framing elements **as bearers**, **joists**, **lintels** and **roof framing**. Due to the extensive usage of these products within the building industry, specific technical literature has been developed for each application and is available either from the Engineered Wood Products Association of Australasia or EWPAA manufacturing members.

7.2 Structural Plywood Flooring and Floor Systems

Structural plywood **has a number of** inherent characteristics **which make it particularly suitable for use as a** platform flooring material**. It has defined and** standardised structural properties**,** good dimensional stability **compared to other timber panel products,** tongued and grooved edges **eliminating the need for nogging, a** permanent Type A phenolic bond **and** high strength and stiffness **capacity suitable for use under the higher design loads** required by **the building codes for** commercial and industrial flooring**. Floor live load requirements for a range of building occupancies are given in AS1170 SAA Structural Design Actions – Part 1: Permanent, imposed and other actions.**

7.3 Design Issues of Flooring

The excellent load re-distribution capabilities **of plywood means uniformly distributed loads are unlikely to govern the design. Structural** plywood flooring **design is** usually governed by **the** concentrated **imposed** loads**. For more** lightly loaded floors deflection **under imposed concentrated loads** governs plywood selection**. Shear strength may govern under higher concentrated loads with closer support spacings.** Concentrated loads **on structural plywood flooring are** treated **as a** line load**. The** distribution width **of the concentrated load** must **therefore** be determined**. TABLE 5.3 in CHAPTER 5 of this Manual** provides standard load distribution widths **for various thicknesses of plywood. Structural** plywood flooring **should be** spanned **with the** face veneer grain **direction** parallel to **the** span **to maximise the plywood capacity.** Support spacings **should be selected to** suit **the plywood sheet** length**, such that the** ends **of the sheet** land on a support**.**

Note closer support spacings with thinner plywood **will** usually **be a** more economical **solution than widely spaced supports with thicker plywood. Long edges of structural plywood flooring are usually manufactured with plastic tongue and groove. The** tested capacity **of the** tongue **and** groove **for EWPAA branded** plywood, under concentrated **load is** 7.5 kN**. If tongued and grooved edges are not used, or where the concentrated load exceeds 7.5 kN, support must be provided to long edges. Finally, in applications where the** plywood surface **will be** subject to abrasive loadings **such as may occur in garage floors and industrial floors subject to wheeled traffic there,** may be a need **for some** surface protection**.**

7.4 Structural Plywood Flooring – Design Methodology

The steps involved in the design of a plywood sheathed floor system are as follows:

1. Select a joist spacing to suit standard plywood flooring sheet lengths :
   a. **Standard sheet lengths** are :
      i. **2400** - suitable joist spacings include 400, 600, 800 mm
      ii. **2250** - suitable joist spacings include 375, 450, 750 mm (limited availability)
      iii. **2700** - suitable joist spacings include 450, 540, 675, 900 mm

   b. To optimise structural plywood performance ensure **plywood** is **supported** continuously **over** a minimum of **two spans**.

2. Set Deflection Limits:

      i. A typical **deflection limit** is **span/200**. Where the plywood flooring will be an underlay to a rigid covering such as tiles, tighter deflection criteria are recommended. **AS/NZS1170.1 Appendix C** recommends a **deflection limit of span/300.**

3. Determine **floor imposed load requirements** from AS/NZS 1170.1 for both concentrated and uniformly distributed loads

      i. Typically, the **load contribution of** the **plywood** flooring itself is **insignificant** when compared to the imposed loads and consequently **is ignored** in strength and stiffness calculations.

4. Determine the **capacity factor** (**Φ**) and **strength modification factors** from AS1720.1-1997 for structural plywood flooring**:**

The relevant factors are: **Refer AS1720.1-1997**
or
CHAPTER 5 **of this manual**

Bending : $(\phi k_1 k_{19} g_{19})$
Shear : $(\phi k1 k_{19} g_{19})$
Deflection: $(j_2\ j_6\ g_{19})$

		AS1720.1-1997 Reference
ϕ = 0.9	for **all structural elements** in houses	**Table 2.6**
k_1=0.94	For **concentrated loads** assuming loads are applied at infrequent intervals such as might arise due to a pallet jack or maintenance type load. Effective duration of peak load = 5 days.	
k_1 = 0.80	For **uniformly distributed loads** assuming loads are typical floor type loads (crowd or vehicle or stored material. Effective duration of peak load = 5 months.	**Clause 2.4.1.1 Table 2.7**
k_{19} = 1.0	As it is not anticipated the plywood moisture content will exceed an average of 15% in a dry interior application. In a **dry interior** application, **moisture content** would be in the **range 8 to 12 %.**	**Table 5.9**
g_{19}= 1.0	Direction of the **face veneers** is **parallel with** the **span direction**. Therefore, g_{19}= 1.0 for bending, shear strength and deflection.	**Table 5.11**
j_2 = 1.0	for **short term concentrated loads** of less than 1 days duration.	**Clauses 2.4.1.2**
j_2 = 2.0	for **longer term uniformly distributed loads**, such as stored materials.	**Table 5.13**
j_6 = 1.0	average **moisture content not** anticipated to **exceed 15%.** (Refer k_{19} above)	**Table 5.15**

5. **Determine** the **critical load action effects** and strength limit state capacity for bending and shear.

6. **Determine** the **serviceability limit state capacity** for bending deflection.

7. Select a suitable structural plywood thickness and stress grade.

7.5 Design Example – Structural Plywood Floor – Specification

Note:
This design example uses AS1170:1998. Please note there are minor changes in AS1170:2002 which will become mandatory in due time.

Design requirements and specification for a structural plywood mezzanine floor for storage is as follows:

Design criteria: **Joists** @ 400 centres;
Plywood to be two span continuous minimum, spanning with face veneer grain direction (panel length direction) parallel with plywood span direction.
Deflection limit: Span/200 = 400/200 = 2mm

1. **Loads:** **AS1170.1 App.B**
   Live
   7kN concentrated load;
   5kPa uniformly distributed load (UDL)

   Dead
   Self weight:
   For 25mm plywood ($\underset{\sim}{~}$600kg/m3 x 9.81E-3 x 0.025) = 0.15 kPa (Not usually considered but included in this design example for completeness)

2. **Load Combinations** **AS1170.1 Cl.2.2**
   Strength limit state:
   1.25G + 1.5Q

   Serviceability limit state: **AS1170.1 Cl.2.4**
   1 x Q (short term)
   G + 0.4Q (long term)

3. **Capacity Factor and Strength Modification Factors**

ϕ	= 0.9	**Table 5.6**
k_1	= 0.94 concentrated live load	**Table 5.1**
k_1	= 0.80 uniformly distributed live load	**Table 5.1**
k_{19}	= 1.0 (MC $\leq$ 15%)	**Table 5.9**
g_{19}	= 1.0 for bending strength	**Table 5.11**
g_{19}	= 0.4 for shear strength	**Table 5.11**

4. **Serviceability Modification Factors**

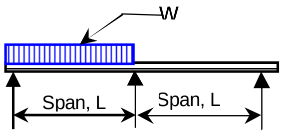
j_2	= 1.0 short term load	**Table 5.13**
j_2	= 2.0 long term load	**Table 5.13**
j_6	= 1.0 (MC $\leq$ 15%)	**Table 5.15**
g_{19}	= 1.0 bending deflection	**Table 5.11**

5. Critical Load Action Effects

Load Case 1 | **Load Case 2**

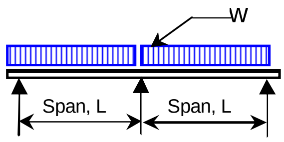
UDL

Load Case 1:



$M_{max} = 49wL^2/512$
$V_{max} = 9wL/16$
$\Delta_{max} = wL^4/(105EI)$

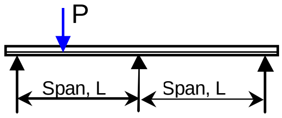
Load Case 2:



$M_{max} = -wL^2/8$
$V_{max} = 5wL/8$
$\Delta_{max} = wL^4/(185EI)$

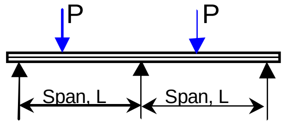
Concentrated Load

Load Case 1:



$M_{max} = 13PL/64$
$V_{max} = -19P/32$
$\Delta_{max} = 3\ PL^3/(200EI)$

Load Case 2:



$M_{max} = 6PL/32$
$V_{max} = 11P/16$
$\Delta_{max} = PL^3/(110EI)$

Note:

The shear strength limits were also considered when high concentrated loads act at, or close to a support joist. Applying the theory for beams on elastic foundations, Paulet (1945) as expanded in "Load Distribution in Wooden Floors Subjected to Concentrated Loads" by N.H. Kloot and K.B. Schuster – Division of Forest Products, CSIRO 1965, indicates that load distribution will result in concentrated loads applied close to supports being less critical for shear strength than a centrally applied concentrated load.

For example, for a floor consisting of 300 x 40mm F8 joists at 400 mm centres, spanning 2400 mm and 24 mm F11 structural plywood flooring, the ratio of joist stiffness to flooring stiffness is $\{E_j.I_j.L^3_{(spacing)} / E_f.I_f.L^3_{(span)}\} = 0.18$, *which results in a reaction on the joist under the concentrated load equal to 60% of the applied load. As 20% of the applied load is transferred to the joist/s either side of the applied load, the expected plywood shear force is 60% of the applied load which is less than the* $11/16^{th}$ *of the applied load used when a central concentrated load is applied on each span. (reference: Assumptions Used and An Example Calculation of Allowable Point Live Loads, 1990, Adkins & Lyngcoln).*

7.6 Structural Plywood Floor – Worked Example

1. Design Action Effects on Member Due to Factored Loads

	M_p	M^*_p	M^*_p/k_1	V_p	V^*_p	V^*_p/k_1	Δ(mm)
DL	$0.15 \times 10^{-3} \times 400^2/8$ = **3**	1.25 x 3 = **3.75**	3.75 / 0.57 = **6.58**	$5 \times 0.1510^{-3} \times 400/8$ = **0.04**	1.25 x 0.04 = **0.05**	0.05 / 0.57 = **0.09**	$\frac{0.15\text{x}10^{-3} \text{x} 400^4}{101.\text{EI}} = \frac{38\text{x}10^3}{\text{EI}}$
LL(UDL)	$5\text{e}10^{-3} \times 400^2/8$ = **100**	1.5 x 100 = **150**	150 / 0.8 = **156**	$5 \times 5\text{x}10^{-3} \times 400/8$ = **1.25**	1.5 x 1.25 = **1.9**	1.9 / 0.8 = **2.3**	$\frac{5\text{x}10^{-3}\text{x}400^4}{101\text{EI}} = \frac{1267\text{x}10^3}{\text{EI}}$
LL(conc)	$\frac{13 \times 7000 \times 400}{64 \times 520}$ =1094	1.5 x 1094 = 1641	1641 / 0.9 = **1823**	$11 \times 7\text{x}10^3 / (16\text{x}520)$ = **9.25**	1.5x9.25 = **13.9**	13.9 / 0.9 = **15.4**	$\frac{7000\text{x}400^3}{64\text{EI}\text{x}520} = \frac{13.5\text{x}10^6}{\text{EI}}$

Notes:

1. Units for moment are Nmm/mm width, units for shear are N/mm width.

2. As the maximum moment and shear due to self-weight are very small, and do not occur at the same location as the maximum live load moment and shear, load action effects for strength due to dead load will be ignored.

3. Load distribution width for concentrated loads has been assumed to be 520mm (refer TABLE 5.2)

2. Strength Limit State – Design Load Combinations

$M^*\text{crit} = M^*(\text{LLconc})$
$= 1823$ Nmm/mm width

$V^*\text{crit} = V^*(\text{LLconc})$
$= 15.4$ N/mm width

3. Shear criteria – (establish minimum f'_sA_s)

$\phi V_p \geq V^*_p$

and $\phi V_p = \phi k_1 k_{19} g_{19}[f'_sA_s]$

⇨ $\phi k_1 k_{19} g_{19}[f'_sA_s] \geq V^*p$

and $[f'_sA_s] \geq V^*p / (\phi k_1 k_{19} g_{19})$

Minimum required $f'sAs = 15.4/(0.9\text{x}0.94\text{x}1.0\text{x}0.4)$
$=$ **45.5** N/mm width

From Appendix 6C, Require minimum **12mm 12-15-5, F14** (f'sAs = 49)

4. Bending criteria – (establish minimum f'_bZ_p)

$\phi M_p \geq M^*_p$

and $\phi M_p = \phi k_1 k_{19} g_{19}[f'_bZ_p]$

⇨ $\phi k_1 k_{19} g_{19}[f'_bZ_p] \geq M^*p$

And $[f'_bZ_p] \geq M^*p / (\phi k_1 k_{19} g_{19})$

Minimum required $f'_bZ_p = 1823/(0.9\text{x}0.94\text{x}1.0\text{x}1.0)$
$=$ **2155** Nmm/mm width

From **TABLE 5.4**, suitable structural plywoods include:

F11, 25mm 25-30-9 (f'_bZ_p = 2468) Nmm/mm width
F14, 25mm 25-30-9 (f'_bZ_p = 2820) Nmm/mm width
F17, 19mm 19-30-7 (f'_bZ_p = 2325) Nmm/mm width
F27, 17mm 17-24-7 (f'_bZ_p = 2680) Nmm/mm width

5. **Serviceability limit state – Design Load Combinations**

1 x Q (short term)
G + 0.4Q (long term)

6. **Deflection criteria – (determine minimum required EI)**

Maximum allowable defection = (span/200)
= 2mm

Under short term load: $\Delta_{max} = j_2 \times g_{19} \times \Delta_{LL\ conc}$

$2mm = 1.0 \times 1.0 \times 13.5\text{x}10^6 / EI$

Required $\mathbf{EI_{min} = 6750 \times 10^3\ Nmm^2/mm\ width}$

From **TABLE 5.5**, any of the following structural plywoods will be suitable:

F11, 25mm, 25-30-9 (EI = 9450 x 10^3) Nmm^2/mm width
F14, 21mm, 21-24-9 (EI = 6780 x 10^3) Nmm^2/mm width
F17, 21mm, 21-30-7 (EI = 7770 x 10^3) Nmm^2/mm width
F27, 19mm, 19-24-9 (EI = 7030 x 10^3) Nmm^2/mm width

Under long term load: $\Delta_{max} = j_2 \times g_{19} \times \Delta\ (_{DL} + {}_{LL\ UDL})$

$2mm = 2.0 \times 1.0 \times (38 + 1267)\ \text{x}10^3 / EI$

$\mathbf{EI_{min} = 1305 \times 10^3\ Nmm^2/mm\ width}$

which is less than required EI under short term load ⇨ **not critical**

7. **Select Suitable Structural Plywood Flooring**

Subject to availability, suitable structural plywoods would **include**:
F11, 25mm, 25-30-9 (EI = 9450 x 10^3, f'_bZ_p = 2820)
F14, 25mm, 25-30-9 (EI = 6780 x 10^3, f'_bZ_p = 2820)
F17, 21mm, 21-30-7 (EI = 7770 x 10^3, f'_bZ_p = 2625)
F27, 19mm, 19-24-9 (EI = 7030 x 10^3, f'_bZ_p = 3160)

Plywood specification:
Specify number of sheets x 2400 x 1200 x 25mm, **structural plywood** to AS/NZS 2269, stress grade F11, (25-30-9), CD – **A bond, EWPAA / JAS-ANZ Product Certified**

7.7 Structural Plywood Flooring

Typical structural plywood thicknesses and stress grades for a range of minimum floor imposed loadings detailed in AS/NZS 1170 are given in TABLE 7.1. Other structural plywood stress grades and thicknesses are available and alternate stress grade/thickness combinations can be designed for and specified. Full design information **on** using structural plywood flooring**, including span tables and fixing details are** provided **in the** EWPAA design manuals T&G Structural Plywood for Residential Flooring **and** Structural Plywood for Commercial & Industrial Flooring **available from the EWPAA.**

Flooring Application	Uniformly Distributed Load (kPa)	Point Load (kN)	Structural Plywood thickness (mm)			
			Stress Grade F11		Stress Grade F14	
			Span 400mm	Span 450mm	Span 400mm	Span 450mm
Residential	1.5	1.8	15	15	15	15
Assembly Areas	3.0 – 5.0*	2.7 – 3.6	17 – 19	19 – 20	17 – 19	19
Public Corridors & Spaces	4.0 – 5.0	4.5*	20	21	19	20
Stages	7.5	4.5	20	21	19	20
Offices	3.0	6.7	25	25	21	25
Retail Sales Areas	5.0*	7.0*	25	25	21	25
General Storage	2.4*/m ht	7.0*	25	25	21	25
Drill Rooms and Halls	5.0*	9.0*	25	27	25	25

Notes:
1. Plywood sheets must be laid with face grain parallel to the span
2. Structural plywood is assumed to be a minimum of two span continuous.
**To be determined but not less than the given value*

TABLE 7.1: Summary of AS/NZS 1170.1 Floor Live Loads & Suitable Structural Plywood Thickness

7.8 Engineered Flooring System

An engineered floor system **for** residential applications**, utilising structural plywood and either LVL or seasoned pine joists and bearers has been** developed as **a cost competitive,** viable alternative to concrete slab on ground floors**, and the traditional unseasoned hardwood bearer and joist flooring system. Full details of the floor system are given in the design manual** LP91 Low Profile Stressed Skin Plywood Floor System which is a free download from the EWPAA website**. This cost and performance** optimised structural plywood platform floor system **is** designed with joists and bearers in the same horizontal plane**. The structural plywood flooring is then glued and nailed to the subfloor members to develop composite action and achieve maximum structural and material efficiency. Maximum grid support spacings of 3.6m x 3.6m are achievable using LVL for the bearer and joist elements, making this floor system particularly suitable for the upper floors of two or more storey buildings.**

7.9 Structural Laminated Veneer Lumber (LVL) and LVL / Plywood I-Beams

Structural LVL **and** LVL/Plywood I-Beams **are used as** joists **and** bearers **in both** residential, commercial **and** industrial flooring **applications. These engineered beams have the advantages of being dimensionally uniform and straight, lightweight, available in long lengths and possessing uniform, consistent and reliable structural properties.**

7.10 Structural Plywood Residential Bracing and Combined Bracing/Cladding

Structural Plywood Wall Bracing Design Manual

Structural plywood bracing systems in timber framed buildings provide designers with flexibility in design. The high bracing capacities **achievable using** structural plywood**, along with the ability to utilise short wall lengths,** facilitates **the use of** large wall openings **while** still maintaining structural adequacy**. With appropriate fixings and framing,** limit state bracing capacities **of up** to 8.7kN/m **can be** achieved for single sided plywood braced walls**;** twice this **capacity can be achieved** where **the** wall is braced both sides**. Plywood bracing allows walls as short as 0.3m to be utilised to achieve the desired bracing capacity. Additionally structural plywood with aesthetic grade faces can serve the dual purpose of bracing and wall claddings both internally and, when preservative treated, externally.** Guidance on **the** design **and use of structural plywood bracing are** given in **the** EWPAA Limit States Design Manual: Structural Plywood Wall Bracing**. Bracing capacities in this manual are based on actual tested systems. Typical failure modes for braced wall systems tested to failure were nail failure and pull through for thicker (7 mm +) plywood bracing and buckling of the plywood for thinner (4.5 mm or less) plywoods. The manual includes details on plywood stress grades and thicknesses, fastener specification and fixing details, bracing capacities of bracing systems, minimum framing requirements, bracing installation including bottom plate fixings and maximum permissible hole sizes through the braced wall for services.**

> ***NOTE:***
> *As a result of re-validation of plywood bracing systems the EWPAA now recommends plywood bracing be a minimum of 6mm thickness. A free downloadable copy of* EWPAA Limit States Design Manual – Structural Plywood Bracing *incorporating changes as a result of re-validation tests is available from the EWPAA website.*

Wall Bracing Testing Methodology

There are many factors **affecting bracing response which are difficult or even impossible to replicate in the testing of** discrete wall panels**. Some of the more obvious of these are:**

- **influence of** gravity loads **due to dwelling** self weight;
- **location of** return walls;
- **effect of** window **and** door openings;
- distribution of the racking load along the top plate.

Hence, to allow designers to use the bracing data to its fullest effect, some of the more important testing procedural aspects are discussed herein.

In the first instance, except for short wall **evaluation, test panels are generally:**

- free standing panels **fixed to the base support by bolts through the bottom plate;**
- **2400 or 2700mm long (depending on plywood width) x 2400mm high;**
- lateral buckling **of the top plate is prevented by supports placed either side of the panel;**
- **except when testing for** combined racking and uplift **the top plate is free of any encumberances;**
- nailing patterns**, fitting of** anti-rotation rods **and** nogging locations **are as given in the Bracing Manual.**

FIGURE 7.1 shows a typical panel arrangement prior to testing. T1 through T5 are transducers **located to measure panel and test frame deflections.**

The 1 and 2 identify two methods of fixing the plywood sheets to the timber framing. Type 1 **would entail the fitting of an** anti-rotation rod **and a** 150/300 nailing pattern **for** both sheets. Type 2 **would not incorporate an anti-rotation rod but would have a** close nailing pattern **along the** top and bottom plates of 50mm **and along** edge **and** internal studs of 150 **and** 300mm **respectively.**

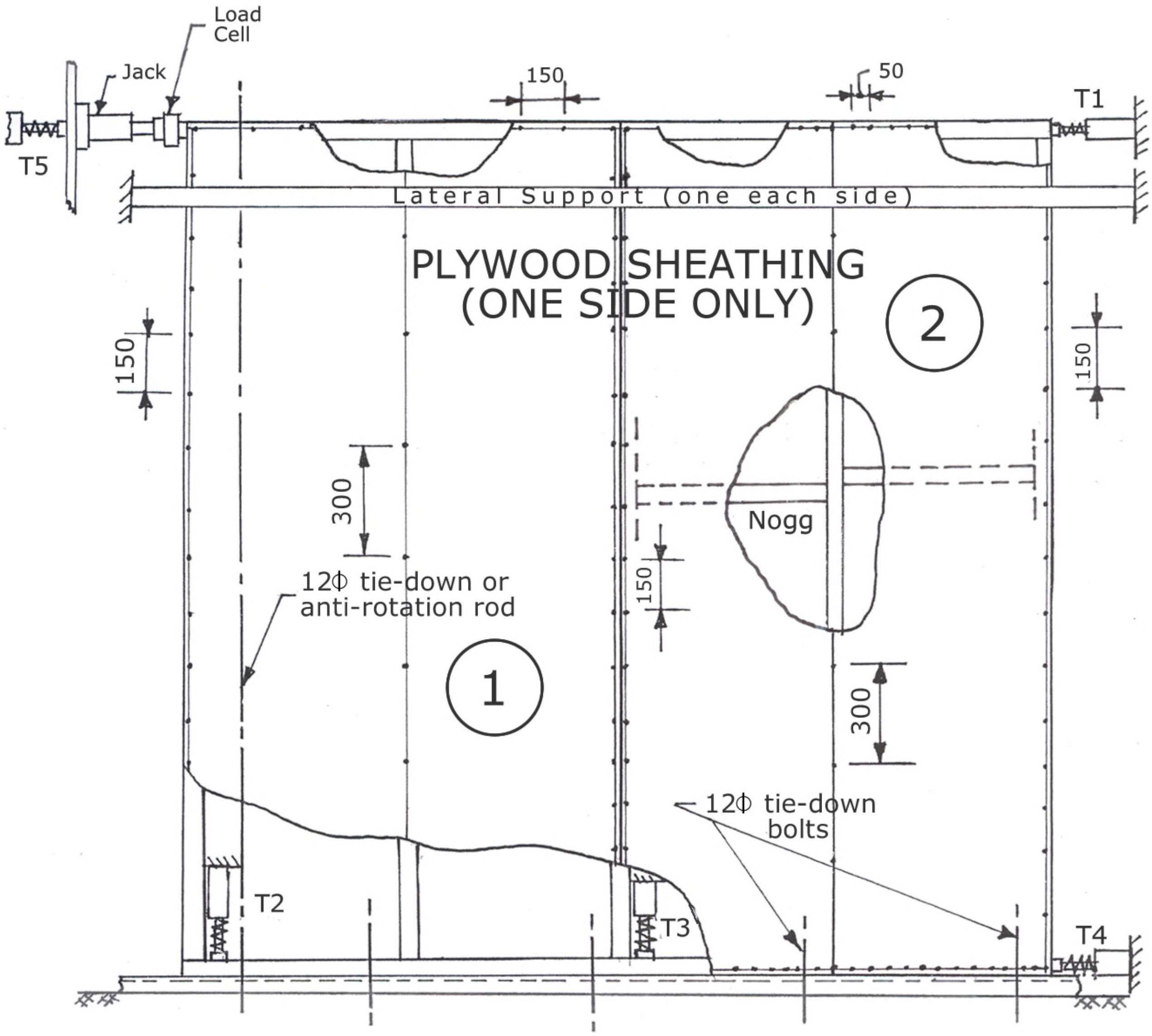


FIGURE 7.1: Typical test panel arrangement

To satisfy **Limit States** design criteria necessitates test panels must be:

- **Stiff enough** to ensure the **serviceability limit state** is satisfied. This is attained by setting a **deflection limit** at **T1** of **panel height / 300;**
- **Strong enough** to satisfy the **strength limit state.** This situation is taken to be satisfied, even though some **connector** and **material distress** may be evident, when the panel is still capable of taking further load.
- **Stable**, i.e. shows **no** significant signs of **buckling at** the **serviceability limit state**.

Because of the obvious difficulty associated with having to attempt an **analytical check** of the **racking deflection** of a dwelling it is essential:

"satisfaction of the **strength limit state** results in **automatic satisfaction** of the **serviceability limit state"**.

The **strength limit state** for EWPAA test panels has been established by determining the **racking load** at a **deflection limit** of **height/100**. To ensure a **reserve of strength:**

- **strength limit state defined by height/100 must be > 0.8 x ultimate racking load.**

To fully quantify the **racking load variables** requires:

- **strength limit state to be ≥ 1.5 x serviceability limit state.**

The bracing topic will be discussed further in CHAPTER 9 on **Shearwalls and Diaphragms.**

7.11 Structural Plywood Lightweight Roofing Systems

Tongued and grooved **structural** plywood **in combination** with overlayed waterproof membranes or shingles **is** used **as** lightweight**,** flat or curved roof systems **in residential, light commercial and industrial buildings. Design information including installation and fixing details for non-trafficable roof systems are contained in the EWPAA design manual** Featuring Plywood in Buildings**.** TABLE 7.2 **gives** minimum structural **plywood** thicknesses for rafter **or** truss spacings **for** non-trafficable roofs**. For** trafficable roof **systems the** plywood **must be** designed as a floor **in accordance with the EWPAA flooring design manuals detailed previously in this chapter. Design issues for structural plywood used in lightweight roofing systems are similar to those detailed for structural plywood flooring. Structural plywood roofing should be spanned with the face veneer grain direction parallel to the span to maximise the plywood capacity. Support spacings should be selected to suit the plywood sheet length, such that the ends of the sheet land on a support.**

Rafter or Truss Spacing	Minimum Allowable Plywood Thickness (mm)		
(mm)	F8	F11	F14
800	13	12	12
900	16	15	15
1200	19	17	16

TABLE 7.2: Minimum Structural Plywood Thickness and Support Spacing for Non-Trafficable Roof Systems Supporting Lightweight Roofing (20 Kg/M2)

7.12 Structural Laminated Veneer Lumber (LVL) Framing Members

EWPAA / JAS-ANZ certified structural LVL **and structural** plywood webbed**,** LVL flanged**,** I-Beams **are seasoned,** engineered timber members **that are dimensionally accurate,** with **very consistent,** defined engineering properties**. The high structural reliability and consistent performance of these engineered products means they have highly predictable strength and deflection characteristics and therefore** can be designed for use in single member, load critical applications, with confidence**. Their high strength to weight ratio and the availability of long lengths (12+ metres) facilitates handling and installation on site. Additionally, being timber, these products can be nailed, screwed and fixed with timber fasteners as well as sawn, drilled or otherwise modified using conventional carpentry tools.**

Design Issues for LVL Framing Members

LVL **is a** generic descriptor **used** to define **a** product fabricated from veneers laminated with adhesive**, in which the grain direction of the outer veneers and most of the inner veneers is in the longitudinal direction. The** mechanical properties **of structural** LVL **are** based **on the** properties of **the** parent material **used in fabrication and** are **therefore** specific for each manufacturer's product**. The manufacturer's brand name is used to identify the particular suite of engineering properties unique to their product. Therefore,** when specifying **a structural LVL product, the** brand name assigned by **the** manufacturer **to their product** must **also** be included in **any** specification **for LVL products.**

Generally, design **of structural** LVL **elements and components is** similar to **that for** sawn timber**. However, structural** LVL **is** differentiated from sawn timber **due to its engineered nature achieved** by randomising **any** naturally occurring timber characteristics throughout the member **and a high degree of process control during manufacture. The** end product **has** highly predictable structural properties **with a** low co-efficient of variation of these properties**. These attributes are reflected in the assignment to structural LVL of the highest possible capacity factor under the Timber Structures Code AS1720.1-1997.**

Structural LVL is manufactured as a seasoned, dimensionally uniform product and for best results, the product should be stored and utilised on site to minimise exposure to moisture.

7.13 Design Example – LVL Lintel Beam - Specification

Design and specification for a **structural lintel beam** supporting roof loads over doors, in a residential application, in a C1 Cyclonic wind classified area. Lintel beam will be **Best By Far (BBF)** brand LVL.

Characteristic Strengths and Elastic Moduli, MPa for BBF LVL as published by the manufacturer of BBF brand LVL		
Bending	f'_b	45 MPa
Tension	f'_t	31 MPa
Shear in beams	f'_s	5.3 MPa
Compression parallel to grain	f'_c	43 MPa
Compression perpendicular to grain	f'_p	11 MPa
Modulus of Elasticity	E	12400 MPa
Modulus of Rigidity	G	620 MPa
Joint Group	JD4	

Specification for the **LVL lintel beam** is as follows:

1. **Design criteria**:
   - Lintel beam is a single span of 3.6m.
   - Lintel beam is supporting rafter loads input as discrete point loads at 900mm centres.
   - Roof and ceiling loads are 40 kg/m^2.
   - Roof load width is 4.8m.
   - Adequate clearance must be maintained over doors. Therefore set deflection limit as follows:
     - Permanent Loads:Span/300 to 9 mm maximum
     - Imposed Loads: Span/250 to 9 mm maximum
     - Downwards Wind Loads:Span/250 to 9 mm maximum
     - Wind Uplift:Span/100 to 50 mm maximum

2. **Loads**

Permanent:

Roof & Ceiling load **= 40kg/m^2**
(40 x 9.81 x 4.8m x 0.9m) x10^{-3}
= 1.7kN/rafter

Self weight **allow 650kg/m^3**

Select Trial size beam 300 x 45 mm

Self weight = 65 x 9.81 x 0.3 x.045
= **0.09 kN/m**

Imposed:
1.4 kN concentrated imposed load (Assume live load directly over centre rafter)

Partial Imposed Loads: 0.25 kPa (Assumed spread over 3.6m width of lintel and 1.2 m roof load width) **AS/NZS 1170.1, Table3.2**

0.25 kPa x 1.2m x 0.9 m = **0.27 kN /rafter** **AS/NZS 1170.1,**

Table3.2

Wind:
(Cyclonic C1 Wind classified Area)

Strength:

Net Uplift = 2.04*kPa x 4.8m x 0.9m — **AS 4055 Table 6**

Net Uplift = 8.8 kN / rafter

Down:

Net pressure co-efficient ($C_{p,n}$) = 1.05

$V_{h,u}$ = 50 m/s — **AS 4055 Table B2**

Pressure $= 0.6 x V_{h,u}^2 \times 10^{-3} \times C_{p,n}$ — **AS 4055 Table 2**

$= 0.6 \times 50^2 \times 1.05/1000$

= 1.58kPa

1.58kPa x 4.8m x 0.9m = **6.8 kN / rafter**

Serviceability:

Net Uplift = 0.58kPa x 4.8m x 0.9m

Net Uplift = 2.5 kN/rafter

Down:

Net pressure co-efficient ($C_{p,n}$) = 1.05 — **AS 4055 Table B2**

$V_{h,s}$ = 32 m/s — **AS 4055 Table 2**

Pressure $= 0.6 \times V_{h,s}^2 \times 10^{-3} \times C_{p,n}$

$= 0.6 \times 32^2 \times 1.05$

= 0.65kPa

0.65kPa x 4.8m x 0.9m = **2.8 kN / rafter**

3. Load Combinations

Strength limit state: = 1.2G + 1.5Q — **AS/NZS1170.0 Cl.4.2.2**

$= 1.2G + W_u + \omega_c Q$

Q = 0 under max. downward wind loading

$= W_u\uparrow - 0.9\ G$

Serviceability limit state: G — **AS/NZS1170.0 Cl.4.3**

$\omega_s Q$ (ω_s = 0.7 for roofs)

W_s — **AS/NZS1170.0 T4.1**

4. Capacity Factor and Strength Modification Factors

The **relevant factors for beam design** are: — **Refer to AS1720.1-1997 or CHAPTER 6 of this manual for the following references**

Bending $(\phi k_1\ k_4\ k_6\ k_9 k_{11} k_{12})$

Shear $(\phi k_1\ k_4\ k_6\ k_{11})$

Deflection $(j_2\ j_6)$ — **AS1720.1-1997**

ϕ **= 0.9** for **LVL** in all **structural elements in houses** — **Table 2.5**

k_1 **= 0.57** for **permanent loads** such as roof self weight — **Table 2.7**

k_1 **= 0.94** for **imposed loads applied at infrequent intervals** such as might arise due maintenance — **Table 2.7** type loads. Effective duration of peak load = 5 days

k_1 = **1.0** for **wind gust loads** — **Table 2.7**

k_4 = **1.0** as it is **not anticipated** the **LVL moisture content will exceed** an average of **15%** in a dry interior application. In a dry interior application, moisture content would typically be in the range 8 to 12 % — **Table 8.1**

k_6 = **0.9 Coastal area of Queensland, north of latitude 25^0 S** — **Clause 8.4.4**

k_9 = **1.0** for all **LVL** used **in parallel systems** — **Clause 8.4.6**

k_{11} = **1.0** for **bending assuming lintel beam depth ≤ 300 mm** — **Clause 8.4.7(a)**

k_{11} = **1.0** for **shear** — **Clause 8.4.7(c)**

k_{12} **based on** value of $\rho_b S$ — **Clause 8.4.8**

ρ_b = $14.71\ (E/f'_b)^{-0.480} r^{-0.061}$. — **Clause 8.4.8**

$(E/f'_b)^{-0.480}$ = $(12400/45)^{-0.480} = 0.067$

r = **temporary** design action effect/**total** design action effect — **App. E2**

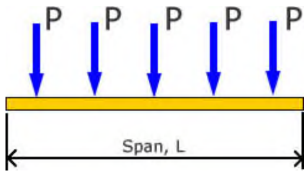
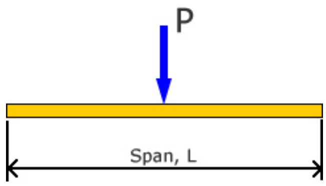
S_1 = $1.25\ d/b\ (Lay/d)^{0.5}$ (downwards loads, Lay = 0.9 m) — **Clause 3.2.3.2a**

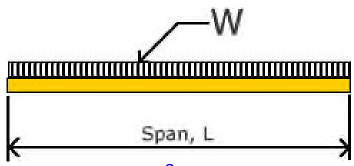
S_1 = $(d/b)^{1.35}\ (Lay/d)^{0.25}$ (wind uplift, Lay =0.9 m) — **Clause 3.2.3.2a**

5. Serviceability Modification Factors

j_2 = **1.0 short term load** — **Clause 8.4.2**
j_2 = **2.0 long term load** — **Clause 8.4.2**
j_6 = **1.0 (MC ≤ 15%)** — **Clause 8.4.3**

6. Critical Load Action Effects

Permanent roof loads (G) & Wind Loads (W_u)	 $M_{max}=PL/2$ $V_{max}=3P/2$ $\Delta_{max}=19PL^3/(384EI)$
Concentrated Imposed Load (Q)	 $M_{max}=PL/4$ $V_{max}= P/2$ $\Delta_{max}= PL^3/(48EI)$

Lintel Beam Self Weight (G)	 $M_{max}=wL^2/8$ $V_{max}= wL/2$ $\Delta_{max}= 5wL^4/(384EI)$

7.14 Structural LVL Lintel Beam : Worked Example

1. Design Action Effects on Member Due to Factored Loads

Loading Criteria	M_D (kNm) (unfactored)	M^*_D (kNm) (factored)	M^*_D/k_1 (kNm)	V_D (kN) (unfactored)	V^*_D (kN) (factored)	V^*_D/k_1 (kN)	Δ(mm)
DEAD **G =** Beam Self Weight + Permanent Roof Load	0.09 x 3.6^2 / B8 = 0.15 1.7 x 3.6/2 = 3.06 **Total** = **3.2**	**1.2 x 3.2** **= 3.8**	**3.8 / 0.57** **= 6.6**	0.09 x 3.6/2 = 0.16 3 x 1.7/2 = 2.55 **Total** **= 2.7**	1.2 x 2.7 = **3.2**	3.2/0.57 = **5.6**	5 x 0.09 x 3600^4 / 384EI 19 x 1.7 x 3600^3 / 384EI **Total** **= 2.0 x10^{11}/ EI**
IMPOSED **Q** (conc load) Q (UDL) **G + Q**	1.4 x 3.6/4 = 1.26 0.27x3.6/2 = 0.49	1.26 x 1.5 = 1.9 0.49 x 1.5 = 0.7 **3.8 + 2.6** **= 6.4**	**6.4 /0.94** **= 6.8**	1.4/2 = 0.7 0.27 x 3.6/2 = 0.5 **Total** **= 1.2**	1.5 x 0.7 = 1.1 1.5 x 0.5 = 0.8 **Total** **= 1.9**	**1.9/0.94** **= 2.0**	1.4 x 3600^3 / 48EI =1.4 **x10^9 / EI** 19 x 0.27 x 3600^3 / 384 EI = **0.62 x10^9 / EI**
WIND UP **W_u↑ 0.9G** **W_u↑ - 0.9G**	**8.8 x 3.6/2** **= 15.8**	1.0 x 15.8 = 15.8 0.9 x 3.2 = 2.9 **Total** **= 12.9**	**15.8 / 1.0** **= 15.8** **2.9 / 0.57** **= 5.1** **Total** **= 10.7**	3 x 8.8 / 2 = 13.2	3 x 8.8/2 = 13.2 0.9 x 3.2 = 2.9 **Total** **= 10.3**	**13.2/1.0** **= 13.2** **2.9/0.57** **= 5.1** **Total** **= 8.1**	19 x 2.5 x 3600^3 / 384EI **= 5.8 x10^9/ EI**
WIND DOWN **W_u↓** **G** **W_u↓ + G**	6.8 x3.6/2 = 12.	**12.2 + 3.8** = **16.0**	**12.2 / 1.0** **= 12.2** **3.8/0.57** **= 6.6** **Total** **= 18.8**	3 x 6.8 / 2 = 10.2	1.0 x 10.2 = 10.2 **3.2** **Total** **=13.4**	**10.2/1.0** **= 10.2** **5.6** **Total** **= 15.8**	19 x 2.8 x 3600^3 / 384EI **= 6.5 x 10^9/ EI**

2. Strength Limit State - Design Load Combinations

M*crit	=	M*($\mathbf{W_u}\downarrow$ + G)	=	18.8 kNm
V*crit	=	V*($\mathbf{W_u}\downarrow$ + G)	=	15.8 kN

Trial Beams:

240 x 45, A =10800 mm^2, I_{xx} = 51.8 x 10^6 mm^4, Z_{xx} = 432.0 x 10^3 mm^3
300 x 45, A =13500 mm^2, I_{xx} =101.3 x 10^6 mm^4, Z_{xx} = 675.0 x 10^3 mm^3

Determine k_{12} based on critical load combination and trial section sizes.

k_{12}: ρ_b = 14.71 $(E/f'_b)^{-0.480} r^{-0.061}$. **AS1720.1-1997, Cl8.4.8**

= 14.71 $(12400/45)^{-0.480}(12.2/18.8)^{-0.061}$
= 1.01

S_1 = 1.25 d/b $(Lay/d)^{0.5}$ (downwards loads, L_{ay} =0.9 m) **AS 1720.1, Cl3.2.3.2**

300 x 45 section size: = $1.25(300/45)(900/300)^{0.5}$ =14.4
ρ_b S_1 = 14.6 **=>** k_{12} = 1.5 – (0.05 x σ_b S_1) **=> k_{12} = 0.77** **AS 1720.1, Cl3.2.4**

240 x 45 section size: = $1.25(240/45)(900/300)^{0.5}$ =11.55
ρ_b S_1 = 11.8 **=>** k_{12} = 1.5 – (0.05 x σ_b S_1)**=> k_{12} = 0.91**

3. Bending criteria - (establish minimum Z_p)

$\boldsymbol{\phi M_p}$ **≥ M^*_p**
and $\phi M_p = \phi\, k_1\, k_4\, k_6\, k_9\, k_{11}\, k_{12}\, [f'_b Z_p]$
=> $\phi k_1\, k_4\, k_6\, k_9\, k_{11}\, k_{12}\, [f'_b Z_p]$ ≥ M^*_p
and Z_p ≥ $M^*_p / (\phi k_1\, k_4\, k_6\, k_9\, k_{11}\, k_{12})\, f'_b$

Try 240 x 45 section size k_{12} **= 0.91**
Minimum required Z_p = 18.8 x 10^6/(0.9x1.0x1.0x0.9x1.0x1.0x0.91) x 45
= **567x10^3 mm^3**
> Z_{xx} = **432.0** x **10^{3}mm^3** for **240 x 45 section size**
=> Not OK

Try 300 x 45 section size k_{12} **= 0.77**
Minimum required Z_p = 18.8 x 10^6/(0.9x1.0x1.0x0.9x1.0x1.0x0.77) x 45
= **670x10^3 mm^3**
< Z_{xx} **= 675.0 x 10^3 mm^3 for 300 x 45 section size**
=> OK

<u>=>REQUIRE 300 x 45 BBF LVL FOR BENDING STRENGTH</u>

4. Shear criteria - (establish minimum A_s)

$\boldsymbol{\Phi V_p}$ **≥ V^*_p**
and ΦV_p = $\Phi k_1\, k_4\, k_6\, k_9\, k_{11}[f'_s A_s]$
=> $\Phi k_1\, k_4\, k_6\, k_9\, k_{11}\, k_{12}\, [f'_s A_s]$ ≥ V^*_p
and $[A_s]$ ≥ $V^*p / (\Phi k_1\, k_4\, k_6\, k_9\, k_{11})\, f'_s$

Minimum required A_s = 15.8x10^3/(0.9x1.0x1.0x0.9x1.0x1.0) x 5.3
= **3680** mm^2
<A **= 13500mm^2 for 300 x 45 section size**
=> OK

5. Serviceability Limit State

Design Load Combinations: **G**
$\boldsymbol{\omega_s Q}$ (ω_s = 0.7 for roofs)
$\mathbf{W_s}$

Deflection criteria:

Maximum allowable defection:

Permanent Loads	**Span/300 to 9 mm maximum**
Imposed Loads & downwards Wind Loads	**Span/250 to 9 mm maximum**
Wind Uplift	**Span/100 to 50 mm maximum**

6. Under permanent load:

Δ_{max} = j_2 x j_6 x Δ_G

Δ_{max} = 2.0 x 1.0 x 2.0 x10^{11} / EI

300 x 45 BBF LVL, E = 12400 MPa

I_{xx} = 101.3 x 10^6 mm^4

Δ_{max} = 2.0 x 1.0 x 2 x 10^{11}/(12400 x 101.3 x 10^6)

Δ_{max} = 0.3 mm

<<< 9mm

=> OK

7. Under Imposed load:

Δ_{max} = j_2 x j_6 x 0.7Δ_Q

= 1.0 x 1.0 x 0.7 x 1.4x 10^9/EI

Δ_{max} = 1.0 x 1.0 x 0.7 x 1.4 x 10^9/(12400 x 101.3 x 10^6)

Δ_{max} = 0.001 mm

<<< 9mm

=> OK

8. Under Wind load:

Δ_{max} = j_2 x j_6 x Δ_{Ws}

= 1.0 x 1.0 x 6.5 x 10^9/EI

Δ_{max} = 1.0 x 1.0 x 6.5 x 10^9/(12400 x 101.3 x 10^6)

Δ_{max} = 0.005 mm

<<<9mm

=> OK

9. **Specification for** a **structural lintel beam supporting roof loads over doors**, in a residential application, in a C1 Cyclonic wind classified area:

LINTEL BEAM TO BE 3.6 M SPAN, 300 X 45 mm BBF BRAND LVL.

Part Three

Plywood Element & System Design Examples

Structural Plywood Webbed Box Beam Design

Structural Plywood Diaphragms & Shearwalls

Structural Plywood / LVL Gusseted Timber Portal Frames

Plywood Stressed Skin Panels

CHAPTER 8

8 STRUCTURAL PLYWOOD WEBBED BOX BEAM DESIGN

8.1 Introduction

Structural **plywood webbed beams** are a **composite timber beam** fabricated **utilising structural plywood as** the **web** of the beam and a **structural timber as** the **continuous beam flanges**. The **flange and web** components are usually **connected with nails and/or glue**. In this design chapter, information is given for the design of nailed plywood webbed box beams. The design method is based on the information contained in the EWPAA Design Guide for Plywood Webbed Beams. The **design aides included** within this section **facilitate** the **design of box beams with** and **structural plywood webs**. However, these design aides will also be useful in estimating the required size of box beam components and other wood based flanges and plywood web stress grades.

Structural plywood webbed beams are used in a wide variety of applications ranging from beams in residential applications, particularly lintel beams, through to rafters, columns, purlins and girts in industrial buildings and box beam portal frames. Although plywood webbed beams will typically need to be deeper to be structurally equivalent to a solid timber or steel beam, they have a number of useful advantages over solid timber and steel beams. Plywood webbed beams are usually designed as parallel flange box, C or I-beams, however they can also be designed and shaped to suit a particular application as tapered, curved or pitched beams. They are hollow and consequently light in weight, facilitating transportation and handling. They are easy to fabricate either as an independent component or, for nailed beams, in situ. Structurally the flanges are designed to carry the bending stresses while the webs transmit the shear. This achieves maximum structural efficiency as well as economy in material usage and overall costs.

FIGURE 8.1 shows the components of a structural plywood webbed box beam.

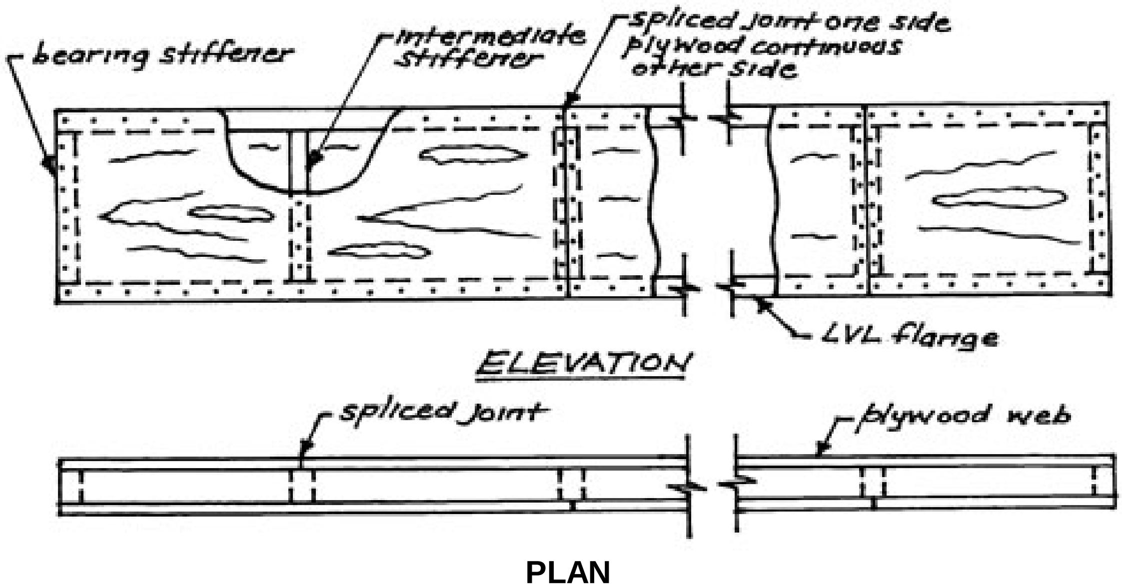


FIGURE 8.1: Plywood webbed box beam

8.2 Beam Components and Materials

Flanges

Flange material can be any structural timber product which complies with AS1720.1-1997 Timber Structures Code. Sawn structural timbers or glulam are suitable. The **flange material** needs to be **one continuous length**, or if this is not possible, seasoned timber or LVL **flanges can be spliced** to form a continuous structural member. A **spliced joint must provide equivalent strength and stiffness to an unjointed flange** of the same material. Two methods of joining seasoned timber flanges are shown in FIGURE 8.2. Alternatively seasoned timber flanges can be spliced with metal nail plates in accordance with the nail plate manufacturer's specification.

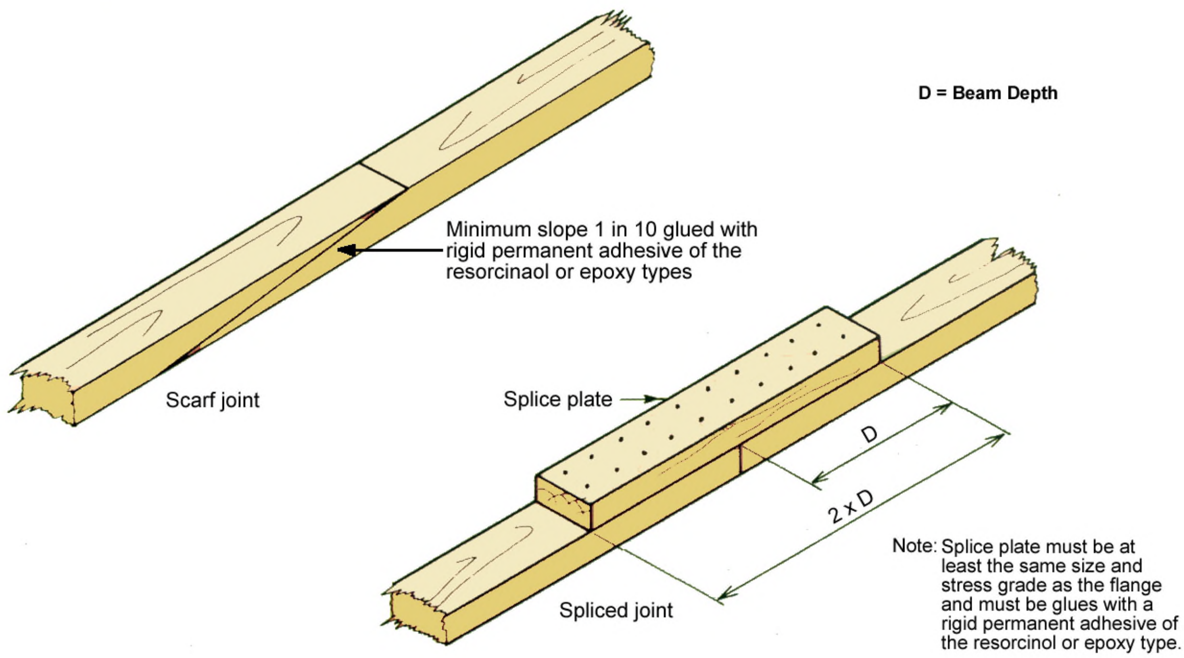


FIGURE 8.2: Jointing of seasoned flanges

Webs

Web material **must be structural plywood** manufactured **to AS/NZS 2269 Plywood – Structural**, and **branded** with the **EWPAA Tested Structural stamp**. **Consideration must be given to** the **direction of the face grain of** the **plywood web**. The Timber Structures Code, AS1720, adopts the parallel ply design method for plywood in which only plies parallel to the direction of stress are considered to contribute to the strength and most of the stiffness of the member.

The most common sheet size for structural plywood is 2400 x 1200 with the face grain running in the 2400 mm direction, but other lengths (2700, 1800) and widths (900) are also available. For efficient material usage with face grain parallel to beam span, plywood webbed beam depths should be 225, 300, 400, 450, 600, 900 or 1200 mm. **Webs with face grain** running **perpendicular to** the **span** are **less common**, but **enable fabrication of beams up to 2400 mm in depth**.

Web Stiffeners

Web stiffeners **are typically made from the same material as the beam flanges and are** used to control buckling of **the** plywood web**. They** will be structurally adequate if **they** extend **the** full depth of the flanges **and** have **the** same cross-sectional area as one of **the** flanges**. AS1720.1-1997 Appendix J2.5 specifies the requirements for web stiffeners. For convenience the** web stiffeners **are** usually located at web butt joint locations. Web stiffeners **should also be** located at **positions of** high load concentration **to counter localized web buckling.**

Adhesives

Beams relying only on an adhesive to connect the flange and web components must achieve a reliable structural bond. The only adhesives with proven structural durability and reliability are the Type A phenolic adhesives. To achieve a reliable bond with these adhesives requires good control over the bonding variables. Typically, beams with adhesive only flange/web bonds require factory controlled conditions to achieve quality bonds. The advantage of glued beams is they become a completely integrated unit with no slippage between the flanges and web, resulting in a stiffer beam. Glued I-beams with plywood web and LVL flanges are commercially available.

Nails

The **simplest method** to fabricate plywood webbed beams is to **nail** the **flange/web connections**. Nails must be flat head structural clouts. Smaller diameter nails at closer spacings are preferable to larger diameter nails widely spaced. The use of a **structural elastomeric adhesive** in conjunction **with nails**, is not a mandatory requirement, but it is **good practice** as it **helps** to **limit nail slip** and increase beam stiffness. **Hot dipped galvanized nails** should be **used in** areas of **high humidity** or **mildly corrosive environments** or **where preservative treated plywood or timber are used** as beam components. The availability of suitable **machine driven flathead nails** should also be considered, but if used, **should not be overdriven.**

8.3 Design of Nailed Plywood Webbed Box Beams - Methodology

The design method for nailed plywood webbed box beams presented in this chapter follows the limit states design methods detailed in AS 1720.1 Timber Structures Code and the design methodology set out in the **EWPAA Design Guide for Plywood Webbed Beams**. Formula for the design of C and I plywood webbed beams can be found in the EWPAA Design Guide for Plywood Webbed Beams. The **plywood webbed beam** is **analysed using transformed section methods and allowances** made **for** the effects of **nail slip**. **TABLES TABLE A8.4(a)** and **TABLE A8.4(b)** provide initial guidance for selecting a beam configuration based on span/depth and depth/width ratios and beam stiffness. Essentially the process for designing a nailed plywood webbed beam has the following steps:

1. **Select an initial beam trial size based on**

   (a) **Span/Depth** (L/D) **ratio** — **Appendix Table A8.4(a) & Fig A8.2**

   (b) **Depth/Breadth** (D/B) **ratio** — **Appendix Table A8.4(b)**

   (c) **Beam deflection** approximated from bending deflection

   Total deflection, Δ_τ in a nailed box beam is the **sum of** the **bending deflection (**Δ_b**), shear deflection (**Δ_s**),** and **nail slip** deflection **(**Δ_{ns}**) :** $\Delta_\tau = \Delta_b + \Delta_s + \Delta_{ns}$

   Typically, **shear** and **nail slip deflection** comprise **50% to 100% of** the **bending deflection**. (Note: In heavily loaded, deep beams, the percentage may be higher). That is:

   Δ_τ **is approximately in the range 1.5 x** Δ_b **to 2.0 x** Δ_b

Therefore a **trial beam size** can be **selected** from an estimate of total beam deflection **based on** the **expected bending deflection**. Bending deflection can be calculated from actual load conditions. Or, as done in the worked example, conservatively estimated from the beam flexibility tables, by determining the deflection of a simply supported, single span beam subjected to a central unit point load.

Appendix Table A8.5 or a uniformly distributed unit load (Appendix Table A8.6).

USING THE BEAM FLEXIBILITY TABLES
A trial beam size can be determined based on the value of F, the flexibility co-efficient, determined from Appendix Tables A8.5 and A8.6. The flexibility co-efficient, F, determined from the tables simply requires multiplication by the actual concentrated load P (kN) or uniformly distributed load w (kN/m) to determine the beam deflection.

Simply supported beam with a centre point load, P:	Simply supported beam with a uniformly distributed load, w:
$\Delta_b = j_2PL^3/48EI$ **$F = L^3/48EI$** **=> $\Delta_b = j_2 P F$** **$\Delta_\tau \approx$ (1.5 to 2.0) x Δ_b** **=> $\Delta_\tau \approx$ (1.5 to 2.0) x j_2 P F** **=>** Select F(max) from Table A8.5 such that $F_{max} \leq \Delta_\tau$(max. allowable) / (1.5 to 2.0)P j_2	$\Delta_b = j_2 5wL^3/384EI$ **$F = 5L^3/384EI$** **=> $\Delta_b = j_2$ w F** **$\Delta_\tau \approx$ (1.5 to 2.0) x Δ_b** **=> $\Delta_\tau \approx$ (1.5 to 2.0) x j_2 w F** => Select F(max) from Table A8.6 such that $F_{max} \leq \Delta_\tau$(max. allowable) / (1.5 to 2.0) w j_2

2. Check Flange Bending Capacity:

Determine critical load case for moment capacity and check flange capacity in tension and compression due to bending

Check **tension flange**:

$$\frac{M^*}{\varphi k_1 k_4 k_6 k_{11}} \leq \frac{2 f'_t (EI)_{xn}}{E_f d}$$

Check **compression flange**:

$$\frac{M^*}{\varphi k_1 k_4 k_6 k_9 k_{11} k_{12}} \leq \frac{2 f'_c (EI)_x}{E_f d}$$

NOTE:
For the above capacities may be conservatives. The above bending capacities do not take into account any impact of nail slip which may be significant in heavily loaded beams.

3. Check Panel Shear Capacity:

Determine critical load case **for shear** and check the **plywood web capacity for panel shear**
Both webs continuous

$$\frac{V^*}{\varphi k_1 k_{12} k_{19} g_{19}} \leq \frac{f'_s (EI)_x . n . t_w}{(EQ)_x}$$

At **web splice**

(i.e. one web continuous only)

$$\frac{V^*(\text{at web splice})}{\varphi k_1 k_{12} k_{19} g_{19}} \leq \frac{f'_s (EI)_{xn} . n . t_w}{(EQ)_x}$$

4. Check Flange - Web Capacity:

Design the **flange-web nailed connection** to transfer the shear flow

Shear flow q $= \frac{V^* (EQ)_{xf}}{(EI)_x . n}$

Design Load per Nail, **Q*** $= \Phi k_1 k_{13} k_{14} k_{16} k_{17} Q_k$

Nail spacing, **s** $= Q^*/q$

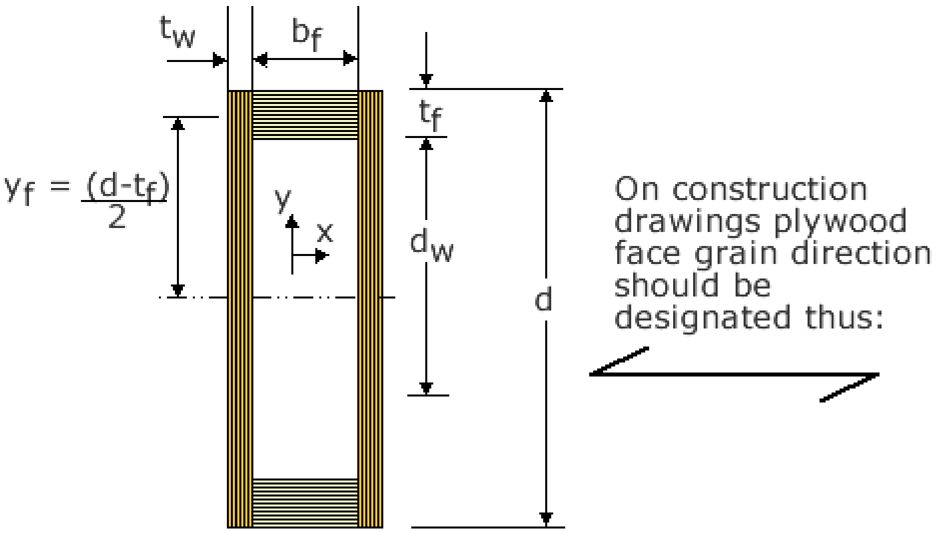
5. Check Beam Stiffness:

Check beam deflection Δ **is not excessive**, where :

$$\Delta_\tau = \Delta_b + \Delta_s + \Delta_{ns}$$

Box Beam Section Property Formula

where
A_f = Area of flange **=** $2\ b_f\ t_f$
A_w = Area of web = $2\ d\ t_w$
A_s = Web shear Area = $2\ d_n\ t_w$
d_n = depth between top and bottom flange-web nailing
= $(d - t_f)$, usually



Plywood Webbed Beam Dimensions

First Moment of Area

Q_{xf} = **First Moment** of Area of the **flange about** the beam **x axis**
= $A_f y_f = b_f\ t_f\ (d-t_f)/2$

Q_{xw} = **First Moment** of Area of the **webs about** the beam **x axis**
= $nk_{34}t_w\ d^2/8$

$(EQ)_x$ = First Moment **of Area** of **the** Box Beam
= **$E_f\ Q_{xf} + E_w\ Q_{xw}$**

$(EQ)_{xn}$ = **$(EQ)_x$ at web butt joint (i.e. only one web continuous, n = 1)**

Second Moment of Area

I_{xf} = **Second Moment** of Area of the **flange about** the beam **x-axis**
= $b_f(d^3-d_w^3)/12$

I_{xw} = **Second Moment** of Area of the **web about** the beam **x-axis**
= $nk_{34}t_w\ d^3/12$

where **n** = number of plywood webs (e.g. 2 for a box beam)

and **k_{34}**= parallel ply factor (note k_{34} is not an AS1720.1-1997 factor) **Table 8.1**

I_{yf} = **Second Moment** of Area of the **Flange about** the beam **y axis**
= $2t_f\ b_f^3/12$

I_{yw} = **Second Moment** of Area of the **Web about** the beam **y axis**
= $k_{34}t_w\ d(b_f + t_w)^2/2$

(using close approximation $2\ A_w\ x_f^2$ where $x_f = t_w/2 + b_f/2$).

Number of plywood veneer layers	k_{34}	
	Plywood face grain parallel to span	**Plywood face grain perpendicular to span**
3 ply	2/3	1/3
5 ply	3/5	2/5
7 ply	4/7	3/7

TABLE 8.1: Parallel Ply Factor, k_{34}

Rigidity in Bending About x-axis where:

$(EI)_x = E_f\ I_{xf} + E_w\ I_{xw}$
E_f = Modulus of Elasticity of the flange
E_w = Modulus of Elasticity of the web
$(EI)_{xn}$ = $(EI)_x$ at web splice (i.e. only one web continuous)

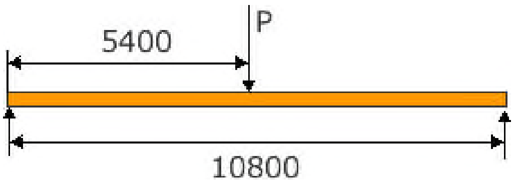
Rigidity in Bending About y-axis:

$$(EI)_y = E_f\ I_{yf} + E_w\ I_{yw}$$

8.4 Design Example – Nailed Plywood Webbed Box Beam

Design an industrial ridge beam that spans 10.8m. The beam supports two 600mm deep box beams that butt either side at mid span thus providing lateral restraint.

Given that:



P_{DL} = 10.8 kN
P_{LL} = 16.2 kN
P_{WL} = -57.8 kN (ult)
= -33.0 kN (serv)

Deflection Limits:

DL: Span/300 to 30mm max.
LL: Span/250 to 30mm max
WL: Span/200 to 50mm max

AS 1720.1 Appendix .B

1. Initial beam trial size:

(a) From Table A8.4(a) and Figure A8.2: Try a 900 mm deep beam which has a L/D ratio in the optimum range of 18:1 to 10:1 **Table 1 (a)**

From Table A8.4(b): Optimum beam width for a 900 deep beam is 90mm to 200mm.

(b) **Select** a **trial beam size based** on **deflection criteria**.
$\Delta_\tau = \Delta_b + \Delta_s + \Delta_{ns}$.
Assume shear and **nail slip** deflection are **75%** of **bending deflection**.
=> Δ_τ **= 1.75 x** Δ_b.

For a **simply supported beam with** a **central point load**:

$$\Delta_b = j_2 \times PL^3 / 48(EI)x$$
$$\&\Delta_\tau \simeq 1.75\, j_2 \times PL^3 / 48(EI)x$$
$$F = L^3/48\ EI \text{ where } F = \text{flexibility co-efficient}$$
$$\Rightarrow \Delta_\tau = 1.75 \times j_2 \times P \times F$$
$$\Rightarrow F_{(max)} \leq \Delta_\tau / 1.75 j_2 P$$

Load Type	Load (kN)	Deflection limit (mm)	j_2	$\Delta_{max}/(1.75 \times j_2 \times P)$ (mm/kN)
DL	10.8	30	2	0.79
LL	16.2	30	1	1.06
WL	-33.0	50	1	0.87

=>select a beam from Appendix Table A8.5 with a maximum beam flexibility of 0.82 mm. From Table A8.5, for a 10.8m span, & F = 0.82 mm/kN, gives a 900mm deep trial box beam with 150 x 35 LVL flanges and 7mm F11 structural plywood webs.

Beam Capacities

Box Beam: 900 mm deep x 150 x 35 LVL Flanges x 7 mm F11 structural plywood webs. Capacities for the various beam actions have been extracted from Table A8.7 and are given in **TABLE 8.2.**

Moment Capacity -Tension Flange		
	$2f'_t(EI)_{xn} / E_f\, d$	= 161 kNm
Moment Capacity - Compression Flange		
	$2f'_c(EI)_x / E_f\, d$	= 242 kNm
Web Shear		
	$f'_s(EI)_x.n.t_w / (EQ)_x$	= 59.3 kN
Web Shear at splice (only one web continuous)		
	$f'_s(EI)_{xn}.\ n.t_w / (EQ)_x$	= 30.7 kN
Unit Shear Flow for Nail Connection		
	$(EQ)_{xf} / (EI)_x$	= 0.94 $\times 10^{-3}$ mm^{-1}

TABLE 8.2 : Beam Capacities

Check factored loads and critical load cases:

Loads	Factored load combinations for strength limit states	Load combination value (kN)	k_1	V* (kN)	V*/k_1 (kN)	M* (kNm)	M*/k_1 (kNm)
DL	1.25G	15.8	0.57	7.9	13.8	42.6	74.7
DL + LL	1.25G +1.5Q	40.1	0.97	20.0	20.7	108.2	111.5
DL + WL	0.8G + Wu	-47.7	1.15	-23.8	-20.7	-128.8	-112.0

Load Combinations AS1170.1 Clause 2.2

k_1:AS1720.1-1997 Table 2.7

2. Flange Bending Capacity

Check Tension Flange $= M^* / \phi k_1 k_4 k_6 k_{11}$
$= 128.8 / (0.85x1.15x1x1x1)$
$= 131.8$ kNm (<161 kNm) **OK**

Check Compression Flange

Calculate k_{12} :

$S_1 = [(5.3L_{ay}(EI)_x)/(h_1D(EI)_y)]^{0.5}$
$= [(5.3x\ 5400x31894x10^9)/(5.5x900x803.7x10^9)]^{0.5}$
$= 15.2$

S_1:Section 8.3 A1.8.1.EI

h_1:AS1720.1-1997 Chapter 6

$\rho = 11.39(E_f/f'_c)^{-0.408}\ r^{-0.074}$
$\rho = 11.39(13200/45)^{-0.408}\ 1^{-0.074}$
$= 1.12$

ρ:AS1720.1-1997 App.E,E2(3)

$k_{12} = 1.5 - 0.05\ \rho\ S_1$
$= 1.5 - 0.05x1.12x15.2$
$\mathbf{k_{12} = 0.65}$

$M^*/\phi k_1 k_4 k_6 k_9 k_{11} k_{12} = 128.8 / (0.85x1.15x1x1x1x1x0.65)$
$= 203$ kNm (< 242 kNm) **OK**

3. Panel Shear Capacity

Calculate k_{12} : From **TABLE 5.9** for 7 x 900 mm webs for buckling strength of plywood webs. **Note** : **k_{12} = 0.52 is slightly conservative.** A check of panel length confirms the plywood web is a short panel and if the appropriate reduction is applied, k_{12} = 0.56.

$V^* / \phi k_1 k_{12} k_{19} g_{19} = 23.8 / (0.8x1.15x0.52x1x1)$
= **49.7 kN** (<59.3 kN) **OK**

Panel Shear at Web Splice (k12 = 1.0 at web splice)

S:AS1720.1-1997 App.J, Table J2

$V^*_{(at\ splice)} / \phi k_1 k_{12} k_{19} g_{19} = 23.8 / (0.8x1x1.15x1x1)$

= 26 kN (< 30.7 kN) **OK**

4. Flange - Web Connection

Design nailed flange-web connection. Use 2.8mm diameter nails:

Check critical load case for fasteners:

Load combinations	V* (kN)	k_1 for connectors	V*/k_1 (kN)
1.25G	7.9	0.57	13.9
1.25G +1.5Q	20.0	0.86	23.3
0.8G + Wu	23.8	1.30	18.3

Required nail spacing **s** = Q*/q, Characteristic capacity, Q_k, of 2.8mm nail in JD4 timber is 665 N

AS1720.1-1997 Table 4.1(B)

Design load per nail Q* $= \phi_1 k_{13} k_{14} k_{16} k_{17} Q_k$.

$= 0.8 \times 0.86 \times 1 \times 1 \times 1.1 \times 1 \times 665$

Q* = 503 N/nail

Shear flow at connection:

$q = V^*(EQ)_{xf} / (EI)_x.n$

$= (20 \times 10^3 \times 0.94 \times 10^{-3})/2$

$q = 9.4$ N/mm

=> $s_{max.}$ = 503/9.4

= **53 mm**

=> Use 2.8 ϕ nails at 50mm spacing

5. Beam stiffness

For a single span, simply supported beam:

Deflection type	Estimated mid-span deflection due to a centre point load, P	Estimated mid-span deflection due to uniformly distributed load, w
Bending	$j_2 \times PL^3/48(EI)_x$	$j_2\ 5\ wL^4/384(EI)_x$
Shear	$j_2 \times PL/4GA_s$	$j_2\ w\ L/8GA_s$
Nail slip*	$\frac{d_n L}{64} \frac{[E_f A_f\ .A.P]^2}{(EI)_x\ A}$	$\frac{d_n L^3}{192} \frac{[E_f A_f\ .S.w]^2}{(EI)_x\ A}$

***Refer Chapter 8** Appendix **for nail slip deflection equations**

Load Type	j2	Load		Estimated Deflection, mm			Total (mm)	
		Point (kN)	UDL (kN/m)	Bending	Shear	Nail Slip		
DL	2	10.8	0.16	18.4	10	1.7	30.1	**OK** (just)
LL	1	16.2		12.5	7	1	20.5	**OK**
WL	1	-33		25.4	14	4.3	43.7	**OK**

So go with initial trial beam selection

A8 Chapter 8 Appendix

Bending / Compressive Strength Stability Factor k_{12}

The stability factor k_{12} reduces the allowable compressive or bending stresses for slender beams that are subject to torsional buckling due to lateral instability of compression flanges. The beam capacity can be increased by providing lateral restraint to compression flanges, full restraint to the tension flange or by using a mores stocky beam.

Calculation of k_{12} for strength reductions for buckling of plywood diaphragms is covered in Appendix E of AS1720.1-1997. A more thorough examination of lateral torsional buckling, slenderness co-efficients and critical elastic buckling moment can be found in Appendix E of AS1720.1-1997 and the EWPAA Design Guide for Plywood Webbed Beams. The approach used in this Manual is to approximate the slenderness co-efficient for box beams using the formula:

$$S_1 = \left(\frac{5.3.L_{ay}.(EI)_x}{h_1.d.(EI)_y} \right)^{0.5} \tag{A8.E1}$$

where:

L_{ay} = distance between effectively rigid buckling restraints
h_1 = constant from AS1720.1-1997 Table E6 reproduced below

Moment parameter β (see diagram below)	Slenderness factor h_1	
	Free restraint condition*	Fixed restraint condition*
1.0	**3.1**	**6.3**
0.5	**4.1**	**8.2**
0.0	**5.5**	**11.1**
-0.5	**7.3**	**14.0**
-1.0	**8.0**	**14.0**

*The buckling restrains must prevent rotation of the beam about the z-axis. The terms 'free' and 'fixed' restraint condition refer to the possibility for rotation of the beam about the y-y axis at the restraint locations, as shown in Figure A8.1.

TABLE A8.1 : Reproduced from Table E6, AS 1720.1-1997

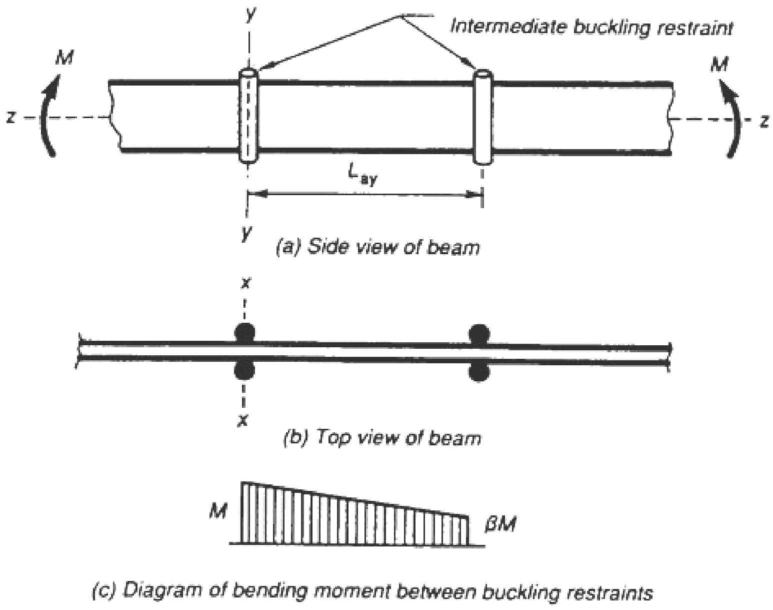


FIGURE A8.1: Lateral Buckling Terminology

Formula A8.E1 is accurate to within approximately 10% and is based on the use of 3 ply webs and the following assumptions:

$d = 1.1(d - t_f)$
$b_f + t_w = 1.08 b_f$
$(EI)_x = 1.25 E_t I_{xf}$
$(EI)_y = 1.6 E_w I_{yw}$

For 5 ply webs, the only change required to Equation A8.E1 is to reduce 5.3 to 5.1

Nail Slip Deflection Equations

In nailed box beams, shear slips may occur between the beam components depending on the effectiveness of the nailed joints. The effect of joint slips are to increase beam bending deflection and to change beam share stress distributions. Nail slip deflection in the design example has been calculated based on the linear elastic solutions for continuous web beams established by R.B. Sandie and published in The Flexural Behaviour of Nail Timber Boxed Beams.

For mid span deflections, for a simply supported beam of span L, deflection due to nail slip is estimated from:

For a central load P $\quad \mathbf{\Delta_{ns}} = \dfrac{\mathbf{d_n.L}}{\mathbf{64}}\left[\dfrac{\mathbf{E_f A_f . s . P}}{\mathbf{(EI)_x A}}\right]^{\mathbf{2}}$

For a UDL of w/unit length $\quad \mathbf{\Delta_{ns}} = \dfrac{\mathbf{d_n L}^3}{\mathbf{192}}\left[\dfrac{\mathbf{E_f A_f . s . W}}{\mathbf{(EI)_x A}}\right]^{\mathbf{2}}$

where:

d_n = distance between nail centres in each flange
s = nail spacing
L = beam span
E_f = Modulus of Elasticity of Flange Material
A_f = Area of Flange
$(EI)_x$ = Beam flexural rigidity about x-axis

$$A = h_{32}\sqrt{\frac{D^{35}.10^3}{J_{12}.44}}\ N.m^{-0.5}$$

where:

h_{32} = stiffness factor from Table C1 of AS 1720.1-1997 – reproduced herein in **TABLE A8.2** .

D = Nail diameter in mm

J_{12} = load duration factor (from Table C2 of AS 1720.1) – reproduced herein in **TABLE A8.3** .

For seasoned timber, substituting appropriate j_{12} values, values for A under short duration loads and long duration loads are:

Short duration: $A_L = 4.767\ h_{32}D^{1.75}$

Long duration: $A_D = 0.5\ A_L$

For example, in the box beam design example given, nail slip deflection due to the central wind point load is:

$$\Delta_{ns} = \frac{d_n.L}{64}\left[\frac{E_f A_f.s.P}{(EI)_x A}\right]^2$$

$$= \frac{865 x 10800 x}{64}\left[\frac{13200x(150x35)x50x16200}{31894x10^9 x21669}\right]^2 m$$

$$\Delta_{ns} = 0.001m$$

where:

$A = 4.767\ h_{32}D^{1.75}$
$= 4.767 \times 750 \times 2.8^{1.75}$
$= 20574\ Nm^{-0.5}$

Initial Moisture Condition	Species joint group	Factor h_{32}
Unseasoned	**J1**	**1450**
	J2	**1050**
	J3	**750**
	J4	**550**
	J5	**410**
	J6	**300**
Seasoned	**JD1**	**1600**
	JD2	**1250**
	JD3	**990**
	JD4	**750**
	JD5	**590**
	JD6	**470**

TABLE A8.2 : Stiffness Factor h_{32} for Nailed and Screwed Joints in Solid Timber

Initial moisture condition	Duration of load	Factor j_{12}	Factor j_{13}
Unseasoned	More than 3 years	9	0.5
	5 months	4	0.7
	Less than 2 weeks	1	1
Seasoned	More than 3 years	4	0.5
	Less than 2 years	1	1

NOTE: If required, intermediate values of j_{12} and j_{13} may be obtained by linear interpolation with log-time

TABLE A8.3 : Duration Factors j_{12} and j_{13}

Panel Shear Slenderness Co-efficient, S and Stability Factor k12 for Edge Shear Forces

The requirements for **strength reductions** to allow **for buckling** of plywood diaphragms is detailed in **Appendix J of AS 1720.1-1997**. The strength of reductions are stated in terms of a stability factor k_{12}, based on the slenderness co-efficient, S of the plywood diaphragm. $\mathbf{k_{12}}$ stability factors for **plywood diaphragms with lateral edges supported** have been **tabulated in** this Manual in **TABLE 5.9**. These factors will be slightly conservative if the plywood diaphragm is a short panel (refer Appendix J, AS 1720.1-1997) or the 0.8 reduction factor is applied where the plywood web is considered “fixed” to the flanges and “pinned” at the web stiffeners.

Guide table for selecting initial trial beam size based on span/depth and depth/breadth ratios

Beam Depth (mm)	(a) Span/Depth ratio				(b) Depth/breadth ratio	
	25 : 1	18 : 1	10 : 1	5 : 1	10 : 1	4.5 : 1
		Optimum range				
	Very lightly loaded beams eg purlins	Lightly loaded beams	Residential type loads	Heavily loaded beams		
		SPAN (m)			BREADTH (mm)	
225	5.6	4.1	2.3	1.1	23	50
300	7.5	5.4	3.0	1.5	30	67
400	10.0	7.2	4.0	2.0	40	89
450	11.3	8.1	4.5	2.3	45	100
600	15.0	10.8	6.0	3.0	60	133
900	22.5	16.2	9.0	4.5	90	200
1200	30.0	21.6	12.0	6.0	120	267

TABLE A8.4: (a)Span/Depth – (b) Depth/Breadth

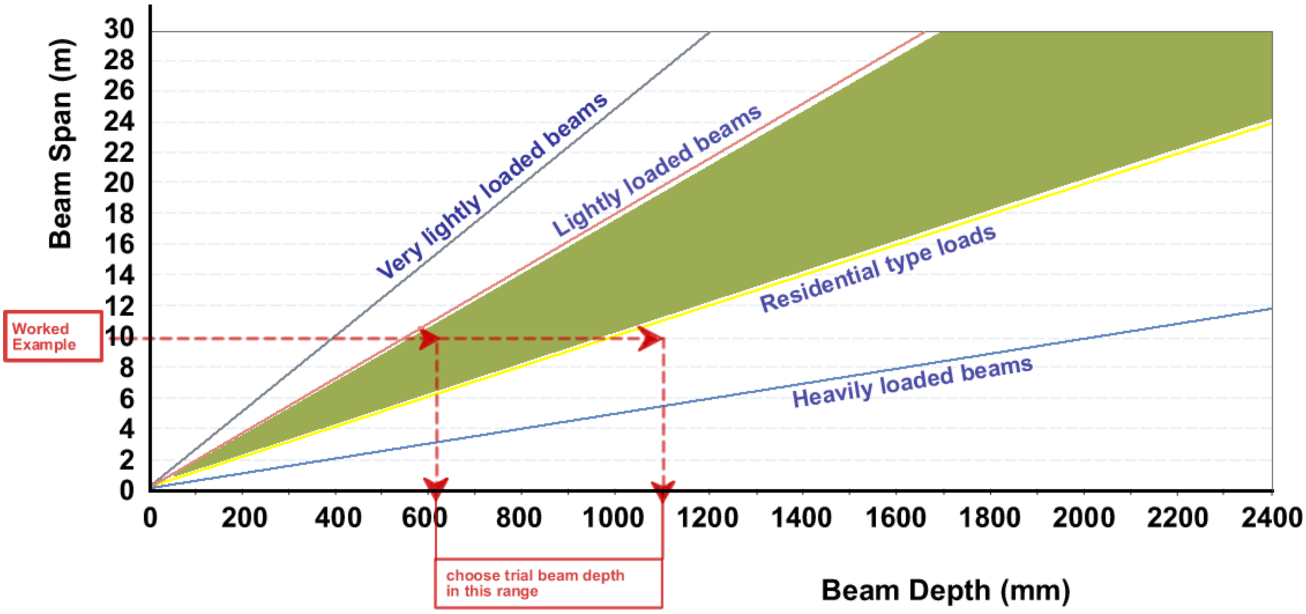


FIGURE A8.2: Guide for Selecting Initial Beam Depth

Table A8.5: Unit-Load-Deflection Span Tables for a Simply Supported Box Beam with a unit Centre Point Load, P = 1kN

Box Beam Components – Structural LVL Flanges and F11 Structural Plywood Webs – 7mm Thick Plywood

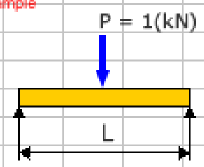
Beam Component	Material	Characteristic Strength (Mpa)				Short Duration Average Modulii (Mpa)		Density (kg/m^3)	Strength Group	Nominal web thickness, tw (mm)	Number of veneers	Number of webs	K34
		f'b	f't	f's	f'c	MOE	MOR						
Flanges	LVL	48	33	5.3	45	13200	660	620	JD4				
Webs	F11 Plywood	35	20	5.3	25	10500	525	550	JD4	7.00	3.00	2.00	0.67

Simply supported beam with a unit centre point load of P= 1 (kN)

Beam deflection per unit kN load (mm/kN)

Depth of Section D (mm)	Flange Width bf (mm)	Flange Depth tf (mm)	1.2	1.8	2.4	3	3.6	4.2	4.8	5.4	6	6.6	7.2	7.8	8.4	9	9.6	10.2	10.8	11.4	12	12.6	13.2	13.8	14.4	15	15.6	16.2	16.8	17.4	18
			Beam Span (m)																												
225	63	35	0.06	0.19	0.46	0.90	1.56	2.47	3.69	5.25																					
225	63	45	0.05	0.17	0.40	0.79	1.37	2.17	3.24	4.61																					
300	63	35		0.10	0.23	0.45	0.78	1.24	1.85	2.63	3.60	4.80	6.23																		
300	63	45		0.08	0.20	0.39	0.67	1.06	1.59	2.26	3.10	4.13	5.36																		
400	63	35		0.05	0.12	0.23	0.39	0.63	0.93	1.33	1.82	2.43	3.15	4.01	5.00	6.16															
400	63	45		0.04	0.10	0.19	0.34	0.53	0.80	1.13	1.56	2.07	2.69	3.42	4.27	5.25	6.37														
400	83	35		0.04	0.09	0.18	0.32	0.50	0.75	1.06	1.46	1.94	2.52	3.20	4.00	4.92	5.98														
400	83	45		0.03	0.08	0.15	0.27	0.42	0.63	0.90	1.23	1.64	2.13	2.71	3.39	4.17	5.06														
450	63	35			0.09	0.17	0.30	0.47	0.71	1.01	1.38	1.84	2.39	3.04	3.79	4.66	5.66	6.79													
450	63	45			0.08	0.15	0.25	0.40	0.60	0.86	1.18	1.57	2.03	2.58	3.23	3.97	4.82	5.78	6.86												
450	83	35			0.07	0.14	0.24	0.38	0.57	0.81	1.11	1.48	1.92	2.44	3.05	3.75	4.55	5.45	6.47												
450	83	45			0.06	0.12	0.20	0.32	0.48	0.68	0.94	1.25	1.62	2.06	2.57	3.16	3.84	4.60	5.46												
600	83	35				0.07	0.12	0.20	0.29	0.42	0.57	0.76	0.99	1.25	1.56	1.92	2.34	2.80	3.33	3.91	4.56	5.28	6.07	6.94							
600	83	45				0.06	0.10	0.16	0.25	0.35	0.48	0.64	0.83	1.05	1.32	1.62	1.97	2.36	2.80	3.29	3.84	4.45	5.11	5.84	6.64	7.50					
600	130	35				0.05	0.09	0.14	0.20	0.29	0.40	0.53	0.68	0.87	1.09	1.34	1.62	1.95	2.31	2.72	3.17	3.67	4.22	4.82	5.47	6.19					
600	130	45				0.04	0.07	0.11	0.17	0.24	0.33	0.44	0.57	0.72	0.90	1.11	1.35	1.62	1.92	2.26	2.63	3.05	3.50	4.00	4.55	5.14					
900	130	35						0.05	0.08	0.12	0.16	0.21	0.27	0.35	0.43	0.53	0.65	0.78	0.92	1.09	1.27	1.47	1.69	1.93	2.19	2.47	2.78	3.11	3.47	3.86	4.27
900	130	45							0.07	0.10	0.13	0.18	0.23	0.29	0.36	0.44	0.54	0.65	0.77	0.90	1.05	1.22	1.40	1.60	1.82	2.06	2.31	2.59	2.89	3.21	3.55
900	130	63							0.05	0.07	0.10	0.14	0.18	0.23	0.28	0.35	0.42	0.50	0.60	0.70	0.82	0.95	1.09	1.25	1.42	1.60	1.80	2.02	2.25	2.50	2.77
900	150	35							0.07	0.10	0.14	0.19	0.24	0.31	0.39	0.48	0.58	0.69	0.82	0.97	1.13	1.31	1.50	1.72	1.95	2.20	2.48	2.78	3.10	3.44	3.81
900	150	45							0.06	0.09	0.12	0.16	0.20	0.26	0.32	0.39	0.48	0.57	0.68	0.80	0.93	1.08	1.24	1.42	1.61	1.82	2.05	2.30	2.56	2.85	3.15
900	150	63							0.05	0.07	0.09	0.12	0.16	0.20	0.25	0.31	0.37	0.44	0.53	0.62	0.72	0.84	0.96	1.10	1.25	1.41	1.59	1.78	1.99	2.21	2.44
1200	130	45									0.07	0.09	0.12	0.15	0.19	0.23	0.28	0.34	0.40	0.47	0.55	0.63	0.73	0.83	0.95	1.07	1.20	1.35	1.50	1.67	1.85
1200	130	63									0.05	0.07	0.09	0.12	0.15	0.18	0.22	0.26	0.31	0.37	0.43	0.50	0.57	0.65	0.74	0.84	0.94	1.05	1.18	1.31	1.45
1200	150	45									0.06	0.08	0.11	0.13	0.17	0.21	0.25	0.30	0.36	0.42	0.49	0.57	0.65	0.74	0.85	0.96	1.08	1.20	1.34	1.49	1.65
1200	150	63									0.05	0.06	0.08	0.10	0.13	0.16	0.19	0.23	0.28	0.33	0.38	0.44	0.51	0.58	0.66	0.74	0.83	0.93	1.04	1.16	1.28
1200	200	45									0.05	0.06	0.08	0.11	0.13	0.16	0.20	0.24	0.28	0.33	0.39	0.45	0.51	0.59	0.67	0.75	0.85	0.95	1.06	1.18	1.30
1200	200	63									0.04	0.05	0.06	0.08	0.1	0.12	0.15	0.18	0.22	0.25	0.3	0.34	0.39	0.45	0.51	0.58	0.65	0.73	0.81	0.9	1

Worked Example



Bending deflection Δ_b per unit load P=1 (kN) =

$$\Delta_b = \frac{L^3}{48*(EI)_x} \text{ mm/kN}$$

Lightly Loaded Beams

Medium / Resedential Loaded Beams

Heavily Loaded Beams

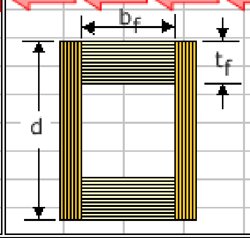


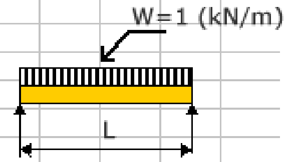
Table A8.6: Unit-Load Deflection Span Tables for a Simply Supported Box Beam with a Uniformly Distributed Load, w = 1 kN/m

Box Beam Components – Structural LVL Flanges and F11 Structural Plywood Webs – 7mm Thick Plywood

Beam Component	Material	Characteristic Strength (Mpa)				Short Duration Average Modulii (Mpa)		Density (kg/m³)	Strength Group	Nominal web thickness, tw (mm)	Number of veneers	Number of webs	K34
		f'b	f't	f's	f'c	MOE	MOR						
Flanges	LVL	48	33	5.3	45	13200	660	620	JD4				
Webs	F11 Plywood	35	20	5.3	25	10500	525	550	JD4	7	3	2	0.67

Simply supported beam with a uniformly distributed unit load of w = 1(kN/m)

Depth of Section	Flange Width	Flange Depth	Beam deflection per unit load, w = 1 kN/m (mm/kN/m) — Beam Span (m)																												
D	bf	tf																													
mm	mm	mm	1.2	1.8	2.4	3	3.6	4.2	4.8	5.4	6	6.6	7.2	7.8	8.4	9	9.6	10.2	10.8	11.4	12	12.6	13.2	13.8	14.4	15	15.6	16.2	16.8	17.4	18
225	63	35	0.04	0.22	0.7	1.7	3.5	6.5	11.1	17.7	27.0																				
225	63	45	0.04	0.19	0.6	1.5	3.1	5.7	9.7	15.6	23.7																				
300	63	35		0.11	0.3	0.8	1.8	3.2	5.5	8.9	13.5	19.8	28.0																		
300	63	45		0.09	0.3	0.7	1.5	2.8	4.8	7.6	11.6	17.0	24.1																		
400	63	35		0.06	0.2	0.4	0.9	1.6	2.8	4.5	6.8	10.0	14.2	19.5	26.3	34.6	44.8														
400	63	45		0.05	0.1	0.4	0.8	1.4	2.4	3.8	5.8	8.5	12.1	16.7	22.4	29.5	38.2														
400	83	35		0.04	0.1	0.3	0.7	1.3	2.2	3.6	5.5	8.0	11.3	15.6	21.0	27.7	35.9														
400	83	45		0.04	0.1	0.3	0.6	1.1	1.9	3.0	4.6	6.8	9.6	13.2	17.8	23.4	30.3														
450	63	35			0.1	0.3	0.7	1.2	2.1	3.4	5.2	7.6	10.7	14.8	19.9	26.2	34.0	43.3	54.4												
450	63	45			0.1	0.3	0.6	1.1	1.8	2.9	4.4	6.5	9.1	12.6	16.9	22.3	28.9	36.8	46.3												
450	83	35			0.1	0.3	0.5	1.0	1.7	2.7	4.2	6.1	8.6	11.9	16.0	21.1	27.3	34.8	43.7												
450	83	45			0.1	0.2	0.5	0.8	1.4	2.3	3.5	5.1	7.3	10.0	13.5	17.8	23.0	29.3	36.9												
600	83	35				0.1	0.3	0.5	0.9	1.4	2.1	3.1	4.4	6.1	8.2	10.8	14.0	17.9	22.4	27.9	34.2	41.6	50.1	59.8	70.9	83.5					
600	83	45				0.1	0.2	0.4	0.7	1.2	1.8	2.6	3.7	5.1	6.9	9.1	11.8	15.0	18.9	23.5	28.8	35.0	42.2	50.4	59.7	70.3					
600	130	35				0.1	0.2	0.4	0.6	1.0	1.5	2.2	3.1	4.2	5.7	7.5	9.7	12.4	15.6	19.4	23.8	28.9	34.8	41.6	49.3	58.0					
600	130	45				0.1	0.2	0.3	0.5	0.8	1.2	1.8	2.6	3.5	4.7	6.2	8.1	10.3	12.9	16.1	19.7	24.0	28.9	34.5	40.9	48.2					
900	130	35							0.2	0.4	0.6	0.9	1.2	1.7	2.3	3.0	3.9	5.0	6.2	7.7	9.5	11.5	13.9	16.6	19.7	23.2	27.1	31.5	36.5	42.0	48.1
900	130	45							0.2	0.3	0.5	0.7	1.0	1.4	1.9	2.5	3.2	4.1	5.2	6.4	7.9	9.6	11.6	13.8	16.4	19.3	22.5	26.2	30.3	34.9	40.0
900	130	63							0.2	0.3	0.4	0.6	0.8	1.1	1.5	1.9	2.5	3.2	4.0	5.0	6.2	7.5	9.0	10.8	12.8	15.0	17.6	20.4	23.6	27.2	31.1
900	150	35							0.2	0.3	0.5	0.8	1.1	1.5	2.0	2.7	3.5	4.4	5.6	6.9	8.5	10.3	12.4	14.8	17.6	20.7	24.2	28.1	32.5	37.4	42.9
900	150	45							0.2	0.3	0.4	0.6	0.9	1.3	1.7	2.2	2.9	3.7	4.6	5.7	7.0	8.5	10.3	12.2	14.5	17.1	20.0	23.3	26.9	31.0	35.5
900	150	63							0.1	0.2	0.3	0.5	0.7	1.0	1.3	1.7	2.2	2.8	3.6	4.4	5.4	6.6	7.9	9.5	11.3	13.3	15.5	18.0	20.9	24.0	27.5
1200	130	45									0.3	0.4	0.5	0.7	1.0	1.3	1.7	2.1	2.7	3.4	4.1	5.0	6.0	7.2	8.5	10.0	11.7	13.7	15.8	18.2	20.8
1200	130	63									0.2	0.3	0.4	0.6	0.8	1.0	1.3	1.7	2.1	2.6	3.2	3.9	4.7	5.6	6.7	7.8	9.2	10.7	12.3	14.2	16.3
1200	150	45									0.2	0.3	0.5	0.7	0.9	1.2	1.5	1.9	2.4	3.0	3.7	4.5	5.4	6.4	7.6	9.0	10.5	12.2	14.1	16.2	18.6
1200	150	63									0.2	0.3	0.4	0.5	0.7	0.9	1.2	1.5	1.9	2.3	2.8	3.5	4.2	5.0	5.9	7.0	8.1	9.5	10.9	12.6	14.4
1200	200	45									0.2	0.3	0.4	0.5	0.7	0.9	1.2	1.5	1.9	2.4	2.9	3.5	4.2	5.1	6.0	7.1	8.3	9.6	11.1	12.8	14.6
1200	200	63									0.14	0.2	0.29	0.4	0.53	0.7	0.91	1.16	1.46	1.81	2.22	2.7	3.25	3.88	4.6	5.42	6.33	7.37	8.52	9.8	11.2



Bending deflection Δ_b per unit load w=1 (kN/m) =

$$\Delta_b = \frac{5L^4}{384*(EI)_x} \quad \text{mm/kN/m}$$

Lightly Loaded Beams

Medium / Resedential Loaded Beams

Heavily Loaded Beams

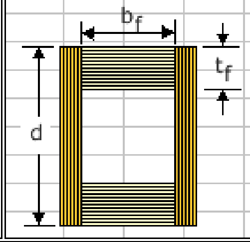
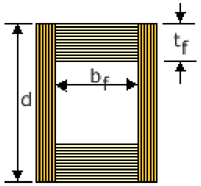


Table A8.7: Section Properties and Beam Capacities – Plywood Box Beam with Structural LVL Flanges and 7mm Thick Structural Plywood Webs

Box Beam Component	Material	Characteristic Properties: MOE MPa	MOR MPa	f'b MPa	f't Mpa	f's Mpa	f'c Mpa	Density kg/m³	Joint: Strength Group	Plywood Webs: Nom. Thick. tw mm	No. of Veneers	No. of Webs	k34
Flanges	LVL	13200	660	48	33	5.3	45	620	JD4				
Webs	F11 Plywood	10500	525	35	20	5.3	25	550	JD4	7	3	2	0.67



LVL Flanges - F11 Structural Plywood Webs									Section Properties												Beam Capacities				
	Flange			Web					About X Axis									About Y Axis			Flange Bending Capacity		Panel Shear Capacity		Flange / web connection
Depth of Section D mm	Width b_f mm	Depth t_f mm	A_f (mm^2)	Area web A_w (mm^2)	Shear area A_s mm^2	d/B ratio	Self Weight kg/m	Volume m^3/m	I_{fx} x 10^6 mm^4	I_{wx} x 10^6 mm^4	$(EI)_x$ x 10^9 Nmm^2	$(EI)_{x(n=1)}$ x 10^9 Nmm^2	Q_{fx} x 10^3 mm^3	E_fQ_{fx} x 10^9 Nmm	Q_{wx} x 10^3 mm^3	E_wQ_{wx} x 10^9 Nmm	$(EQ)_{x(n=1)}$ x 10^9 Nmm	I_{fy} x 10^6 mm^4	I_{wy} x 10^6 mm^4	$(EI)_y$ x 10^9 Nmm^2	Tension flange kN/m	Compression flange kN/m	Max kN	At web splice* kN	$E_fQ_{xf}/(EI)_x$ x 10^{-3} mm^{-1}
225	63	35	4410	3150	2660	2.9	5.1	0.008	40	9	624	578	209.5	2.8	59	3.4	3.1	1.5	2.6	46.4	13	19	13.7	7.0	4.43
225	63	45	5670	3150	2520	2.9	5.9	0.009	47	9	712	665	255.2	3.4	59	4.0	3.7	1.9	2.6	51.9	15	22	13.2	6.7	4.73
300	63	35	4410	4200	3710	3.9	5.8	0.009	78	21	1248	1138	292.2	3.9	105	5.0	4.4	1.5	3.4	55.4	19	28	18.7	9.6	3.09
300	63	45	5670	4200	3570	3.9	6.6	0.010	93	21	1450	1340	361.5	4.8	105	5.9	5.3	1.9	3.4	60.9	22	33	18.3	9.3	3.29
400	63	35	4410	5600	5110	5.2	6.7	0.010	147	50	2467	2206	402.4	5.3	187	7.3	6.3	1.5	4.6	67.4	28	42	25.2	13.0	2.15
400	63	45	5670	5600	4970	5.2	7.5	0.011	180	50	2893	2632	503.2	6.6	187	8.6	7.6	1.9	4.6	72.9	33	49	25.0	12.8	2.30
400	83	35	5810	5600	5110	4.1	7.7	0.011	194	50	3085	2823	530.2	7.0	187	9.0	8.0	3.3	7.6	123.6	35	53	25.6	13.1	2.27
400	83	45	7470	5600	4970	4.1	8.8	0.013	237	50	3646	3385	663.0	8.8	187	10.7	9.7	4.3	7.6	136.1	42	62	25.3	12.9	2.40
450	63	35	4410	6300	5810	5.8	7.1	0.011	190	71	3257	2884	457.5	6.0	236	8.5	7.3	1.5	5.2	73.5	32	49	28.4	14.7	1.85
450	63	45	5670	6300	5670	5.8	8.0	0.012	233	71	3826	3454	574.1	7.6	236	10.1	8.8	1.9	5.2	79.0	38	58	28.2	14.5	1.98
450	83	35	5810	6300	5810	4.6	8.1	0.012	251	71	4054	3682	602.8	8.0	236	10.4	9.2	3.3	8.5	133.5	41	61	28.8	14.9	1.96
450	83	45	7470	6300	5670	4.6	9.3	0.014	308	71	4804	4432	756.3	10.0	236	12.5	11.2	4.3	8.5	146.1	49	73	28.6	14.7	2.08
600	83	35	5810	8400	7910	6.2	9.5	0.014	464	168	7892	7010	820.7	10.8	420	15.2	13.0	3.3	11.4	163.3	58	90	38.4	19.9	1.37
600	83	45	7470	8400	7770	6.2	10.7	0.016	576	168	9374	8492	1036.5	13.7	420	18.1	15.9	4.3	11.4	175.9	71	107	38.4	19.8	1.46
600	130	35	9100	8400	7910	4.2	12.0	0.018	727	168	11363	10481	1285.4	17.0	420	21.4	19.2	12.8	26.3	445.3	87	129	39.4	20.3	1.49
600	130	45	11700	8400	7770	4.2	13.9	0.020	903	168	13683	12801	1623.4	21.4	420	25.8	23.6	16.5	26.3	493.6	107	155	39.3	20.1	1.57
900	130	35	9100	12600	12110	6.3	15.0	0.022	1703	567	28435	25458	1967.9	26.0	945	35.9	30.9	12.8	39.4	583.4	141	215	58.8	30.5	0.91
900	130	45	11700	12600	11970	6.3	17.1	0.024	2140	567	34204	31228	2500.9	33.0	945	42.9	38.0	16.5	39.4	631.7	173	259	59.1	30.5	0.97
900	130	63	16380	12600	11718	6.3	20.9	0.029	2870	567	43894	40860	3427.5	45.2	945	55.2	50.2	23.1	39.4	718.2	227	333	59.0	30.2	1.03
900	150	35	10500	12600	12110	5.5	16.2	0.023	1964	567	31894	28900	2270.6	30.0	945	39.9	34.9	19.7	51.8	803.8	161	242	59.3	30.7	0.94
900	150	45	13500	12600	11970	5.5	18.6	0.026	2469	567	38551	35574	2885.6	38.1	945	48.0	43.1	25.3	51.8	878.0	198	292	59.6	30.7	0.99
900	150	63	18900	12600	11718	5.5	22.9	0.032	3316	567	49731	46754	3954.8	52.2	945	62.1	57.2	35.4	51.8	1011.6	260	377	59.4	30.3	1.05
1200	130	45	11700	16800	16170	8.3	20.3	0.029	3904	1344	65645	58589	3378.4	44.6	1680	62.2	53.4	16.5	52.6	769.8	244	373	78.3	40.7	0.68
1200	130	63	16380	16800	15918	8.3	24.4	0.033	5299	1344	84063	77007	4656.0	61.5	1680	79.1	70.3	23.1	52.6	856.8	321	478	78.9	40.7	0.73
1200	150	45	13500	13800	16170	7.3	22.0	0.030	4505	1344	73573	66517	3898.7	51.5	1680	69.1	60.3	25.3	69.1	1059.3	277	418	79.0	40.9	0.70
1200	150	63	18900	16800	15918	7.3	26.7	0.036	6115	1344	94825	87769	5372.3	70.9	1680	88.6	79.7	35.4	69.1	1192.9	366	539	79.5	40.8	0.75
1200	200	45	18000	16800	16170	5.6	26.1	0.035	6006	1344	93393	86337	5197.5	68.6	1680	86.2	77.4	60.0	120.0	2052.2	360	531	80.3	41.4	0.73
1200	200	63	25200	16800	15918	5.6	32.4	0.042	8153	1344	121729	114673	7163.1	94.6	1680	112.2	103.4	84.0	120.0	2369.0	478	692	80.5	41.2	0.78

Worked Example

Flange Bending Capacity (tension flange) = $\frac{2f'_t(EI)_{xn}}{E_f\,d}$

Flange Bending Capacity (comp flange) = $\frac{2f'_c(EI)_{xn}}{E_f\,d}$

Panel Shear Capacity (Max) = $\frac{f'_s(EI)_x.n.t_w}{(EQ)_x}$

Panel Shear Capacity (at web splice) = $\frac{f'_s(EI)_{xn}.n.t_w}{(EQ)_x}$
(one web continuous only)

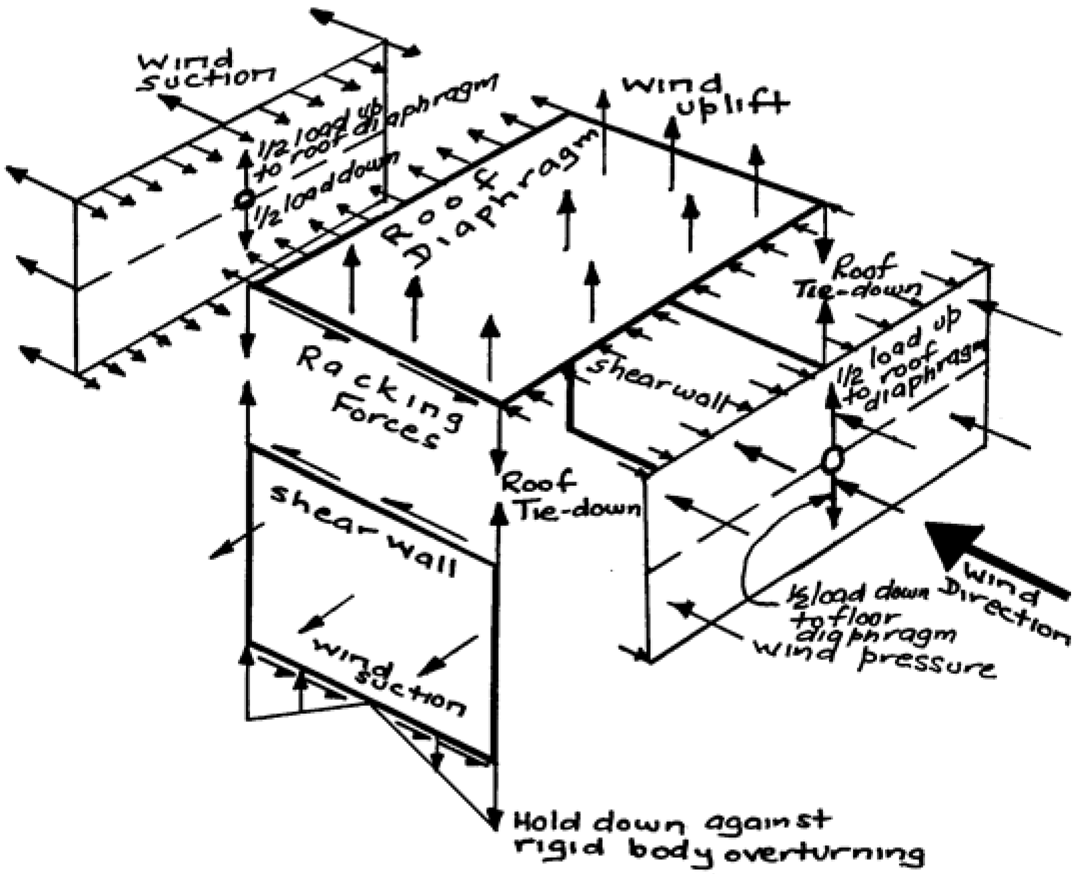
CHAPTER 9

9 STRUCTURAL PLYWOOD DIAPHRAGMS & SHEARWALLS

9.1 Introduction

Diaphragms **and** shearwalls **are engineered building elements designed to resist lateral loads. They are essentially the same type of structure except** shearwalls **are located in a** vertical **or** inclined plane **and** diaphragms **are situated in a** horizontal **or** near horizontal plane**.** Lateral loads **are loads applied horizontally to a building. The most common lateral load types are due to** high winds**,** impact **or** seismic **(earthquake) forces.**

FIGURE 9.1 shows a diagrammatic representation of a basic building subjected to lateral wind.



Wind Forces on Low-Rise Structures

FIGURE 9.1: Shows location and function of the shearwalls and diaphragm

Half the wind load normal to the windward and leeward walls is transferred to the **horizontal roof diaphragm** which in turn is then transferred to the **vertical shearwalls** and then to the **foundations.**

However, as previously stated diaphragms and shearwalls do not necessarily have to be horizontal and vertical but can take a range of orientations and forms as shown in FIGURE 9.2.

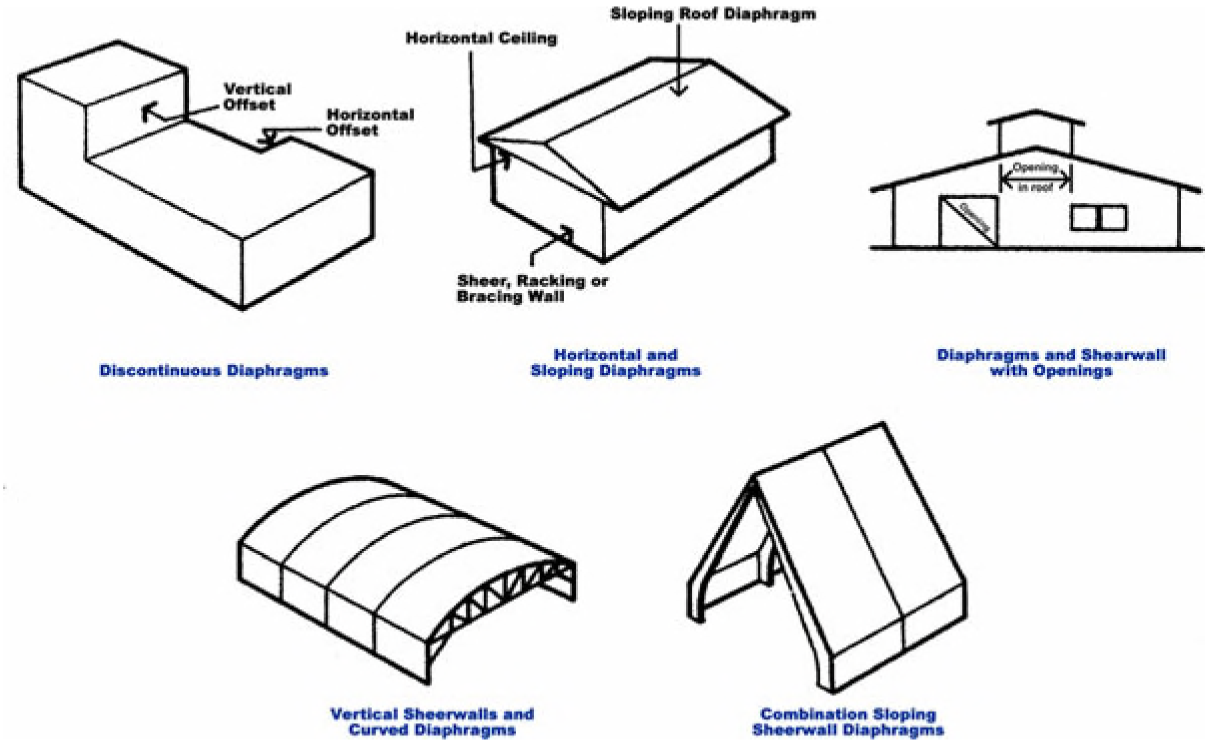


FIGURE 9.2: Shearwall and diaphragm applications

Shearwalls and diaphragms are **multifunctional** structural components, e.g:

Shearwalls may also act as a:

- **deep beam** when transferring **roof gravity loads** to ground via stumps;
- **flexural panel** when subjected to **suction** or **normal forces** due to wind loading;
- **tension panel** when required to resist **wind uplift** from the roof.

diaphragms may also act as a:

- **floor system** with loads normal to its plane;
- **structural ceiling** and/or **roof** system.

In general a **well designed shearwall** or **floor system** will perform the other functions adequately. Problems do arise when **holes** are cut in shearwalls and diaphragms and the designer has not been forewarned of this possibility.

9.2 Fundamental Relationship

Shearwalls and diaphragms are constructed by fixing plywood sheathing (of various thickness) to timber framing (of various joint strength groups). The **load transferring capabilities** of the resulting structural components becomes **dependent upon** the development of **shear flow**, i.e. **UNIT SHEARS** around the framing.

FIGURE 9.3 shows a **plywood panel nailed** to a **pin-jointed timber frame**.

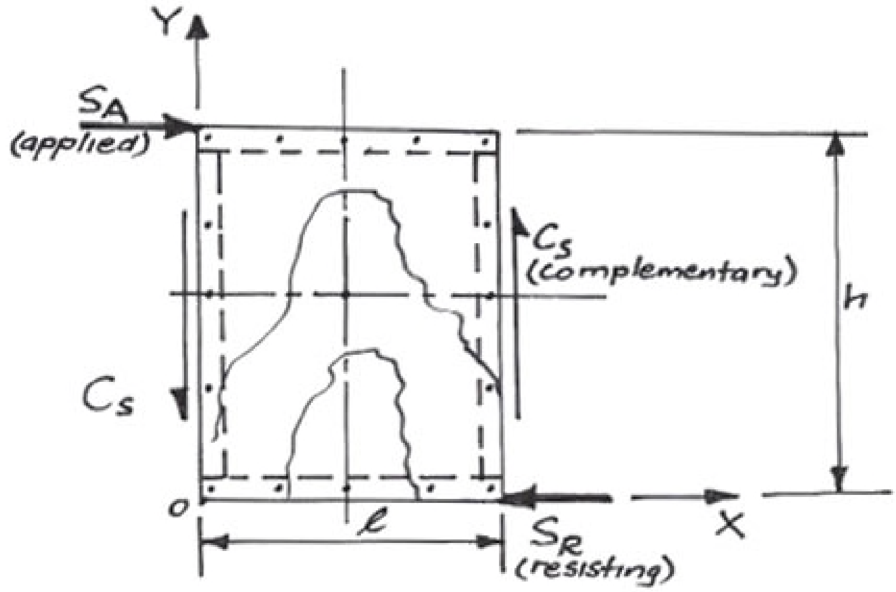


FIGURE 9.3: Shows panel subjected to shear

From statics:

$\Sigma F_x = 0 : S_A = S_R$
$\Sigma F_y = 0 : C_s = C_s$ (complimentary shears)
$\Sigma M_o = 0 = S_A.h - C_s.\ell$

$S_A / \ell = C_5 / h$

That is, the UNIT SHEAR along HORIZONTAL EDGES equals UNIT SHEAR along VERTICAL EDGES

NOTE:
The **Unit Shear** concept is of **FUNDAMENTAL IMPORTANCE** when re-distributing **SHEARS** around **OPENINGS.**

9.3 Diaphragm Design – Diaphragm Action

Diaphragm capacity will vary considerably depending on **nail frequency** and **capacity**, and whether the diaphragm is "**blocked**" or "**unblocked**". Blocking consists of lightweight framing, usually 90 x 45 timber framing, located between the joists or other primary structural supports, for the specific purpose of connecting the edges of the plywood panels. The use of blocking to connect panels at all edges facilitates shear transfer and increases diaphragm capacity. **Unblocked diaphragm** capacity is governed by **buckling** of unsupported panel edges, such that above a maximum load, increased nailing will not increase diaphragm capacity. The capacity of **blocked diaphragms is 1.5 to 2 times** the capacity of an equivalently nailed **unblocked diaphragm**. Additionally, blocked diaphragms can be designed to carry lateral loads many times greater than those for unblocked diaphragms.

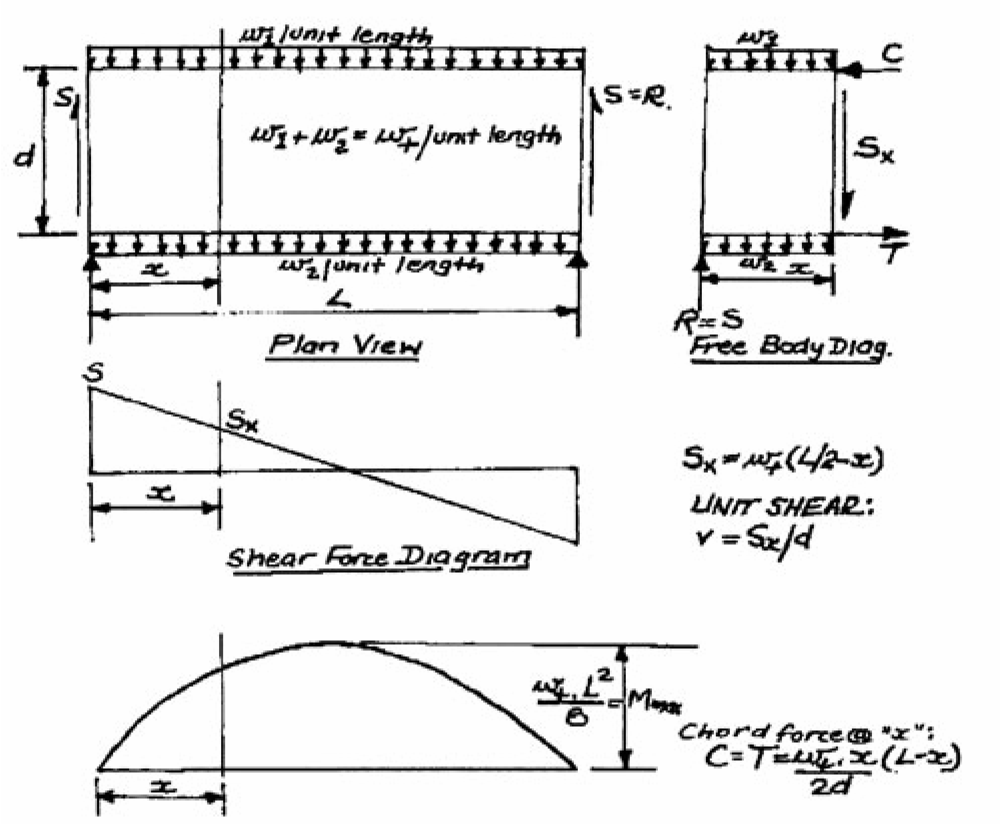
Diaphragm action **differs** from simple beam action in that **shear stresses** have been shown to be essentially **uniform** across the depth rather than displaying the **parabolic distribution** associated with shallow beam webs.

Also, the **chord members** are responsible for transfer of **bending moments,** acting in **uniaxial tension and compression.** Chord members must, however, be **continuous** over the length of the diaphragm. The advent of LVL, being available in long straight lengths has reduced the need for incorporating **spliced joints** along the chord lengths. Chord members of plywood sheathed, timber framed diaphragms are not restricted to timber members. They could also be the face of a concrete or masonry wall, a reinforced or masonry beam or a steel beam.

The recommended maximum span to depth ratio for plywood systems blocked or unblocked is 4 : 1.

A case for extreme caution exists when designing diaphragms in which rotation is possible. Such cases arise when a glass facade, for example, is located in one of the walls or the building has one end open. This situation will not be pursued further herein.

Figure 9.4 illustrates the application of the normal assumptions made in the analysis of a plywood sheathed, timber framed diaphragm.



Bending Moment Diagram

FIGURE 9.4: Diaphragm design formula for lateral loading

9.4 Diaphragm Design – Methodology

The **design method** and values presented in this Manual are **based on** the **extensive testing** conducted **by** the **Engineered Wood Panel Association** (formerly the American Plywood Association). The design method allows a conventionally framed roof, floor or wall to function as a structural diaphragm with only slight design modifications. TABLE 9.1 provides nailing and plywood thickness details for horizontal diaphragms.

Lateral loads can be applied to a building from any direction, however they can be resolved into two orthogonal force systems acting in the direction of its two primary orthogonal axes. The **worst case loading** in either of the buildings two primary directions will **govern** the **diaphragm design**.

The following are the **design steps** to be followed in the design of a **structural plywood sheathed**, **timber framed diaphragm**.

1. **Calculate the magnitude of the wind loads on the roof diaphragm in each direction as shown in** FIGURE 9.5**.**

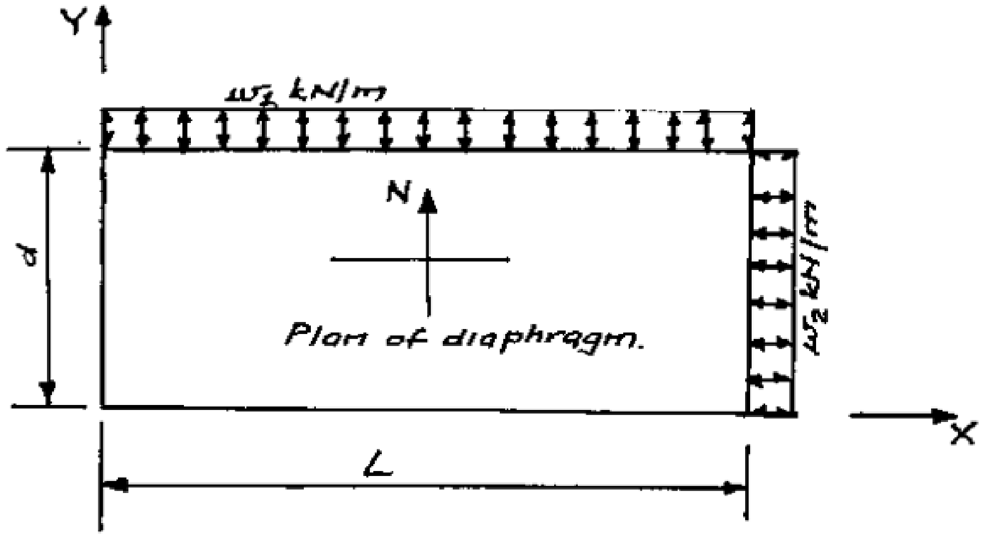


FIGURE 9.5 : Wind forces on diaphragm

2. **Determine the design UNIT SHEAR on the diaphragm in each co-ordinate direction.**
3. **Determine a suitable PLYWOOD PANEL LAYOUT and NAILING SCHEDULE from Table 9.1.**
4. **Determine diaphragm CHORD FORCES and design adequate CHORD SPLICES.**
5. **Calculate diaphragm DEFLECTION and check it against acceptable SIDE WALL DEFLECTION.**
6. **Other factors to be considered by the designer:**
7. **Diaphragm/wall interconnection** which will depend on the type of construction.
8. **Shear in shearwalls,** particularly where openings occur, requires the design of the shearwalls.
9. **Drag strut forces and connections.**
10. **Wall hold down forces and connections.**

9.5 Design Example 1 - Diaphragms

Structural Plywood Diaphragm in One Storey Buildings

Wind Loads as per AS/NZS 1170.2: 2002 Structural Design Actions, Part 2: Wind actions.
Building Location in Region B; Regional wind speed for Strength Limit State: Wind Speed 60 m/s

Given Details: The building dimensions and openings as shown in FIGURE 9.6. The exterior walls consist of timber stud wall framing with F11 structural plywood clad shearwalls and corrugated sheet metal exterior cladding. Timber framing members are minimum joint strength group JD4.

Building: 12m wide x 36m long x 5.4m high, One end wall has a 3.6 wide x 4.8 high door

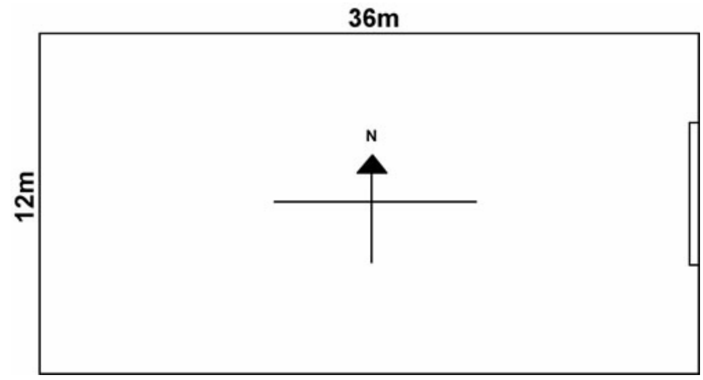


FIGURE 9.6: Plan of building

Diaphragm – Worked Example 1

1. Lateral Wind Loads on Roof Diaphragm

Design wind pressure acting normal to a surface, Pa	$P = 0.6[V_{des,0}]^2 C_{fig} C_{dyn}$	**1170.2 Cl 2.4.1**
Building orthogonal design wind speeds	$V_{,des,0} = V_{sit,\beta}$	**1170.2 Cl 2.3**
Site wind speeds	$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$	**1170.2 Cl 2.2**
Regional 3 s gust wind speed for annual probability of exceedance of 1/R.	V_R	**1170.2 Cl 3.2**
Regional wind speed for strength calculations	$V_{1000} = 60$ m/s	**1170.2 Table 3.1**
Regional wind speed for serviceability calculations	$V_{20} = 38$ m/s	**1170.2 Table 3.1**
Wind directional multiplier	$M_d = 0.95$	**1170.2 Cl 3.3**
Terrain and height multiplying factor	$M_{(5.4,2.5)} = 0.84$	**1170.2 Cl 4.2**
Shielding multiplier, Table 3.2.7, AS 1170.2	$M_s = 1$	**1170.2 Cl 4.3**
Topographic multiplier, Table 3.2.8, AS1170.2	$M_t = 1$	**1170.2 Cl 4.4**
Dynamic response factor	$C_{dyn} = 1$	**1170.2 Cl 2.4.1**
Aerodynamic shape factor	$C_{fig} =$	**1170.2 Cl 5.2**
For external pressures	$C_{fig} = C_{p,e} K_a K_c K_\ell K_p$	
For internal pressures	$C_{fig} = C_{p,i} K_c$	
External pressure co-efficient, windward wall	$C_{p,e} = 0.7$	**1170.2 Table 5.2(A)**
Internal pressure co-efficient, windward wall	$Cp_{,} = -0.65$	**1170.2 Table 5.1(B)**
Area reduction factor for roofs and side walls	$K_a = 1$	**1170.2 Cl 5.4.2**
Combination factor	$K_c = 1$	**1170.2 Cl 5.4.3**
Local pressure factor for cladding	$K_{\ell} = 1$	**1170.2 Cl 5.4.4**
Reduction factor for permeable cladding	$K_p = 1$	**1170.2 Cl 5.4.5**
Aerodynamic shape factor for external pressures	$C_{fig,ext\ v} = 0.7 \times 1 \times 1 \times 1 \times 1 = 0.7$	
Aerodynamic shape factor for internal pressures	$C_{fig,int} = 0.65 \times 1 = 0.65$	

ULTIMATE LIMIT STATES STRENGTH:

Design wind speed for ultimate limit states strength	$v_{des,\theta} = 60 \times 0.95 \times 0.84 \times 1 \times 1$ **= 47.9 m/s**
Design wind pressure for ultimate limit states strength	**P** $= 0.6 \times 47.9^2 \times 10^{-3}(0.7+0.65)$ **= 1.86kPa**
Wind force, w on diaphragm	w = 1.86 X (5.4/2)

(half of wind load on 5.4 m wall is transferred directly to foundations)	**= 5.0kN/m**
Total wind force, W on roof diaphragm, in North-South direction	$\mathbf{W_{N\text{-}S}}$ = 5.0 kN/m x 36 m **= 180kN**
Total wind force, W on roof diaphragm, in East-West direction	$\mathbf{W_{E\text{-}W}}$ = 5.0 kN/m x 12 m **= 60kN**

Unit Shear, v in the roof diaphragm in each direction:

Diaphragm design unit shear from Figure 9.4	$V = (w_t.L/2)/d$
Diaphragm design **unit shear** in North-South direction	$v_{N\text{-}S} = (5.0 \times 36/2)/12$ **= 7.5kN/m**
Diaphragm design **unit shear** in East-West direction	$v_{E\text{-}W} = (5.0 \times 12/2)/36$ **= 0.83kN/m**

LIMIT STATES SERVICEABILITY:

Design wind speed for limit states serviceability	$v_{des,0} = 38 \times 0.95 \times 0.84 \times 1 \times 1$ **= 30.3 m/s**
Design wind pressure for limit states serviceability	**P** $= 0.6 \times 30.3^2 \times 10^{-3}(0.7+0.65)$ **= 0.74kPa**
Wind force, w on diaphragm (half of wind load on 5.4 m wall is transferred directly to foundations)	$w = 0.74 \times (5.4/2)$ **= 2.0kN/m**

Unit Shear, v in the roof diaphragm in each direction:

Diaphragm design unit shear from Figure 9.4	$v = (w_t.L/2)/d$
Diaphragm design unit shear in North South direction	$V_{N\text{-}S} = (2.0 \times 36/2)/12$ **= 3.0kN/m**
Diaphragm design unit shear in East West direction	$V_{E\text{-}W} = (2.0 \times 12/2)/36$ **= 0.33kN/m**

2. Determine a Suitable Structural Plywood Panel Layout and Nailing Schedule

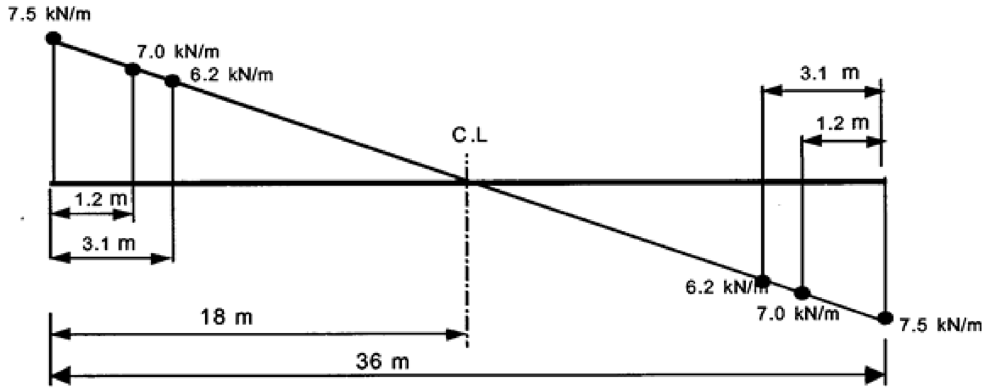
Extract from Table 9.1

Factored Limit State Shear Capacities (kN/m) x k_1 = 1.14

Plywood Thicknesses (mm)	Flathead Nail Size	Min. Nail Penetration into Framing (mm)	Frame width (mm)	Blocked Diaphragm Nail Spacing (mm) at Boundary/Other Edges				Unblocked	
				150/150	100/150	65/100	50/75	Case 1	Case 2 to 6
12	3.75mm dia x 75mm long	40	75	7.0	9.3	14.0	16.0	6.2	4.7

Shear Force Diagram for the roof diaphragm in the N-S direction

7.5 kN/m to 7 kN/m : Blocked Case 1, Nail spacing 100/150 : capacity = 9.3 kN/m
7.0 kN/m to 6.2 kN/m : Blocked Case 1, Nail spacing 150/150 : capacity = 7.0 kN/m
$\leq$6.2 kN/m : Unblocked Case 1, Nail spacing 150/300 : capacity = 6.2 kN/m



Change in Shear Locations:
$18/7.5 = (18-x_1)/7$; $x_1 = 1.2$ m
$18/7.5 = (18-x_2)/6.2$; $x_2 = 3.1$ m say 3.6 m

Shear force in the roof diaphragm in the E-W direction:
0.83 kN/m << 4.7 kN/m = > Unblocked Case 3

Converted to Limit States Capacity
Conversion factor used was 1.3 i.e. allowable shear capacities were multiplied up by 1.3

Minimum Structural Plywood Thickness (mm)	Flathead Nail Size (mm)	Minimum Nail Penetration into Framing (mm)	Minimum Nominal Width of Framing Member (mm)	Factored **Limit States** shear capacities (kN/m)					
				Blocked Diaphragms				Unblocked Diaphragms	
				Nail spacing (mm) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)				Nails spaced 150 mm maximum at supported edges	
				150	**100**	**65**	**50**	Case 1 (no Unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2,3,4 5 & 6)
				Nail spacing (mm) at other plywood panel edges (Cases 1,2,3 & 4)					
				150	**150**	**100**	**75**		
7	2.87 dia. x 50	32	50	3.5	4.7	7.1	8.0	3.1	2.4
			75	4.0	5.3	8.0	9.0	3.5	2.7
9	3.33 dia. x 65	38	50	5.1	6.8	10.1	11.4	4.6	3.4
			75	5.7	7.6	11.4	12.8	6.0	3.8
12	3.75 dia. x 75	41	50	6.1	8.1	12.1	13.9	5.4	4.1
			75	7.0	9.3	14.0	16.0	6.2	4.7

(a) Timber joint strength group shall be JD4 or better and plywood a minimum of F11
(b) Space nails 300 o.c. along intermediate framing members for roofs and 250 o.c. for floors.
(c) Framing shall be 75mm nominal or deeper, and nails shall be staggered where nails are spaced 50 mm or 65 mm o.c. and where 3.75 dia. nails having penetration into framing of more than 40 mm are spaced 75 mm o.c.
(d) Maximum joist spacing shall be 600 mm.

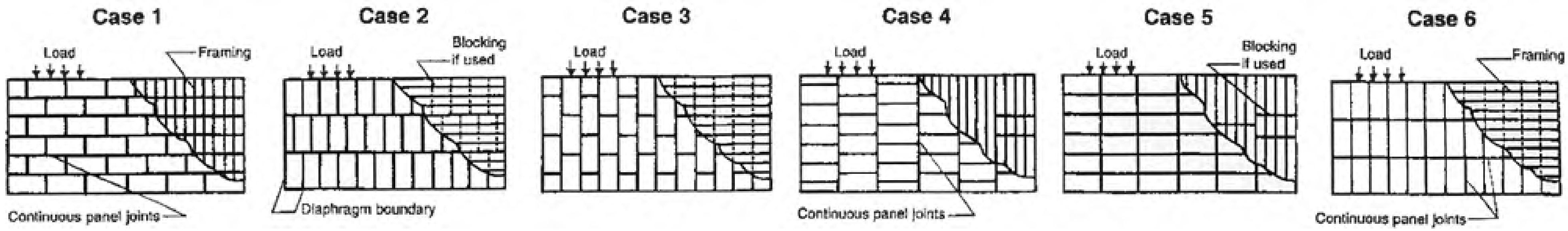
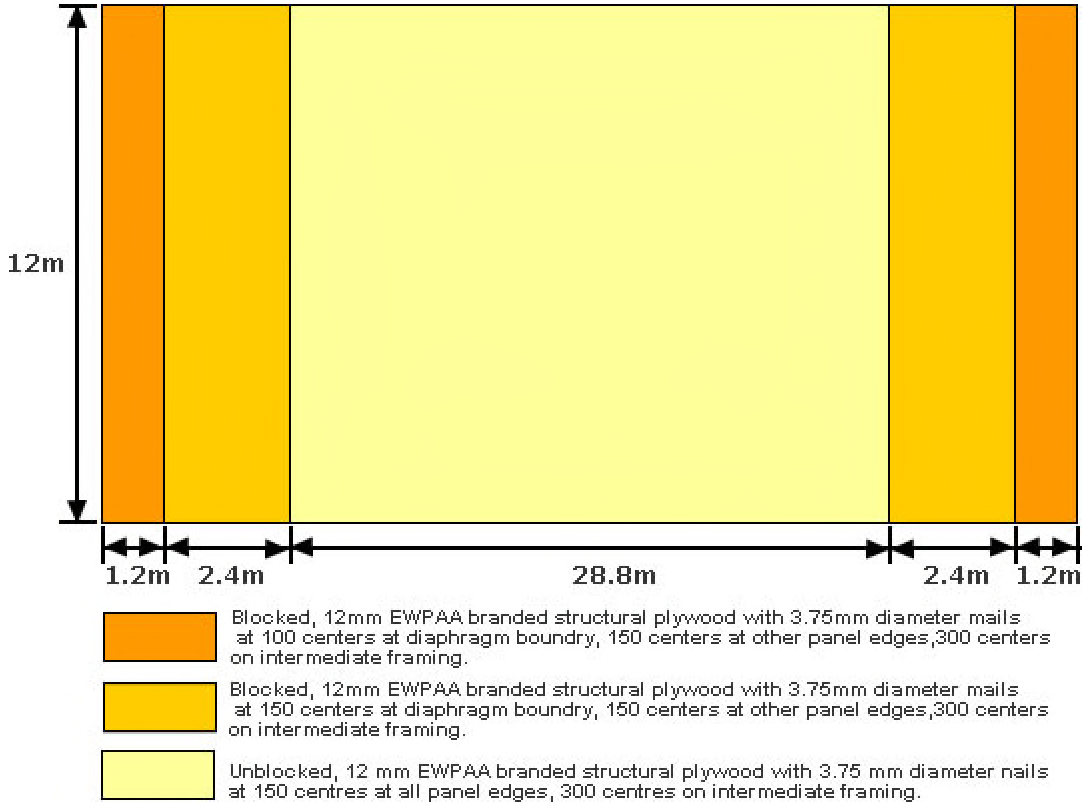
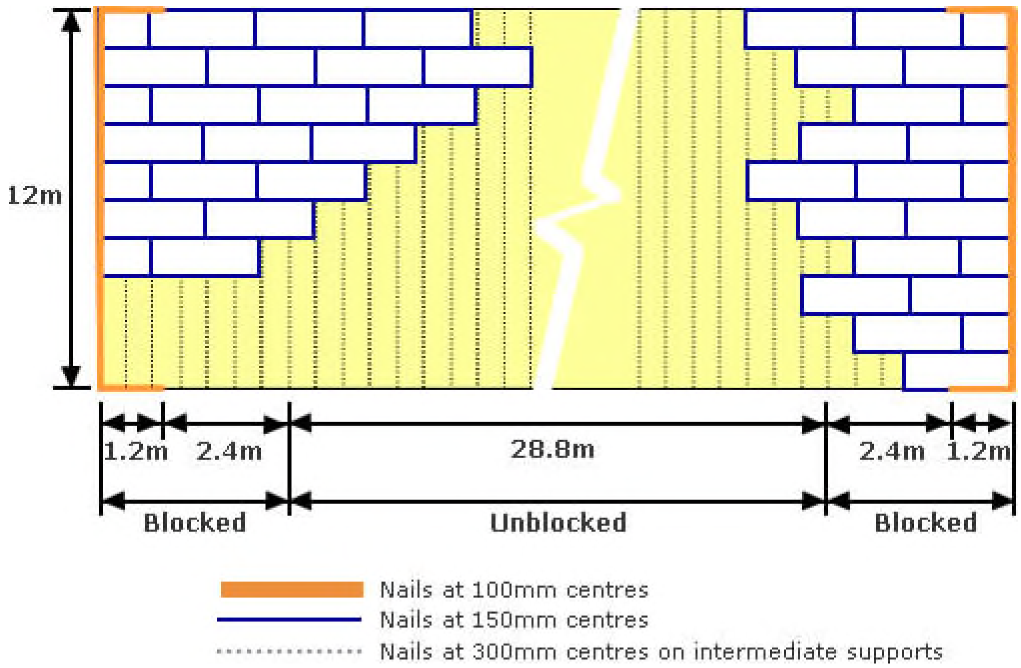


TABLE 9.1 : Shear capacities in kN/m for Horizontal Plywood Diaphragms

3. Roof Framing and Structural Plywood Diaphragm Layout

All framing members to be a minimum of JD4 joint strength group and structural plywood to be F11 x 12mm EWPAA Structural Plywood fastened with 3.75 diameter x 75 mm long flathead nails.





4. Chord Size and Splices

The chords must be continuous and therefore must be spliced for wind in the NS direction assuming LVL is available in only 12m lengths.

Two layers of 130 x 45 LVL x 12 m lengths will be used as the diaphragm chord. Assume E = 13200 MPa and $\mathbf{f_t'} = \mathbf{33MPa}$.

For high tension forces in splice joints it is usually more efficient to splice the joint with metal side plates and bolts acting in double shear. For low chord forces, splicing can often be achieved by nailing.

Moment in diaphragm, M:

Chord Force = C = T= $(w_t.x/2d)(L-x)$ **from Figure 9.4.**

Wind Direction N-S	Total Diaphragm Design Load (kN/m)	Location of Chord Splice "x" (m)	L-x (m)	Diaphragm width "d" (m)	Chord Force C or T $(w_t.x/2d)(L-x)$ (kN)
N-S	5	12	24	12	60
E-W	not applicable – no join				

Design tensile capacity required $= \phi N_t$

where:

$$\phi N_t = \phi k_1 k_4 k_6 k_{11} f_t' A_t$$

and: $\phi N_t \geq N^*_t$

⇨ $\phi N_t \geq 60$

Required $A_t \geq \phi N_t / \phi k_1 k_4 k_6 k_{11} f_t'$

$= 60 \times 10^3 / (0.85 \times 1.15 \times 1.0 \times 1.0 \times 1.0 \times 33)$

$\mathbf{A_t} = \mathbf{1860\ mm^2}$

Chord:

Effective cross-sectional area of LVL: 130 x 45, 2 layers:

2 x [130-(2 x 12)] x 45 = 9540 mm^2

>1860 **OK**

Allow for two rows of M12 bolts (Allow for hole diameter of 12 mm + 10%)

Splice:

Determine number, n of M12 bolts required each side of the joint

$\phi N_j \geq N^*$

$= \phi k_1 k_{16} k_{17} nQsk$

⇨ n $= 60 / (0.65 \times 1.3 \times 1.0 \times 1.0 \times 1.0 \times 19.2)$

n = 3.7 use 4 **bolts each side** of centre splice

Design metal splice plates for tension (at net section) compression (buckling between bolts each side of joint) and tear out.

Number of bolts may be reduced towards end of diaphragm, in proportion to moment if applicable.

5. Diaphragm Deflection

Δ *(diaphragm)*	= $\sum$(bending deflection, Δ_b + shear deflection, Δ_s + nail slip, Δ_{ns} + chord splice, Δ_c)
	= 5 v L^3/(96EAd) + v L/(4Gt) +0.188e_nL +$\sum$(Δ_cX)/(2b)
where v	= unit shear kN/m
L	= diaphragm length (m)
d	= diaphragm width (m)
A	= area of chord cross-section (mm^2)
E	= Modulus of Elasticity of the chord material (MPa)
G	= Modulus of Rigidity of the diaphragm material (MPa)
t	= Effective plywood thickness for shear (mm)
e_n	= nail deformation (mm) from TABLE 9.2 at calculated load per nail on perimeter of interior panels, based on shear per meter divided by number of nails per meter. If the nailing is not the same in both directions, use the greater spacing.
$\sum$(ΔcX)	= sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by the distance (v) of the splices to the nearest support.
Δ_c	= Half the allowable hole tolerance in excess of the bolt diameter M12 bolts : Permitted hole tolerance of + 10% of bolt diameter **AS1720 cl 4.4.1**

Diaphragm deflection in the N-S direction:

v	= 7.5 kN/m
L	= 36 m
b	= 12 m
A	= 11700 mm^2
E	= 13200 MPa
G	= 660 MPa
t	= 7.2 mm
$V_{(3.75)}$	= 7.5/8 = 0.94 kN/nail (8 nails per metre)
e_n	= 1.194 mm **Table 9.2**
Δ_c	= 0.6 mm
M12 bolts	=> 10% of 12 mm = 1.2mm Half of this = 0.6mm
X	= 12 m

Δ (diaphragm) = 5 v L^3/(96EAd) + v L/(4Gt) + 0.188 e_nL + $\sum$(ΔcX)/(2d)

Δ (bending) = 5 v L^3/(96 EAd)
= (5 x 7.5 x 36000^3)/(96 x 13200 x 11700 x 12000)
= 9.8 mm

Δ (shear) = v L/(4Gt)
= (7.5 x 36000)/(4 x 660 x 7.2)
= **14.2 mm**

Δ (nail slip) = 0.188 e_n L
= 0.188.1.194.36
= **8.1 mm**

Δ (chord splice) = $\sum$(ΔcX)/(2d)
= (4 x 0.6mm x 12m)/(2 x 12)
= **1.2 mm**

Δ (diaphragm) = 9.8 + 14.2 + 8.1 + 1.2
= **33.3 mm**

Load/Nail (N)	Nail Deformation (mm)		
	2.87	3.3	3.76
267	0.305	0.203	0.152
356	0.508	0.305	0.254
445	0.762	0.457	0.330
534	1.143	0.584	0.457
623	1.723	0.787	0.584
712	2.590	1.041	0.737
800	-	1.422	0.940
890	-	1.880	1.194
979	-	2.438	1.524
1068	-	-	1.778

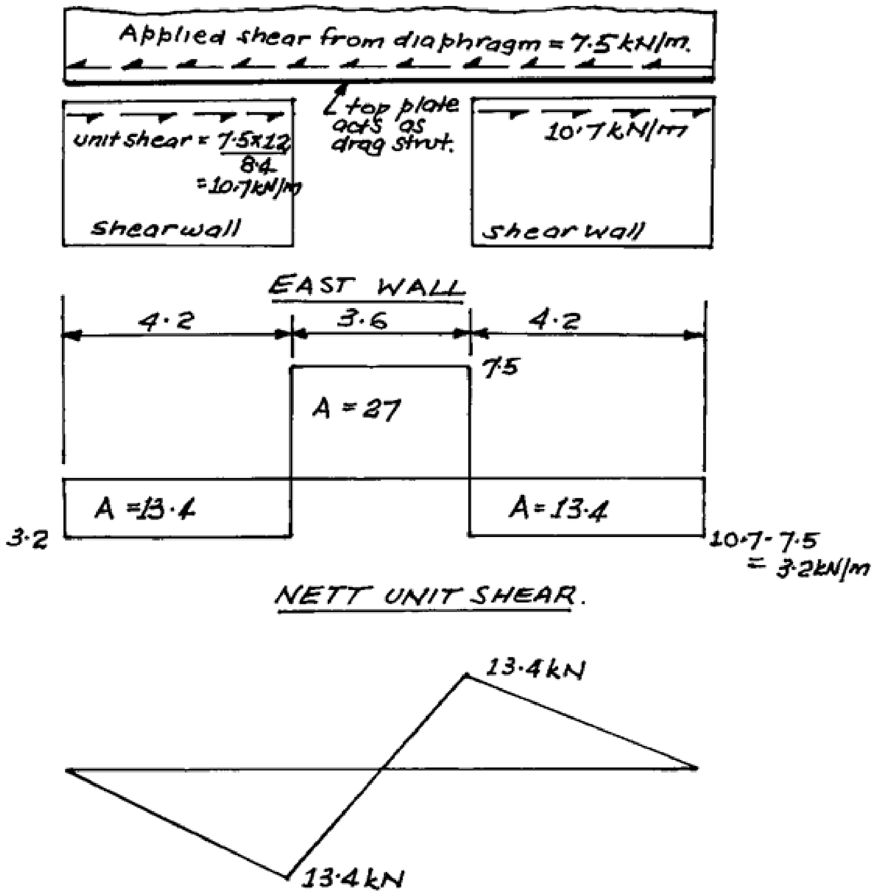
- Load/nail = (maximum shear per meter) / (number of nails per meter at interior panel edges).
- Decrease value 50% for unseasoned timber

TABLE 9.2 : e_n values (mm) for calculating nail slip in diaphragms

Drag Strut Forces

Drag struts are required over openings in shearwalls to **redistribute shear forces from** the **diaphragm to** the **shear wall.**

FIGURE 9.7 shows how the presence of an opening results in a build of force in the drag strut which, with no opening, would be equal and opposite.



Axial Force in Drag Strut

FIGURE 9.7: Build-up of axial force in drag strut due to opening

Since the drag force (13.4 kN) is much less than the splice force (60 kN) there is no need for any modifications.

9.6 Diaphragm Variations

As shown in FIGURE 9.2 diaphragms may have:

- **openings** which may be large or relatively small;
- discontinuities resulting in **horizontal and vertical offsets.**

Worked examples will be presented illustrating methods of dealing with the above contingencies.

9.7 Design Example 2 – Diaphragms - Openings

Diaphragm shear capacity is 3.5 kN/m.
Computed forces due to the opening are additive to the basic shears.
The opening is relatively small compared to the overall dimensions of the diaphragm, i.e. 1.2 x 2.4 m opening located at the centre of the diaphragm.
Overall diaphragm dimensions are 6 x 12 m.
Total wind load applied across the opening is 1.5 kN/m.

Diaphragm – Worked Example 2

Assume the distribution of shear above and below the opening is proportional to the depth of diaphragm resisting the load. That is:

$$\frac{\mathbf{6}}{\mathbf{4.8}}\mathbf{x\,1.5} = \mathbf{1.9\ kN/m}$$

FIGURE 9.8 shows the distribution of shears around the opening due to the applied wind loading.

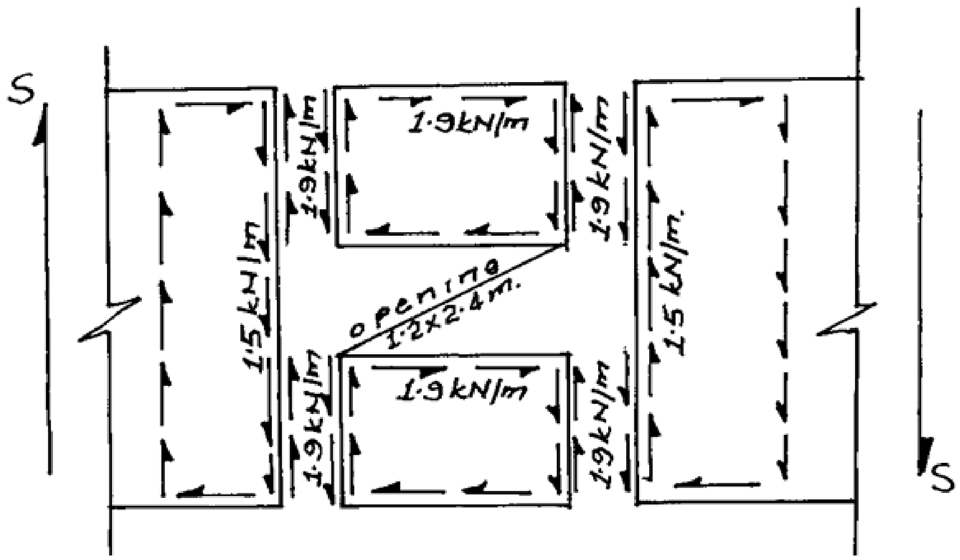


FIGURE 9.8: Distribution of shears around opening

From the distribution of shears shown in FIGURE 9.8 :

- due to the discontinuity created at the **right side of the opening** a force of **1.2 x 1.5 = 1.8 kN** cannot be directly transferred;
- hence, a **collector member** must extend far enough above and below the opening to **introduce 0.9 kN** into the sheathing;
- a **similar member** is required on the **left side** of the opening.
- due to the **fundamental relationship**, because of the development of the vertical shears of 1.9 kN/m a **horizontal shear** along the edge of the opening is required for segment equilibrium.

- The **horizontal force** is **1.9 x 2.4 = 4.6 kN**

- This 4.6 kN can be:

  - o **distributed equally either side** of the opening as a **lower bound;**
  - o **totally** to one side, say the **left side** as an **upper bound.**

- Since the **shear capacity** in the diaphragm is **3.5 kN/m** then:

  - o for the **vertical member** the distance it must extend above and below the hole is:

$$\frac{\mathbf{0.9}}{\mathbf{3.5 - 1.9}} = \mathbf{0.56\,m}$$

  - o for the **horizontal member** assuming the **lower bound** the distance is:

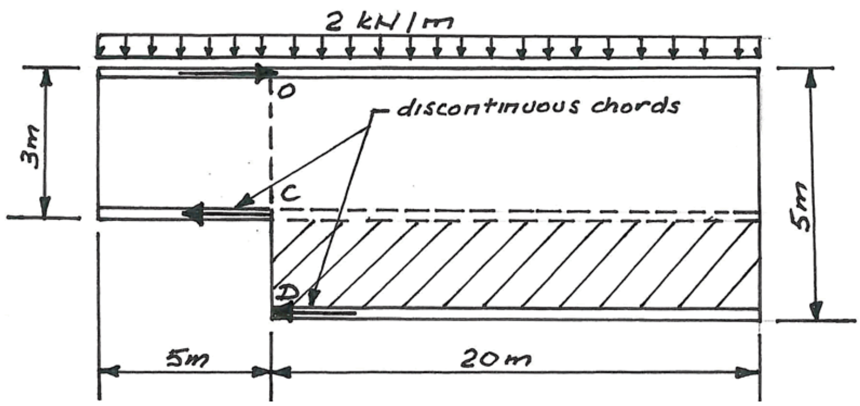
$$\frac{\mathbf{4.6}}{\mathbf{2}} \bullet \frac{\mathbf{1}}{\mathbf{3.5 - 1.5}} = \mathbf{1.15\,m}$$

These lengths are not considered to be excessive and could be further reduced if a **higher diaphragm shear** capacity was chosen.

However, should the distances required to develop the forces due to the opening become excessive, not only do the **shears** have to be redistributed but also **axial forces** due to **member bending** have to be included.

9.8 Design Example 3 - Diaphragms Horizontal Offsets

FIGURE 9.9 shows a diaphragm with a horizontal offset and discontinuous chord members. Diaphragm loading is 2 kN/m.



Diaphragm Plan

FIGURE 9.9: Diaphragm with horizontal offset

Diaphragms – Worked Example 3

Possible solutions:

- if permissible, provide **bracing under**, along the **line OCD**. This will result in the diaphragm being able to be considered as **two simply supported beams** thus **eliminating the discontinuity;**

- another possibility is related to the **axial force** developed **in** the **chord of** the **smaller diaphragm**. One approach is to determine the **fixed end moment at** the **discontinuity** and **divide** this by the **depth** of the **shallowe**r diaphragm to give the **force** to be **absorbed.**

- **ignore the hatched portion** of FIGURE 9.9 and consider the shallow section of **depth 3 m** as being effective;

- treat the diaphragm as a **notched beam** which will be considered in detail in this example.

Notched Beam Solution

Requires determining the effect of the offset on the **distribution of shears** throughout the diaphragm. Taking this approach requires the **absorption** of the **chord force F_{ch}** into the **sub-diaphragm** shown hatched in FIGURE 9.10.

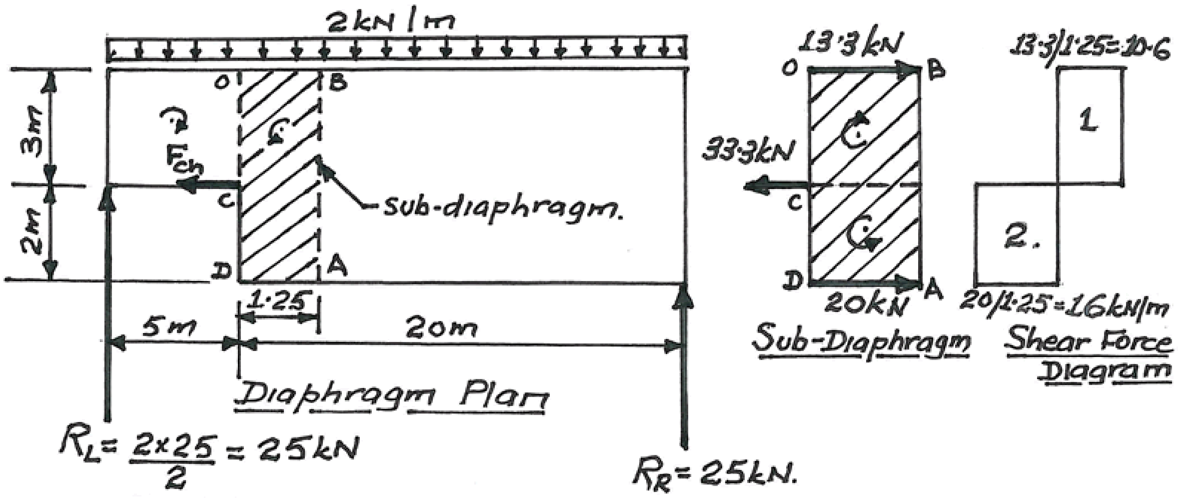


FIGURE 9.10: Shows sub-diaphragm in offset diaphragm

Sub-diaphragm shears:

These result from the chord force F_{ch} and are determined by considering the **free body diagram** to the **left of OCD** in FIGURE 9.10.

$$\Sigma M_{ol} = 0$$

$$= 25 \times 5 + F_{ch} \times 3 - 2 \times 5 \times \frac{5}{2}$$

$$F_{ch} = (125 - 25)/3$$

$$F_{ch} = 33.3\,kN$$

Consider the **isolated sub-diaphragm** of FIGURE 9.10 and taking moments about A:

$$\Sigma M_A = 0$$

$$= R_B \times 5 - 33.3 \times 2$$

$$R_B = (33.3 \times 2)/5$$

$$R_B = 13.3kN$$

$$R_A = 20\,kN$$

The resulting **shear flows** within the sub-diaphragm become:

$\mathbf{v_1}$ = 13.3/1.25
= **10.6 kN/m;**

$\mathbf{v_2}$ = 20/1.25
= **16 kN/m**

Shear flows at the discontinuity due to the **actual loading** will be:

S_{ol} = 25 – 5 x 2
S_{ol} = 15 kN S_{or} = 15 kN
$\mathbf{v_{ol}}$ = 15/3
= **5 kN/m** $\mathbf{v_{or}}$ = 15/5 = **3 kN/m**

FIGURE 9.11 **summaries** the **shear flows** and shows the result of **superposing** the two effects.

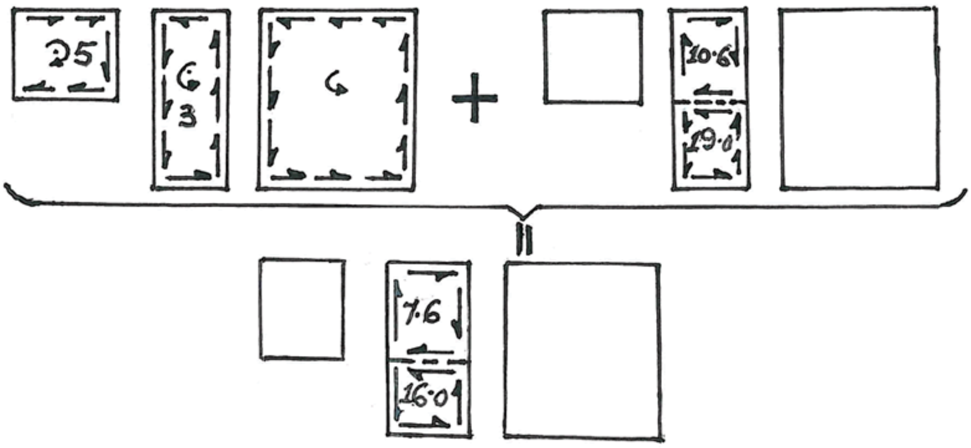


FIGURE 9.11: Shows the effect of Superposing Shear Flows

A suitable **nailing schedule** can now be chosen.

9.9 Vertical Offsets

Diaphragms - Worked Example 4

The first diagram in FIGURE 9.2 shows a diaphragm with a **vertical offset**. Evidently there are different design loads applied to the two diaphragms.

The obvious deficiency in such a structural configuration is the **lack of continuity of the chord members,** a fundamental requirement for the satisfactory functioning of a diaphragm.

Solutions to the problem do exist including:

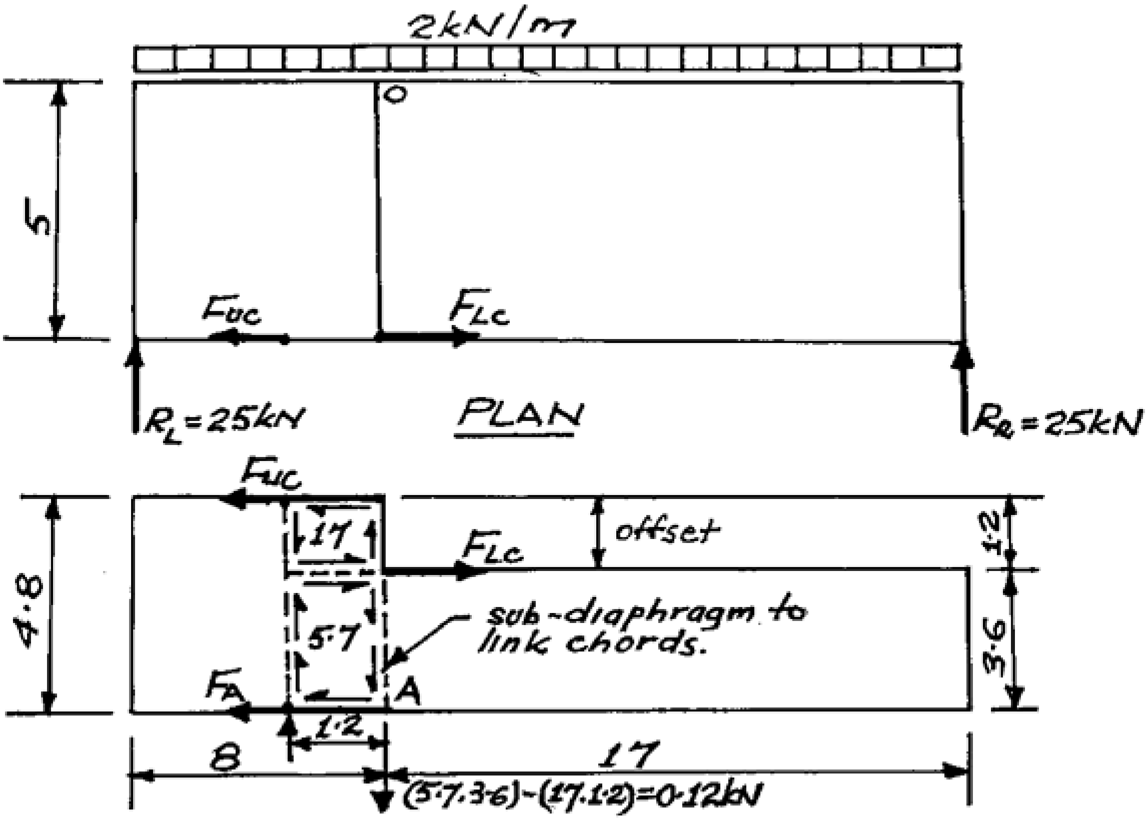
- providing a **vertical bracing element** at each level. This could be a solid wall, however **windows** in this region would preclude this possibility;
- incorporating a **rigid frame** of some type;
- use of **diagonal bracing**

If none of the above provide a satisfactory solution an **alternative is sought.** Such a solution requires to **effectively splice** the **two chord members** by utilising the **plywood wall sheathing** to do so.

Lower diaphragm chord forces : From the free body diagram to the right of the offset:

$$\begin{aligned}
\Sigma M_{OR} &= 25 \times 17 + F_{Lc} \times 5 - 2 \times 17 \times \frac{17}{2} \\
&= 0 \\
F_{Lc} &= \frac{136}{5} \\
&= 27.2\,\mathrm{kN}
\end{aligned}$$

Sub-diagram forces : From the elevation shown in FIGURE 9.12:



ELEVATION

FIGURE 9.12: Shows shear flows and chord forces

$$\sum M_A = 27.2 \times 3.6 - F_{UC} \times 4.8 = 0$$

$$F_{UC} = \frac{27.2 \times 3.6}{4.8}$$

$$= 20.4\,kN$$

$$\sum F_X = 27.2 - 20.4 - F_A$$

$$= 0$$

$$F_A = 6.8kN$$

Resulting shear flows within sub-diaphragm :

upper section : v_u = 20.4/1.2
= **17 kN/m**

lower section : v_L = 6.8/1.2
= **5.7 kN/m**

Although the **anchorage force** at A, directly under the vertical offset is small in this instance (0.12kN) it will be increased by :**(25 – 8 x 2)x1.2/5 = 2.2 kN** i.e. the **9 kN shear force** acting on the 1.2 x 5 m offset along the **line OB.** This, of course, excludes any **restoring influence** due to the weight of the offset wall.

9.10 Shearwall Design - Panel Response

In general a shearwall is a **cantilever like** structure which is required to resist **two components of load** due to the application of a **lateral force**, i.e.:

- **rigid body overturning;**
- a **pure shear** load.

Of course the above definition excludes the possibility of **bending** of the wall due to **lateral wind forces.**

For a **nailed only plywood sheathed, timber framed shearwall** these force components result in the **deformations** shown in FIGURE 9.13.

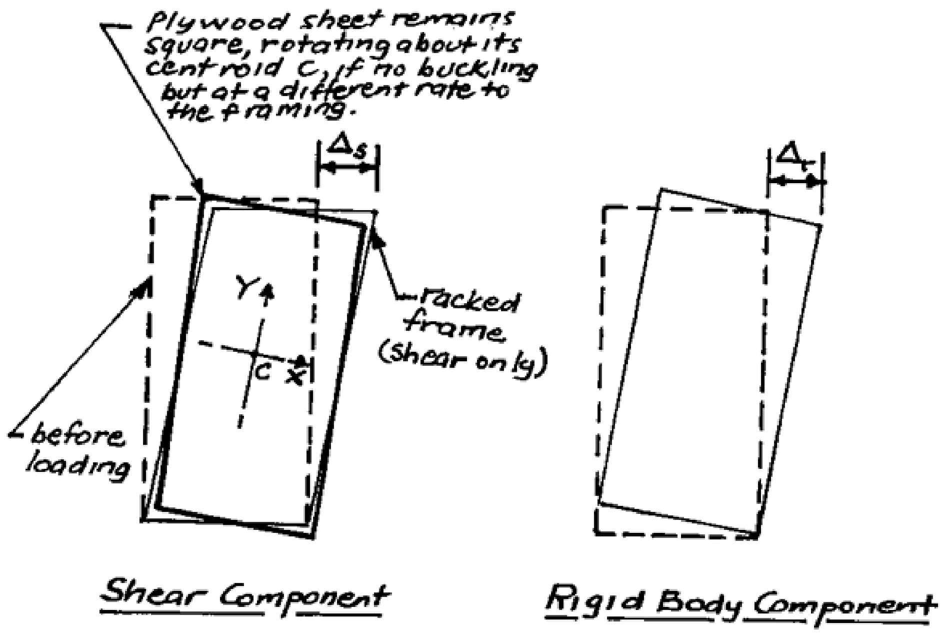


FIGURE 9.13: The two major racking deflection components

In the **nailed only bracing system** it is **nail group stiffness** (or lack thereof) which dominates panel response. This is opposed to **in-plane sheathing torsional rigidity (GJ)** as expected in the shear component diagram of FIGURE 9.13 or the **flexural rigidity (EI)** of the sheathing which is of significance when **buckling** becomes an issue.

The **nailed only** system also offers certain advantages, e.g.:

- **failure** is **gentle** resulting in fairly large deformations compared to the **rapid catastrophic failure** associated with **glued** joints;

- offers the possibility of designing a **plastic moment joint** and its associated advantages where seismic loads occur.

The **classical stiffness relationship** for a **nailed only** plywood sheathed, timber framed bracing panel is given by:

$$\mathbf{k} = \frac{c}{h^2}\left[\frac{I_x I_y}{I_x + I_y}\right] \qquad \textbf{(9.1)}$$

where:

h = **height** of the **bracing panel**;

I_x = **second moment of area** of the nail group about the X-axis, see FIGURE 9.13;

I_y = **second moment of area** of the nail group about the Y-axis;

$\frac{I_x I_y}{I_x + I_y}$ **is** equivalent **to the** second moment of area (I) **for the panel and can be seen to be entirely dependent upon** nailing density**;**

c = takes into account **material aspects** and is equivalent to the **modulus of elasticity (E).**

As previously mentioned the **shearwall** was **described** as **cantilever like** but not as a cantilever per se. To further emphasize this point compare the **classical stiffness relationship** of Equation 9.1 with the **stiffness** of a **simple cantilever beam** loaded in **flexure**, i.e.

$$\mathbf{k} = \frac{\mathbf{3\,EI}}{\mathbf{h^3}}$$

where:

E is **modulus of elasticity** of the **plywood sheathing**;
I is the **second moment of area** of the **sheathing**;
h is the **height** of the **bracing panel.**

Rigid body overturning tendencies contribute significantly to the forces required to be resisted by the first (6) or so nails along the bottom plate at the loaded end of a bracing panel.

Incorporation of **anti-rotation rods** at panel ends eliminates the need for **any nails** having to accommodate **overturning** forces, making their **full capacity** available for **shear transfer.** This is evident in viewing the bracing capacities of the EWPAA wall panels given in **Tables 6 and 8** of the Structural Plywood Wall Bracing Limit States Design Manual. **Nailed only** has a capacity of **3.4 kN/m** and **with** anti-rotation **rod** fitted the capacity is 6.4 kN/m.

It should be noted the **EWPAA Racking Test Procedure** does not incorporate the application of any simulated gravity load from the roof to the top plate. This is not the case for other test procedures, e.g. the American Society for Testing Materials. Reasoning behind the EWPAA testing protocol was that **lightweight roofs** offered **little resistance** to **wind uplift.**

9.11 Shearwall Design - Methodology

Generally the design process is straightforward. The steps involved require:

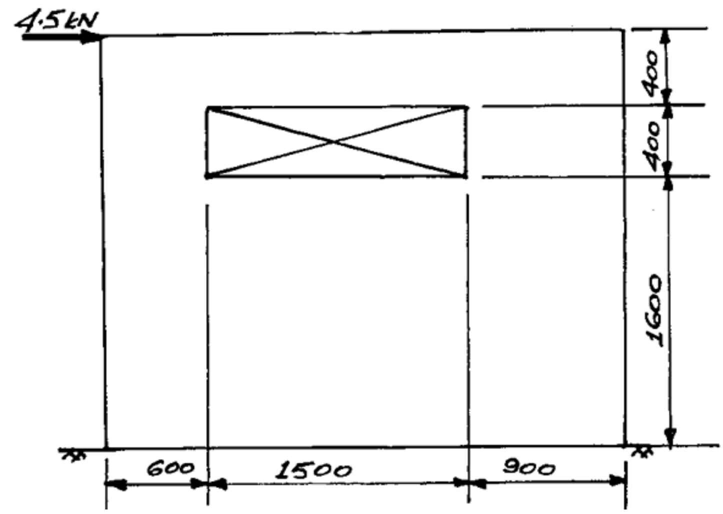
- determining the **diaphragm reactions** to be transferred to the shearwalls;
- determining the **unit shear** to be transferred by the shearwalls;
- choosing a suitable **structural plywood panel layout and fastener schedule** e.g., as per the EWPAA Structural Plywood Wall Bracing Limit States Design Manual. Panel layouts for **single wall heights** are usually arranged with **plywood face grain parallel** to the **studs.** The alternative, with no penalty in shear capacity is for the **face grain** to be **perpendicular** to the **studs**;
- decide if the **structural configuration** will allow **advantage** to be taken of:
    - o location of **return walls**,
    - o influence of **first floor construction** on ground floor bracing response, i.e. **gravity loads** reducing overturning tendencies.
- ensure an **efficient distribution** of **shearwalls**, i.e. locate panels in **corners** if at all possible and distribute them as evenly as possible throughout the building. Doing this will combat any tendency towards **diaphragm rotation;**
- assess the effect any **openings** may have on bracing response.

Since the shearwalls **without openings** present no real design challenges the example will consider a shearwall with an opening.

9.12 Design Example 1 - Shearwalls

Figure 9.14 shows a shearwall subjected to a racking load of 4.5kN. The wall has a window opening of 400 x 1500 located as shown.

The **initial solution** will follow the usual approach, i.e. by **discretisation** of the **panels either side of** the **opening.**



ELEVATION

FIGURE 9.14: Shearwall with opening

Shearwalls Worked Example 1 - Accepted Solution

Considers the 600 and 900 lengths of shearwall to act independently of each other. The racking load per panel being in the ratio of their width, i.e. the 600 panel would take 600/1500 of the 4.5 kN **(1.8kN)** and the 900 panel would take 900/1500 of 4.5 kN **(2.7 kN).**

Tie-down at the ends of the panels (can be loaded in either direction) is:

$$\frac{1.8 \times 2.4}{0.6} = 7.6\text{kN}; \quad \frac{2.7 \times 2.4}{0.9} = 7.6\text{kN}$$

The **unit shear** to be resisted in each panel is **1.8 / 0.6 = 3 kN/m and 2.7/0.9 = 3 kN/m**. However, the **uplift** at the panel end is **7.6 kN**

For the 600 panel to attain 3 kN/m would require it to be fitted with coach screws and washers at its four corners as per Table 9 EWPAA Wall Bracing Limit State Design Manual.

Shearwalls – Worked Example 1 - Alternative Solution:

An alternative approach takes into account the contribution made by the **panel under** the **window.** FIGURE 9.15 shows the free body diagram of the shearwall, **neglecting** the contribution of the section **above the window.**

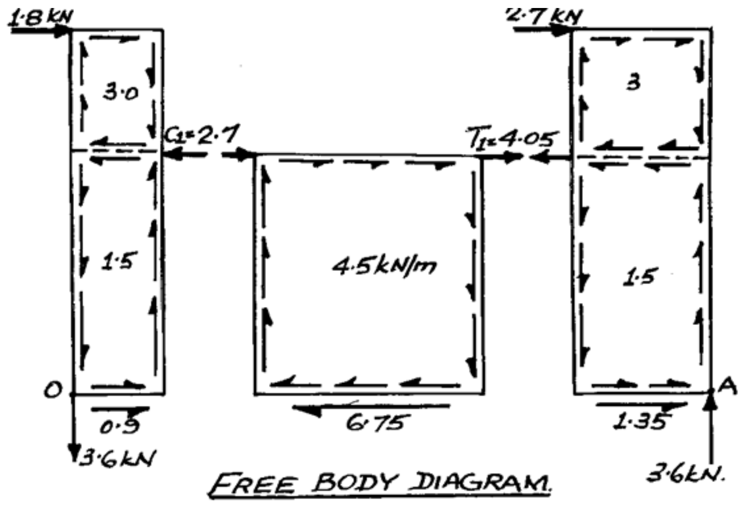


FIGURE 9.15: Free body diagram of shearwall

Assuming the applied racking load to be distributed in proportion to the panel widths as shown in FIGURE 9.15 then for the section to the **left** of the **opening:**

$$\Sigma M_O = 1.8 \text{x} 2.4 - C_1 \text{x} 1.6$$
$$= 0$$
$$C_1 = \frac{1.8 \text{ x } 2.4}{1.6}$$
$$C_1 = 2.7 \text{kN}$$

For the section to the **right of the opening:**

$$\Sigma M_A = 2.7 \text{x } 24 - T_1 \text{x} 1.6$$
$$T_1 = \frac{2.7 \text{x } 2.4}{1.6}$$
$$T_1 = 4.05 \text{kN}$$

The **shear flows** in the various sections of the shearwall are as shown in **FIGURE 9.15.**

The unit shears vary, the highest being **4.5 kN/m** which is **significantly larger** than the **3 kN/m** but **uplift** at **3.6 kN** is **significantly less than 7.6 kN.**

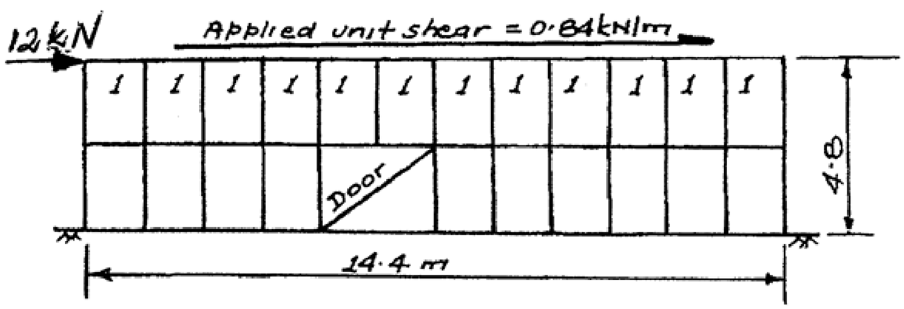
> **NOTE:**
> Should it be considered the section above the opening to be a significant contributor to panel response the analytical difficulties are increased significantly. The situation becomes analogous to that of the large opening in a diaphragm.

9.13 Design Example 2 - Shearwalls

FIGURE 9.16 shows a shearwall subjected to a racking load of 12kN applied at top plate level. The resultant **unit shear** is **0.84 kN/m**.

It is required to assess the distribution of **timber framing forces and panel shears** due to the inclusion of the **door opening** in the shearwall.

The method of analysis chosen is the **Shear Transfer** method due to Dean et al.



ELEVATION

FIGURE 9.16: Loaded shearwall with door opening location

Shearwalls - Worked Example 2

FIGURE 9.17 shows a free body diagram of the sections of shearwall adjacent to the door opening.

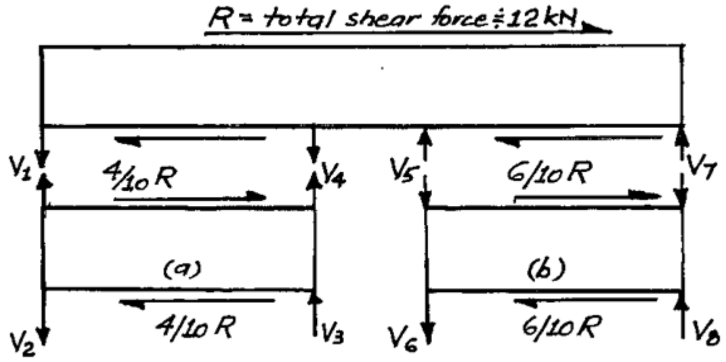


FIGURE 9.17: Free body diagram

The free body diagram shows there are:

8 unknown reactions

However, there are only **3 equations of statics.** The problem becomes solvable because it is possible to **distribute the shears** due to the **well behaved response** of the **nailed sheathing**.

As done previously:

i. when **length a ≈ length b** the **shears** can be **distributed** in the **ratio** of **panel lengths**.

ii. when **length a << length b** the **shears** can be **distributed** according to the **relative stiffnesses of the wall sections.** If sheathing **varies** use **EI's.**

NOTE: The **distributed shears of** (i) and (ii):

- **must satisfy equilibrium**;
- can be resisted by adjusting the **nailing density of the sheathed panels**;
- must result in the **axial forces in the framing members** being **in equilibrium** with **nail forces** transferred from the sheathing;
- **Finite Element Analysis** shows the procedure to be legitimate

applied unit shear :

$$v_u = \frac{12}{14.4}$$

$$v_u = \mathbf{0.84\,kN/m}$$

horizontal panel forces = p_h
= (0.84 x 1.2) for 1200 wide panels
= **1 kN**

distributing shears according to(i)

$$S_L = \frac{4}{10} \times 12$$

$$S_L = \mathbf{4.8\,kN}$$

$$S_R = \frac{6}{10} \times 12$$

$$S_R = \mathbf{7.2\,kN}$$

allocation of panel shears, adjacent to opening such that:

$\sum p_h$ to left ≈ 4.8 kN (actually (2 x 1.5) + (2 x 1) = 5 kN)
$\sum p_h$ to right ≈ 7.2 kN (actually 8 kN)

In this instance **let the two panels either side** of the opening have:

$$p_h = 1.5 \text{ kN}$$

NOTE: The values of p_n can be of **any magnitude** usually greater than 1.

The above choice of **horizontal panel force** results in the **accumulation of nail force** in the **timber framing members** shown in FIGURE 9.18.

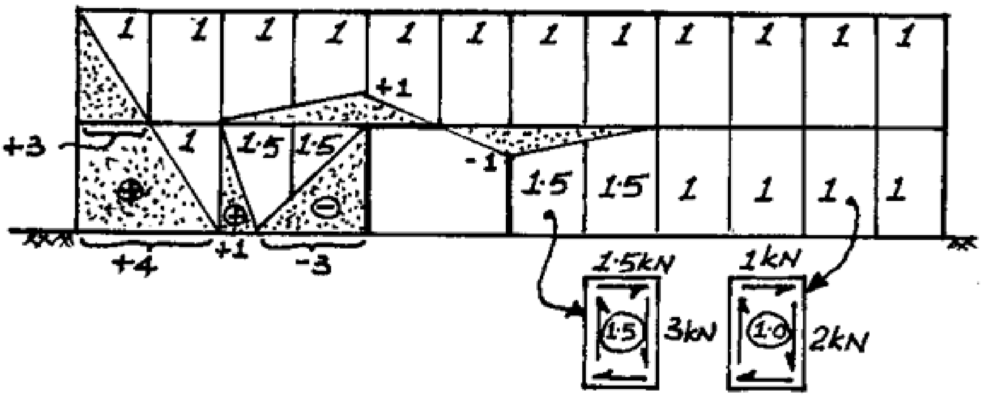


FIGURE 9.18: Accumulation of nail forces

Accumulation of nail forces in the **vertical and horizontal framing members** is demonstrated by referring to FIGURE 9.18. To do this consider the **two 1.5 kN and one 1 kN panels** to the **left** of the door opening.

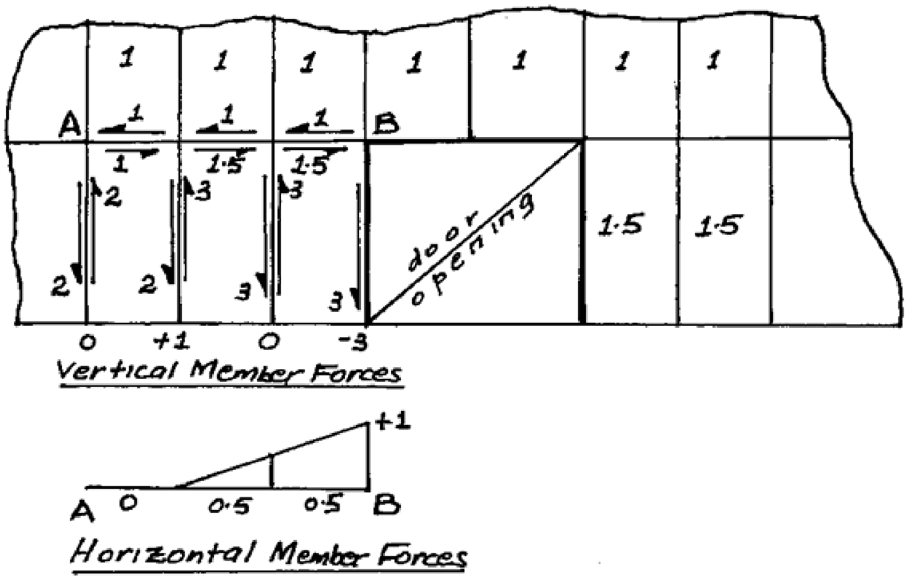


FIGURE 9.19: Shows panel shear flows and resulting nail force accumulation

Summation of the nail forces along the vertical members allows V_2, V_3, etc. to be evaluated as shown in FIGURE 9.17.

9.14 Photographs

Appendix A1.9 illustrates some practical examples of diaphragms and shear walls.

Plate 1

Plate 2

Plate 3

Plate 4

Plate 5

Plate 6

Plate 7

Plate 8

Plate 9

Plate 10

Plate 11

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CHAPTER 10

10 STRUCTURAL PLYWOOD / LVL GUSSETED TIMBER PORTAL FRAMES

10.1 Introduction

Portals can be of **rigid, two** or **three hinged** construction as shown in FIGURE 10.1. The **rigid portal** minimises column / rafter cross-sectional dimensions but provides challenges regarding the development of **full moment resistance** at the **column base**. The **three hinged portal** results in maximum column / rafter dimensions and **maximum bending moment** having to be resisted at the portal **eaves** (haunch or knee) **join**t. The **two hinged portal** provides a suitable structural compromise thus **eliminating** the **column base connection problem** and the member oversizing by incorporation of a **ridge moment joint**.

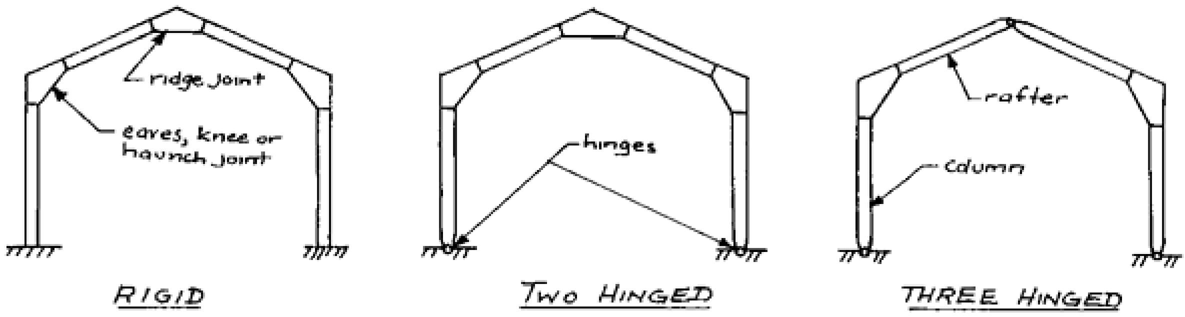


FIGURE 10.1: Common single storey, single spanning portal frames

The plywood or LVL gusseted timber portal, in its multitude of structural forms, provides an excellent solution to a wide range of building requirements. **Plywood** or **LVL gussets** nailed to the framing elements at the eaves and ridge of the portal frame are **economical**, **easy to fabricate** and provide an **effective method** of **developing moment resisting joints** at these locations. The two main design components for structural plywood or LVL gussets are:

- sizing of the **structural plywood / LVL gussets;**
- design of the **nailed connection** to transmit the applied **column** / **rafter forces** developed at the portal **eaves** and the **rafter / rafter moment** joint at the **ridge**.

10.2 Materials

Portal frame **gussets** can be fabricated from **structural plywood** or **structural Laminated Veneer Lumber (LVL).**

Structural plywood produced in Australia and New Zealand is typically manufactured from a pine species; Radiata, Slash or Hoop Pine in Australia and Radiata Pine in New Zealand, with F8, F11 and F14 being the most readily available stress grades for these species. The most common sheet size for structural plywood is 2400 x 1200 mm, but other lengths (2700, 1800) and widths (900) are also available. Suitable face veneer grades for gussets would be DD, or possibly CD where appearance is also a consideration. Structural plywood is available in a range of thicknesses and constructions. 5 TABLE 5.3 in Chapter 5 of this Manual details standard structural plywood thicknesses and constructions. The construction or lay-up of the plywood must be specified in the gusset design, as AS 1720 utilises parallel ply theory in evaluation of the strength capacity of structural plywood loaded in-plane in bending. Therefore, thickness of cross-band veneers is not included in the overall structural plywood thickness used to evaluate gusset strength capacity.

Structural Laminated Veneer Lumber manufactured in Australia and New Zealand is typically manufactured from Radiata Pine. Strength capacities for LVL vary from manufacturer to manufacturer with each individual manufacturer publishing their products specific structural properties. Properties are identified by the manufacturer's brand name and this brand name needs to be included in any specification for structural LVL gussets. Structural LVL is available in widths up to 1200 mm and lengths up to 25m. Veneer grades for LVL are in accordance with the manufacturer's specification, and are based on structural properties rather than aesthetic considerations. Face veneer grades for LVL would be comparable to a plywood D or C quality face.

Typical thicknesses for LVL are 35, 36, 45, 63 and 75 mm. Structural LVL is usually manufactured with no cross-bands, however when used as gussets, a cross-band immediately below the face veneers will improve "nailability" by increasing resistance to splitting out at edges, ends and between nails. Structural plywood with cross-bands is not standard stock and would need to be ordered in advance. Where cross-band layers are included in structural LVL, parallel ply theory applies, with the cross-band veneer thickness not included in the overall structural LVL thickness used to evaluate gusset strength capacity.

Nails used to fix plywood gussets must be flat head structural nails or clouts. Nail sizes should be specified to suit installation with a nail gun. Hot dipped galvanised nails should be used in areas of high humidity or mildly corrosive environments or where preservative treated plywood, LVL or timber are used as components.

10.3 Plywood / LVL Gusset Design – Gusset Action

Gussets

Joint configuration, i.e. the intersection of the column / rafter members and whether **joints** are **internal** or **external** significantly **influences** the **stress distribution** likely to occur across the **critical section** of a plywood / LVL gusset. Quantifying such distributions has been the result of considerable research effort worldwide.

Mitred Internal Knee Gusset

Irrespective of whether the **internal gusseted joint** is **opening** or **closing** the **actual stress distribution** will be of the form shown in FIGURE 10.2(a). The **idealised stress distribution** is shown in FIGURE 10.2 (b).

Assuming a **balanced gusset construction,** i.e. that the **depth** of the **plywood gusset** is **twice** the **depth** of the **column** / **rafter member**, allows the **applied moment** to be expressed in terms of the **gusset strength and cross-sectional geometry.** The **centroid** of the stress distribution is taken to be a **distance** (**D**) from the gusset point, along its **centreline** (critical stress line).

The **moment / bending stress** relationship developed for the stress distribution of FIGURE 10.2 closely approximates the **classical linear distribution**, resulting in the **flexure formula.**

To obtain the **plywood gusset thickness** requires **manipulation** of the relationship, $\mathbf{f_b = M/Z}$, thus :

$$t_{II} \geq \frac{6M_p^*}{\Phi.k_1.k_{19}.g_{19})f_b' \times D^2} \qquad (10.1)$$

where:

M^*_p = in-plane **design 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 = **depth of column / rafter member**.

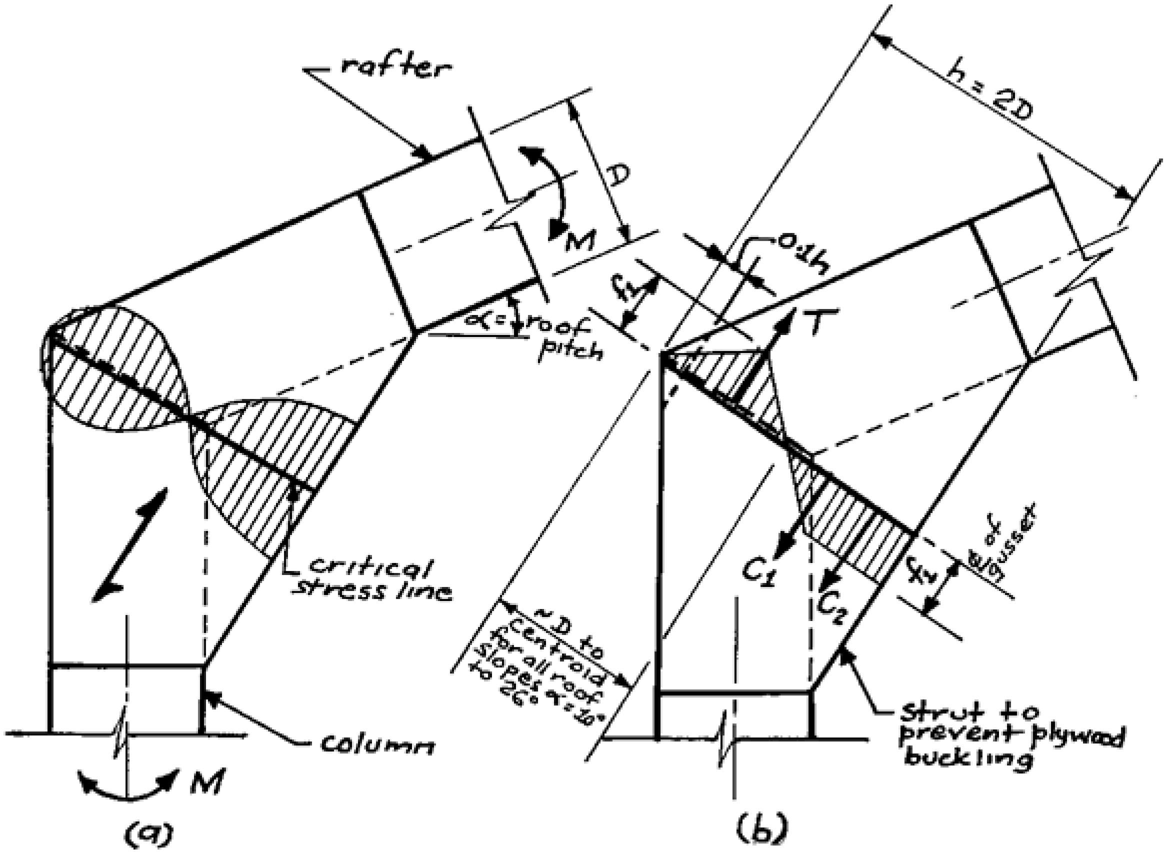


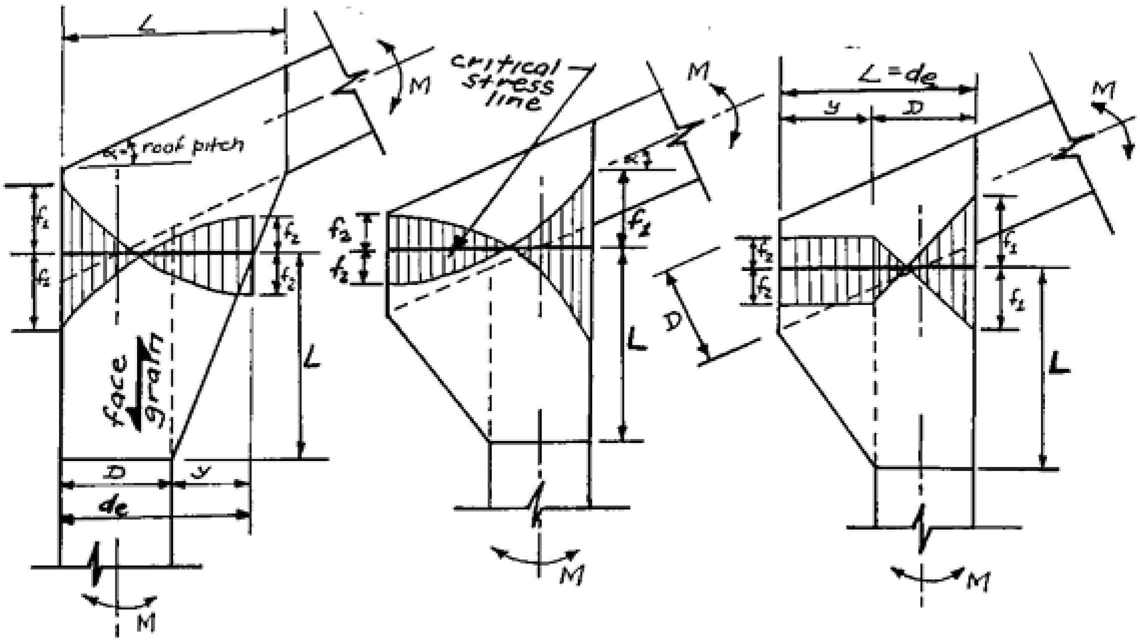
FIGURE 10.2: Mitred internal knee gusset and stress distributions

Internal and External Haunch (Knee) Gussets

The **external haunched gusseted joint** shown in FIGURE 10.2 provides the attractive alternative of being able to locate the plywood / LVL gusset **external** to the building.

Comparison of the **stress distributions** on the **critical stress lines** for the **external gusset** (FIGURE 10.3 (b)) and the **internal gusset** (FIGURE 10.3(a)) show:

- **internal stresses f_1** on the **external joint** equal the **external stresses f_1** on the **internal joint;**
- **likewise** for the **stresses f_2;**
- **stresses f_1** are **2** to **3 times greater than** stress **f_2.**



(a) (b) (c)

FIGURE 10.3: Actual and idealised stress distributions on the critical stress line

From the **idealised gusset stress distribution** shown in FIGURE 10.3(c) the following relationships have been developed:

$$\mathbf{f_1} = \frac{\mathbf{24\,Mk(1-k)}}{\mathbf{t_{11}D^2(4k-1)}} \qquad (10.2)$$

$$\mathbf{f_1} = \mathbf{f_1}\frac{\mathbf{(1-k)}}{\mathbf{k}}$$

where:

f_1 = **fibre stress** in MPa;
M = **total applied moment** on the joint N-mm;
D = **depth of column/rafter member** in mm;
$K = \frac{(y + D/2)}{(y + d)}$
$\mathbf{t_{II}}$ **= effective thickness of plywood**

For an **internal gusset:**

$$\boldsymbol{y} = \frac{\mathbf{L-D}}{\mathbf{1+(1-D/2\,L)\tan\propto}} \qquad (10.3)$$

where: $\propto$ **= roof slope**

For an **external gusset:**

$$\mathbf{y = L\text{-}D} \qquad (10.4)$$

Ridge (Apex) Mitre Joint

The design procedure is **similar** to that employed in the design of the **mitred internal knee gusset.** FIGURE 10.4 shows a **ridge joint.**

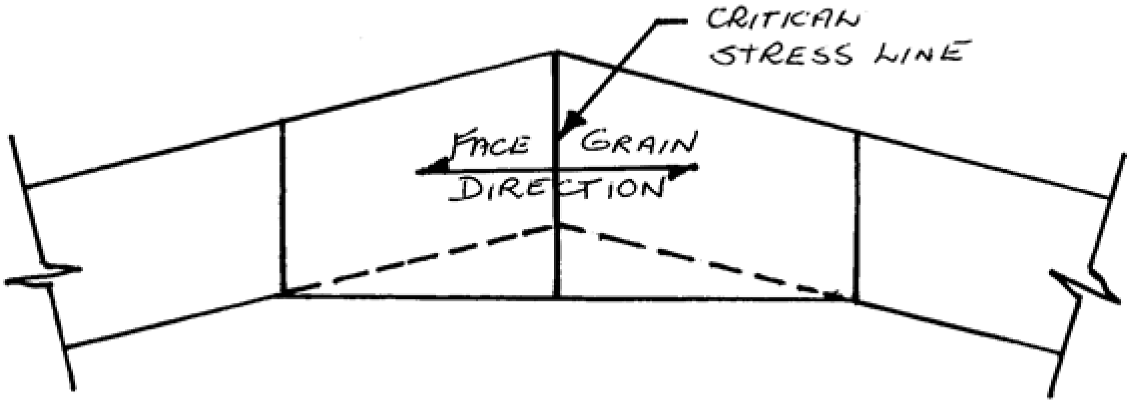


FIGURE 10.4: Ridge Joint

Nail Joint Action

Nail forces are evaluated through application of the classic **torsion relationship,** i.e. τ **= Tρ/I$_p$.** FIGURE 10.5 shows a simple nailed joint subjected to the combined **torsional moment (T)** and **shear force (P).** The **ith nail** of the nail group is subjected to a force **p_{in}**, at a **radius ρ** from the **centroid C** of the group.

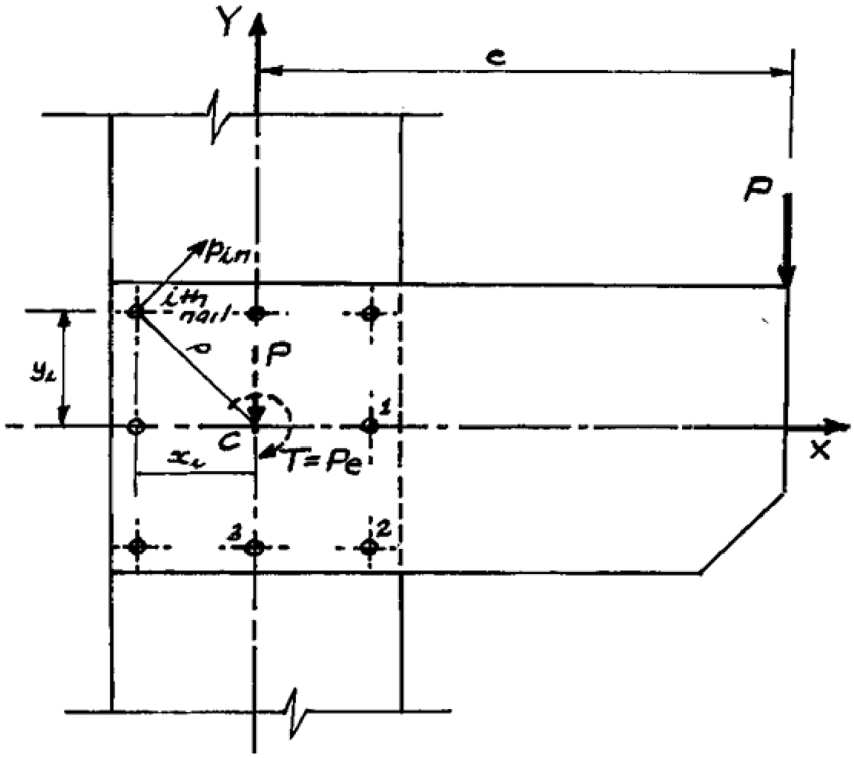


FIGURE 10.5: Nail group subjected to torsional moment

The **centroid** of the nail group can be found from the relationships:

$$\bar{x} = \sum_{i=1}^{n} x_i / n$$
$$\bar{y} = \sum_{i=1}^{n} y_i / n \qquad (10.5)$$

where x_I and y_I = nail co-ordinates in mm;
n = number of nails in the group.

The **polar moment** of the nail group is given by:

$$\mathbf{I_p = I_x + I_y} \qquad (10.6)$$

Where $I_X = A\sum_{i=1}^{n} y_i^2 \; \& \; I_y = A\sum_{i=1}^{n} x_i^2$ (10.7)

A = nail cross-sectional area in mm^2.

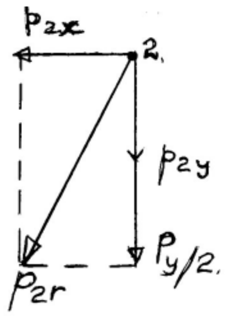
Re-arranging the **torsion equation** results in:

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

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:

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

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

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

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.

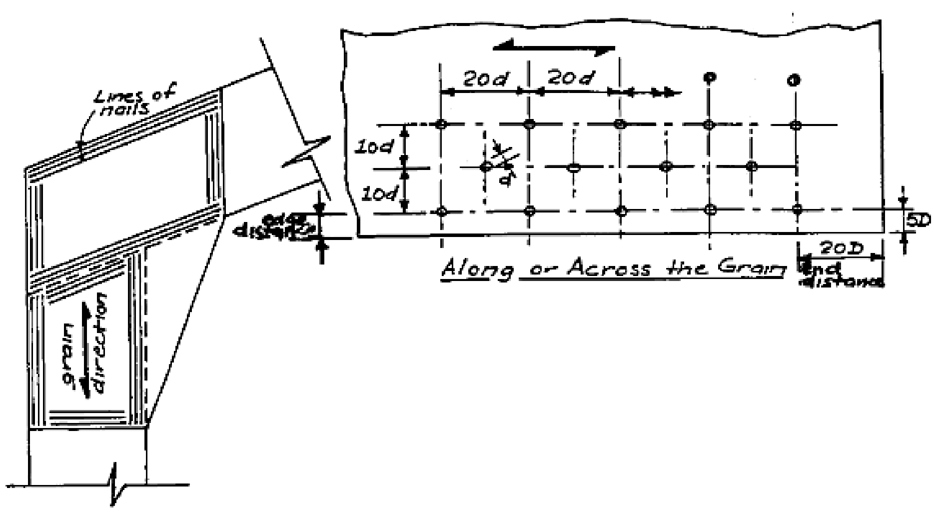


FIGURE 10.6: Typical nailing pattern and an idealised line representation

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

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

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.**

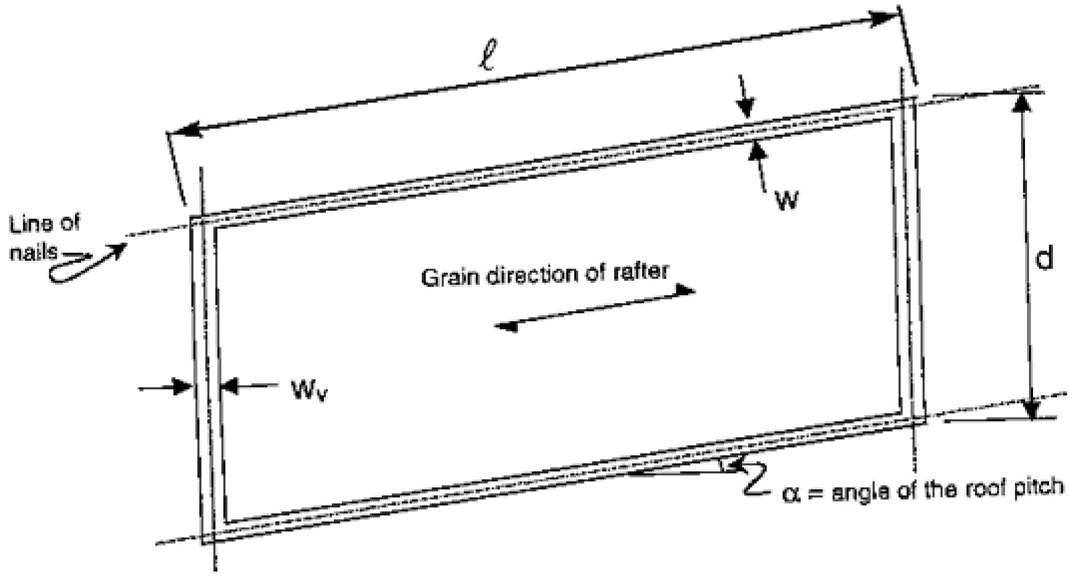


FIGURE 10.7: Smeared lines of nails for determining Ip

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:

$$I_p o = \frac{w\,\ell^3}{12} + w\,\ell\,r^2$$

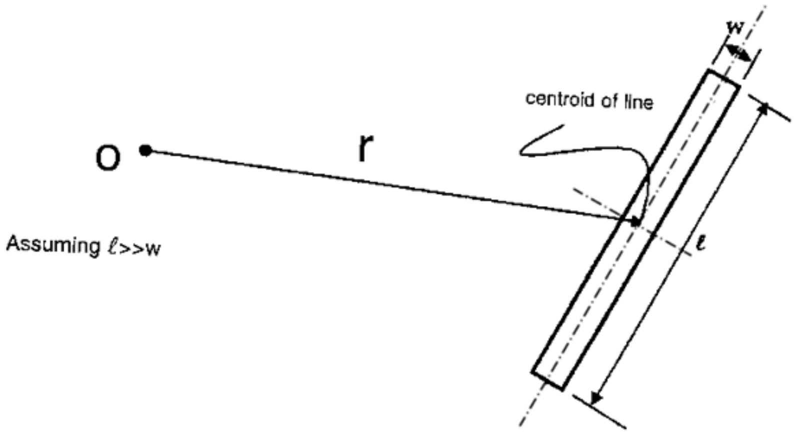


FIGURE 10.8: Polar moment of a line width w

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:

$$W_v = \left(\frac{\textbf{nail spacing parallel to grain}}{\textbf{nail spacing perpendicular to grain}}\right)\cos\alpha$$

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.**

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

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 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.**

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

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

$$d_e = L - y$$

$$\text{where } \propto = \text{roof slope}$$

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

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**

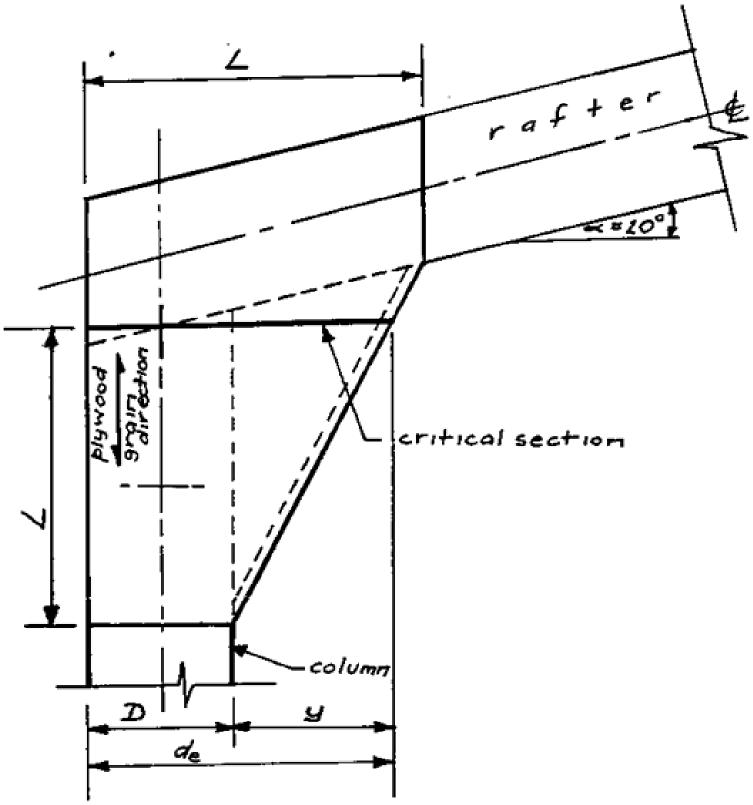


FIGURE 10.9: Defines the major dimensions of the knee joint

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:

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

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

= 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, reinstating **Equation 10.11** which allows the determination of (I_P) for a **smeared single ring** (rectangle) **of nails** as shown in Figure 10.7, results in:

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

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:

$$\mathbf{w_v} = \frac{\mathbf{60}}{\mathbf{30}}\mathbf{cos10^\circ}$$
$$= \mathbf{1.97}$$

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

$$\ell_1 = \mathbf{1140}\ (\textit{\textbf{which is equally divisible by 60}})$$

$$\mathbf{d_1} = \frac{\mathbf{D}}{\mathbf{cos10^\circ}} - \mathbf{2\,x\,20}$$

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

Therefore for the first ring of nails:

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

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

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

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

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

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

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

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

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

where:

s_{II} = 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 x_m and 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**

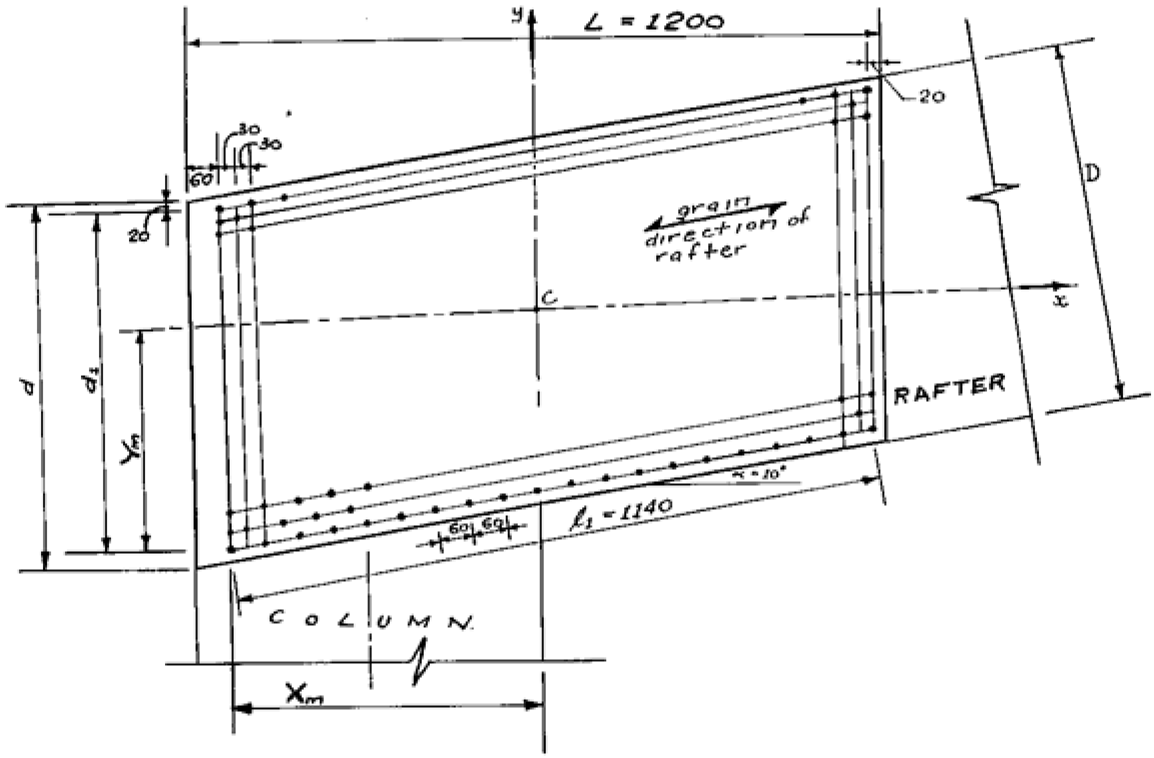


FIGURE 10.10: Shows nailing pattern for nail group in column or rafter

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:**

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

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

$$= 578 + 48$$

$$p_{ix} = 626N$$

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

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

$$= 813 + 131$$

$$p_{iy} = 944N$$

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

$$p_{ir} = 1132N$$

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 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 Photographs of Portals, Moment Joints

Plate 1

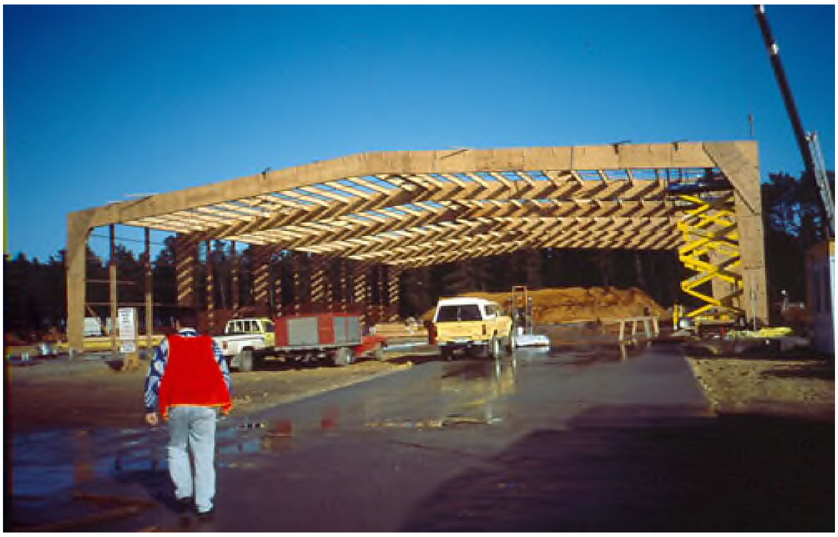


Plate 2

Plate 3

Plate 4

Plate 5

Plate 6

Plate 7

Plate 8

Plate 9(a)

Plate 9(b)

10.6 Photographs of Pinned Bases

Plate 10

Plate 11

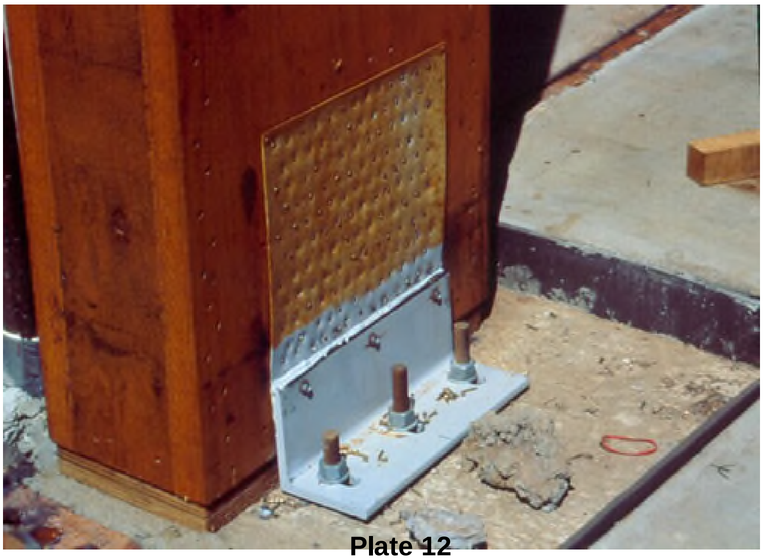


Plate 12

REFERENCES CITED:

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3. **A Study of the Structural Behaviour and Performance of Pitched Timber Portals**, A. Kermani, Second International Workshop on Full Scale Behaviour of Low Rise Buildings, James Cook Cyclone structural Testing Station, Townsville, July, 1994.

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.

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

CHAPTER 11

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.

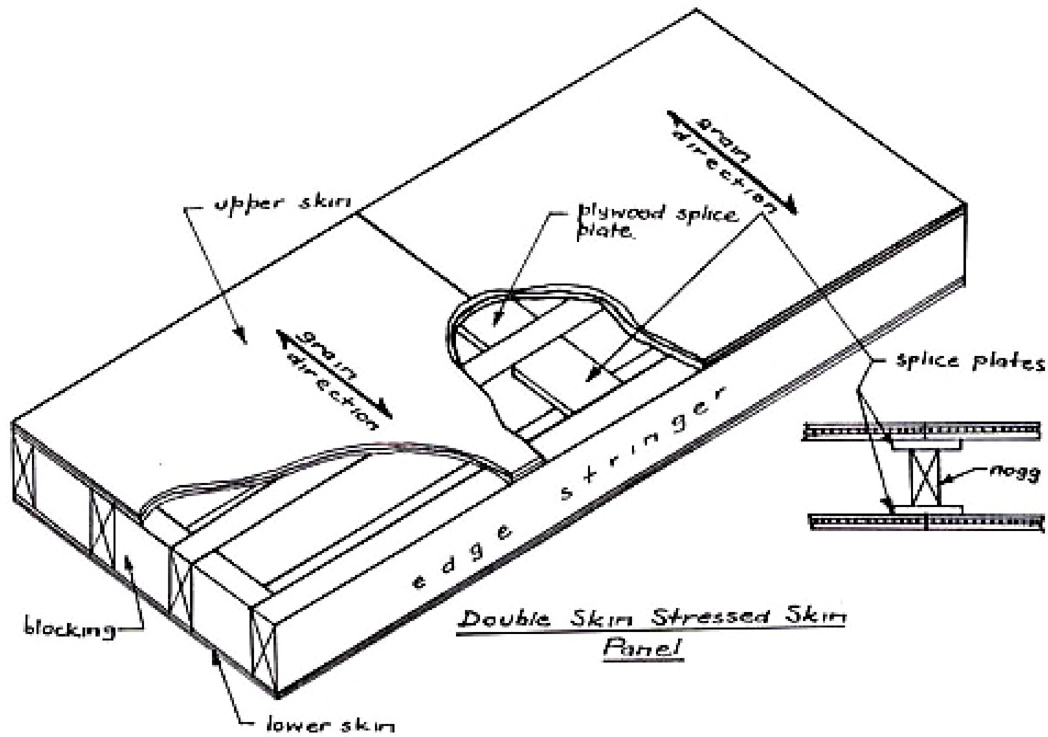


FIGURE 11.1: Component parts of a stressed skin panel

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.

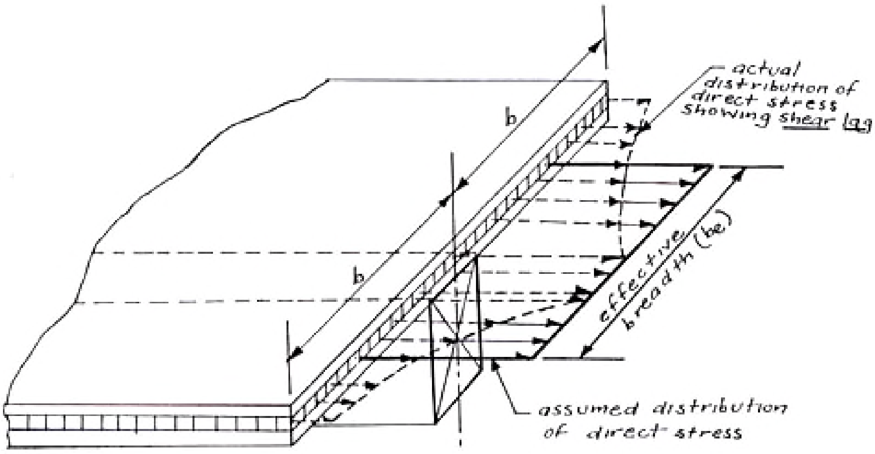
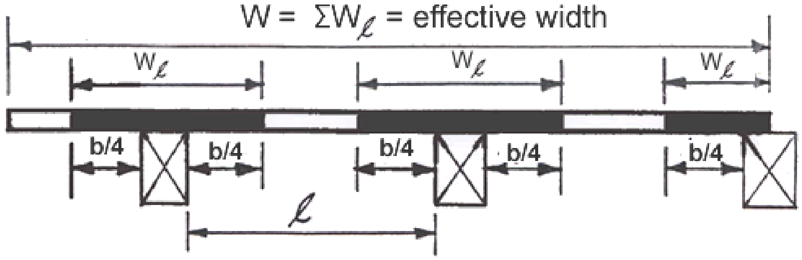


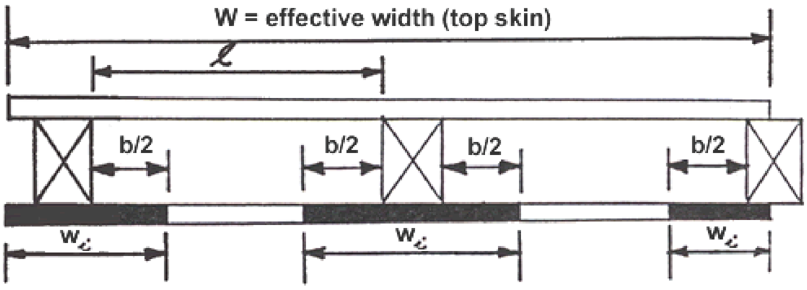
FIGURE 11.2: Distribution of flange stresses

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.**



b = basic spacing ≤ ℓ ; wℓ = effective flange width
Single Sided Panel
(a)



b = basic spacing ≤ ℓ
Double Sided Panel
(b)

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**.

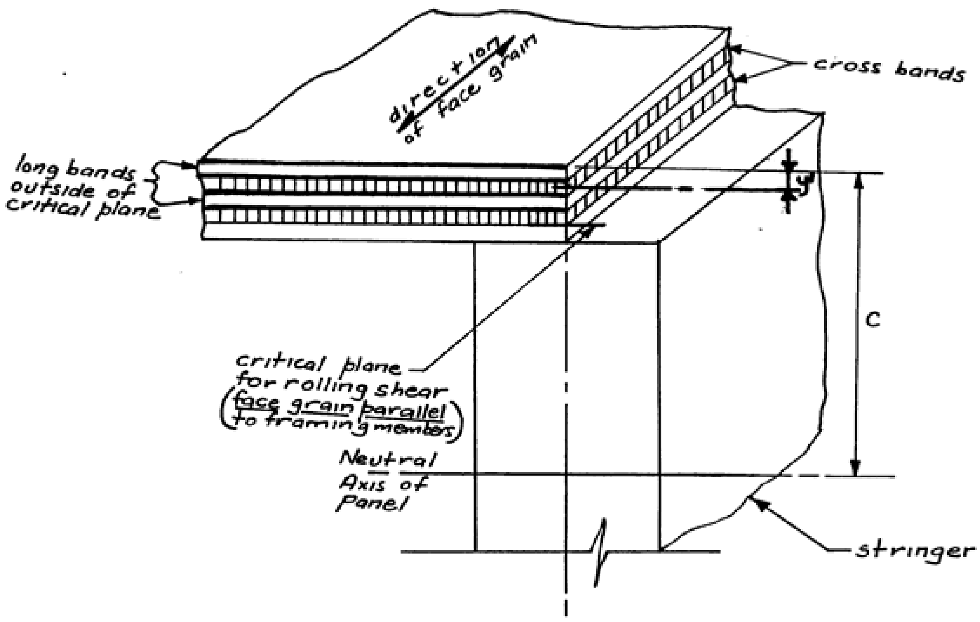


FIGURE 11.4: Position of critical plane for rolling shear

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:

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

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.

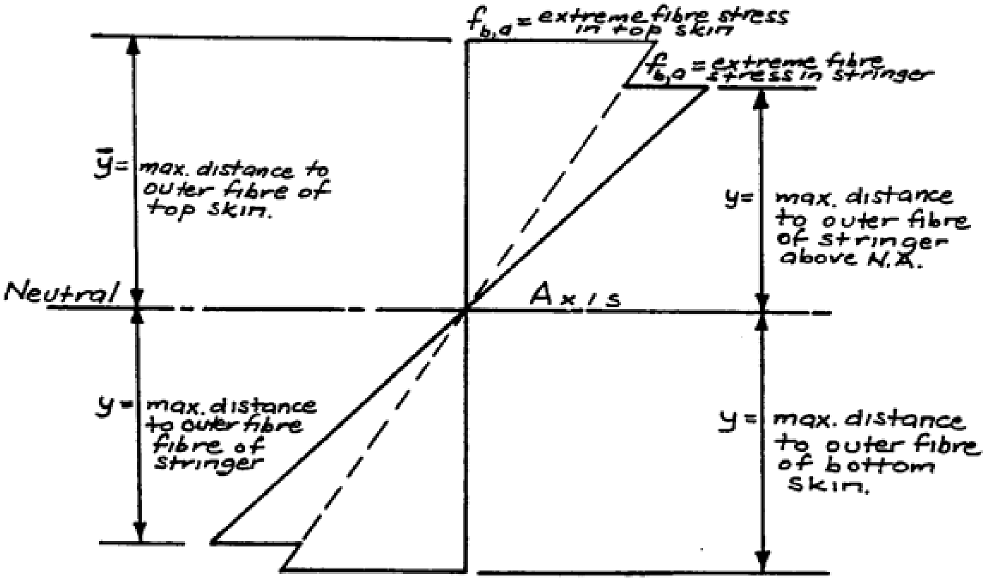


FIGURE 11.5: Bending stresses in stressed skin panel

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 :

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

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

F_{sp} = splice force

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

where:

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

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

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 $\mathbf{f_{sp}}$ 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.

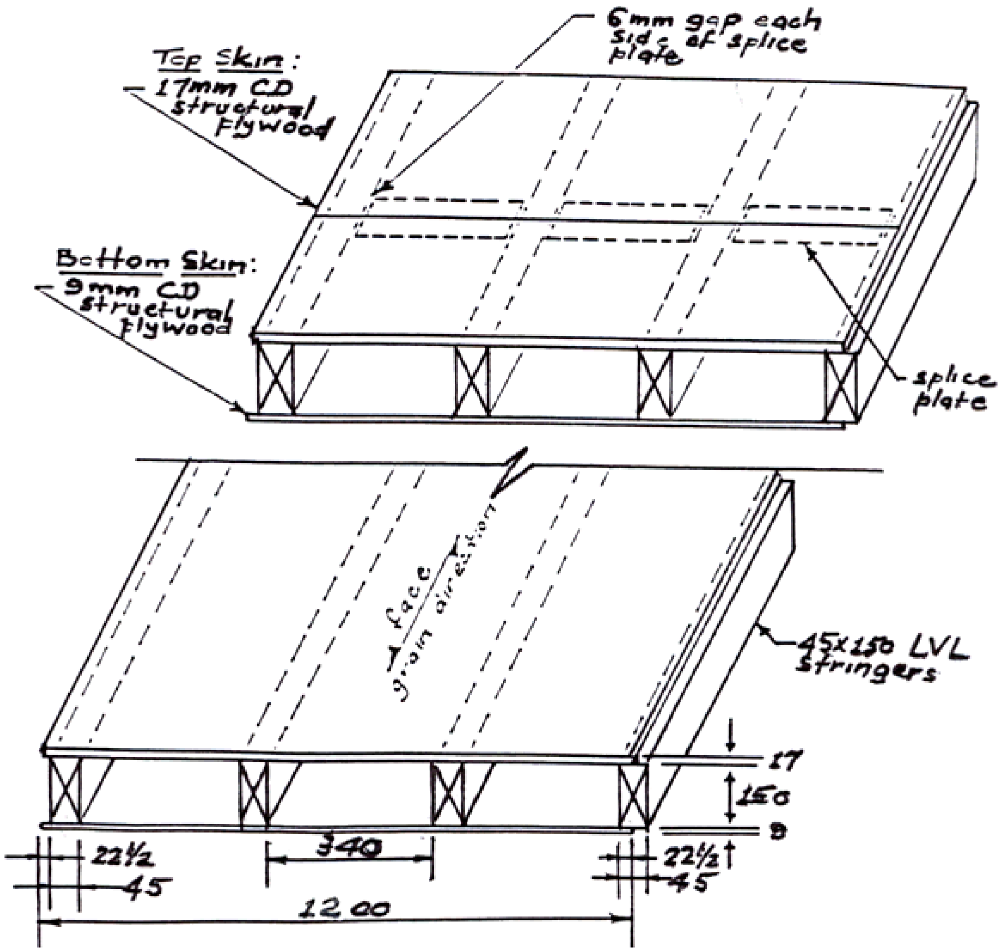


FIGURE 11.6: Stressed skin panel trial section

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

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

Section Properties - Deflection

Element	MoE	A_Π	$A_\Pi E$	Y	$A_\Pi Ey$
Top Skin Stringer Bottom Skin	10600 13200 10500	1.2 x 9600 = 11530 4 x 6750 x 1.26 = 34020 1.2 x 6000 = 7200	121×10^6 449×10^6 75.6×10^6	167.5 84 4.5	20.9×10^3 37.72×10^3 0.34×10^3
		Σ	645.6×10^6	Σ	58.900×10^3

TABLE 11.3: Gives procedure for determining section centroid

$$\bar{y} = \frac{\sum A_D \, x E x y}{\sum A_D \, x E} = \frac{58.96 \times 10^9}{645.6 \times 10^6}$$

$$\bar{y} = 91.3\,\text{mm}$$

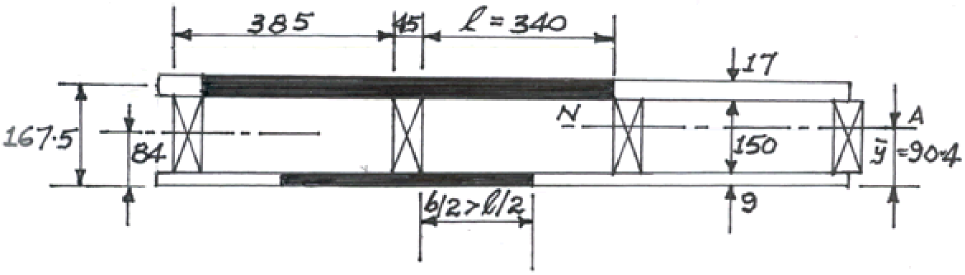


FIGURE 11.7: Shows the neutral axis and "b" relative to "ℓ"

Element	$A_\Pi E$	I_o	EI_o	d	$A_\Pi Ed^2$	$EI_o + A_\Pi Ed^2$
Top Skin Stringer Bottom Skin	121×10^6 449×10^6 75.6×10^6	295 x 1200 = 0.34×10^6 12.66×10^6 = 50.6×10^6 60 x 1200 = 0.07×10^6	3.59×10^3 667.9×10^3 0.74×10^3	67.7 7.3 86.8	55.46×10^{10} 2.39×10^{10} 56.96×10^{10}	55.82×10^{10} 69.18×10^{10} 57.03×10^{10}
					ΣEI_g	182.03×10^{10}

TABLE 11.4: Gives procedure for determining (EI_g)

EI_g = 182.03×10^{10} N-mm^2/1200 width

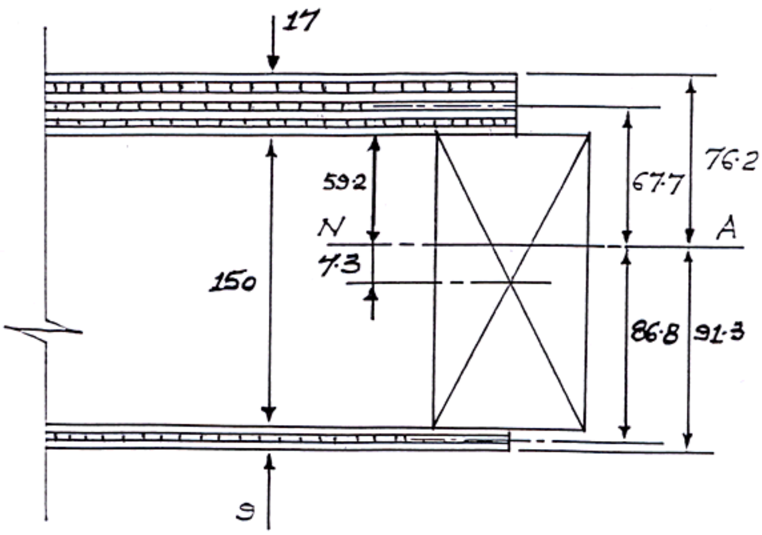


FIGURE 11.8: Shows relevant panel cross-section dimensions

Long Term Serviceability

Flexural Deflection:

For long term

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

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

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

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

$$= \frac{2 \times 5 \times 1.3 \times 1.2 \times (5)^4 \times 10^{12}}{384 \times 182.03 \times 10^{10}}$$

$$\Delta_b = 14\ mm$$

Shear Deflection:

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

$$= \frac{2 1.8 \times 1.3 \times 1.2 \times 5 \times 5 \times 10^6}{4 \times 6750 \times 880}$$

$$\Delta_s = 6\ mm$$

$$\Delta_b + \Delta_s = \mathbf{14 + 6 = 20mm}$$

$$\Delta_b + \Delta_s = \mathbf{20mm < 20.8mm, \text{ i.e. span/240 so OK}}$$

Top Skin Deflection:

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

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

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

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

where:

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

Bending Moment

Member Design Capacity: Strength Limit State

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

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 x 10^6	11530	67.7^2	52.8 x 10^6
Stringer	50.6 x 10^6	34020	7.3^2	1.8 x 10^6
Bottom skin	0.07 x 10^6	7200	86.8^2	54.2 x 10^6
			ΣI_g=	108.8 x 10^6

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

Ig **=** 108.8 x 10^6mm^4 for 1200 wide panel

For the top skin :

$$Z_t = \frac{I}{y} = \frac{I_g}{76.2} = \frac{108.8 \times 10^6}{76.2}$$

Z_t = 1.43 x 10^6mm^3

ϕM = (0.72 x 35 x 1.43 x 10^6)N-mm

ϕM = 36.0 kN-m

For bottom skin:

$$Z_b = \frac{Ig}{91.3} = \frac{108.8 \times 10^6}{91.3}$$

Z_b = 1.19 x 10^6mm^3

ϕM = 30 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 + Ψ_cQ	(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:

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

$$= \left[\frac{2 \times 1.2 \times 5^2}{8} \times \frac{10500}{182.03 \times 10^{10}} \times 91.3 \times \frac{984}{1200} \times 10^6\right]$$

$$f_{st} = 3.24\,MPa$$

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

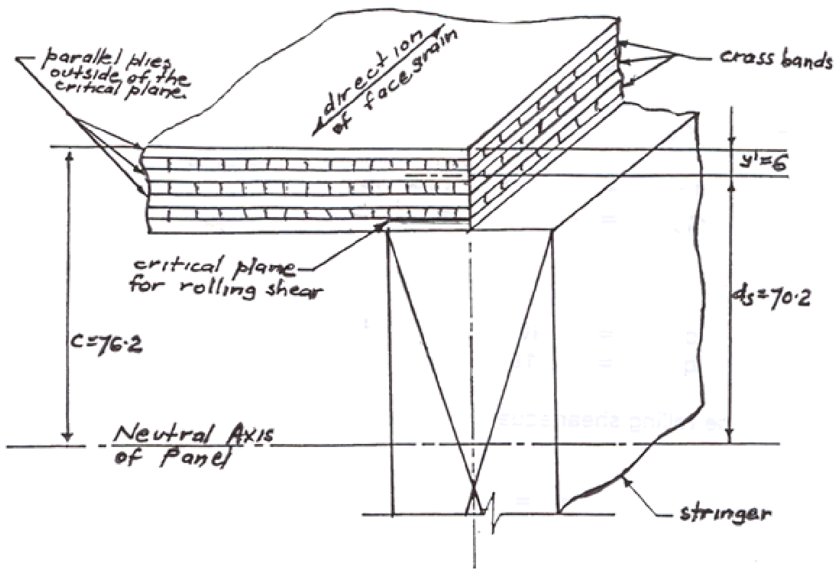


FIGURE 11.9: Shows dimensions for obtaining Q_s

$$\tau_r = \frac{SxQs}{I \times b}$$

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

The statical moment Qs is obtained thus:

$Q_s = A_{II} \times d_s$
$= 3(2400 \times 1.2)70.2$
$\mathbf{Q_s = 606528mm^3}$

From FIGURE 11.6:

b_s = 3 x stringer width + 0.5 x stringer width
= (3 x 45) + (0.5 x 45)
$\mathbf{b_s}$ **= 157.5mm**

Writing ($\tau_r \times b_{sf}$) = q, a shear flow,
where:

b_{sf} = 45 + (0.5 x 45)
= **67½mm for edge stringers**
$\mathbf{b_{sf}}$ = **(2 x 45)**
= 90mm for internal stringers.

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

$\tau_r = \phi \times k_1 \times k_{19} \times g_{19} \times f'_s$

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):

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

$$\mathbf{1.2\,w = \left[\frac{168.2 \times (182.03 \times 10^{10}) \times 1}{1 \times 5 \times 10500 \times 606528} \times \frac{1}{10^3}\right] kN/m}$$

$$\mathbf{1.2\,w = 9.6\,kN/m}$$

or

$$\mathbf{w = 8kPa, >2kPa, \therefore O.K.}$$

Horizontal Shear

From Equations 11.8 and 11.9 and FIGURE 11.8:

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

$$= 4(45 \times 59.2 \times \frac{59.2}{2}) + (\frac{10500}{13200} \times 1.2 \times 9600 \times 67.7)$$

$$Q_H = 935796mm^3$$

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

$$= \left(\frac{10.9 \times 10^3 \times 935796 \times 13200}{132.03 \times 10^{10} \times 4 \times 45}\right) MPa$$

$$\tau_H = \mathbf{0.41MPa < 1.7MPa.\ \ O.K.}$$

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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12. **Structural Wood Panels**, Thomas E. McLain, from Wood Engineering & Construction Handbook, Keith F. Faherty & Thomas G. Williamson, McGraw-Hill, 1989.

Part Four

Exotic Structures & Connection Design

Exotic Structural Forms

Connection Design – Plywood and LVL

CHAPTER 12

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.

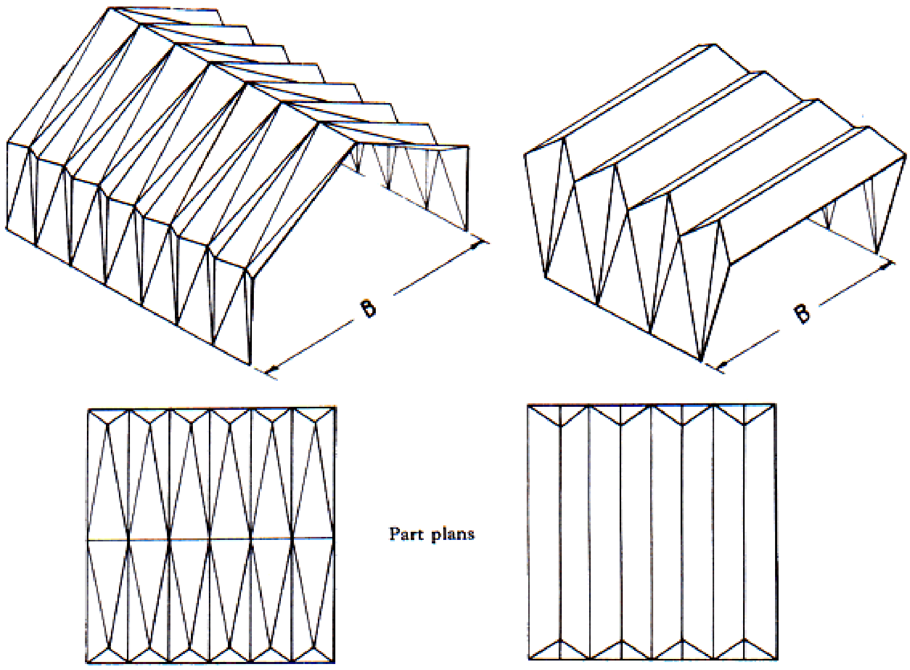


FIGURE 12.1: Various types folded plate structures

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.

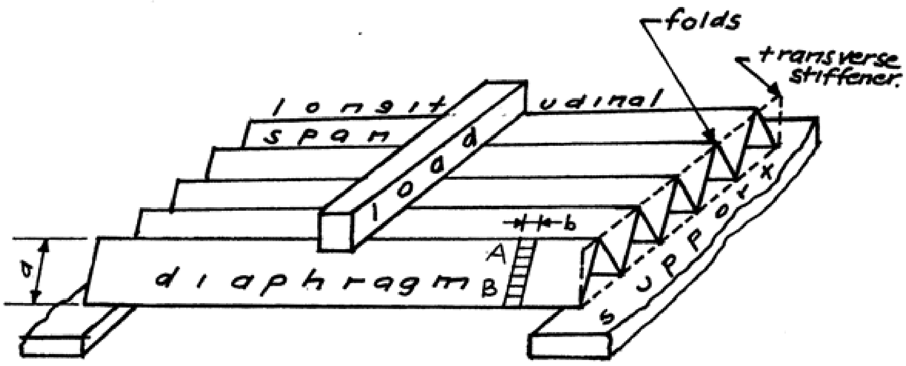


FIGURE 12.2: Sheet of paper with folds supporting a load

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.

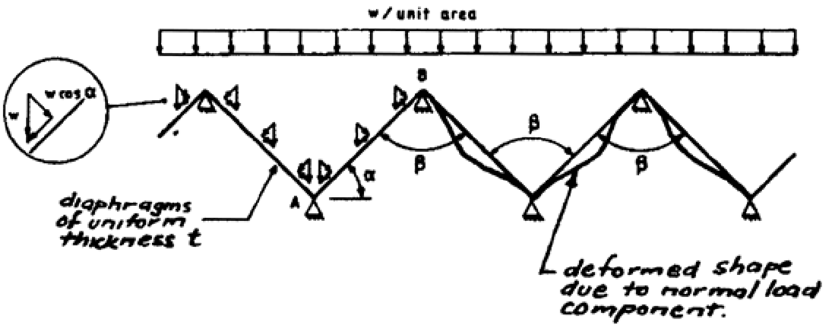


FIGURE 12.3: Load components on a transverse strip

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:

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

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

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

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

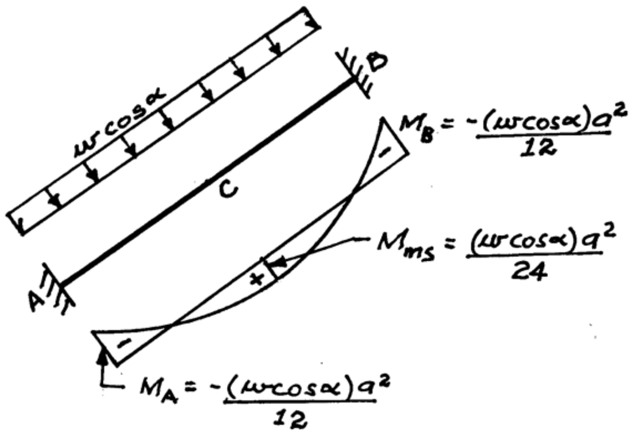


FIGURE 12.4: Isolated transverse element under load

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

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

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

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

$$\left.\begin{aligned} \mathbf{\sigma_{A/B}} &= \mathbf{\pm\frac{w\cos\alpha}{2}\left(\frac{a}{t}\right)^2} \\ \mathbf{\sigma_c} &= \mathbf{\frac{w\cos\alpha}{4}\left(\frac{a}{t}\right)^2} \end{aligned}\right]$$

(12.1)

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

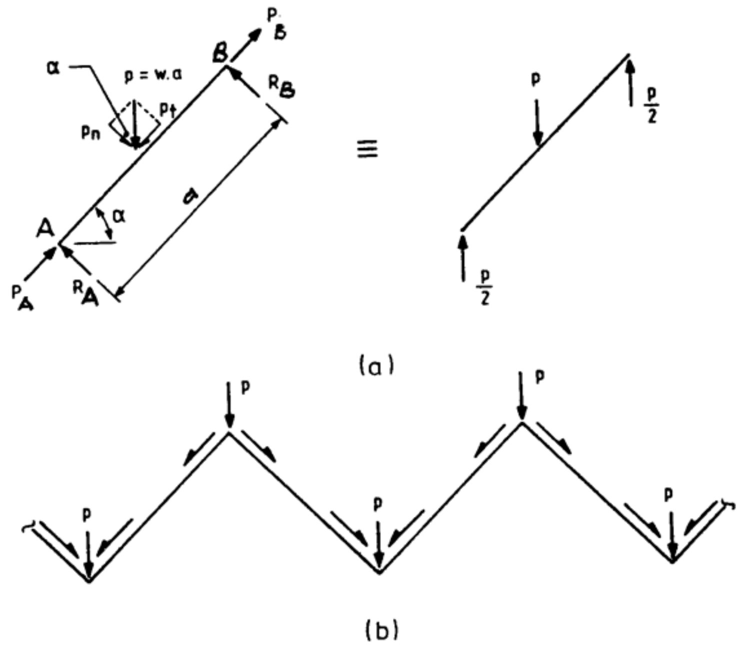


FIGURE 12.5: Determination of longitudinal load on the system

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**$\propto$

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

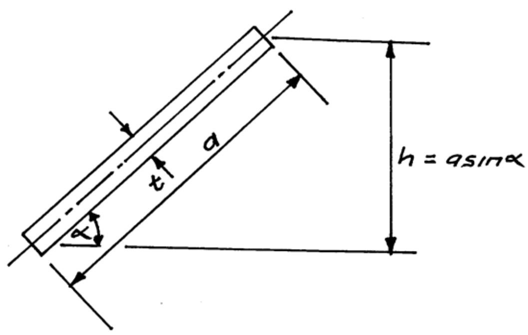


FIGURE 12.6: Inclined diaphragm

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

$$
\begin{aligned}
\mathbf{I} &= \frac{\mathbf{bh^3}}{\mathbf{12}} \\
&= \frac{\mathbf{1}}{\mathbf{12}}\left(\frac{\mathbf{t}}{\mathbf{sin}\propto}\right)(\mathbf{asin}\propto)^3 \qquad \mathbf{(12.2)} \\
&= \frac{\mathbf{1}}{\mathbf{12}}.\mathbf{ta^3}\,.\,\mathbf{sin^2}\propto
\end{aligned}
$$

The section modulus (Z) is:

$$
\begin{aligned}
Z &= \frac{I}{y} \\
&= \frac{1}{12}\left(\frac{t}{\sin\propto}\right)(a\sin\propto)^3.\frac{1}{h/2} \\
&= \frac{1}{6}.ta^2.\sin\propto
\end{aligned}
\qquad (12.3)
$$

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

$$
\begin{aligned}
\sigma_{max} &= \frac{pL^2/8}{Z} \\
&= \frac{p.L^2/8}{t.a^2\sin\propto/6} \\
&= \frac{6.w.a.L^2}{8.t.a^2.\sin\propto} \\
&= \frac{3}{4}.\frac{wL^2}{th}
\end{aligned}
\qquad (12.4)
$$

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.

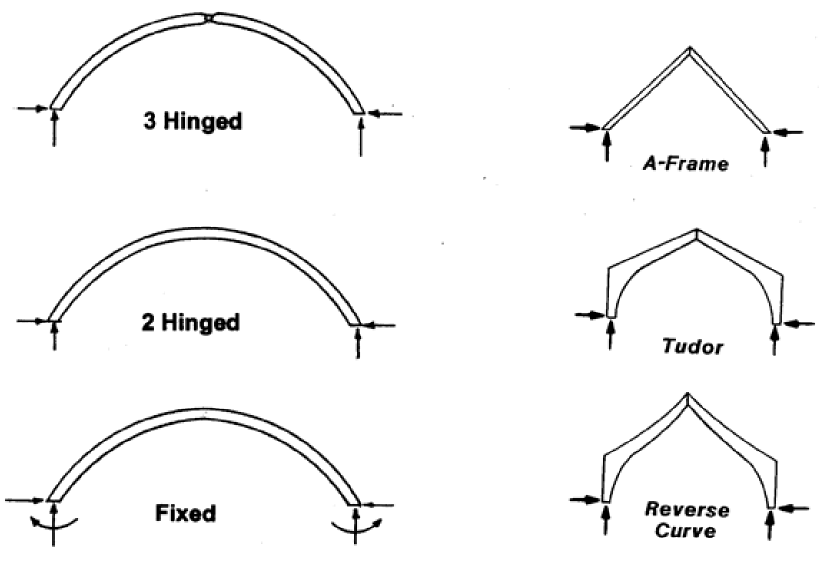
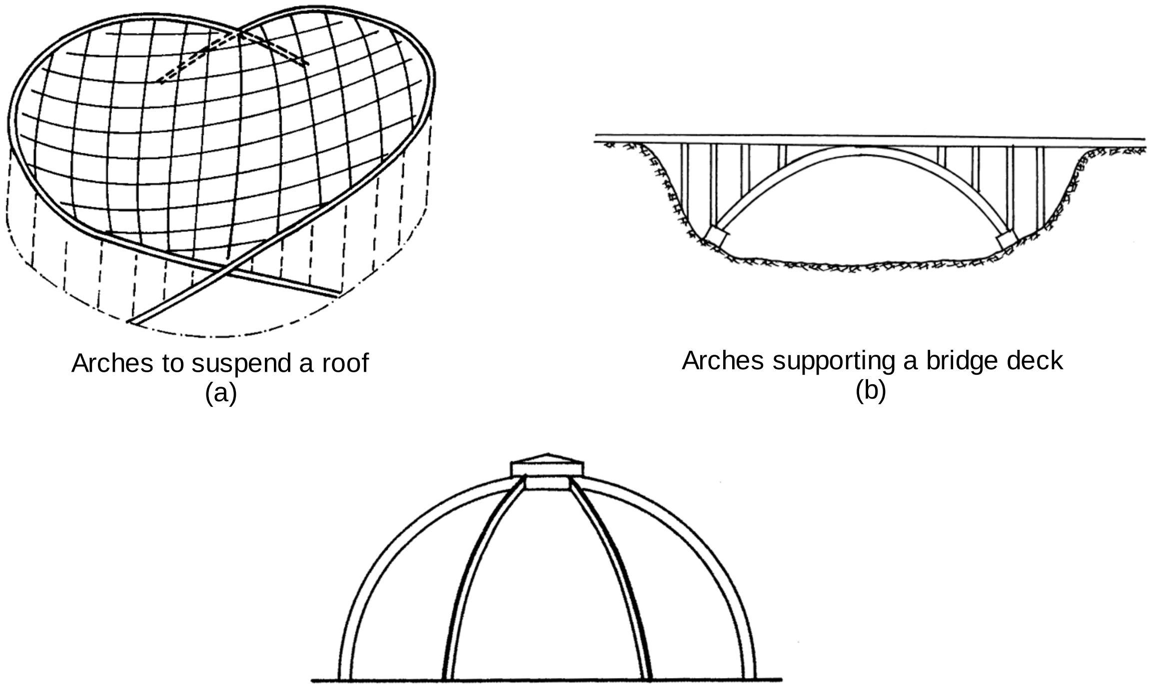


FIGURE 12.7: Basic arches and some portal derivatives

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 a **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:

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

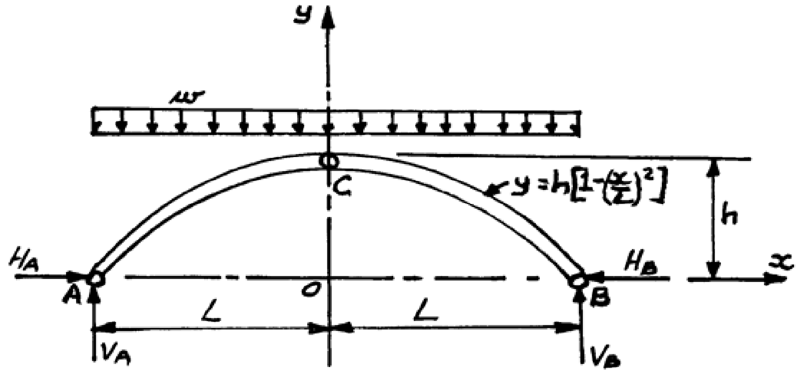


FIGURE 12.9: A parabolic arch (not to scale)

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

$$\Sigma F_y = 0 : V_A + V_B = 2wL$$

$$\Sigma F_x = 0 : H_A = H_B$$

$$\Sigma M_{CR} = 0 : H_B \times h - V_B \times L + \frac{wL^2}{2} = 0$$

$$\Sigma M_{CL} = 0 : V_A \times L - H_A \times h - \frac{wL^2}{2} = 0$$

(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.

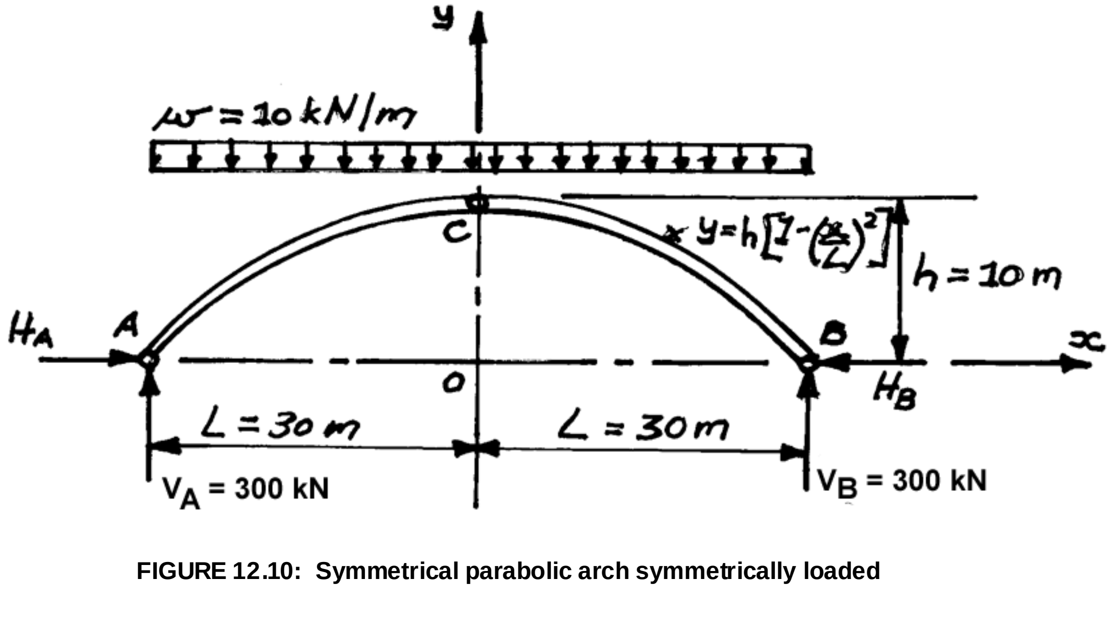


FIGURE 12.10: Symmetrical parabolic arch symmetrically loaded

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:

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

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

$$\text{From } \Sigma M_{CR} = 0$$

$$= 10H_B\ \ 300 \times 30 + \frac{10 \times 30^2}{2}$$

$$= 0$$

$$\mathbf{H_B} = \left(\frac{\mathbf{9000 - 4500}}{\mathbf{10}}\right)\mathbf{kN}$$

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

Hence:

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

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.

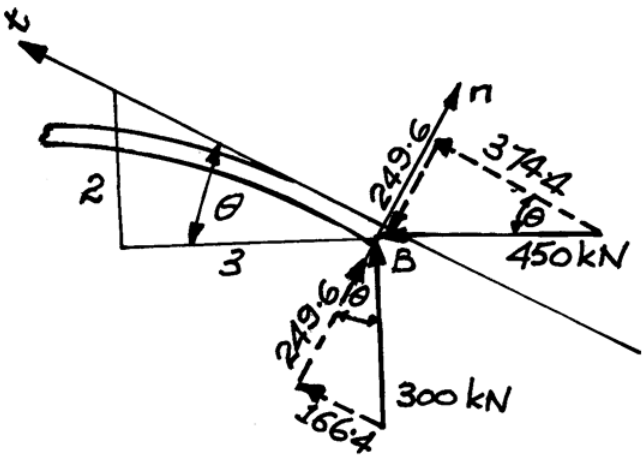


FIGURE 12.11: Components of shear and axial force

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:

$$\frac{dy}{dx} = -2\left(\frac{x}{L^2}\right)h$$
$$= \frac{-2 \times 15 \times 10}{30^2}$$
$$= -1/3$$

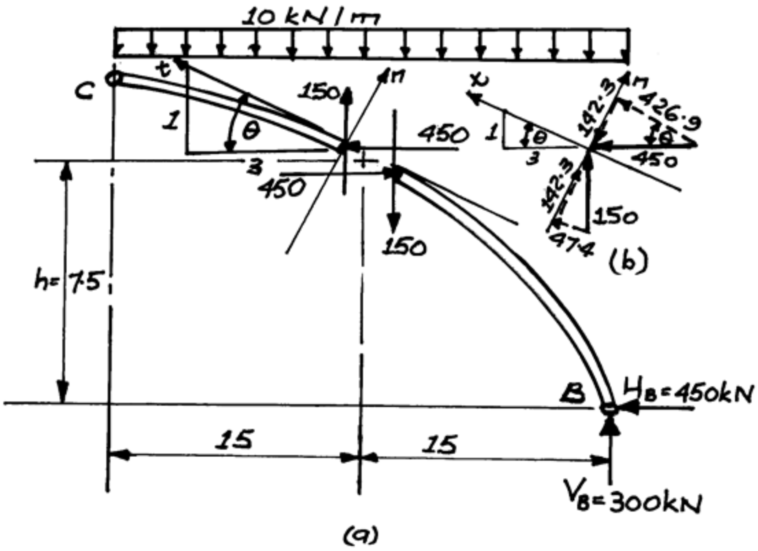


FIGURE 12.12: Exposed cross-section at mid-length and axial and shear force components

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:

F_A = (426.9 + 47.4)kN
F_A = 474.3kN(compression)
F_S = (142.3 – 142.3)kN
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.

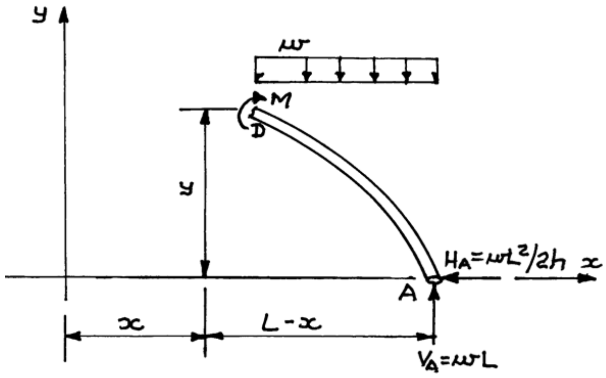


FIGURE 12.13: Free body diagram of part of arch

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).**

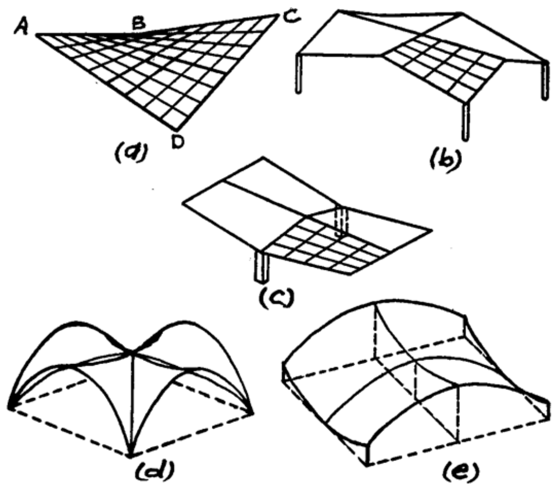


FIGURE 12.14: Various hypar configurations

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:**

$$z = kxy \qquad \textbf{(12.7)}$$

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

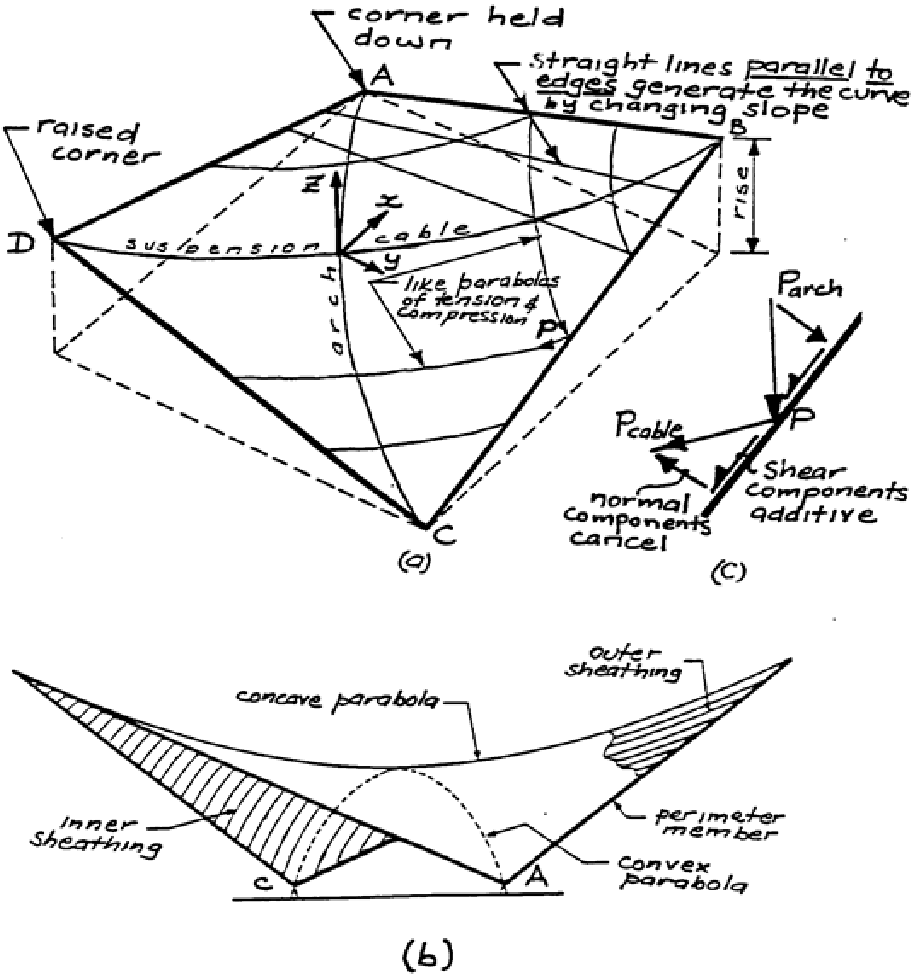


FIGURE 12.15: Two views of a single hyperbolic-paraboloid shell

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 :

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

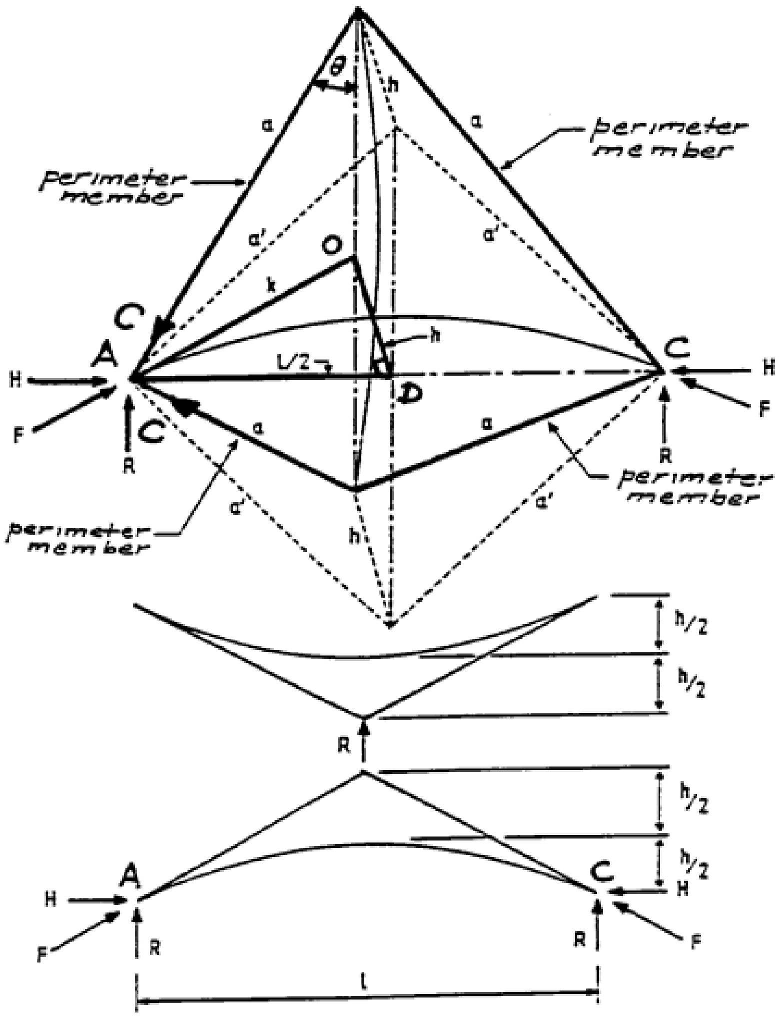


FIGURE 12.16: Reactive force components and resultant

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:

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

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.

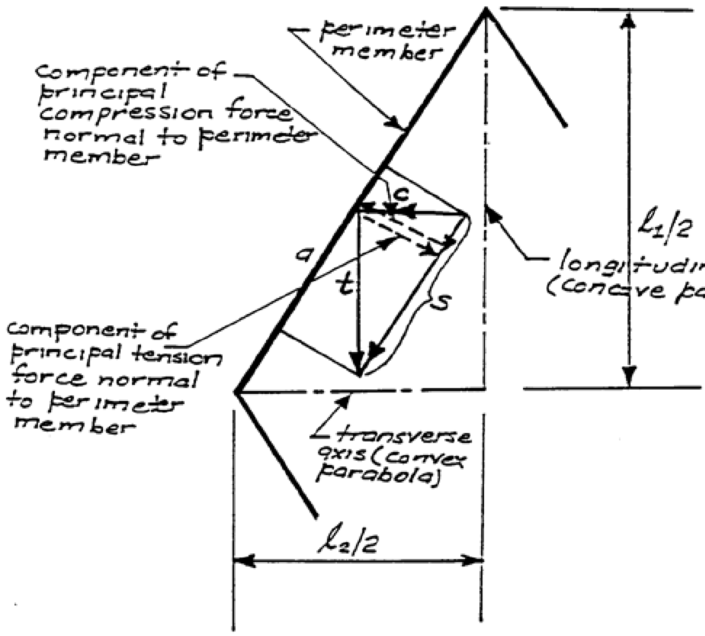


FIGURE 12.17: Resolved components of the tension and compression forces

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:

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

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

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

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

Hence:

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

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:

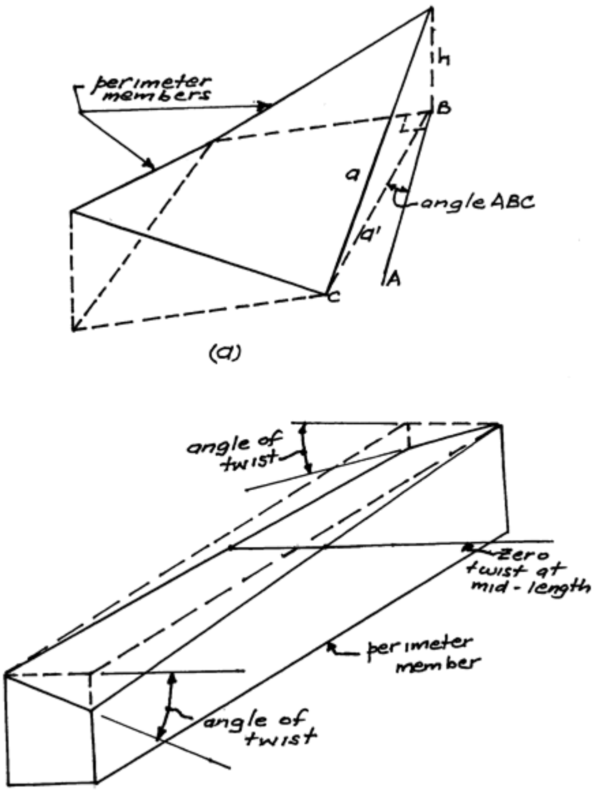
$$\textbf{tan}\,\textbf{\textit{of angleof twist}} = \frac{\textbf{\textit{ha}}}{\textbf{(\textit{a}')}^2\,\textbf{cos}\,\angle \textbf{ABC}}$$

where:

$\angle ABC$ 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).

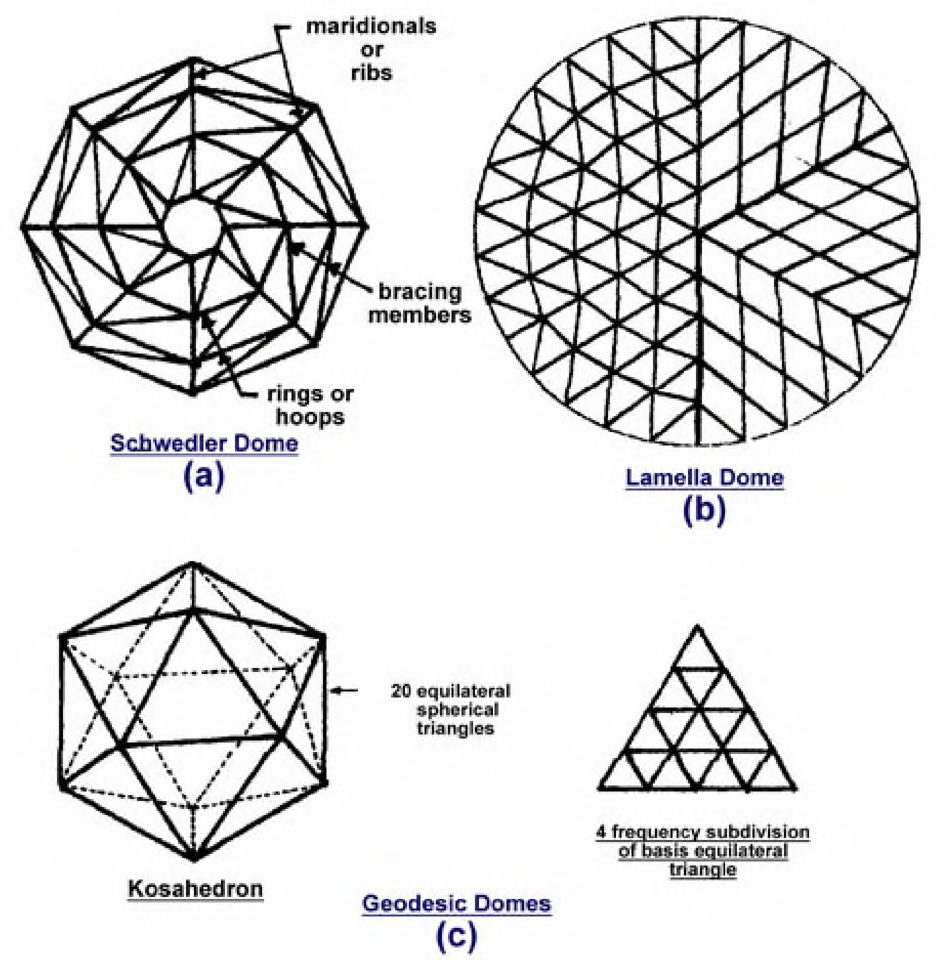


FIGURE 12.19: Shows some different dome geometries

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 N_x, N_y and 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).

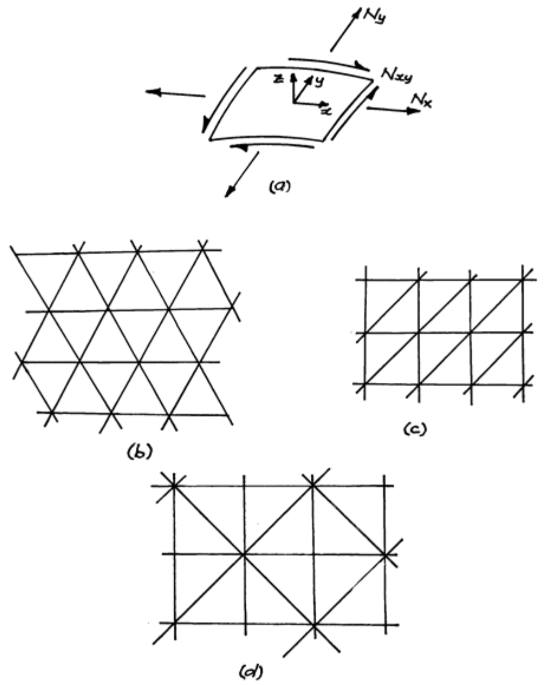


FIGURE 12.20: Membrane forces and reticulated spatial systems

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:

$$N_{xy} = 0$$

The **hoop force:**

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

and the **meridional force:**

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

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.

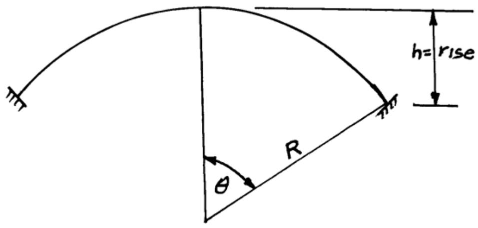


FIGURE 12.21: Shows θ = angle to crown and R = radius of curvature

when:

$$\frac{1}{1+\cos\theta}-\cos\theta \qquad = 0 \text{ from Equation 12.16}$$

Then: $\theta \approx 52°$
and $N_x = 0$

With further **increase** in **θ**, 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.

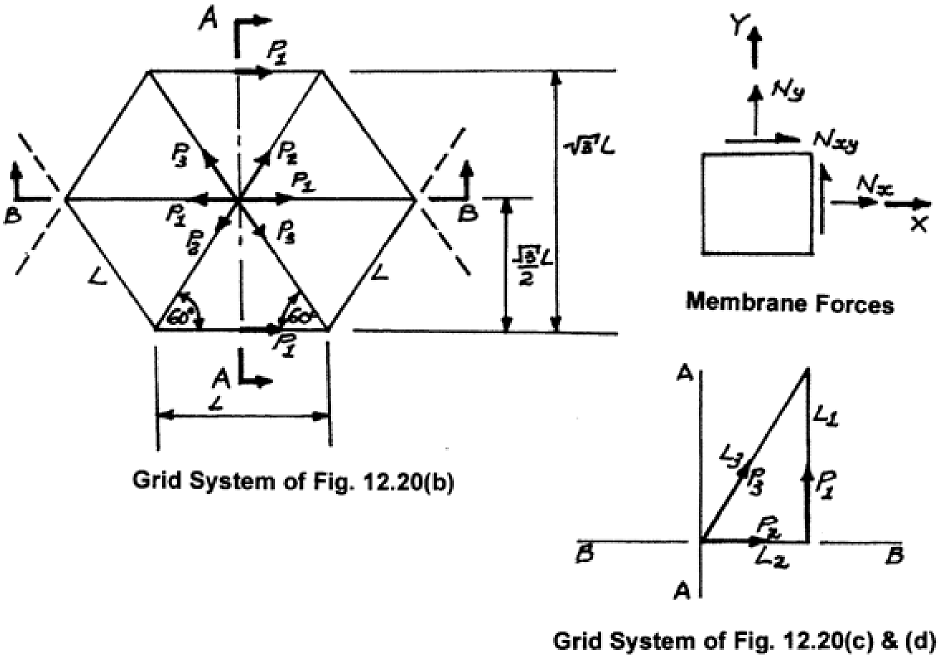


FIGURE 12.22: Grid systems and membrane forces

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:

$$\Sigma Fx = 0$$

$$\frac{P_1}{2} + \frac{P_1}{2} + P_1 + P_2 \cos 60° + P_3 \cos 60° = 2\sqrt{3} L \times N_x$$

Hence :

$$4P_1 + P_2 + P_3 = 2\sqrt{3} \times L \times N_x \qquad \textbf{(12.18)}$$

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

In the Y-direction:

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

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

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

$$\Sigma \mathbf{F_X} = \mathbf{0}$$

$$\mathbf{P_2 - P_3 = 2LN_{xy}} \qquad \textbf{(12.20)}$$

Re-arranging the above equations:

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

Inverting Equations 12.21 gives:

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

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

$$\boldsymbol{P}_1 = \boldsymbol{P}_2 = \boldsymbol{P}_3 = -\boldsymbol{L.R.w.} / 2\sqrt{3}$$

NOTE:
*The above relationships were derived from **static equilibrium requirements** and are therefore **independent** 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:

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

And

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

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.

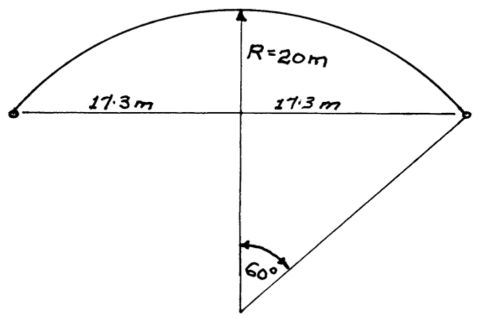


FIGURE 12.23: Dome dimensions

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

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

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

From Equations 12.22 :

$$P_1 = \frac{L}{2\sqrt{3}}(3N_x - N_y)$$

$$= \frac{2}{2\sqrt{3}}(3.5 + 60)$$

$$= 43.3 \text{ kN}$$

$$P_2 = \frac{L}{\sqrt{3}}\left(N_y + \sqrt{3} \text{ x } N_{xy}\right)$$

$$= \frac{2}{\sqrt{3}}. -60$$

$$= -69.3 \text{ kN}$$

$$P_3 = \frac{L}{\sqrt{3}}(N_y - \sqrt{3}\, N_{xy})$$

$$= \frac{2}{\sqrt{3}} \text{x} - 60$$

$$= -69.3 \text{ kN}$$

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.

$$f_c^{'} = \frac{P}{A}$$

$$A = \frac{69.3 \text{x} 10^3}{20}$$

$$= 3465 \text{mm}^2$$

Assuming an LVL thickness of 45 mm:

$$d = \frac{3465}{45}$$

$$= 77 \text{mm}$$

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.

REFERENCES CITED:

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2. **Architectural Structures : An Introduction to Structural Mechanics**, H.J. Cowan, 1st Metric Ed., Pitman, 1980.

3. **Steel Space Structures**, Z.S. Makowski, Michael Joseph, 1965.

4. **Wood Engineering & Construction Handbook**, K.F. Faherty & T.G. Williamson, McGraw-Hill, 1989.

5. **The Thames Barrier Roofs & Other Timber Structures Part II**, V.C. Johnson, Journal of the Institute of Wood Science, No. 48, December, 1980.

6. **Arches**, P. Smith, New Zealand Timber Industry Federation (Section B-7).

7. **Timber Shells**, L.O. Keresztesy, Timber Research & Development Association, High Wycombe, Bucks, June 1970.

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9. **An Experimental Investigation into the Stress & Displacement Distribution in Hyperbolic Paraboloid Timber Shell**, L.O. Keresztesy, TRADA, High Wycombe.

10. **Hyperbolic Paraboloid timber Shell Roofs**, L.G. Booth, The Architect & Building News, August, 1959.

11. **Membrane Forces & Buckling in Reticulated Shells**, D.T. Wright, Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, 1965.

12. **Reticulated Space Structures**, J.O. Crooker & K.P. Buchert, Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, 1970.

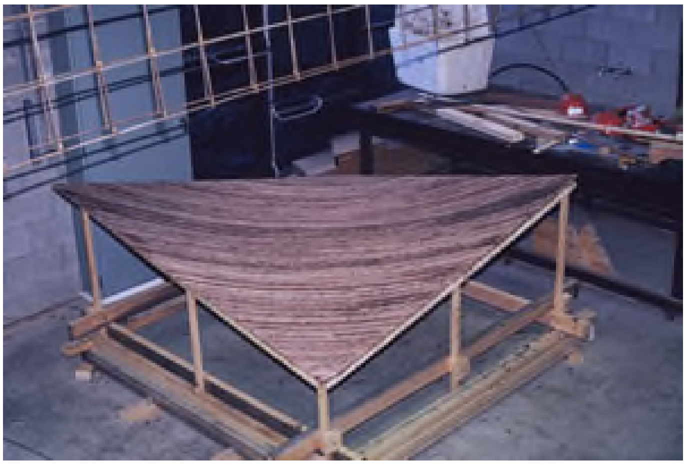
13. **Structural Form**, H. Bier, New Zealand Timber Industry Federation (Section A-1).

A12 Chapter 12 Appendix

EXAMPLES OF EXOTIC STRUCTURES

Folded Plates

Arches



Hypars

Domes

EXOTIC STRUCTURAL FORMS DESIGN AIDS

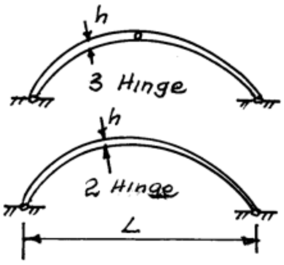
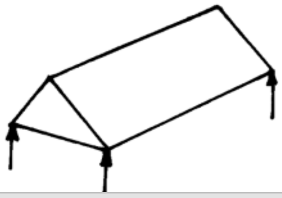
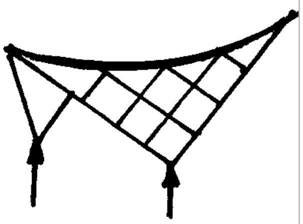
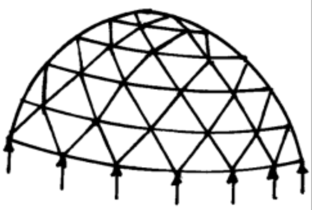
STRUCTURE	SPACING (m)	SPAN (m)	DEPTH
ARCHES			
	5-15 5-20	20-60 20-100	h = L/50 h = L/55
FOLDED PLATE		**Max. Likely (m x m)**	**COMMENTS**
		20 x 10	Stiffening ribs may be required
HYPAR			
		25 x 25	LVL edge beams with boarded membrane
DOMES			
		30 Φ	Ribbed dome with plywood membrane
		100 + Φ	Braced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

TABLE A12.1: Preliminary design information

CHAPTER 13

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).

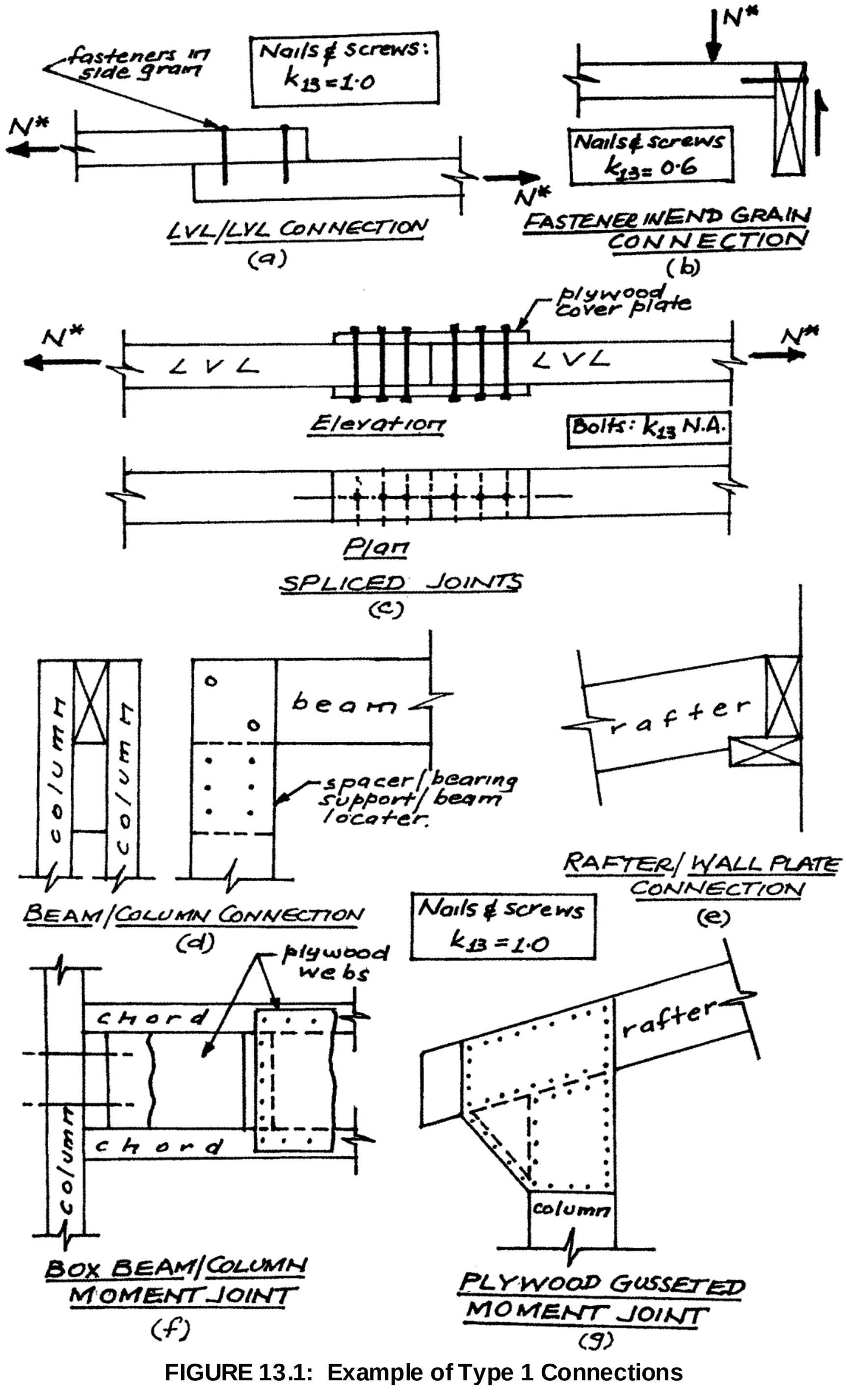


FIGURE 13.1: Example of Type 1 Connections

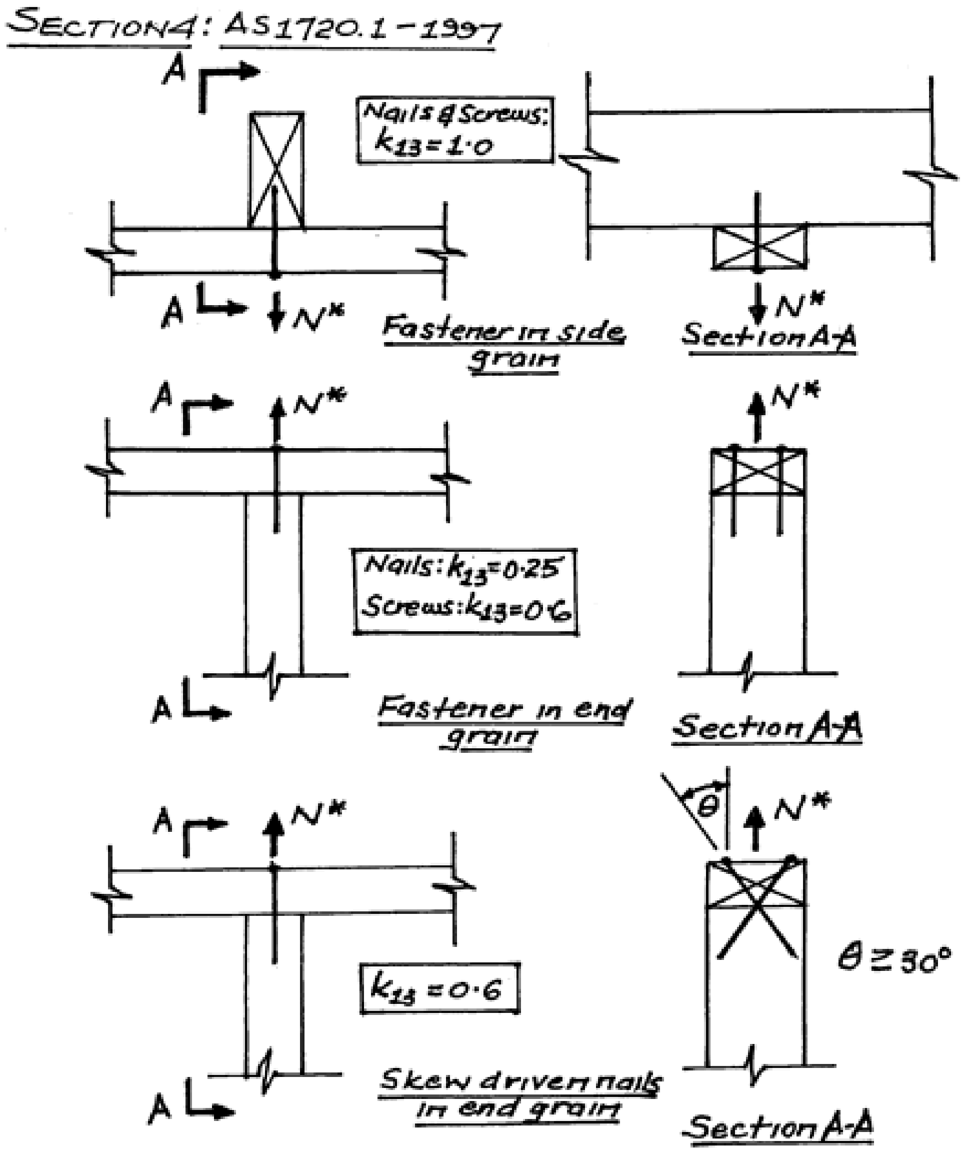


FIGURE 13.2: Examples of Type 2 Connections for nails and screws

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.

SECTION 4: AS1720.1-1997: Clauses 4.1.1 to 4.3.6

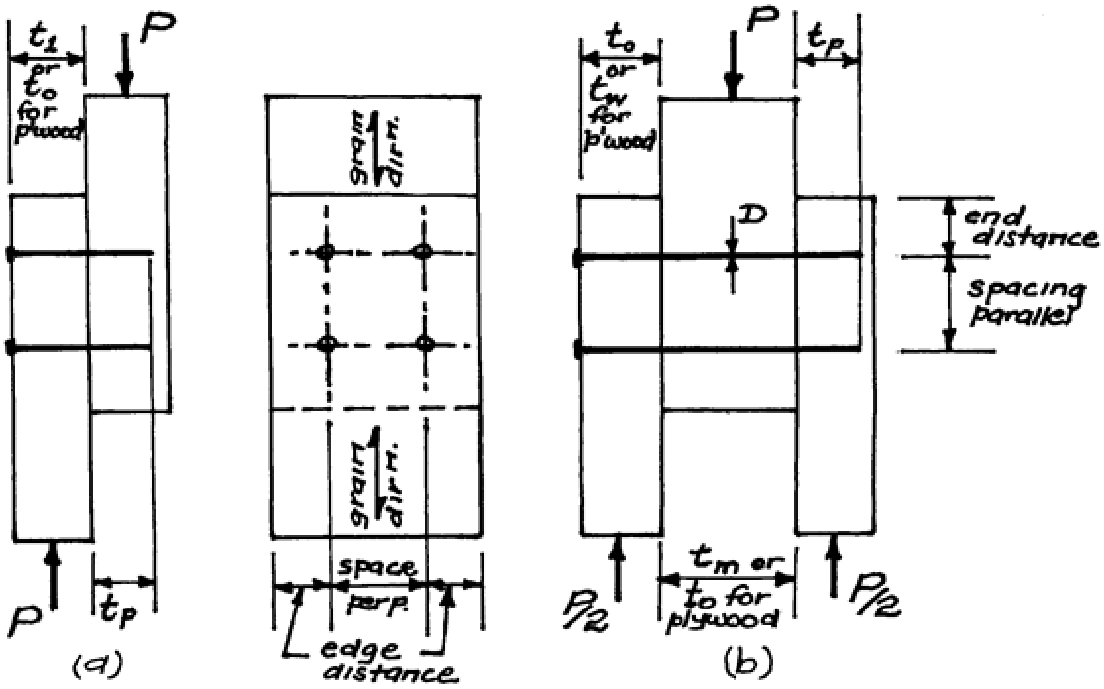


FIGURE 13.3: Two and Three member Type 1 nailed and screwed connections

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 dimeter 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**.

SECTION 4 : AS 1720.1-1997 : Clauses 4.4.1 to 4.5

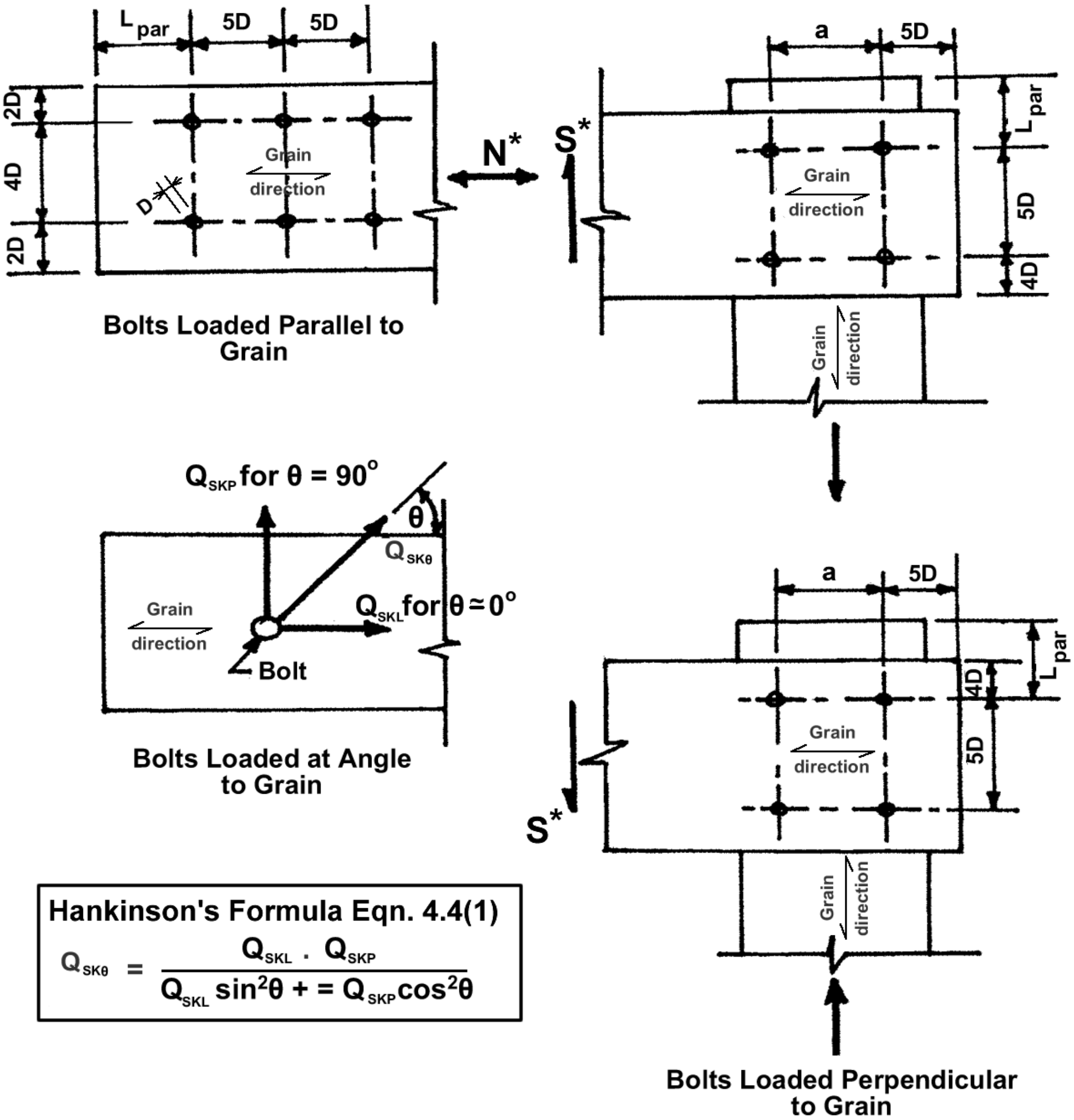


FIGURE 13.4: Critical bolt spacings and distances

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 **strength** 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).

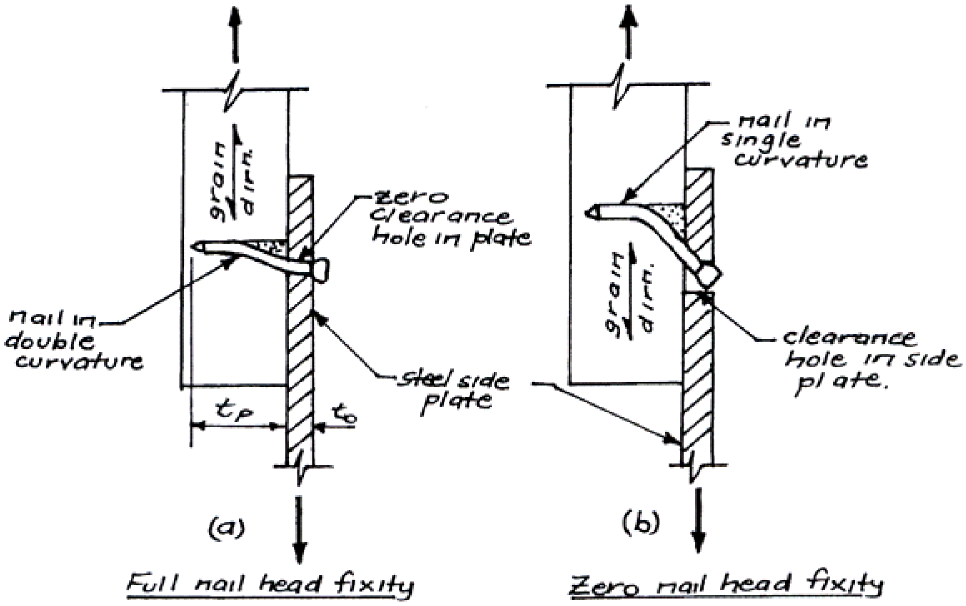


FIGURE 13.5: Nail head fixity

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 k_{16}

Multiple nail factor (k_{17})

Multiple nail factor (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 (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.

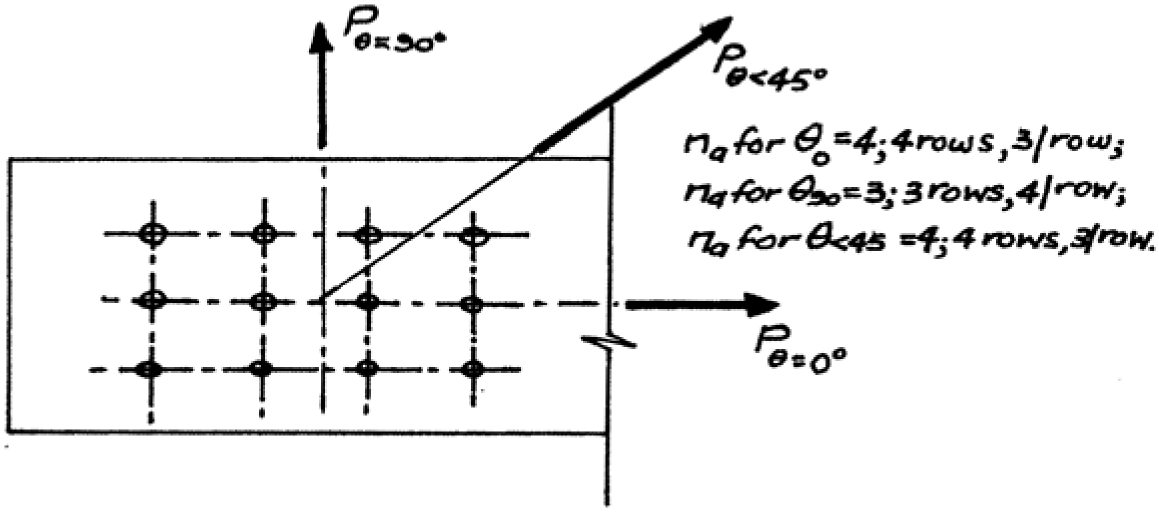


FIGURE 13.6 :Shows how rows are defined relative to applied load

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:

$$\Phi N_j \geq N^* \quad \textbf{(13.2)}$$

where

$$\Phi N_j = \Phi k_1 k_{13} k_{14} k_{16} k_{17} n Q_k \quad \textbf{(13.3)}$$

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

$$\Phi N_j = \mathbf{\Phi} k_{13} . \ell_p . n Q_k \quad \textbf{(13.4)}$$

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 (C*ℓ*.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:

$$\Delta = \left[\frac{44\,\boldsymbol{x}\,\boldsymbol{j}_{12}}{\boldsymbol{D}^{3.5}}\right]\left[\frac{\boldsymbol{Q^*}}{\boldsymbol{h}_{32}}\right]^2 \quad \boldsymbol{for}\,\Delta < 0.5\boldsymbol{mm} \tag{13.5}$$

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:

Δ = 0.5mm and Δ = 2.5mm

where

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**.

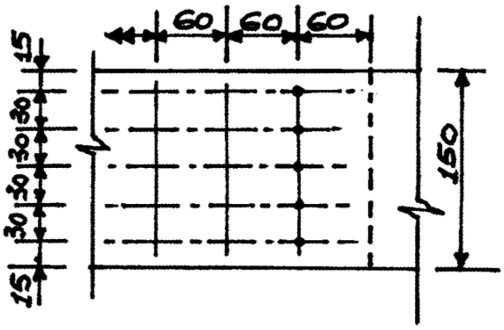
Distance	**Dimension**	**Minimum**	**Actual**
End distance	20D	56	60
Edge distance	5D	14	15
Along grain spacing	20D	56	60
Across grain spacing	10D	28	30

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:

$$n_r = 5$$



<u>**Connector Capacity Factors**</u>

- **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:

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

<u>**Number of Nails**</u>
From Equation 13.3:

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

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

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

<u>**Number of Rows**</u>
The **number of rows** of nails can be found from:

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

$$= \frac{42}{5}$$

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

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

Error! Reference source not found. shows the connection with the nailing pattern chosen.

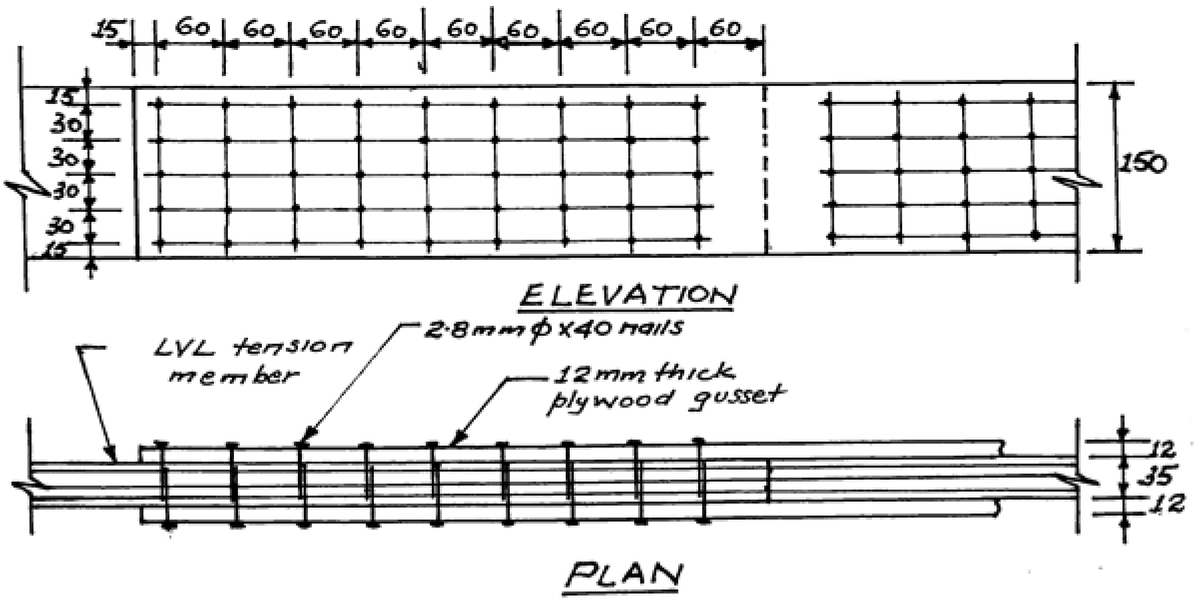


FIGURE 13.7: LVL / Plywood spliced joint

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:

$$\Phi N_j \geq N^* \tag{13.8}$$

where

$$\Phi N_j = \Phi k_1 k_{13} k_{14} k_{16} k_{17} n Q k \tag{13.9}$$

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:

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

where ΦN_j is the lesser of

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

OR

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

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 CHAPTER 10 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).

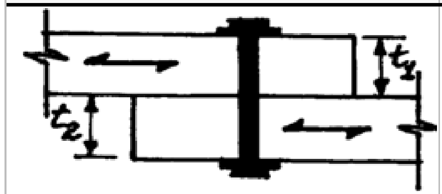
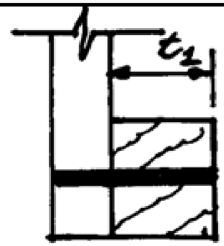
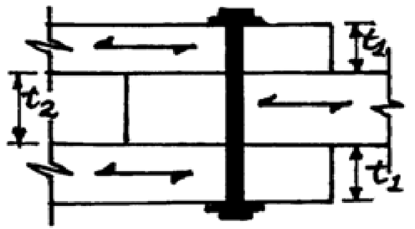
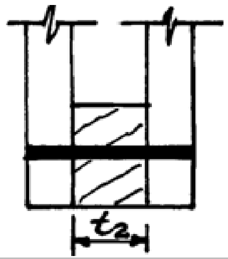
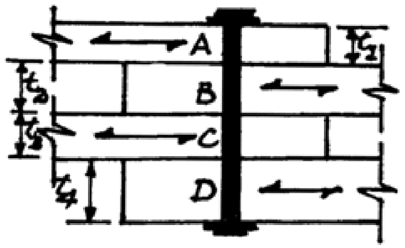
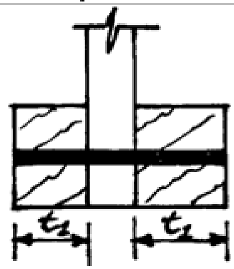
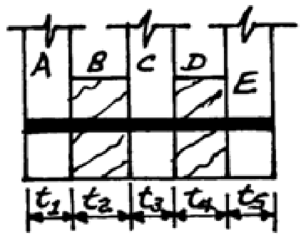
CHARACTERISTIC CAPACITIES & EFFECTIVE TIMBER THICKNESS FOR SINGLE BOLTS					
Parallel to Grain			Perpendicular to Grain		
Type of Joint	**Effective thickness (b_{eff})**	**System Capacities $Q_{sk\ell}$**	**Type of Joint**	**Effective thickness (b_{eff})**	**System Capacities Q_{skp}**
	b_{eff} = smaller 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.

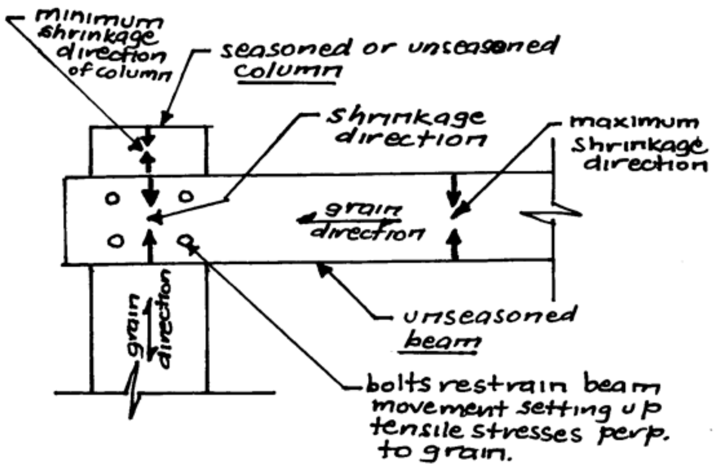


FIGURE 13.9: Lateral restraint stresses

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 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 b_{eff} to be determined, hence $Q_{sk\ell}$ and/or 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
    - b_{eff} for each **member/bolt interface**;

- **Evaluate modification factors** except for 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:**

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

where

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

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:

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

where ΦNj is the lesser of

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

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

$$\Phi N_j = \Phi.k_1.k_7.nf\text{"}pj.Aw \ (13.17)$$

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 members, for example, those of a **truss joint do not intersect at a 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.

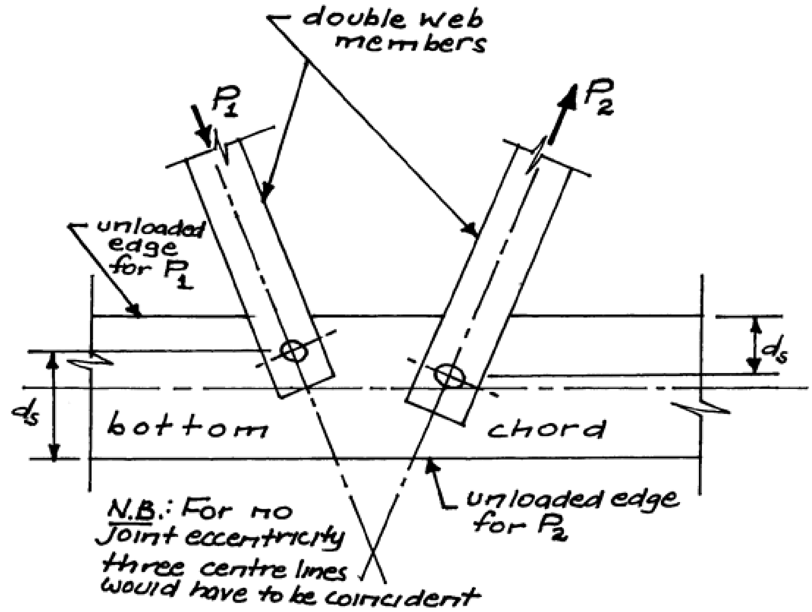


FIGURE 13.10: Eccentric joint

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.

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

where

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

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.**Error! Reference source not found.** for $\mathbf{d_s}$**.**

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.**

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

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

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 **Error! Reference source not found.** 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.

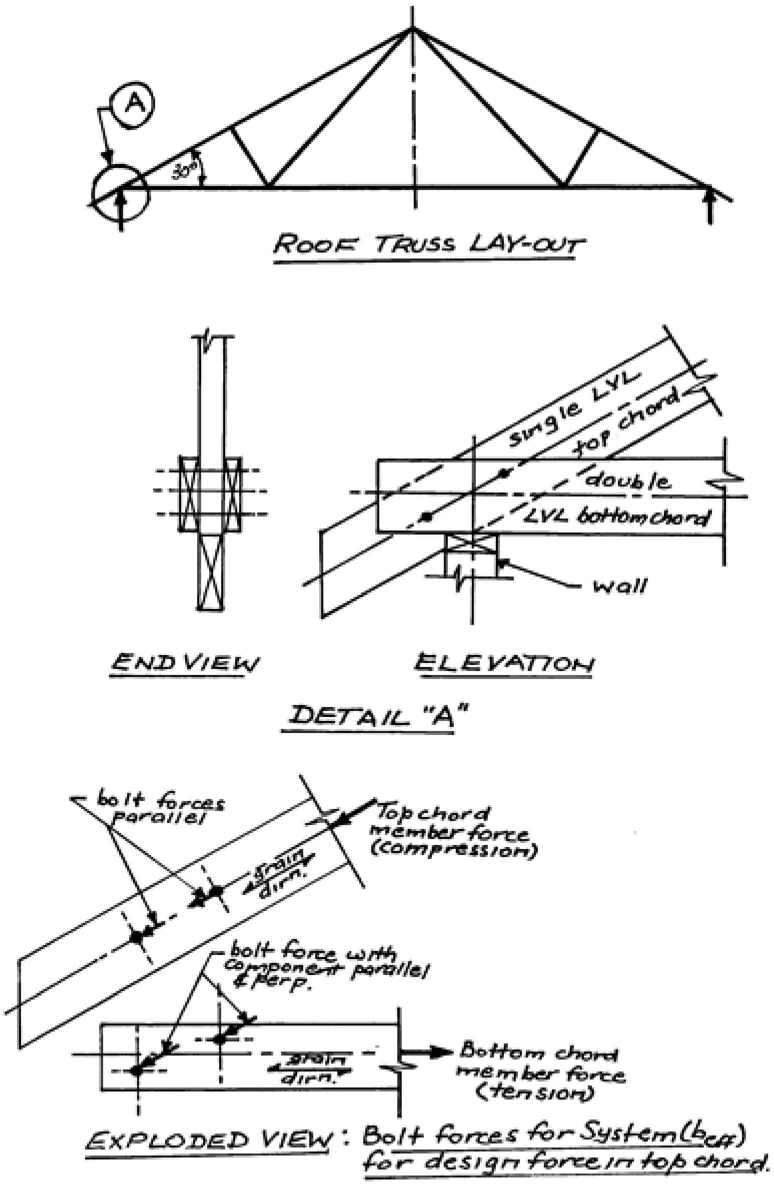


FIGURE 13.11: Bolted truss joint

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 **Error! Reference source not found.**. 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:**

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

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

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

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.**

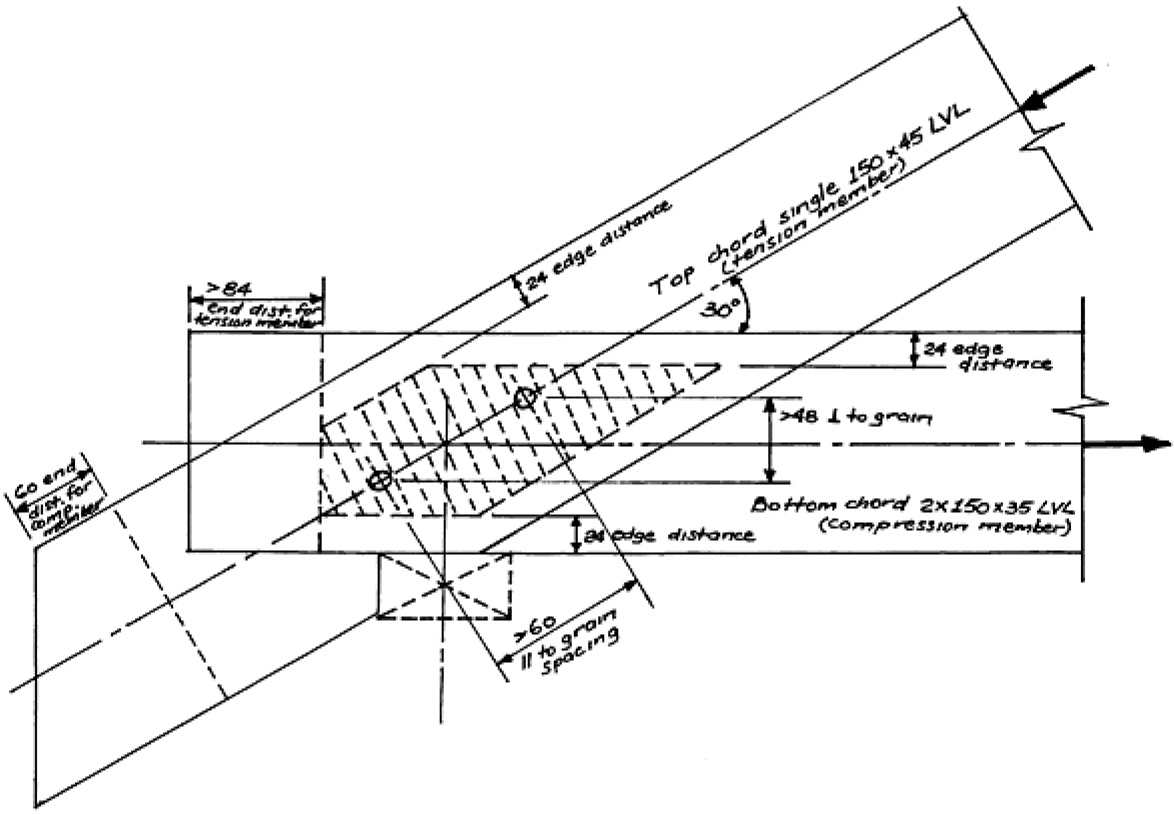


FIGURE 13.12: Edge, end and bolt spacings

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):

  $\mathbf{t_t} \geq \mathbf{3D_s}$

  where:

  $\mathbf{D_s}$ = **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.**

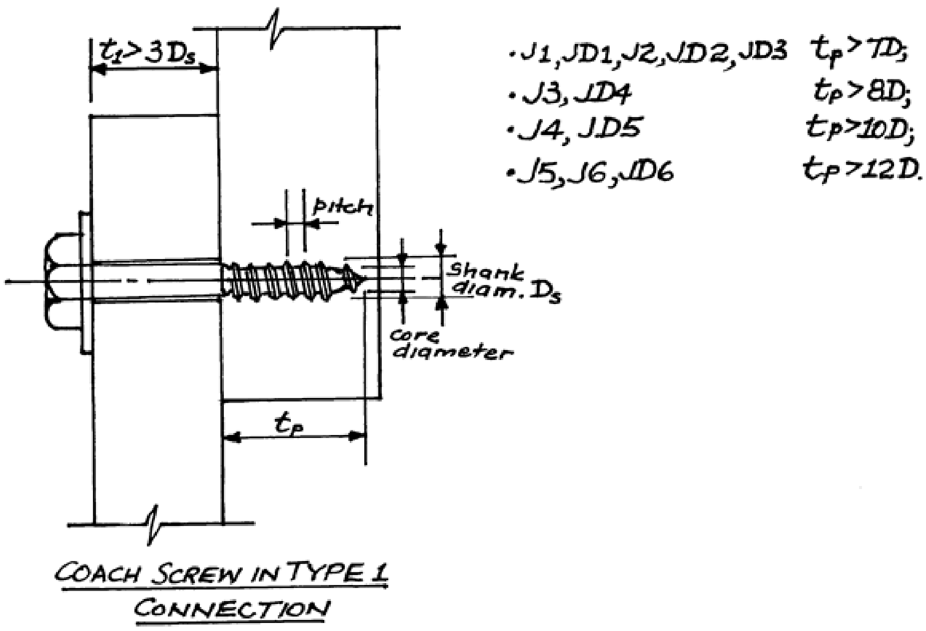


FIGURE 13.13: Coach screw depth of penetration and timber thickness

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:

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

where **ΦN_j** is the **lesser** of

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

Or

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

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

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

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 A1.13 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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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

CHAPTER 14

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:**

$$SPL = 10.\log_{10}\left(\frac{p}{p_{ref}}\right)^2 \quad \textbf{(14.1)}$$

where

p = **sound pressure;**
$\mathbf{p_{ref}}$ = **reference sound pressure of 20µPa**

Sound energy is related to SPL thus:

$$\left(\frac{p}{p_{ref}}\right)^2 = 10^{\,(spl-10)} \quad \textbf{(14.2)}$$

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.

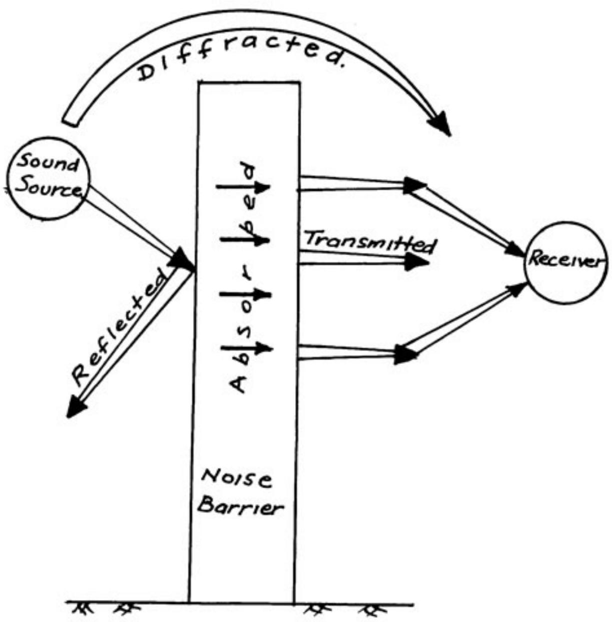

FIGURE 14.2: Sound response on meeting a barrier

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:

$$\textbf{TL} = \textbf{20 log (M.f)} - \textbf{47.2dB} \qquad \textbf{(14.3)}$$

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.

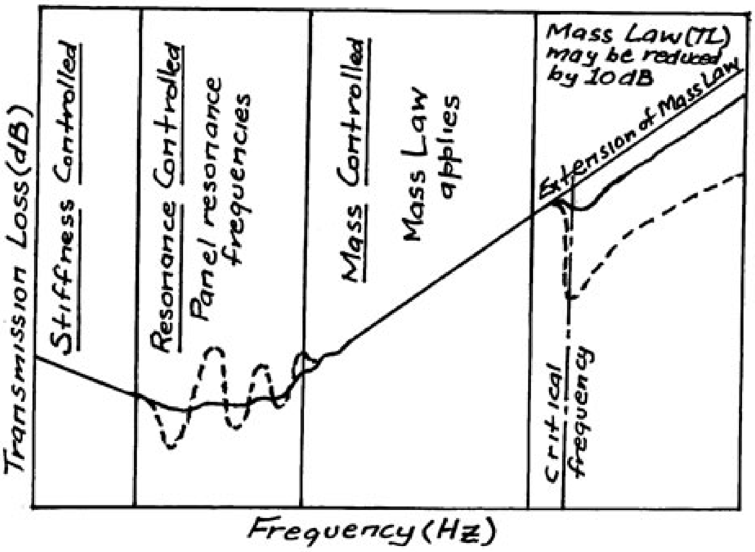

FIGURE 14.3: Shows various regions of performance for single leaf partition

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.

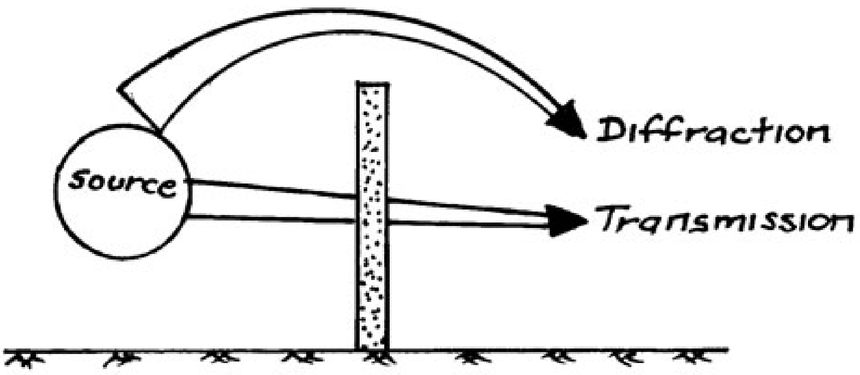

FIGURE 14.4: Sound paths

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.

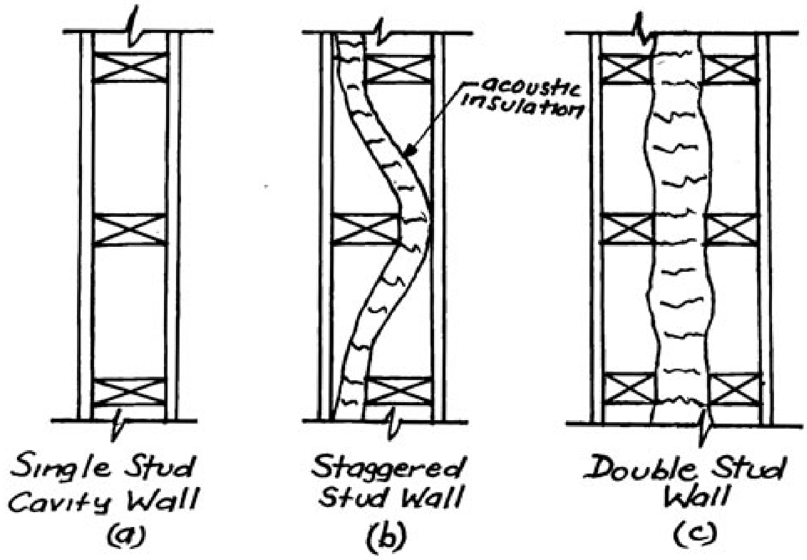

FIGURE 14.5: Types of cavity walls

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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CHAPTER 15

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**.

$$R_h = \frac{p_w}{p_s} x100 \qquad \textbf{(15.1)}$$

  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.

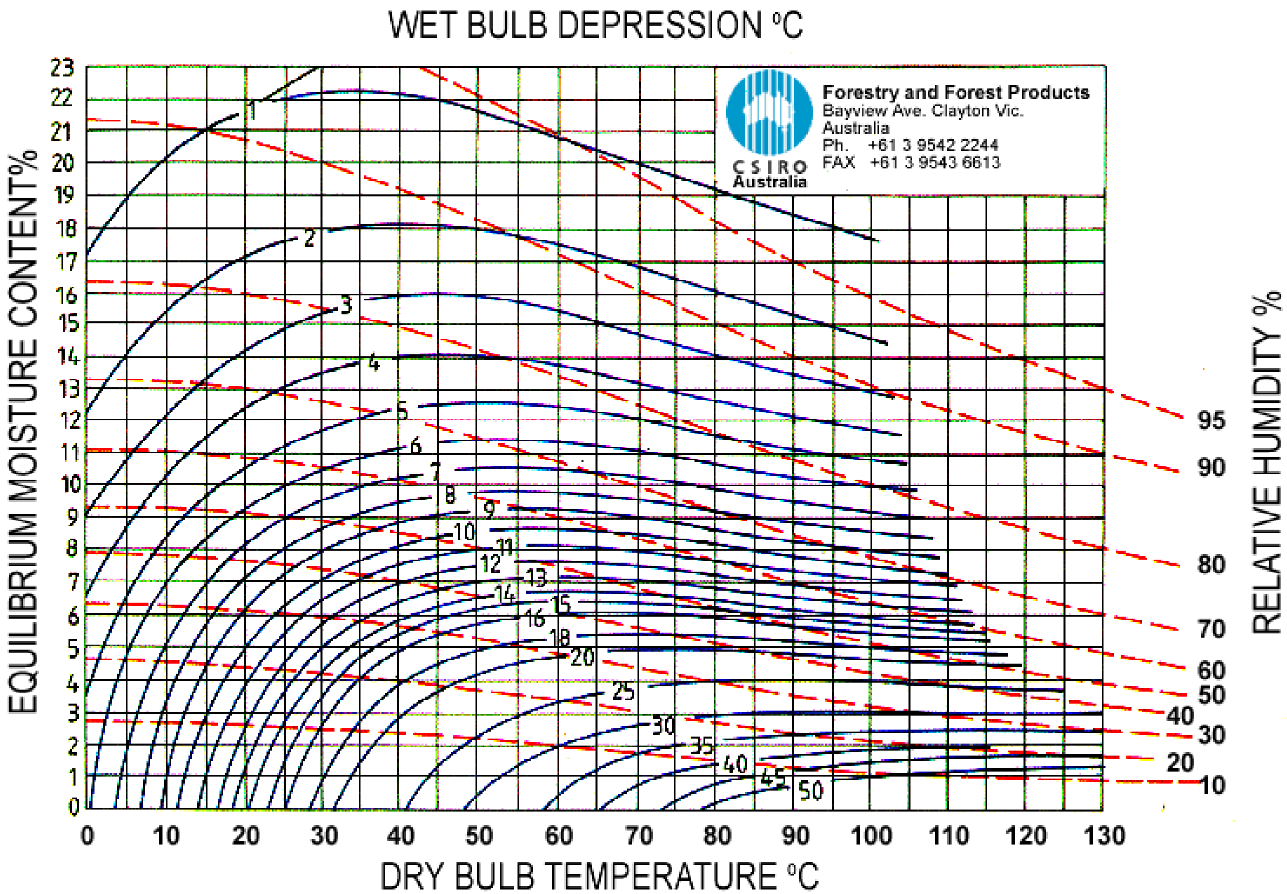

FIGURE 15.1: Equilibrium Moisture Content of Wood as a function of Dry Bulb Temperature, Wet Bulb Depression and Relative Humidity

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.

$$\mathbf{R} = \frac{\mathbf{1}}{\mathbf{k}}\,\mathbf{m^{\circ}C/W} \qquad \mathbf{(15.2)}$$

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:

$$r_i = \frac{\text{material thickness (metres)}}{\text{unit thermal conductivity}} = \frac{T_i}{k_i}$$

where T_i = **thickness** of the **barrier (m)**
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:

$$U = \frac{1}{R_t} \, W/m^{2\circ}C \qquad (15.4)$$

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.**

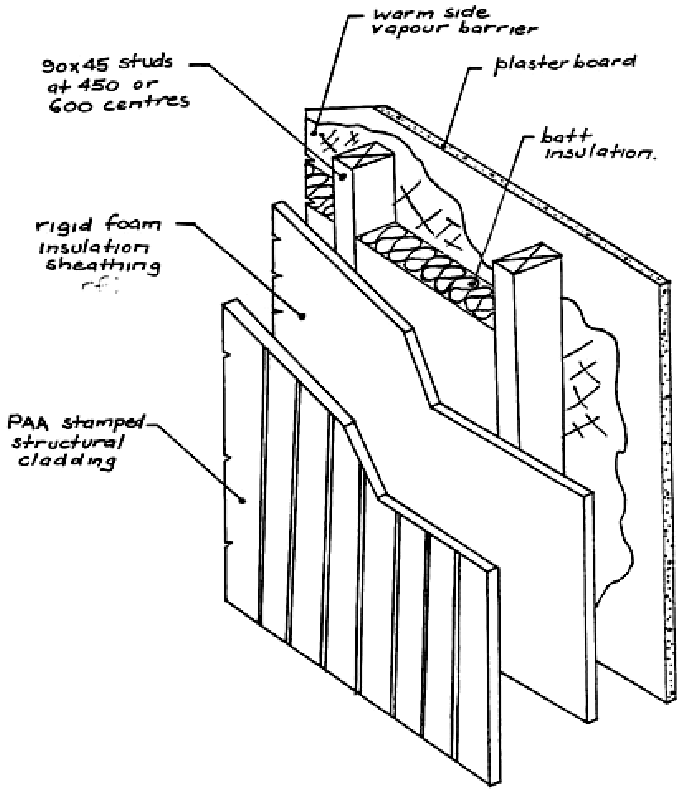

FIGURE 15.2: Sample Wall

$$\mathbf{R_t} = \mathbf{r_{oa}} + \frac{\mathbf{T_{ply}}}{\mathbf{k_{ply}}} + \mathbf{r_{rf}} + \frac{\mathbf{T_{batt}}}{\mathbf{r_{batt}}} + \mathbf{r_{vb}} + \frac{\mathbf{T_{pb}}}{\mathbf{r_{pb}}} + \mathbf{r_{ia}}$$

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**

$$R_t = 0.04 + \frac{0.012}{0.13} + 0.5 + \frac{0.095}{0.05} + 0 + \frac{0.012}{0.22} + 0.12$$
$$= 0.04 + 0.092 + 0.5 + 1.9 + 0 + 0.05 + 0.12$$

$$R_t = 2.25\,m^{2\circ}C/W$$

$$U = \frac{1}{R_t} = \frac{1}{2.25} \qquad \textbf{(15.5)}$$

$$U = 0.44\,W/m^{2\circ}C\ (\text{between studs})$$

If the heat flow **through** the **studs** was being **considered** $\mathbf{r_{stud} = T_{stud}/k_{timber}}$, which in this case would be **0.09/0.13 = 0.69**, would have to be **included in** the **calculation** for $\mathbf{R_t}$.

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.

REFERENCES CITED:

1. **Condensation**, CSIRO Building Technology file, 1995.

2. **Condensation Causes and Control**, APA The Engineered Wood Products Association, 1997.

3. **Modern Air Conditioning Heating and Ventilating Carrier**, W.H., Cherne, R.E., Grant, W.A., and Roberts, W.H., Pitman, 1959.

4. **Guide to the Insulation Regulations for Residential Buildings**, BCA Victoria Appendix – Part 6, 1992.

5. **Energy Efficient Design : theory and the real world,** Task 3 – Lightweight Construction and the Victorian thermal Insulation Regulations and Vichers, Williamson, T., Coldicut, S., Bennets, H., and Rees, J., The University of Adelaide, 1995.

6. **BCA 2007**, Class 2 to Class 9 Buildings, Volume One.

7. **Insulation Solutions to Enhance the Thermal Resistance of Suspended Timber Floor Systems in Australia**, Williamson, T., and Beauchamp, B., for Forest & Wood Products Research & Development Corporation. Project No. PN05, 1014. Web:www.fwprdc.org.au

CHAPTER 16

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.

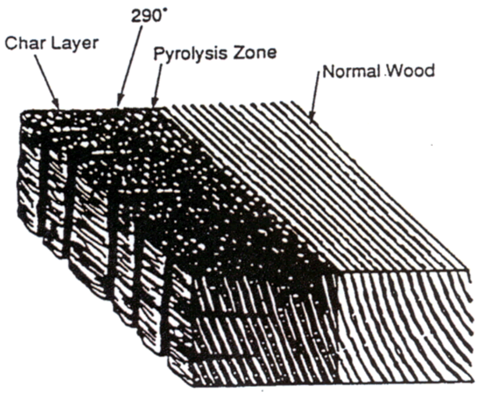

FIGURE 16.1: Zones of burning wood

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. 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 20** for **ignitability index** and from **0 best to 10 worst** for the **others.**

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**.

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.**

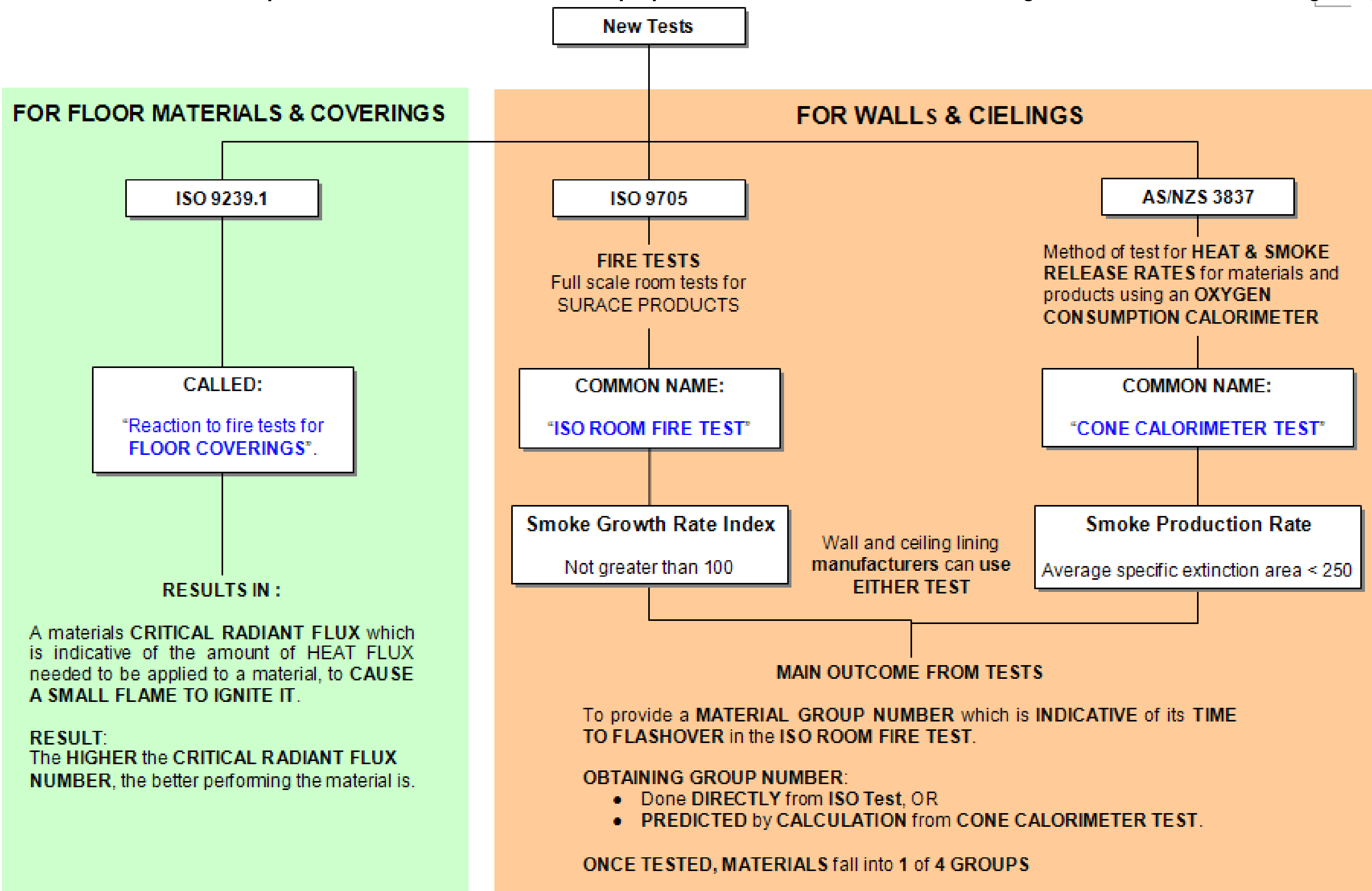

FIGURE 16.2: Summary of Floor, Wall and Ceiling Fire Tests

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)

16.3 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.

16.4 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:

$$c = dh/dt = 0.4 + (280/\rho)^2 \qquad \textbf{(16.1)}$$

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:

$$d_c = ct + 7.5 \quad (16.2)$$

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 d_c from all fire-exposed surfaces of the timber member as shown in FIGURE 16.3

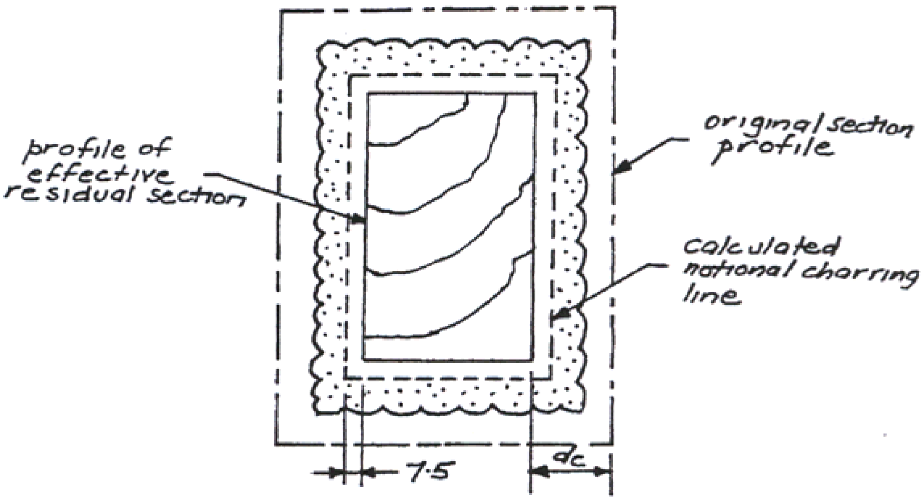

FIGURE 16.3: Shows loss of section due to charring

- 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 16.4 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 16.4: Minimum beam thicknesses

When **determining** the **strength** of the **effective residual section** take k_1 = **5 hours.**

16.5 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.

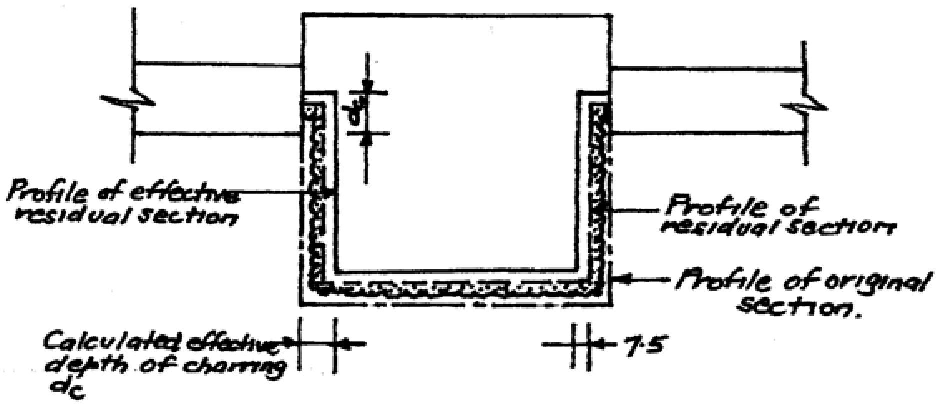

FIGURE 16.4: Charring at junction with fire proof barrier

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:

$$T_p = t_i + t_m \qquad (16.3)$$

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.***

16.6 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.**

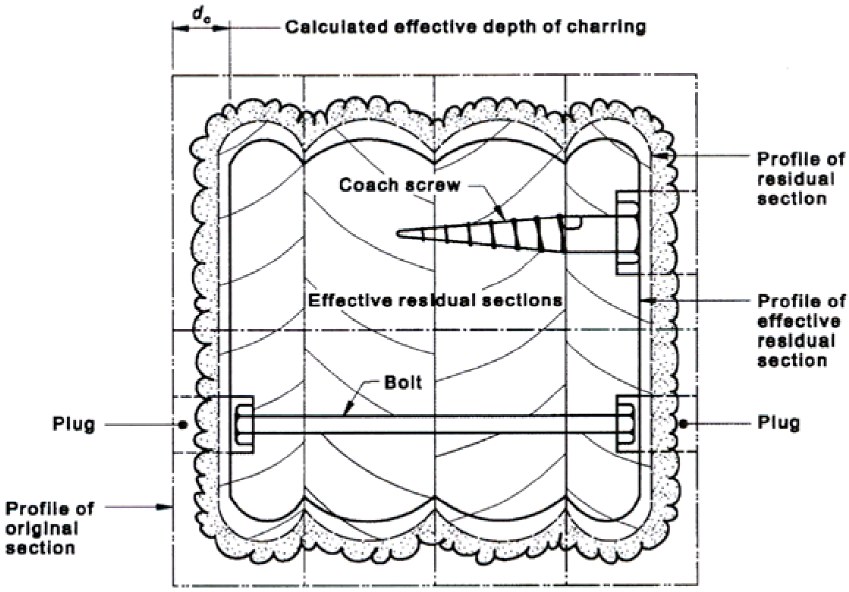

FIGURE 16.5: Fire protected connectors

16.7 Plywood Performance

TABLE 16.5 provides the most recent available data on **early fire hazard** test data for both **untreated and fire retardant treated plywood (which is no longer acceptable).** To obtain **current information** on this topic the designer is **referred to companies involved** in **fire retardant treatment or** contact **EWPAA.**

PLYWOOD (UNTREATED)						
Face veneer's Common Name	**Botanical Name**	**Ignitability Index (0-20)**	**Spread of Flame Index (0-10)**	**Heat Evolved Index (0-10)**	**Smoke Developed Index (0-10)**	**Report Reference**
Australian Red Cedar	Toona australis	13	9	9	9	E.B.S. 5/10/76 E.4248
Australian Red Cedar (grooved)	Toona australis	13	8	7	2	E.B.S 5/10/78 E.4250
Blackbean	Castanospermum australe	13	9	10	3	E.B.S 5/10/78 E.4238
Coachwood	Ceratopetalum apetalum	15	8	8	2	E.B.S. 5/10/78 E.4235
Hickory Ash (grooved)	Flindersia ifflaiana	13	8	9	3	E.B.S. 5/10/78 E.4249
Klinkii pine	Aurancaria hunsteinii	15	8	10	4	E.B.S. 5/10/78 E.4245
Lauan	Parashorea Spp. Shorea Spp.	14	8	10	3	E.B.S. 5/10/78 E.4244
Meranti	Shorea Spp.	14	8	10	2	E.B.S. 5/10/78 E.4240
Pacific Maple	Shorea Spp.	14	8	10	2	E.B.S. 5/10/78 E.4240
Queensland Maple	Flindersia brayleyana	13	8	8	2	E.B.S. 5/10/78 E.4239
Queensland Walnut	Endiandra palmerstoni	14	8	10	3	E.B.S. 5/10/78 E.4241
Radiata Pine	Pinus radiata	14	8	9	2	E.B.S. 5/10/78 E.4237
Radiata Pine (scorched and brushed surface)	Pinus radiata	14	7	7	2	E.B.S. 5/10/78 E.4246
Sapele	Entandrophragma cylindricum	13	8	8	2	E.B.S. 5/10/78 E.4243
Silver Ash	Flindersia bourjotiana	13	8	9	3	E.B.S. 5/10/78 E.4242
Tasmanian Oak	Mixture of: Euc. obliqua Euc. delegatensis Euc. regnans	14	8	8	2	E.B.S. 5/10/78 E.4236
Teak	Tectona grandis	14	8	10	3	E.B.S. 5/10/78 E.4247
Victorian Ash	Mixture of: Euc. regnans Euc. delegatensis	14	8	8	2	E.B.S. 5/10/78

TIMBERS AND PLYWOODS TREATED WITH FIRE RETARDANTS						
Timber Species	**Treatment**	**Ignitability Index (0-20)**	**Spread of Flame Index (0-10)**	**Heat Evolved Index (0-10)**	**Smoke Developed Index (0-10)**	**Report Reference**
Hoop Pine	Retardant Impregnated	0	0	0	2	(a)
Redwood	Surface coated with 3 coats of fire retardant	14	0	4	5	E.B.S. 23/2/79 E.4362
	Surface coated with 1 coat of fire retardant	14	8	6	4	E.B.S. 23/2/79 E.4361
Western Red Cedar	Surface Coated with 3 coats of fire retardant	14	0	5	4	E.B.S. 23/2/79 E.4360
	Surface coated with 1 coat of fire retardant	15	8	6	4	E.B.S. 23/2/79
Yellow Walnut	Retardant impregnated	0	0	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 16.5: Early Fire Hazard Data for Untreated and Fire Retardant Treated Plywood

TABLE 16.6 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 16.6: Fire Resistance Level for Structural Stability for Non-load Bearing Plywood

The **thicker radiata pine panels** are based on a char rate of 0.8mm per minute as opposed to the comparable thicknesses to the Douglas fir plywood having a char rate of 0.65mm per minute.

TABLE 16.7 gives **early fire hazard properties** for 12mm thick Carter Holt Harvey radiata pine plywood in various conditions. The early fire hazard properties of the untreated, uncoated plywood align closely with those obtained in **Australian Laboratories** for untreated radiata pine plywood.

Plywood Condition	Ignitability Index Ig	Spread of Flame Index SFI	Heat Evolved Index HEI	Smoke Developed Index SDI
Untreated, Uncoated Ecoply Clearline	14	7	7	3
H3 CCA treated, Ecoply Structural Plywood	14	6	4	3
Fire retardant treated plywood	Paint or fire retardant coating cannot be used to make a surface comply with the requirements			
Plywood coated with intumescent coating				

TABLE 16.7: Early fire hazard properties of 12mm thick Ecoply plywood

A Warrington Fire Research (Aust) Pty Ltd Report No. 45980 on:

> "Assessment of Solid Timber, Plywood & Timber Veneers on MDF and Particleboard for use as Wall & Ceiling Lining with respect to the Building Code of Australia Specification C1.10a."

for the Timber Development Association, Sydney, 2006 gave the following results for plywood.

Plywood Species	Minimum Thickness (mm)	Group No.	Average Specific Extinction Area (m^2/kg)
Pine, Radiata – Pinus Radiata	6mm or greater	3	<250
Lauan – Shorea agsaboensis	6mm or greater	3	<250

16.8 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.

16.8.1 Durability of the Adhesive

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**.*

16.8.2 Durability of the Timber Veneers

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**

<u>**Decay:**</u> **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).

<u>**Wood**</u> 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 **(**CHAPTER 15**) shows timber moisture content relative to temperature and humidity.**

<u>**Surface Moulds:**</u> **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.

16.9 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 16.8. 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 16.8: Hazard Class Selection Guide for Preservative Treatments
(from AS1604 Specification for preservative treatment, Part 3: Glued wood veneer-based products)

CHAPTER 17

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.

<u>**Where paint systems**</u> 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.

<u>**Rigid paint systems**</u>, 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.

<u>**Edge sealing**</u> 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.

<u>**The back or unexposed face of plywood**</u> should be **left unsealed** if possible **to prevent moisture being trapped** within the panel.

<u>**Orientation**</u> 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.

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