Design Guide

Structural Plywood and LVL

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ewpaa

Disclaimer

The advice provided in this publication is general in nature and is intended as a guide only. Whilst the information in this guide was accurate at the time of publication, it is the users' responsibility to confirm alignment with current standards and/or building codes. It is also the users' responsibility to confirm that they are using the current version of this publication.

You should consult the manufacturer of each product for the latest, most accurate information about the product and its use. You should check with the designer or specifier for the project you are working on to ensure the product and its method of installation are suitable for that project.

We provide this publication to you on condition that, to the maximum extent permitted by law, we will not be liable for any claim you might make for damage arising directly or indirectly from your use of the information in the publication.

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Preface

This Guide 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 Guide 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.

It is hoped to also provide the user with, under the one cover as nearly as practicable, all the design information required for the solution of a range of problems. Australian Standards, e.g. AS 1720.1, 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 has been attained and the design concepts have been fully digested, the contents of the Guide will provide a more than adequate solution procedure.

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 of prime importance, and therefore, is the 'centre of gravity' of the Design Guide. 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 Guide and are also available from the EWPAA website.

Happy and fruitful designing,

Mick McDowall

January 2007
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Acknowledgements – Original Version

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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 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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This Manual has been produced for the design and construction industry by the Engineered Wood Products Association of Australasia Ltd. The information, opinions, advice and recommendations have been prepared with due care and are aimed at providing useful background data to assist professionals in the design of safe and economical structures.

Whilst every effort has been made to ensure that this Manual is in accordance with current technology, the document is not intended to be exhaustive in its coverage of all issues that affect structural plywood and LVL design and construction. The Engineered Wood Products Association of Australasia Ltd accepts no responsibility for errors or omissions from the Manual, or for structural plywood and LVL design or construction done or omitted to be done in reliance on this Manual.

Edited By

James MacGregor, BE, MIE(Aust), CPEng. sometime Production Manager, Merchandising Manager, Fabrication Manager, Market Manager – Tubemakers, Market Development Engineer and Consulting Timber Engineer

Acknowledgements – Revised Version

Since its initial appearance in January 2007, the Design Guide has undergone only one minor revision. This was in July 2009 and was mainly cosmetic and involved correcting obvious errors identified by users. During the ten years of existence of the Design Guide, significant changes have occurred to:

- Relevant Codes and Standards;
- Characteristic properties of the timber resource used in Plywood and LVL manufacture.

In order to satisfactorily incorporate the many changes due to the above resulted in 2015 calling upon the professional expertise of Mrs Claudia Roder, a German educated Timber Engineer, to weave her exceptional skills throughout the Guide.

A considerable portion of the revision task was necessarily devoted to the reworking of Design Tables and worked examples to include any changes to the material characteristic values. Revision of the 2007 version of the Building Code of Australia also required significant researching to identify necessary changes. In 2014 it was subjected to a partial name change to become the National Construction Series Sub-titled the Building Code of Australia for Volumes One and Two and the Plumbing Code of Australia Volume Three. The Building Code of Australia being such a comprehensive publication precludes an in-depth coverage of its contents in such a document as this Design Guide. The Building Code of Australia's "appropriate performance requirements" requires special attention by the Designer when entering a new area of endeavour.

Mrs Roder's single minded and knowledgeable approach to the Guide's revision was a necessary ingredient to ensure a satisfactory completion of the revision process and her effort is most appreciated.

Mr James MacGregor, Consulting Engineer is thanked for his contributions regarding the re-working of the Box Beam example.

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Last but by no means least thanks to Mrs Sonia Moore, Executive Assistant (retired) for happily keeping all concerned fully informed on the "state of play" regarding the revision and Mrs Clair Borg for carrying on the good work.

Mick McDowall

Consulting Timber Engineer

June 2018

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 manufacturing members whose products carry the EWPAA / JAS-ANZ brand is available from the EWPAA website.

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.

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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.

Veneer ribbon exiting lathe after peeling

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.

Sanding, Trimming and Branding

LVL Production Line

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.

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

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.

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 series AS/NZS 2269 Plywood - Structural. These Standards set 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 series 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 Standards AS/NZS 2269.

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.

> 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 Plywood Structural Specifications AS/NZS 2269.0 and listed in Table 5-1 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.

> 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 5-3.

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

Face grain usually runs in length direction of panel
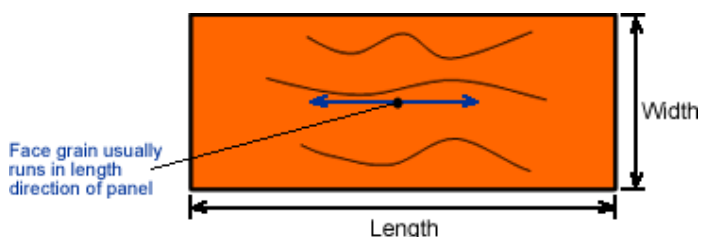
Other panel lengths are available including 1800, 2100, and 2250. Panel widths of 900 mm are also available from some manufacturers. Panel lengths may be intended to suit a particular end application. For example, 2250 mm length plywood is manufactured as flooring to suit the standard floor joist spacing 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 (for additional options please refer to AS/NZS 2269.0). 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	$\pm$7%
Sanded sheets over 7.5 mm thick and up to 17.5 mm thick	$\pm$4%
Sanded sheets over 17.5 mm thick	$\pm$3%
Sanded sheets over 25 mm thick	not specified
Unsanded sheets	as per sanded sheet tolerances plus an additional tolerance of +0.3 mm
Length and Width	+1.5 mm

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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
Loaded sheets:	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
Manufacturer	ACB- Plywood Manufacturer
Formaldehyde emission class	E_0
Treatment branded in accordance with AS/NZS1604.3	AS/NZS 1604.3-H3

Table **2-2**: Specification Guide for Plywood

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 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 Plywood and Blockboard for Interior Use) and Exterior Plywood (manufactured to AS/NZS 2271 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 in conjunction with structural applications given in this manual.

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 series. These Standards set 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/NZS 4357 and is suitable for use in all permanent structural applications.

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.

> 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

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.

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 4357.0, 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	$\frac{Length\ (mm)\ x\ Width\ (mm)}{3500\ x\ Thickness\ (mm)}$
Squareness of Section	1 mm in 100 mm
Moisture Content	8 – 15 %

Table 3-1: Tolerances for LVL

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 Certified.
Grade	Proprietary (or F-grade)
Treatment level	AS/NZS 1604.4 - H3
Formaldehyde emission class	E_0
Limitations of use if applicable	Floor members only

Table 3-2**: Specification Guide for LVL**

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 length. 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.

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.

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

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	‖	0.016	0.009	0.006	0.011
		⊥	0.021	0.008	0.005	0.011
15	5	‖	0.016	0.008	0.004	0.010
		⊥	0.022	0.010	0.009	0.013
17	7	‖	0.017	0.009	0.005	0.011
		⊥	0.022	0.010	0.010	0.014
22	9	‖	0.017	0.012	0.004	0.012
		⊥	0.018	0.010	0.008	0.014

Direction ‖ 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 mm

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.

Early fire hazard indices 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 indices 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 indices 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 coefficient of thermal expansion of plywood is in the range 4.5×10^{-6} to $7 \times 10^{-6} mm/mm/°C$.

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 $k = 0.1154 \frac{W\,m}{m^2\,°C}$ for softwood timbers is sufficiently accurate for determining the overall coefficient of heat transmission (U value) of a construction assembly.

Thermal Resistance

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

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. Acoustic 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 coefficient 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 coefficients 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 Coefficients 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 glue line 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, alkalis, 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.

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 (refer to AS/NZS 2269.0 Plywood Structural Specification). These values must be modified in accordance with the in-service factors in AS1720.1.

Stress Grade	Characteristic values, MPa					
	Bending (f'_b)	Tension (f'_t)	Panel Shear (f'_s)	Compression in the plane of the sheet (f'_c)	Short duration average modulus of elasticity (E)	Short duration average modulus of rigidity (G)
F34	90	54	6.0	68	21500	1075
F27	70	45	6.0	55	18500	925
F22	60	36	5.5	45	16000	800
F17	45	27	5.1	36	14000	700
F14	36	22	4.8	27	12000	625
F11	31	18	4.5	22	10500	525
F8	25	15	4.2	20	9100	455
F7	20	12	3.9	15	7900	395
F5	14	9.6	3.7	12	6900	345
F4	12	7.7	3.4	9.6	6100	305

Table 5-1: Structural Plywood - Characteristic Properties for F-Grades

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

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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 A5.
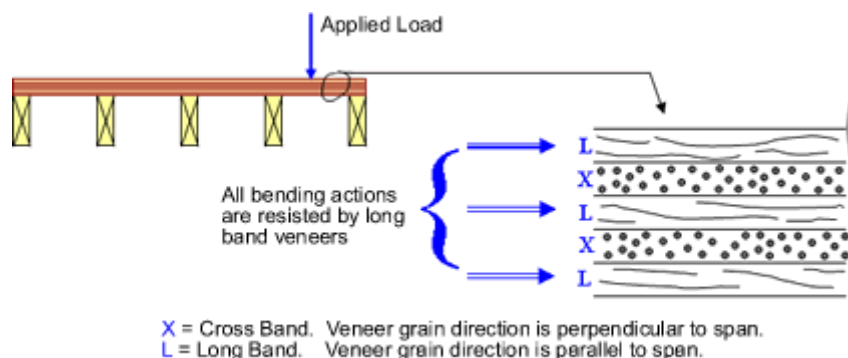
Figure 5-1: Parallel Ply Theory

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.

Section Properties – Standard Plywood Layups

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.
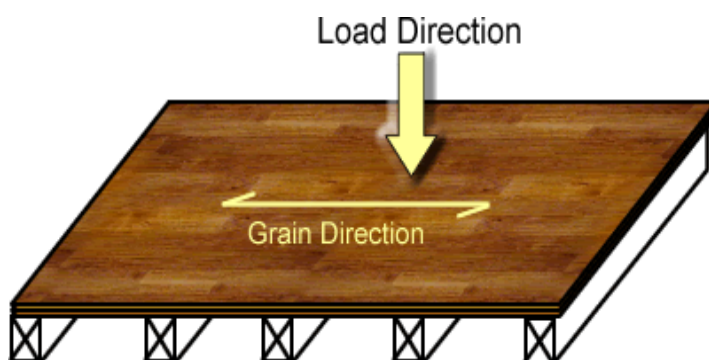
TABLE 5-3 gives section properties for plywood loaded normal to the plane of the plywood panel. These are for standard thicknesses and constructions of structural plywood specified in AS/NZS 2269 together with some additional thicknesses made by some manufacturers. A method for calculating the section modulus (Z) and second moment

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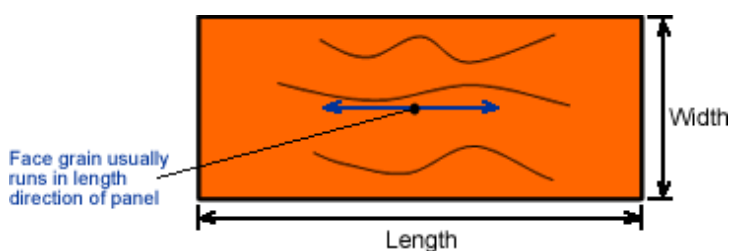
of area (I), for structural plywood loaded normal to the face, is detailed in Appendix I of AS1720.1 Timber Structures Code, see Appendix A5 of this manual.

Load Distribution Width

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 furthest 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 (see 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 ($f_b^{'} \, Z$) 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:

$$A_s \qquad = \frac{2}{3} bt \text{ (derived from the basic beam shear equation)}$$

where: $\quad b \qquad$ = load distribution width (refer TABLE 5-2);

and: $\quad t \qquad$ = 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:

$$A_s \qquad \text{= perimeter of loaded area x full thickness of the panel}$$

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, by the use of an assembly factor g_{19} in the calculation of both interlamina and punching shear capacity.

TABLE 5-4 provides interlamina shear strengths ($0.4 \times f_s^{'} \ A_s$) 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.
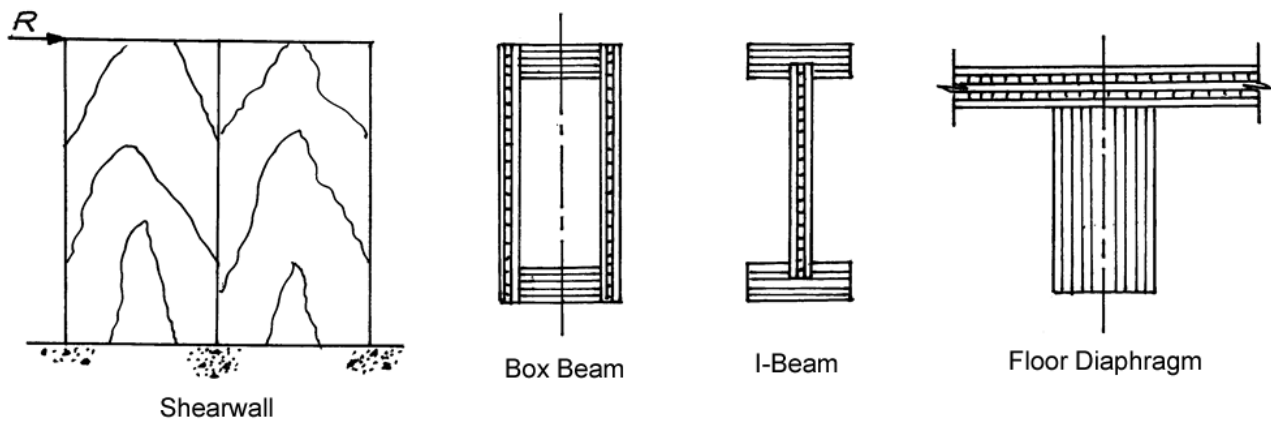
Figure 5-2: Structural Plywood Loaded in its Plane

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 $As = \frac{2}{3}td$ and for local shear $As = dt$, 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 (t_p), Second Moment of Area (I_p) and Section Modulus (Z_p)

Nominal Thickness	Nominal Mass	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_9)	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	3.0	7.3	3.3	1.5	0.5	0.4
6	3.6	6-15-3	1.5/3.0/1.5	3.0	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	1.0
7.5	4.5	7.5-25-3	2.5/2.5/2.5	5.0	34	9.0	2.5	2.3	1.0
9	5.4	9-15-5	1.5/1.5/3.0/1.5/1.5	6.0	45	10.0	3	17.0	5.3
9	5.4	9-15-5	1.5/2.4/1.5/2.4/1.5	4.5	47	10.0	4.8	22.0	6.5
9	5.4	9-30-3	3.0/3.0/3.0	6.0	60	13.0	3	4.0	1.5
12	7.2	12-15-5	1.5/3.0/3.0/3.0/1.5	6.0	85	14.5	6	60.0	13.0
12	7.2	12-24-5	2.4/2.4/2.4/2.4/2.4	7.2	115	19.0	4.8	33.0	8.3
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	120.0	19.0
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	85.0	15.0
15	9	15-30-5	3.0/3.0/3.0/3.0/3.0	9.0	225	29.5	6	65.0	13.0
17	10.2	17-15-7	1.5/3.0/2.4/3.0/2.4/3.0/1.5	7.8	220	25.5	9	190.0	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	120.0	19.0
18	10.8	18-15-7	1.5/3.0/3.0/3.0/3.0/3.0/1.5	9.0	270	29.5	9	230.0	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	125.0	19.0
19	11.4	19-24-7	2.4/3.0/2.4/3.0/2.4/3.0/2.4	9.6	360	38.0	9	190.0	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	200.0	26.5
19	11.4	19-30-7	3.0/2.4/3.0/2.4/3.0/2.4/3.0	12.0	450	46.5	7.2	155.0	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	12.0	565	51.5	9.6	300.0	33.5
21	12.6	21-30-7	3.0/3.0/3.0/3.0/3.0/3.0/3.0	12.0	555	52.5	9	240.0	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	15.0	900	70.5	9.6	380.0	38.0
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	791	65.0	10.8	395.3	41.3
27	16.2	27-30-9	3.0/3.0/3.0/3.0/3.0/3.0/3.0/3.0/3.0	15.0	1110	81.0	12	580.0	52.5
31	18.6	31-24-13	2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4	16.8	1590	100.0	14.4	1020.0	74.0
33	19.8	33-30-11	3.0/3.0/3.0/3.0/3.0/3.0/3.0/3.0/3.0/3.0/3.0	18.0	1940	115.0	15	1150.0	81.0

Notes:

1. The subscript "p" in I_p and Z_p 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 x f'$_s$ A$_s$ (N/mm width) | | | | | | |
| | | Face Grain Parallel to Span = f'$_b$Z$_{parallel}$ | | | | | | | 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	F34
4.5	4.5-15-3	83	102	119	149	198	231	297	10	12	14	18	24	28	36	5	5	6	6	7	7	7
6	6-15-3	133	164	191	239	318	371	477	38	47	54	68	90	105	135	7	7	8	8	9	10	10
7	7-24-3	208	257	299	374	498	581	747	25	31	36	45	60	70	90	8	8	9	10	10	11	11
7.5	7.5-25-3	225	279	324	405	540	630	810	25	31	36	45	60	70	90	8	9	10	10	11	12	12
9	9-15-5	250	310	360	450	600	700	900	133	164	191	239	318	371	477	10	11	12	12	13	14	14
9	9-15-5	250	310	360	450	600	700	900	163	202	234	293	390	455	585	10	11	12	12	13	14	14
9	9-30-3	325	403	468	585	780	910	1170	38	47	54	68	90	105	135	10	11	12	12	13	14	14
12	12-15-5	363	450	522	653	870	1015	1305	325	403	468	585	780	910	1170	13	14	15	16	18	19	19
12	12-24-5	475	589	684	855	1140	1330	1710	208	257	299	374	498	581	747	13	14	15	16	18	19	19
15	15-15-7	563	698	810	1013	1350	1575	2025	475	589	684	855	1140	1330	1710	17	18	19	20	22	24	24
15	15-24-7	688	853	990	1238	1650	1925	2475	375	465	540	675	900	1050	1350	17	18	19	20	22	24	24
15	15-30-5	738	915	1062	1328	1770	2065	2655	325	403	468	585	780	910	1170	17	18	19	20	22	24	24
17	17-15-7	638	791	918	1148	1530	1785	2295	663	822	954	1193	1590	1855	2385	19	20	22	23	25	27	27
17	17-24-7	838	1039	1206	1508	2010	2345	3015	475	589	684	855	1140	1330	1710	19	20	22	23	25	27	27
18	18-15-7	738	915	1062	1328	1770	2065	2655	738	915	1062	1328	1770	2065	2655	20	22	23	24	26	29	29
18	18-30-7	1038	1287	1494	1868	2490	2905	3735	475	589	684	855	1140	1330	1710	20	22	23	24	26	29	29
19	19-24-7	950	1178	1368	1710	2280	2660	3420	663	822	954	1193	1590	1855	2385	21	23	24	26	28	30	30
19	19-24-9	988	1225	1422	1778	2370	2765	3555	663	822	954	1193	1590	1855	2385	21	23	24	26	28	30	30
19	19-30-7	1163	1442	1674	2093	2790	3255	4185	538	667	774	968	1290	1505	1935	21	23	24	26	28	30	30
21	21-24-9	1288	1597	1854	2318	3090	3605	4635	838	1039	1206	1508	2010	2345	3015	24	25	27	29	31	34	34
21	21-30-7	1313	1628	1890	2363	3150	3675	4725	738	915	1062	1328	1770	2065	2655	24	25	27	29	31	34	34
25	25-30-9	1763	2186	2538	3173	4230	4935	6345	950	1178	1368	1710	2280	2660	3420	28	30	32	34	37	40	40
25	25-30-9	1625	2015	2340	2925	3900	4550	5850	1033	1281	1488	1860	2480	2893	3720	28	30	32	34	37	40	40
27	27-30-9	2025	2511	2916	3645	4860	5670	7290	1313	1628	1890	2363	3150	3675	4725	30	32	35	37	40	43	43
31	31-24-13	2500	3100	3600	4500	6000	7000	9000	1850	2294	2664	3330	4440	5180	6660	35	37	40	42	45	50	50
33	33-30-11	2875	3565	4140	5175	6900	8050	10350	2025	2511	2916	3645	4860	5670	7290	37	40	42	45	48	53	53

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	$EI \times 10^3$ 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	50	58
7	7-24-3	273	315	360	420	480	555	645	19	22	25	29	34	39	45
7.5	7.5-25-3	309	357	408	476	544	629	731	21	24	28	32	37	43	49
9	9-15-5	410	473	540	630	720	833	968	155	179	204	238	272	315	366
9	9-15-5	428	494	564	658	752	870	1011	200	231	264	308	352	407	473
9	9-30-3	546	630	720	840	960	1110	1290	36	42	48	56	64	74	86
12	12-15-5	774	893	1020	1190	1360	1573	1828	546	630	720	840	960	1110	1290
12	12-24-5	1047	1208	1380	1610	1840	2128	2473	300	347	396	462	528	611	710
15	15-15-7	1547	1785	2040	2380	2720	3145	3655	1092	1260	1440	1680	1920	2220	2580
15	15-24-7	1866	2153	2460	2870	3280	3793	4408	774	893	1020	1190	1360	1573	1828
15	15-30-5	2048	2363	2700	3150	3600	4163	4838	592	683	780	910	1040	1203	1398
17	17-15-7	2002	2310	2640	3080	3520	4070	4730	1729	1995	2280	2660	3040	3515	4085
17	17-24-7	2594	2993	3420	3990	4560	5273	6128	1092	1260	1440	1680	1920	2220	2580
18	18-15-7	2457	2835	3240	3780	4320	4995	5805	2093	2415	2760	3220	3680	4255	4945
18	18-30-7	3413	3938	4500	5250	6000	6938	8063	1138	1313	1500	1750	2000	2313	2688
19	19-24-7	3276	3780	4320	5040	5760	6660	7740	1729	1995	2280	2660	3040	3515	4085
19	19-24-9	3458	3990	4560	5320	6080	7030	8170	1820	2100	2400	2800	3200	3700	4300
19	19-30-7	4095	4725	5400	6300	7200	8325	9675	1411	1628	1860	2170	2480	2868	3333
21	21-24-9	5142	5933	6780	7910	9040	10453	12148	2730	3150	3600	4200	4800	5550	6450
21	21-30-7	5051	5828	6660	7770	8880	10268	11933	2184	2520	2880	3360	3840	4440	5160
25	25-30-9	8190	9450	10800	12600	14400	16650	19350	3458	3990	4560	5320	6080	7030	8170
25	25-30-9	7200	8308	9494	11077	12659	14637	17011	3597	4151	4744	5534	6325	7313	8499
27	27-30-9	10101	11655	13320	15540	17760	20535	23865	5278	6090	6960	8120	9280	10730	12470
31	31-24-13	14469	16695	19080	22260	25440	29415	34185	9282	10710	12240	14280	16320	18870	21930
33	33-30-11	17654	20370	23280	27160	31040	35890	41710	10465	12075	13800	16100	18400	21275	24725

5.6. Structural Plywood - Design Procedures

Limit State Design to AS 1720.1

The design capacity of structural plywood designed in accordance with the limit states design format of AS 1720.1, 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.

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:

$$R_d > S^*$$

where

R_d = design capacity of the plywood member

S^* = design action effect, e.g. bending moment, M^*, shear force, V^*, etc.

and

$$R_d = \phi\, k_{mod} f_o^{'}\, X$$

where

ϕ = capacity factor

k_{mod} = product of relevant modification factors (e.g. 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_3, j_6, and g_{19}, as appropriate). Guidance on serviceability limit states are given in Appendix B of AS 1720.1.

5.7. Strength & Stiffness limit states design capacities

Loading Normal to the Plane of the Plywood Panel

FIGURE 5-3 shows loading normal to the plane of the plywood panel.
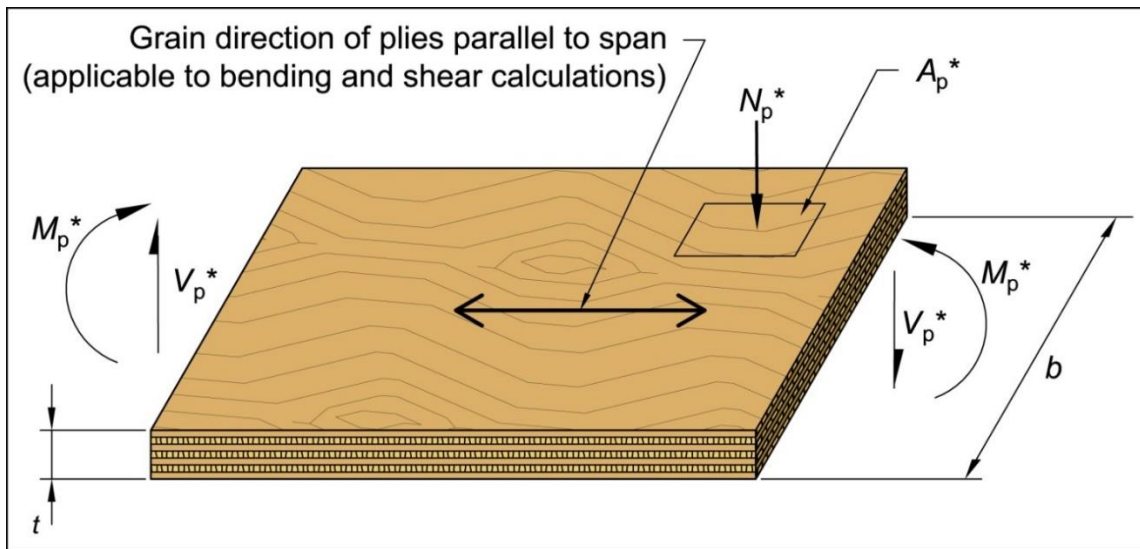
Figure 5-3**: Notation for bearing and shear normal to the face of the plywood panel and for flatwise bending plywood**

Strength Limit State

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied when	AS 1720.1 Reference
Bending	M_p^*	$M_{d,p} = \phi\, k_1\, k_{19}\, g_{19}\, f_b'\, Z_p$	$M_{d,p} \geq M_p^*$	Cl. 5.4.2
Shear	V_p^*	$V_{d,p} = \phi\, k_1\, k_{19}\, g_{19}\, f_s'\, A_s$	$V_{d,p} \geq V_p^*$	Cl. 5.4.3
Bearing	N_p^*	$N_{d,p} = \phi\, k_1\, k_7\, k_{19}\, g_{19}\, f_p'\, A_p$	$N_{d,p} \geq N_p^*$	Cl. 5.4.4

where:

M_p^*, V_p^*, N_p^* = Design action effect in bending, shear and bearing respectively

$M_{d,p}, V_{d,p}, N_{d,p}$ = Design capacity in bending, shear and bearing respectively

ϕ = Capacity factor for plywood

k_1 = Duration of load strength modification factor

k_7 = Length and position of bearing modification factor

k_{19} = Moisture content 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 $= \frac{I_p}{y_p}$

A_s = shear plane area $= \frac{2}{3}(bt)$ for shear in bending

   = full shear area for local (punching) shear.

A_p = bearing area under the design load.

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Serviceability Limit State

Calculated bending deflection $\times\ j_2 \times j_6 \times g_{19} \leq deflection\ limit$ Cl. 5.4.5

<div align="center">where:</div>

$$j_2 \quad = \text{Duration of load creep modification factor}$$
$$j_6 \quad = \text{Moisture content stiffness modification factor}$$
$$g_{19} \quad = \text{Plywood assembly modification factor}$$

Loading in Plane of the Plywood Panel

FIGURE 5-4 shows loading in the plane of the plywood.
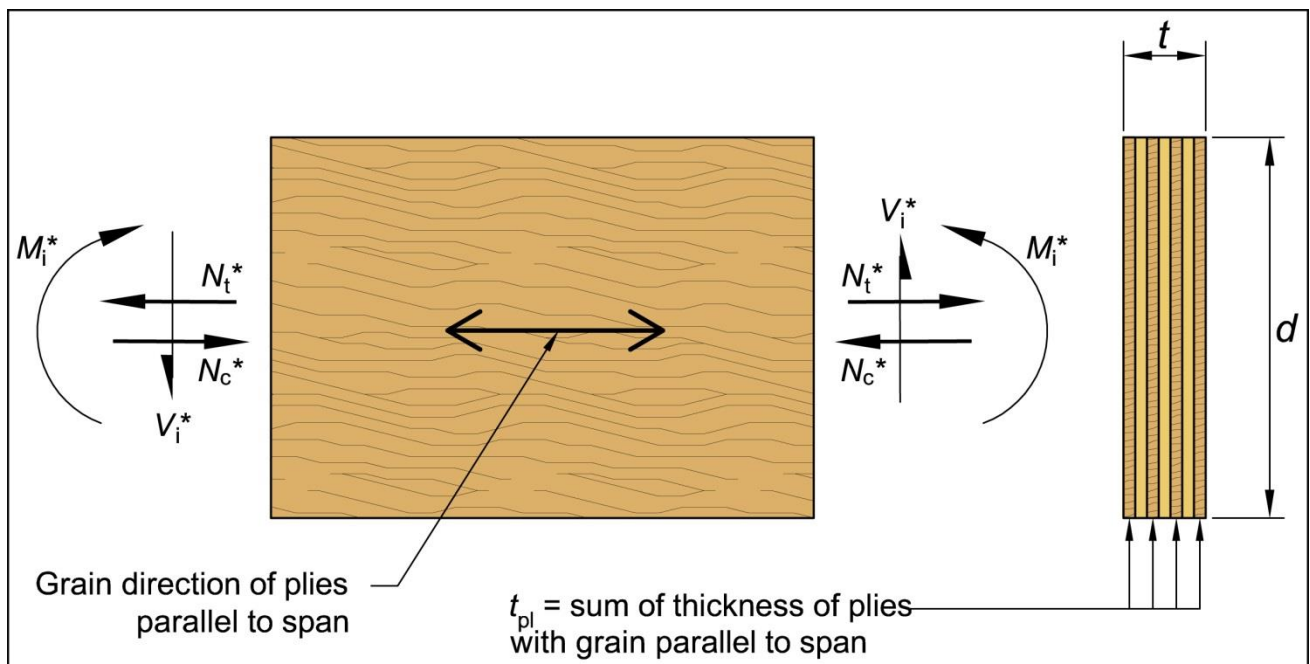
Figure 5-4: Notation for shear, compression and tension acting in the plane of a plywood panel and for edgewise bending

Strength Limit State

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied When:	AS 1720.1 Reference
Bending	M_i^*	$M_{d,i} = \phi\, k_1 k_{12} k_{19}\, g_{19}\, f_b' Z_i$	$M_{d,i} \geq M_i^*$	Cl. 5.5.2
Shear	V_i^*	$V_{d,i} = \phi\, k_1 k_{12} k_{19}\, g_{19}\, f_s' A_s$	$V_{d,i} \geq V_i^*$	Cl. 5.5.3
Tension	N_t^*	$N_{d,t} = \phi\, k_1 k_{19} g_{19}\, f_t' A_t$	$N_{d,t} \geq N_t^*$	Cl. 5.5.4
Compression	N_c^*	$N_{d,c} = \phi\, k_1\, k_{12} k_{19}\, g_{19}\, f_c' A_c$	$N_{d,c} \geq N_c^*$	Cl. 5.5.5

where:

$M_i^*, V_i^*, N_t^*, N_c^*$ = Design action effect in edgewise bending, shear, tension and compression in the plane of the plywood panel, respectively.

$M_{d,i}, V_{d,i}, N_{d,t}, N_{d,c}$ = 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 content 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 $= \dfrac{t_{pl}\, d^2}{6}$

A_s = shear plane area $= \dfrac{2}{3}(dt)$ for shear in bending

= (dt) for localised shear where

d = overall depth of plywood

t = full thickness of plywood

A_t, A_c = Effective cross-sectional area

= $t_{pl} \times d$ for load applied parallel sum of the thickness of veneers with grain parallel to span

= $t \times d$ for load applied at 45° to plywood grain direction

= t_{pl}

Serviceability Limit State:

(Calculated bending deflection $\times j_2 \times j_6 \times g_{19}$) + (Calculated shear deflection $\times j_2 \times j_6 \times g_{19}$) $\leq$ deflection limit

Cl. 5.5.6 & 5.5.7

where:

j_2 = Duration of load creep modification factor

j_6 = Moisture content stiffness modification factor

g_{19} = Plywood assembly modification factor

5.8. Factors

Capacity Factor, ϕ

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, to structural materials, is based on current knowledge of product structural performance, intended structural application and material reliability. The

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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		Capacity Factor, ϕ for Structural Plywood (AS/NZS 2269.0)
Category 1	Structural members for houses for which failure would be unlikely to affect an area* greater than 25 m² OR Secondary members in structures other than houses	0.95
Category 2	Primary structural members in structures other than houses; OR Elements in houses for which failure would be likely to affect an area* greater than 25m²	0.85
Category 3	Primary structural members in structures intended to fulfil an essential service or post disaster function	0.75

NOTES: Refer to definitions: secondary structural element to Cl. 1.7.2.22 AS1720; primary structural element in Cl. 1.7.2.18 AS 1720.

* In this context, area should be taken as the plan area.

Table 5-6**: Capacity Factor, ϕ**

k_1 – Duration of Load Strength Modification Factor

The 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 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. Please refer to Appendix G, AS1720.1.

LOAD DURATION FACTORS FOR TYPICAL LOAD COMBINATIONS FOR STRENGTH LIMIT STATE

Type of load (action)	Load combinations as per AS 1170.0	k_1*	
		For strength of Solid timber	For strength of joints using laterally loaded fasteners[†]
Permanent action (dead load)	1.35 G	0.57	0.57
Permanent and short-term imposed actions	1.2 G + 1.5 Q		
(a) Roof live load— Distributed		0.94	0.77
(b) Roof live load— Concentrated		0.97	0.86
(c) Floor live loads— Distributed		0.80	0.69
(d) Floor live loads— Concentrated		0.94	0.77
Permanent and long-term[†] imposed action	1.2 G + 1.5 ψ_lQ	0.57	0.57
Permanent, wind and imposed action	1.2 G + W_u + ψ_cQ	1.00	1.14
Permanent and wind action reversal	0.9 G + W_u	1.00	1.14
Permanent, earthquake and imposed action	G + E_u + ψ_cQ	1.00	1.14
Fire	G + ψ_lQ	0.94	0.77

* Typical values of k_1 for various load combinations are given in Table G1, Appendix G AS1720.1.
[†] For the strength of joints with fasteners loaded in withdrawal and for the strength of steel in joints, $k_1 = 1.00$.

Table 5-7: Duration of Load Strength Modification Factor

Factor k_6 – Ambient temperature factor

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 $k_6 = 0.9$.

Factor k_7 – Factor for length and position of bearing

The k_7 bearing factor given in TABLE 5-8 may be used to increase the bearing capacity perpendicular to the grain where rectangular bearing areas are located 75mm or more from the end of a piece of timber. The length of bearing shall be measured parallel to the grain of the loaded member.

Length of bearing of Member (mm)	12	25	50	75	125	150 or more
Value of k_7	1.75	1.40	1.20	1.15	1.10	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

The 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 I of AS 1720.1 I2.2)
Diaphragms with Lateral Edges Supported & Subjected to Uniform Edge Forces.

k_{12} - Bending

F14 k_1 = 1.0   (Non conservative for k_1=1.0, F grades=F14)

Nominal Thickness (mm)	ID Code	Face grain direction is horizontal (θ=0°) Depth of Web (mm)										Face grain direction is vertical (θ=90°) 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	1.00	0.57	0.41	0.36	0.29	0.25	0.24	0.23	0.22	1.00	1.00	0.94	0.62	0.53	0.38	0.30	0.28	0.27	0.25
7	7-24-3	1.00	1.00	1.00	0.70	0.59	0.42	0.32	0.30	0.28	0.26	1.00	1.00	1.00	1.00	1.00	0.65	0.45	0.40	0.36	0.31
7.5	7.5-25-3	1.00	1.00	1.00	0.77	0.65	0.45	0.34	0.31	0.29	0.26	1.00	1.00	1.00	1.00	1.00	0.71	0.49	0.43	0.38	0.33
9	9-30-3	1.00	1.00	1.00	1.00	0.85	0.57	0.41	0.36	0.33	0.29	1.00	1.00	1.00	1.00	1.00	0.94	0.62	0.53	0.47	0.38
12	12-24-5	1.00	1.00	1.00	1.00	1.00	1.00	0.84	0.70	0.61	0.48	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.80	0.62
15	15-30-5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.84	0.64	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.86
17	17-24-7	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.77	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

k_{12} - Compression

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

Nominal Thickness (mm)	ID Code	Face grain direction is horizontal (θ=0°) Depth of Web (mm)										Face grain direction is vertical (θ=90°) 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	0.43	0.33	0.26	0.23	0.23	0.21	0.21	0.21	0.21	0.20	0.66	0.46	0.31	0.26	0.25	0.23	0.22	0.21	0.21	0.21
7	7-24-3	0.76	0.51	0.34	0.28	0.26	0.23	0.22	0.22	0.21	0.21	1.00	0.82	0.48	0.36	0.32	0.27	0.24	0.23	0.22	0.22
7.5	7.5-25-3	0.84	0.56	0.36	0.29	0.27	0.24	0.22	0.22	0.21	0.21	1.00	0.92	0.52	0.38	0.34	0.28	0.24	0.24	0.23	0.22
9	9-30-3	1.00	0.72	0.43	0.33	0.30	0.26	0.23	0.23	0.22	0.21	1.00	1.00	0.66	0.46	0.40	0.31	0.26	0.25	0.24	0.23
12	12-24-5	1.00	1.00	0.87	0.58	0.50	0.37	0.29	0.27	0.26	0.24	1.00	1.00	1.00	0.76	0.64	0.45	0.34	0.31	0.29	0.26
15	15-30-5	1.00	1.00	1.00	0.79	0.66	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
17	17-24-7	1.00	1.00	1.00	0.95	0.80	0.53	0.39	0.35	0.32	0.28	1.00	1.00	1.00	1.00	1.00	0.70	0.48	0.42	0.38	0.32

k_{12} - Shear

F8, k_1 = 1.0   (Non conservative for k_1>1.0, F grades <F8)

Nominal Thickness (mm)	ID Code	Face grain direction is horizontal (θ=0°) Depth of Web (mm)										Face grain direction is vertical (θ=90°) 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	0.86	0.70	0.59	0.55	0.54	0.52	0.51	0.51	0.51	0.51	1.00	1.00	0.84	0.69	0.65	0.58	0.55	0.54	0.53	0.52
7	7-24-3	1.00	0.99	0.72	0.62	0.60	0.55	0.53	0.52	0.52	0.51	1.00	1.00	1.00	0.96	0.86	0.70	0.62	0.59	0.57	0.55
7.5	7.5-25-3	1.00	1.00	0.75	0.64	0.61	0.56	0.54	0.53	0.52	0.52	1.00	1.00	1.00	1.00	0.92	0.73	0.63	0.60	0.58	0.56
9	9-30-3	1.00	1.00	0.86	0.70	0.66	0.59	0.55	0.54	0.53	0.52	1.00	1.00	1.00	1.00	1.00	0.84	0.69	0.65	0.62	0.58
12	12-24-5	1.00	1.00	1.00	1.00	1.00	0.86	0.70	0.66	0.63	0.59	1.00	1.00	1.00	1.00	1.00	1.00	0.86	0.79	0.73	0.66
15	15-30-5	1.00	1.00	1.00	1.00	1.00	1.00	0.82	0.75	0.70	0.64	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.86	0.75
17	17-24-7	1.00	1.00	1.00	1.00	1.00	1.00	0.91	0.82	0.76	0.68	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.82

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.
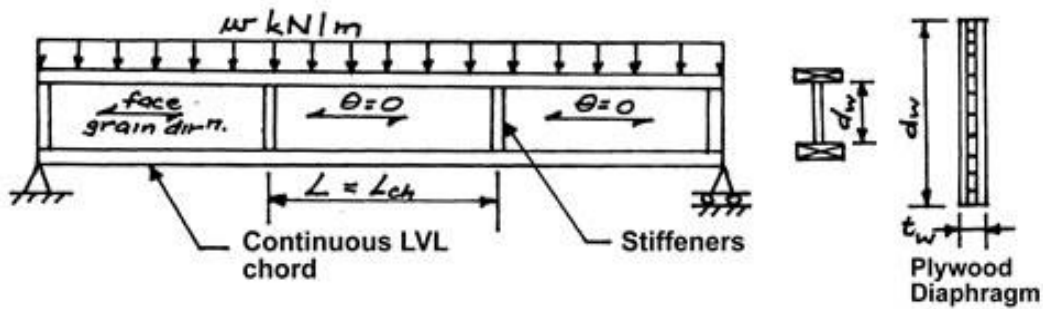
Figure 5-5: Diaphragm Buckling Parameters

k_{19} – Moisture content factor

The 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

The 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 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 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 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 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	1.2
	5 ply or more perpendicular to span	1.0
	Parallel to span	1.0
Shear strength	any orientation	0.4
Bearing strength	any orientation	1.0
Bending deflection	parallel or perpendicular to span	1.0
Shear deformation	parallel or perpendicular to span	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 plies with respect to the stress direction or span	Portion of cross-section considered (effective cross-section area)	Assembly factor (g_{19})
Compression and bending strength	parallel or perpendicular	Parallel plies only	1.0
	±45°	Full cross-section	0.34
Tension strength	parallel or perpendicular	Parallel plies only	1.0
	±45°	Full cross-section	0.17
Shear strength	parallel or perpendicular	Full cross-section	1.0
	±45°	Full cross-section	1.5
Bending Deflection	parallel or perpendicular	Parallel plies only	1.0
Deformation in compression or tension	parallel or perpendicular	Parallel plies only	1.0
	±45°	Full cross-section	0.17

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)

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 month	3 months	6 months	9 months	≥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)

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

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 j₆

Table 5-15: Plywood in Service Moisture Content Factor j_6 for Stiffness

6. Structural 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. 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.

$$I_{xx} = \frac{bd^3}{12} \qquad Z = \frac{bd^2}{6}$$
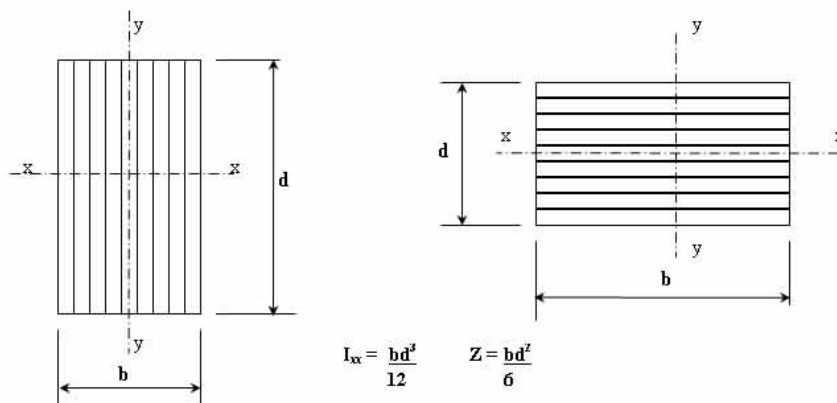
Figure 6-1: Section Properties for LVL with all Veneers Orientated in the Longitudinal Direction

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 A5). 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.

$$I_{xx} = \frac{(b-2t_x)d^3}{12}$$

$$Z = \frac{(b-2t_x)d^2}{6}$$

$$A_t, A_c = (b-2t_x)d$$
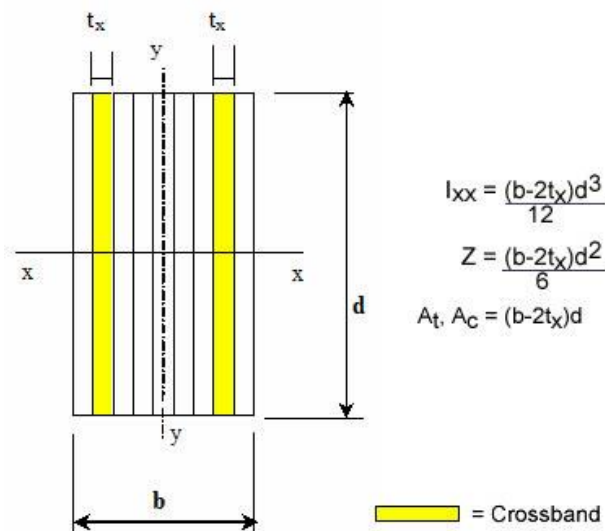
= Crossband

Figure 6-2: Cross-banded LVL section properties for edgewise bending, tension, compression and flexural rigidity

I (neutral axis (NA)-Second moment of area for calculation of bending rigidity (EI)

$$I_R = \left(\frac{bd_1^3}{12}\right) + 2\left[\left(\frac{bd_2^3}{12}\right) + (bd_2 \times (y_1)^2)\right] +$$

$$0.03 \; x \; 2 \; \left[\left(\frac{bd_x^3}{12} + (bd_x \times (y_x)^2)\right)\right.$$

Second moment of area of calculation of strength

$$I_C = \left(\frac{bd_1^3}{12}\right) + 2\left[\left(\frac{bd_2^3}{12}\right) + (bd_2 \times (y_1)^2)\right]$$

Section modulus

$$Z = \frac{I_C}{0.5 \; d}$$
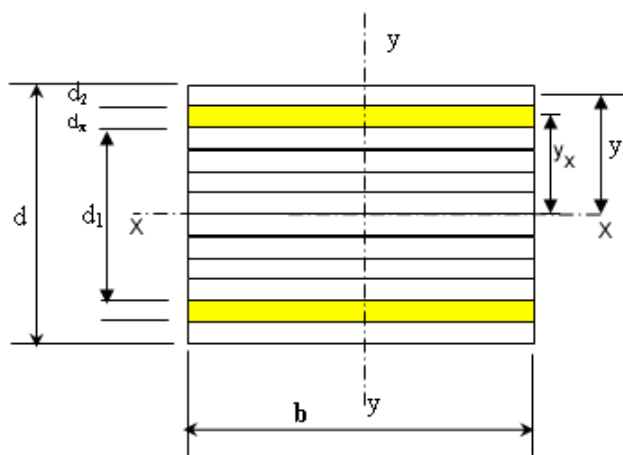
Figure 6-3: Example of cross-banded LVL section properties for on flat bending, bending deflection and shear

6.4. LVL – Design Methodology

Limit States Design to AS1720.1

The design capacity of structural LVL, designed in accordance with the limit states design format of AS1720.1, 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 1170.1.

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:

$$R_d > S^*$$

where

$\quad R_d$ = design capacity of the plywood member

$\quad S^*$ = design action effect, e.g. bending moment, M^*, shear force, V^*, etc.

and

$$R_d = \phi k_{mod} f_o' X$$

where

$\quad \phi$ = capacity factor

$\quad k_{mod}$ = product of relevant modification factors (e.g. $k_1, k_4, k_6, k_7, k_9, k_{12}, k_{19}, g_{19}$)

$\quad f_o'$ = appropriate characteristic strength

$\quad X$ = geometric section property.

Characteristic bending and tension strength may need to be reduced depending on cross section. For calculation of the reduction factor (k_x) please refer to manufacturers' specification. For the following examples an assumption was made.

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.

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.
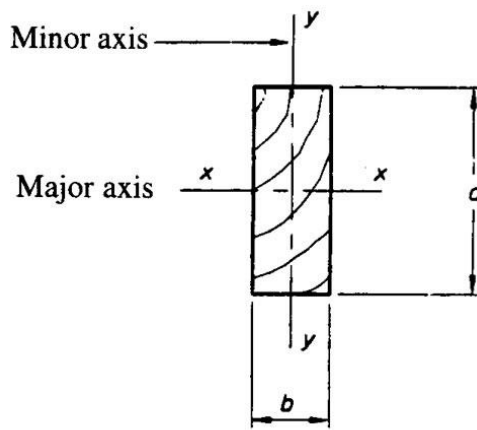
Figure 6-4: Major and Minor Axes of Bending

Strength Limit State

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied when:	AS1720.1 Reference
Bending	M^*	$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, k_x\, f_b'\, Z$	$M_d \geq M^*$	Cl. 3.2.1.1
For beams that can bend about both the major and minor axes simultaneously:			$\dfrac{M_x^*}{M_{d,x}} + \dfrac{M_y^*}{M_{d,y}} \leq 1.0$	Cl. 3.2.1.2
Shear	V^*	$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$	$V_d \geq V^*$	Cl. 3.2.5
Bearing perpendicular to grain	N_p^*	$N_{d,p} = \phi\, k_1\, k_4\, k_6\, k_7\, f_p'\, A_p$	$N_{d,p} \geq N_p^*$	Cl. 3.2.6.1
parallel to grain	N_l^*	$N_{d,l} = \phi\, k_1\, k_4\, k_6\, f_l'\, A_l$	$N_{l,p} \geq N_l^*$	Cl. 3.2.6.2

$M^*,\ V^*, N_p^*,\ N_l^*$	=	Design action/load effect in bending, shear and bearing respectively
$M_d, V_d, N_{d,p}, N_{l,p}$	=	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.
$M_{d,x}, M_{d,y}$	=	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 and position of bearing modification factor
k_9	=	Strength sharing modification factor
k_{12}	=	Stability modification factor

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k_x	=	Size modification factor for bending in accordance with manufacturers specification (volume factor) otherwise modification factor of AS1720.1 Cl. 8.3.1 apply
$f_b{}', f_s{}', f_p{}', f_l{}'$	=	Characteristic strengths in bending, panel shear and bearing normal to the plane of the panel respectively
Z	=	LVL beam section modulus where $Z_x = \frac{bd^2}{6}$ and $Z_y = \frac{db^2}{6}$ with $b = $ breadth and $d = $ depth
A_s	=	shear plane area $= \frac{2}{3}(bd)$ 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 $\times\ j_2 \times j_6 \leq deflection\ limit$

where:

$j_2 = $ Duration of load creep modification factor

$j_6 = $ Moisture content stiffness modification

FIGURE 6-5 defines the design parameters referred to when satisfying strength limit states in column design.
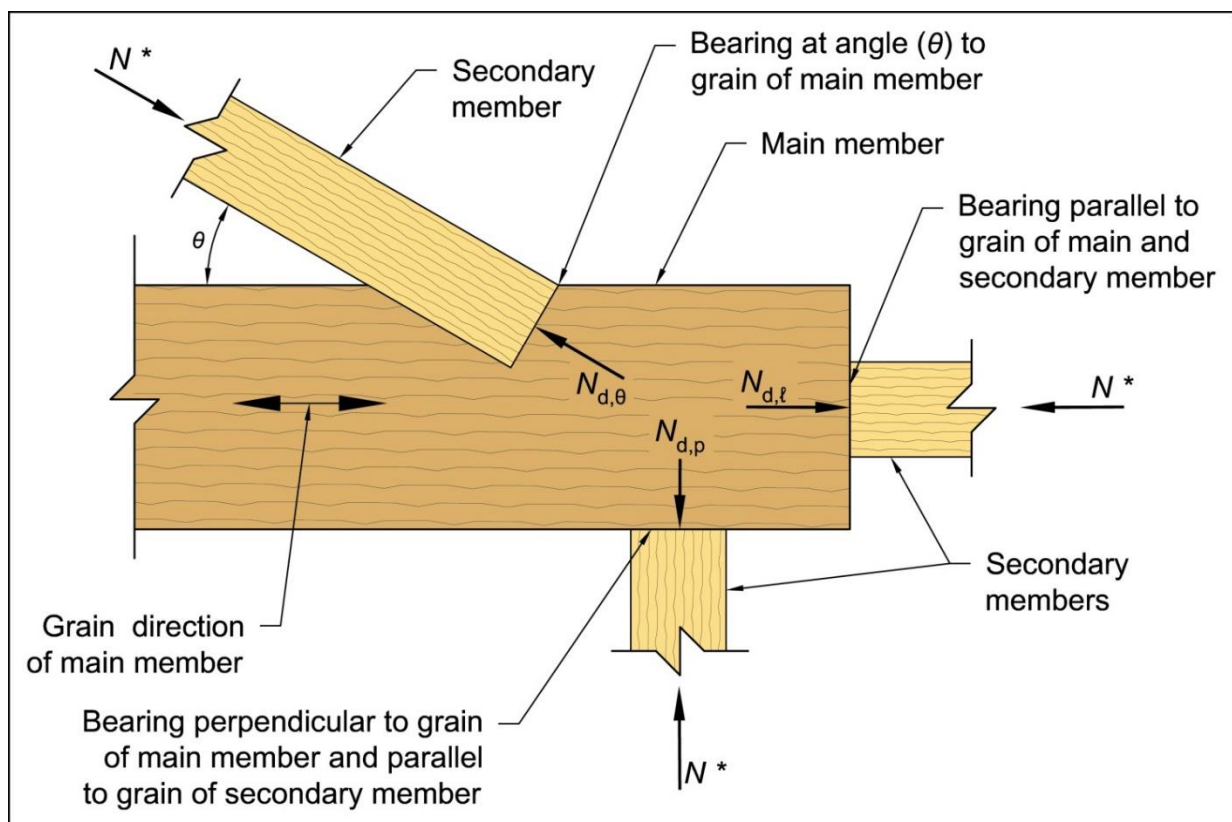
Figure 6-5: Notation for Bearing

6.6.   Column Design

Strength Limit State

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied when:	AS 1720.1 Reference
Compression	N_c^*	$N_{d,c} = \phi\, k_1\, k_4\, k_6\, k_{12}\, f_c'\, A_c$	$N_{d,c} \geq N_c^*$	Cl. 3.3.1
For columns that can buckle about both axes:			$N_{d,cx} \geq N_c^*$ and $N_{d,cy} \geq N_c^*$	Cl. 3.3.1.2

N_c^* = Design action effect in compression

$N_{d,c}$ = Design capacity in compression parallel to the grain

$N_{d,cx}, N_{d,cy}$ = 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 condition modification factor

k_6 = Temperature modification factor

k_{12} = Stability modification factor

f_c' = Characteristic strength 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.
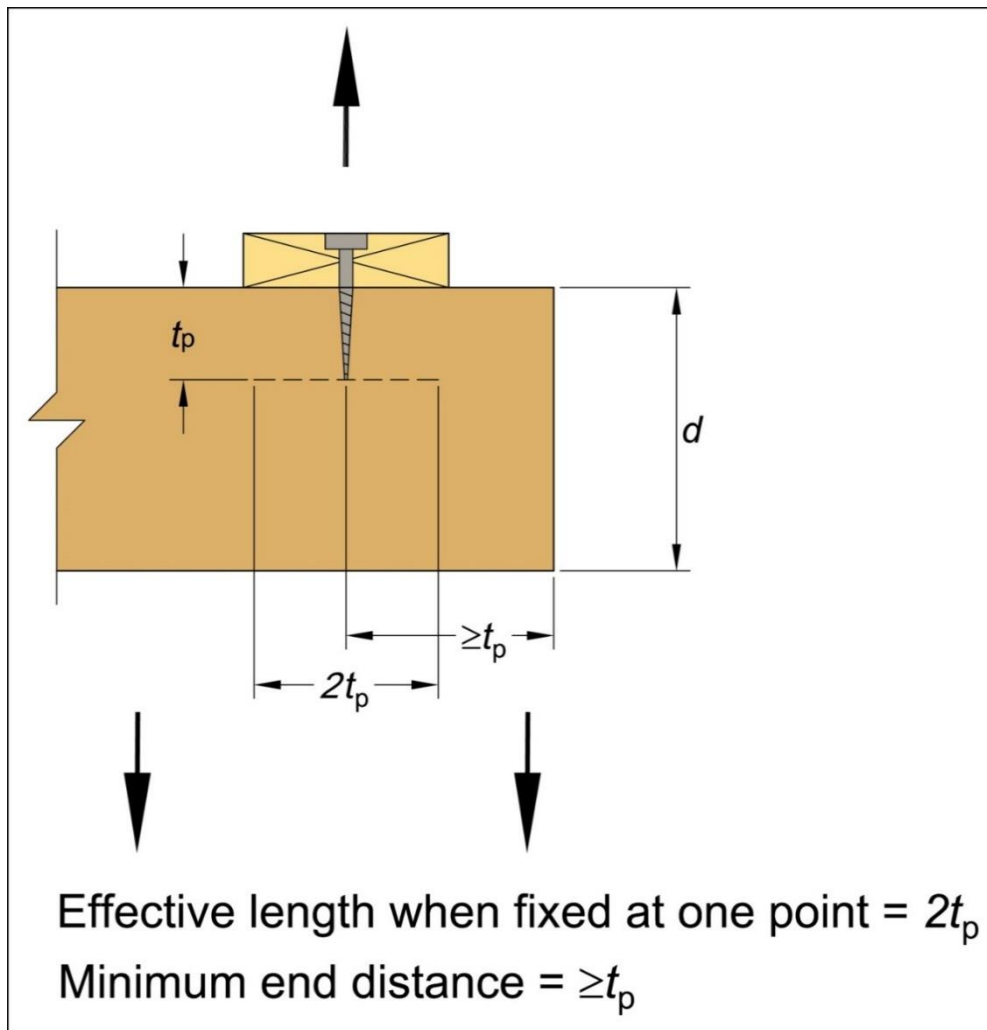
Figure 6-6: Effective Length Stressed in Tension Perpendicular to Grain

Strength Limit State

Strength Limit State	Design Action Effect	Design Capacity	Strength Limit State Satisfied when:	AS 1720.1 Reference
Tension parallel to grain	N_t^*	$N_{d,t} = \phi\, k_1\, k_4\, k_6\, k_x\, f_t'\, A_t$	$N_{d,t} \geq N_t^*$	Cl. 3.4.1
Tension perpendicular to grain	N_{tp}^*	$N_{d,tp} = \phi\, k_1\, k_{11}\, f_{tp}'\, A_{tp}$	$N_{d,tp} \geq N_{tp}^*$	Cl. 3.4.2

where:

N_t^*, N_{tp}^* =   Design action effect in tension parallel and perpendicular to grain respectively

$N_{d,t}, N_{tp}^*$ =   Design capacity in tension parallel and perpendicular to grain respectively

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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_{11}	=	Modification factors for the effect of volume $= \left(\frac{V_o}{V}\right)^{0.2}$ AS1720.1, Cl. 3.4.2, with $V_o = 10^7$ (a reference volume) and V is the volume, in mm^3, stresses in tension perpendicular to the grain at a level of stress greater than 80% of the maximum stress.
k_x	=	Size modification factor for tension in accordance with manufacturers specification (volume factor) otherwise modification factor of AS1720.1 Cl. 8.3.1 apply
$f_t^{'}$, $f_{tp}^{'}$	=	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 Reference
Combined bending and compression about the x axis	M_x^* N_c^*	$(\frac{M_x^*}{M_{d,x}})^2 + (\frac{N_c^*}{N_{d,cy}}) \leq 1$ and $(\frac{M_x^*}{M_{d,x}})^2 + (\frac{N_c^*}{N_{d,cx}}) \leq 1$	Cl. 3.5.1
Combined bending and tension actions	M^* M_x^* N_t^*	$(\frac{k_{12} M^*}{M_d}) + (\frac{N_t^*}{N_{d,t}}) \leq 1$ and $(\frac{M_x^*}{M_{d,x}}) - (\frac{Z}{A} \times \frac{N_t^*}{M_{d,x}}) \leq 1$	Cl. 3.5.2

where:

M_x^*	=	Design action effect in bending about a beam's major principal x-axis.
M^*	=	Design action effect in bending about a beam's appropriate axis.
N_c^*, N_t^*	=	Design action compression and tension respectively.

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		Design capacity in bending about a beam's major principal x-axis.
$M_{d,x}$	=	Design capacity in bending about a beam's major principal x-axis.
M_d	=	Design capacity in bending about a beam's appropriate axis.
$N_{d,cy}, N_{d,cx}$	=	Design capacity in compression for buckling about a beam's major y-axis and x-axis respectively.
$N_{d,t}$	=	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, 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		Structural LVL Capacity Factor, ϕ (AS/NZS 4357.0)
Category 1	Structural members for houses for which failure would be unlikely to affect an area* greater than 25 m² OR Secondary members in structures other than houses	0.95
Category 2	Primary structural members in structures other than houses; OR Elements in houses for which failure would be likely to affect an area* greater than 25m²	0.90
Category 3	Primary structural members in structures intended to fulfil an essential service 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 Reference
Strength modification factors	
k_1 = Factor for load duration	Cl. 8.4.2 & 2.4.1
k_4 = Factor for in-service moisture content	Cl. 8.4.3
k_6 = Factor for temperature effects	Cl. 8.4.4 & 2.4.3
k_7 = Factor for bearing length	Cl. 8.4.5, 2.4.4 & 3.2.6
k_9 = Factor for load sharing in grid systems	Cl. 8.4.6
k_{11} = Factor for volume effect	Cl. 3.4.2
k_{12} = Factor for instability	Cl. 8.4.7, Sect. 3
Stiffness modification factors:	
j_2 = Factor for duration of load for bending, compression and shear	Tab. 2.4
j_3 = Factor for duration of load for	Tab. 2.4
j_6 = Factor for in service moisture content	Cl. 8.1

k_1 – Duration of Load Strength Modification Factor

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. Please refer to Appendix G, AS1720.1.

LOAD DURATION FACTORS FOR TYPICAL LOAD COMBINATIONS FOR STRENGTH LIMIT STATE

Type of load (action)	Load combinations as per AS 1170.0	k_1^*	
		For strength of Solid timber	For strength of joints using laterally loaded fasteners[†]
Permanent action (dead load)	1.35 G	0.57	0.57
Permanent and short-term imposed actions (a) Roof live load— Distributed	1.2 G + 1.5 Q	0.94	0.77
(b) Roof live load— Concentrated		0.97	0.86
(c) Floor live loads— Distributed		0.80	0.69
(d) Floor live loads— Concentrated		0.94	0.77
Permanent and long-term[†] imposed action	1.2 G + 1.5 ψ_lQ	0.57	0.57
Permanent, wind and imposed action	1.2 G + W_u + ψ_cQ	1.00	1.14
Permanent and wind action reversal	0.9 G + W_u	1.00	1.14
Permanent, earthquake and imposed action	G + E_u + ψ_cQ	1.00	1.14
Fire	G + ψ_lQ	0.94	0.77

* Typical values of k_1 for various load combinations are given in Table G1, Appendix G AS1720.1.
† For the strength of joints with fasteners loaded in withdrawal and for the strength of steel in joints, $k_1 = 1.00$.

Table 6-2: Duration of Load Strength Modification Factor

k_4 – Moisture Content Factor

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%	15% to 25%	≥ 25%
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

$k_6 = 1.0$ 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.
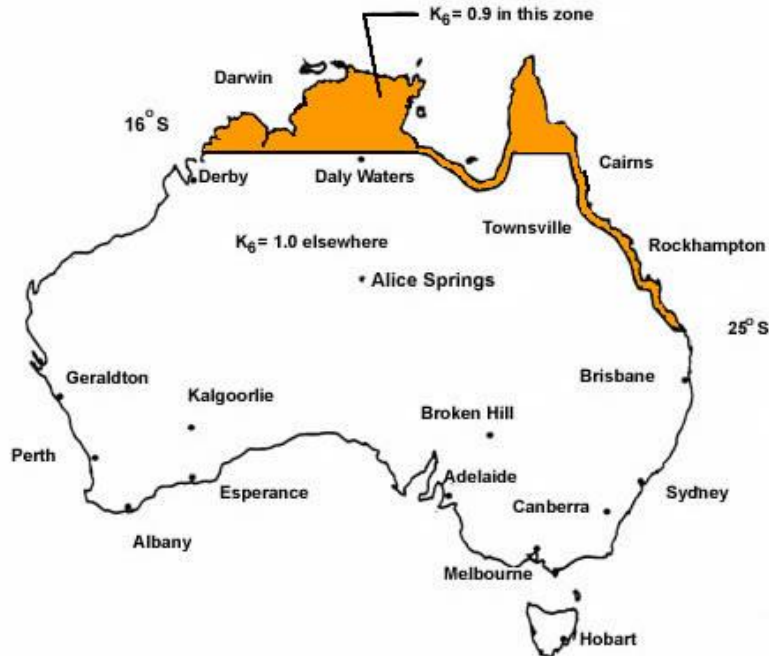
Figure 6-7: Map for Application of k_6

k_7 – Factor for Length and Position of Bearing

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	1.75	1.40	1.20	1.15	1.10	1.00

Table 6-4: Factor for Length and Position on Bearing

k_9 – Strength sharing modification factor for grid systems

$k_9 = 1.0$ for LVL used in parallel systems

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 (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.
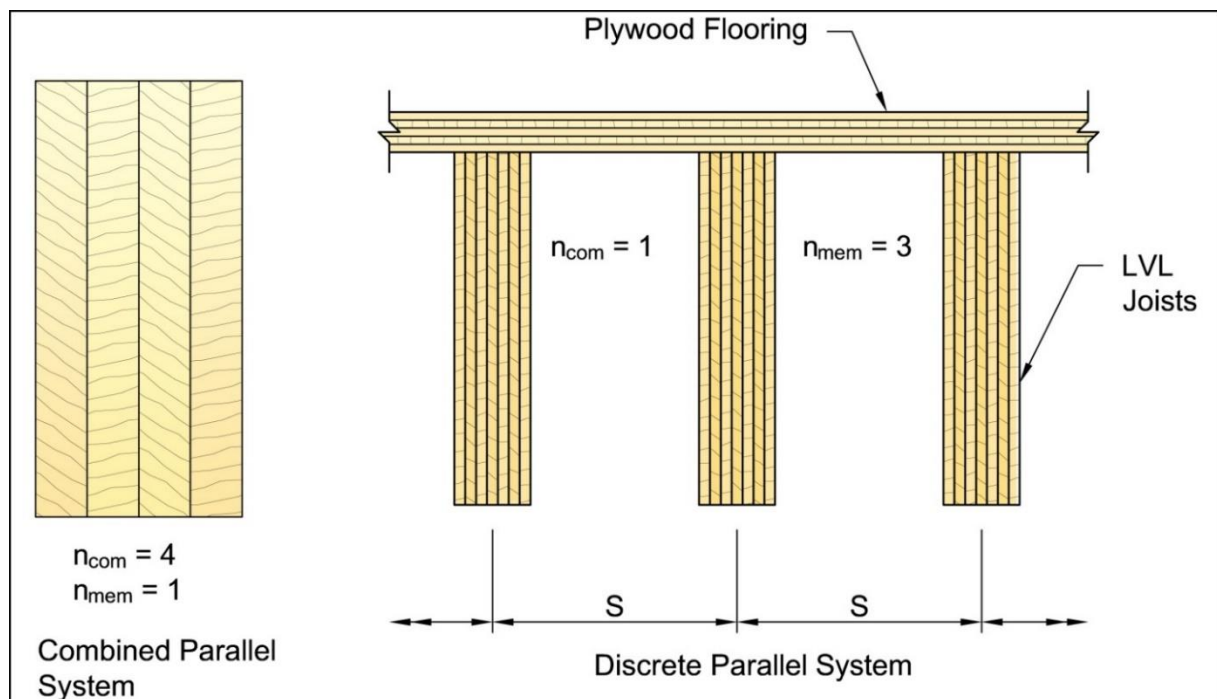
Figure 6-8: Parallel and Grid Systems

k_x – Volume effect Factor

The tensile behaviour of timber modelled on brittle fracture mechanics results in reduced tensile strength in larger cross section members. Tensile strength is a contributing factor to the bending performance of members. Therefore, the volume effect factor, k_x, is used to adjust the bending and tensile performance of larger members.

Australasian LVL manufacturers typically specify proprietary volume adjustment factors for their products, indexed off a 95mm reference size for bending, and 150mm reference size for tension. Manufacturer specific adjustment factors are the result of extensive testing and analysis of their products and are not transferrable to other products.

Volume effect adjustment factors are expressed as:

$k_x = 1.0$ for member depths up to the reference depth, d_{ref}

$k_x = (\dfrac{d_{ref}}{d_{actual}})^a$ for member depths greater than the reference depth

Reference depth and modification factor, a, may vary by manufacturer. Reference depths and modification factors for LVL manufactured in Australasia are summarised in Table 6-5.

Manufacturer	Bending		Tension	
	Reference depth, d_{ref} (mm)	Modification factor, a	Reference depth, d_{ref} (mm)	Modification factor, a
Carter Holt Harvey LVL	95	0.154	150	0.167
Juken New Zealand	300	0.167	150	0.167
Nelson Pine Industries	95	0.167	150	0.167
Wesbeam	95	0.140	95	0.140
AS 1720.1	300	0.167	150	0.167

Table 6-5: Volume Adjustment Factors for LVL Products Manufactured in Australasia

NOTE: Refer to manufacturer literature to confirm that the above factors are current.

AS 1720.1 provides a generic volume adjustment for bending based on a 300mm reference size. For simplicity, calculations in this manual are based on the reference size and modification factor published in AS 1720.1

k_{12} – Stability Factor

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 coefficient.

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

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

(b) For: $10 \leq \rho_b S \leq 20$ $k_{12} = 1.5 - 0.5\,\rho_b S$

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

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Material constants for LVL are:

For beams:

$$\rho_b = 14.71 \left(\frac{E}{f_b'}\right)^{-0.480} \times r^{-0.061}$$

For columns:

$$\rho_b = 11.39 \left(\frac{E}{f_c'}\right)^{-0.480} \times r^{-0.074}$$

The maximum value of ρ_b or ρ_c used need not exceed the value computed for the case $r = 0.25$. In case of beams where a temporary load causes stress reversal, the value of ρ_b or ρ_c to be used is that corresponding to $r = 1.0$.

NOTES: These equations are the same as those given for seasoned timber in AS1720.1 Appendix E. When a member is normally subjected to axial tension effect but may act in compression due to temporary design loads such as wind loads, the material constant (ρ) may be calculated as for beams (see Eq. 8.1, AS1720.1) for the case $r = 1.0$.

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

The j_2 load factor given in 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 6-6: 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)

The 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 **6-7**: Duration of Load Factor j_3 for Creep Deformation for Tension Members

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 AS1170.1 Structural Design Actions – Part 1: Permanent, imposed and other actions.

7.3. Design Issues of Flooring

The excellent load re-distribution capabilities of plywood mean 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 Guide 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.

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/NZS1170.0 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 for structural plywood flooring:

The relevant factors are

Bending: $(\phi\, k_1 k_{19}\, g_{19})$

Shear: $(\phi\, k_1 k_{19}\, g_{19})$

Deflection: $(j_2\, j_6\, g_{19})$

Refer AS1720.1

Chapter 5 of this Guide

$\phi = 0.95$	Category 1, for structural members for houses for which failure would be unlikely to affect an area greater than 25m^2, or secondary members in structures other than houses & structural plywood is compliant with AS/NZS 2269.0	AS1720.1 Reference Table 2.1
$k_1 = 0.94$	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.	Cl. 2.4.1.1
$k_1 = 0.80$	For uniformly distributed loads assuming loads are typical floor type loads (crowd or vehicle or stored material). Effective duration of peak load = 5 months.	Table 2.3
$k_1 = 0.57$	For permanent and long-term components of imposed actions = 50 years or more.	
$k_{19} = 1.0$	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 5.2 A
$g_{19} = 1.0$	Direction of the face veneers is parallel with the span direction. Therefore, $g_{19} = 1.0$ for bending, 0.4 for shear strength and 1.0 for deflection.	Table 5.3
$j_2 = 1.0$	for short term concentrated loads of less than 1 days duration.	Cl. 2.4

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$j_2 = 2.0$	for longer term uniformly distributed loads, such as stored materials.	
$j_6 = 1.0$	average moisture content not anticipated to exceed 15%. (Refer k_{19} above)	

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

Design requirements and specification for a structural plywood mezzanine floor for general 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:

$$\frac{Span}{300} = \frac{400}{300} = 1.3mm$$

Assumption:

2m storage height

1. Loads:
Imposed action = Q (Live load)
Assumption: General storage
7kN concentrated load;
5kPa uniformly distributed load (UDL), where
$UDL = 2.4 \, kPa \times 2 = 4.8 \, kPa$ (assume 5kPa as conservative value)
Permanent action = G (Dead load)
Self-weight:
For 25mm plywood (Not usually considered but included in this design example for completeness
$(G = 600 \frac{kg}{m^3} \times 9.81 \frac{m}{s^2} \times 0.025 \, m)/1000 = 0.15 \, kPa$

2. Load Combinations
Strength limit state:

$$1.35G$$

$$1.2G + 1.5Q \, \psi_l$$

$$1.2G + 1.5Q$$

With

$$\psi_l = 0.6 \, for \, point \, and \, uniform \, loads$$

Serviceability limit state:

$$1 \, x \, Q \, (short \, term)$$

Point load $\qquad G + \psi_l Q \, (long \, term)$

Uniform load $\qquad G + \psi_l Q \, (long \, term)$
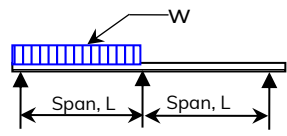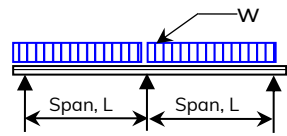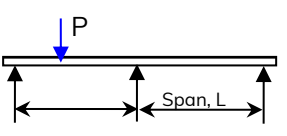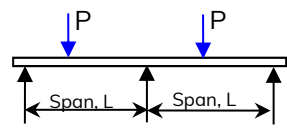
3. Capacity Factor and Strength Modification Factors

$$\phi \quad = 0.95 \qquad \text{Table 2.1}$$

$$k_1 \quad = 0.94 \text{ concentrated live load} \qquad \text{Table 2.3}$$

$$k_1 \quad = 0.80 \text{ uniformly distributed live load} \qquad \text{Table 2.3}$$

$$k_1 \quad = 0.57 \text{ permanent and load term load} \qquad \text{Table 2.3}$$

$$k_{19} \quad = 1.0 \text{ (MC} \leq 15\%) \qquad \text{Table 5.2A}$$

$$g_{19} \quad = 1.0 \text{ for bending strength} \qquad \text{Table 5.3}$$

$$g_{19} \quad = 0.4 \text{ for shear strength} \qquad \text{Table 5.3}$$

4. Serviceability Modification Factors

$$j_2 \quad = 1.0 \text{ short term load} \qquad \text{Table 2.4}$$

$$j_2 \quad = 2.0 \text{ long term load} \qquad \text{Table 2.4}$$

$$j_6 \quad = 1.0 \text{ (MC} \leq 15\%) \qquad \text{Table 5.2B}$$

$$g_{19} \quad = 1.0 \text{ bending deflection} \qquad \text{Table 5.3}$$

5. Critical Load Action Effects

| Load Case 1 | Load Case 2 |

UDL

$$M_{max} = \frac{49wL^2}{512}$$

$$V_{max} = \frac{9wL}{16}$$

$$\Delta_{max} = \frac{wL^4}{72.3EI}$$

$$M_{max} = \frac{-wL^2}{8}$$

$$V_{max} = \frac{5wL}{8}$$

$$\Delta_{max} = \frac{wL^4}{185EI}$$

Concentrated Load

$$M_{max} = \frac{13PL}{64}$$

$$V_{max} = \frac{-19P}{32}$$

$$\Delta_{max} = \frac{PL^3}{66.7EI}$$

$$M_{max} = \frac{6PL}{32}$$

$$V_{max} = \frac{11P}{16}$$

$$\Delta_{max} = \frac{PL^3}{110EI}$$

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 11/16th 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^*	$\frac{M_P^*}{k_1}$
DL	$\frac{0.15 \times 10^{-3} \times 400^2}{8} = 3$	$1.2 \times 3 = 3.6$	$\frac{3.6}{0.57} = 6.32$
LL (UDL)	$\frac{5 \times 10^{-3} \times 400^2}{8} = 100$	$1.5 \times 100 = 150$	$\frac{150}{0.8} = 188$
LL (conc)	$\frac{13 \times 7000 \times 400}{64 \times 520} = 1094$	$1.5 \times 1094 = 1641$	$\frac{1641}{0.94} = \mathbf{1746}$

	V_p	V_P^*	$\frac{V_P^*}{k_1}$
DL	$\frac{5 \times 0.15 \times 10^{-3} \times 400}{8} = 0.04$	$1.2 \times 0.04 = 0.05$	$\frac{0.05}{0.57} = 0.09$
LL (UDL)	$\frac{5 \times 5 \times 10^{-3} \times 400}{8} = 1.25$	$1.5 \times 1.25 = 1.88$	$\frac{1.88}{0.8} = 2.35$
LL (conc)	$\frac{11 \times 7000}{16 \times 520} = 9.25$	$1.5 \times 9.25 = 13.88$	$\frac{13.88}{0.94} = \mathbf{14.77}$

	$\Delta\ (mm)$
DL	$\frac{0.15 \times 10^{-3} \times 400^4}{72.3\ EI} = \frac{53 \times 10^3}{EI}$
LL (UDL)	$\frac{5 \times 10^{-3} \times 400^4}{72.3\ EI} = \frac{1770 \times 10^3}{EI}$
LL (conc)	$\frac{7000 \times 400^3}{66.7 \times 520 \times EI} = \frac{13 \times 10^6}{EI}$

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 5-3).

2. Strength Limit State – Design Load Combinations

$$M^*_{crit} = M^*_{(LLconc)}$$
$$= (3.6 + 1641)/0.94 = 1750 \frac{\text{N mm}}{\text{mm width}} \left(\frac{M^*_p}{k_1}\right)$$

$$V^*_{crit} = V^*_{(LLconc)}$$
$$= (0.05 + 13.88)/\ 0.94 = 14.8 \frac{N}{mm\ width} \left(\frac{V^*_p}{k_1}\right)$$

3. Shear criteria – (establish minimum $f_s' \ A_s$)

$$V_{d,p} \geq V^*_p$$

and $\quad V_{d,p} = \phi \ k_1 \ k_{19} \ g_{19} \ f_s' \ A_s$

$$f_s' \ A_s = \frac{14.8}{(0.95 \ \times 1.0 \ \times 0.4)} = 38.9 \ \frac{N}{mm\ width}$$

$\Rightarrow \quad\quad \phi \ k_1 \ k_{19} \ g_{19} \ f_s' \ A_s \geq V^*_p$

and $\quad f_s' \ A_s \geq \frac{V^*_p}{(\phi \ k_1 \ k_{19} \ g_{19})}$

Minimum required $f_s' \ A_s = \frac{14.8}{(0.95 \ \times 1.0 \ \times 0.4)} = 38.9 \ \frac{N}{mm\ width}$

From Table 5-4 Require minimum 12mm 12-15-5, F14 ($f_s' \ A_s = 45$), please note this Table includes $g_{19} = 0.4$.

4. Bending criteria – (establish minimum $f_b' \ Z_p$)

$$M_{d,p} \geq M^*_p$$

and $\quad M_{d,p} = \phi k_1 \ k_{19} \ g_{19} \ f_b' \ Z_p$

$\rightarrow \ \phi k_1 \ k_{19} \ g_{19} \ f_b' \ Z_p \ \geq M_{d,p}$

And $\quad f_b' \ Z_p \geq \frac{M_{d,p}}{\phi \ k_1 \ k_{19} \ g_{19}}$

Minimum required $f_b' \ Z_p = \frac{1750}{(0.95 \ \times 1.0 \ \times 1.0)} = 1842 \ \frac{N\ mm}{mm\ width}$

From TABLE 5-4, suitable structural plywoods include:

F11, 25mm 25-30-9 ($f_b' \ Z_p = 2186 \ \frac{N\ mm}{mm\ width}$)

F14, 25mm 25-30-9 ($f_b' \ Z_p = 2538 \ \frac{N\ mm}{mm\ width}$)

F17, 19mm 19-30-7 ($f_b' \ Z_p = 2093 \ \frac{N\ mm}{mm\ width}$)

F27, 17mm 17-24-7 ($f_b' \ Z_p = 2345 \ \frac{N\ mm}{mm\ width}$)

Serviceability limit state – Design Load Combinations

$$1 \; x \; Q \; (short \; term)$$

Point load $\qquad G + 0.6 \, Q \; (long \; term)$

Uniform load $\qquad G + 0.6 \, Q \; (long \; term)$

5. Deflection criteria – (determine minimum required EI)

Maximum allowable defection $\qquad \dfrac{Span}{300} = \dfrac{400}{300} = 1.3mm$

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

$$1.3mm = \frac{1.0 \times 1.0 \times 13.0 \times 10^6}{EI}$$

Required $\qquad EI_{min} = 10000 \times 10^3 \, \dfrac{N \, mm^2}{mm \, width}$

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

$$1.3 \; mm = \frac{2.0 \times 1.0 \times (53 + 1770) \times 10^3}{EI}$$

$$EI_{min} = 1715 \times 10^3 \, \frac{N \, mm^2}{mm \, width}$$

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

$$\Delta_{max} = j_2 \times g_{19} \times \Delta_{(DL+LL \; conc)}$$

$$1.3 \; mm = \frac{2.0 \times 1.0 \times (53 \times 10^3 + 0.6 \times 13.0 \times 10^6)}{EI}$$

$$EI_{min} = 12081 \times 10^3 \, \frac{N \, mm^2}{mm \, width}$$

which is higher than required EI under short term load and long term uniformly distributed load ⇨ critical

6. Select Suitable Structural Plywood Flooring
    Subject to availability, suitable structural plywoods would include (see TABLE 5-5):

F14, 27mm, 27-30-9 $\qquad (EI = 13320 \, \dfrac{N \, mm^2}{mm \, width} \; ; f_b{'} \, Z_p = 2916 \, \dfrac{N \, mm}{mm \, width})$

F17, 25mm, 25-30-9 $\qquad (EI = 12600 \, \dfrac{N \, mm^2}{mm \, width} \; ; f_b{'} \, Z_p = 3173 \, \dfrac{N \, mm}{mm \, width})$

F14, 25mm, 25-30-9 $\qquad (EI = 16650 \, \dfrac{N \, mm^2}{mm \, width} \; ; f_b{'} \, Z_p = 4935 \, \dfrac{N \, mm}{mm \, width})$

Plywood specification:

Specify number of sheets x 2400 x 1200 x 25mm, structural plywood to AS/NZS 2269, stress grade F14, (27-30-9), CD – A bond, EWPAA / JAS-ANZ Product Certified

7.6. Structural Plywood Flooring

Typical structural plywood thicknesses and stress grades for a range of minimum floor-imposed loadings detailed in AS/NZS 1170.1 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 (these documents have been archived and are available by contacting 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 Table 3.1 Imposed loads & Suitable Structural Plywood Thickness

7.7. 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 archived document LP91 Low Profile Stressed Skin Plywood Floor System which is available by contacting the EWPAA.

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.

7.8. 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.

7.9. Structural Plywood Residential Bracing and Combined Bracing/Cladding

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 (this document has been archived and is available by contacting the EWPAA). 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.

> As a result of re-validation of plywood bracing systems, the EWPAA now recommends plywood bracing be a minimum of 6mm thickness. An archived copy of EWPAA Limit States Design Manual – Structural Plywood Bracing incorporating changes as a result of re-validation tests is available by contacting the EWPAA.

Racking tests were done in October 2009 at Central Queensland University in Rockhampton on panels framed in 90 x 45mm MGP 10, JD5 and 70 x 45mm MGP 10 JD5 framing, sheathed both sides with 7mm F11 DD structural plywood. The purpose of the tests was top check the adequacy of the top and bottom plates when panels were subjected to a racking load of 17.4 kN/m (which is twice the 8.7 kN/m system sheathed one side only). The panels produced satisfactory results provide they were restricted to a 2.4m long for the 90 x 45mm framing and 1.8m long for the 70 x 45mm system.

For further detail concerning these panels, an archived copy of EWPAA Structural Plywood Wall Bracing – Limit States Design manual is available by contacting the EWPAA.

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 encumbrances;

- 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.
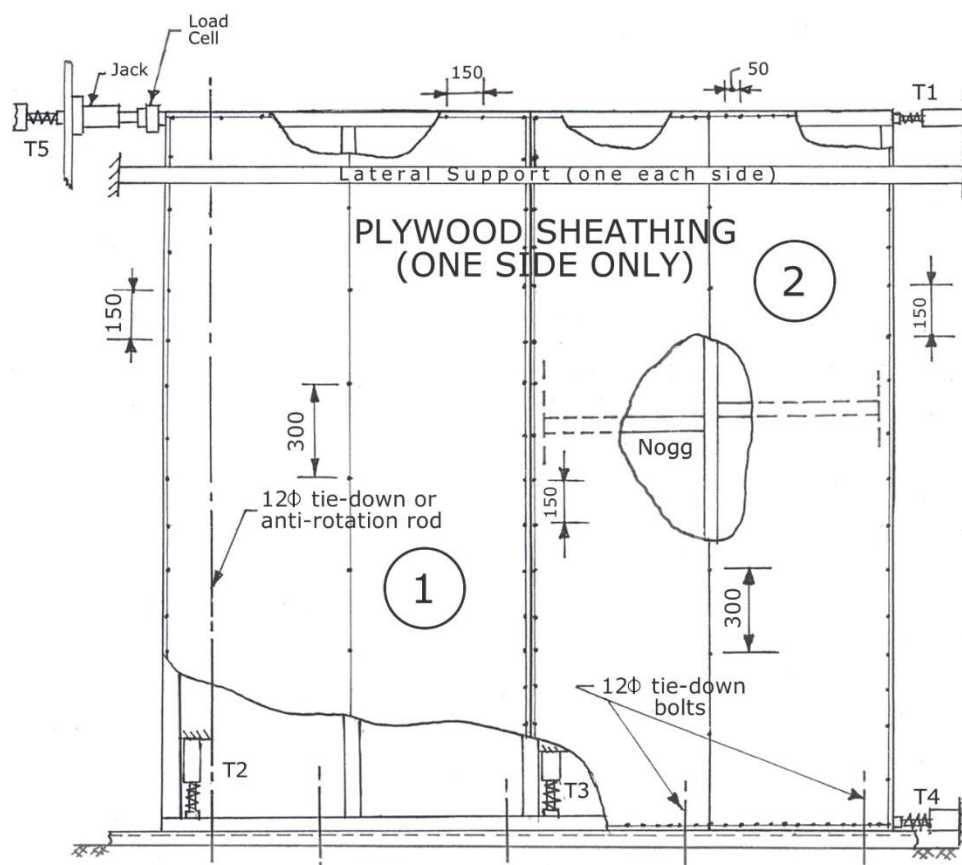
Figure 7-1: Typical Test Panel Arrangement

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.

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To fully quantify the racking load variables requires:

- strength limit state to be $\geqslant$ 1.5 x serviceability limit state.

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

7.10. 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 (this document has been archived and is available by contacting the EWPAA). 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 (these documents have been archived and are available by contacting the EWPAA). 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)

7.11. 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 coefficient 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.

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.

7.12. 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'	$48\ MPa$
Tension	f_t'	$25\ MPa$
Shear in beams	f_s'	$4.6\ MPa$
Compression parallel to grain	f_c'	$38\ MPa$
Compression perpendicular to grain	f_p'	$10\ MPa$
Modulus of Elasticity	E	$13200\ MPa$
Modulus of Rigidity	G	$660\ MPa$
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/m².
    - Roof load width is 4.8m.
    - Adequate clearance must be maintained over doors. Therefore, set deflection limit as follows:
        - Permanent Loads: Span/300 to 10 mm maximum
        - Imposed Loads: Span/250 to 15 mm maximum
        - Downwards Wind Loads: Span/200 to 9 mm maximum
        - Wind Uplift: Span/200 to 50 mm maximum
2. Loads

    Permanent:

    Roof & Ceiling load $= 40\,\frac{kg}{m^2}$

    Load per rafter $= \left(40\,\frac{kg}{m^2} \times 9.81\,\frac{m}{s^2} \times 4.8m \times 0.9\,m\right) \times 10^{-3}$

    $= 1.7\,\frac{kN}{rafter}$

    Self-weight

    $allow\ 650\,\frac{kg}{m^3}$

    Select Trial size beam 300 x 45 mm

$$\text{Self-weight} = 650 \frac{kg}{m^3} \times 9.81 \frac{m}{s^2} \times 0.3\ m \times 0.045\ m$$

$$= 0.09 \frac{kN}{m}$$

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)

$$0.25\ kPa \times 1.2\ m \times 0.9\ m = 0.27 \frac{kN}{rafter}$$

Wind: (Cyclonic C1 Wind classified Area)

Strength:

$$\text{Net Uplift} = 2.08\ kPa \times 4.8m \times 0.9m$$

$$\text{Net Uplift} = 9.0 \frac{kN}{rafter}$$

Down:

Net pressure coefficient $(C_{p,n})$ = 1.05

Design wind gust speed $(V_{h,u})$ = $50 \frac{m}{s}$

Design wind pressure = $q_u \times C_p$

where

$$q_u = \frac{0.5\ \rho_{air}\ [V_h]^2}{1000}, with\ \rho_{air} = 1.2 \frac{kg}{m^3}$$

$$q_u = \frac{0.5 \times 1.2 \frac{kg}{m^3} \times \left[50 \frac{m}{s}\right]^2}{1000} = 1.5 \frac{kg}{m\ s^2}$$

$$= 1.5 \frac{kg}{m\ s^2} \times (1.05)$$

$$= 1.58 \frac{kg}{m\ s^2}$$

$$1.58 \frac{kg}{m\ s^2} \times 4.8\ m \times 0.9\ m = 6.8 \frac{kN}{rafter}$$

Serviceability:

$$\text{Net Uplift} = 0.28\ kPa \times 4.8\ m \times 0.9\ m$$

$$\text{Net Uplift} = 1.2 \frac{kN}{rafter}$$

Down:

Net pressure coefficient ($C_{p,n}$) = 1.05

$$V_{h,s} = 32 \frac{m}{s}$$

Pressure = $q_u \times C_p$

where

$$q_u = \frac{0.5\ \rho_{air}\ [V_h]^2}{1000}, with\ \rho_{air} = 1.2 \frac{kg}{m^3}$$

$$q_u = \frac{0.5 \times 1.2 \frac{kg}{m^3} \times \left[32 \frac{m}{s}\right]^2}{1000} = 0.61 \frac{kg}{m\ s^2}$$

$$= 0.61 \frac{kg}{m\ s^2} \times (1.05)$$

$$= 0.65 \frac{kg}{m\ s^2}$$

$$0.65 \frac{kg}{m\ s^2} \times 4.8\ m \times 0.9\ m = 2.8 \frac{kN}{rafter}$$

3. Load Combinations

> Strength limit state

$$1.35G$$
$$1.2G + 1.5Q$$
$$1.2G + W_u + \psi_c Q$$
$$-0.9\,G + W_u$$

$Q = 0$ under max. downward wind loading and
$\psi_c = 0\ (point\ and\ uniform)$

> Serviceability limit state

$$G$$
$$\psi_s Q\ \text{or}\ \psi_l Q$$

$\psi_s = 0.7\ (uniform)\ and\ 1.0\ (point)\ for\ roofs$
$\psi_l = 0$

4. Capacity Factor and Strength Modification Factors
The relevant factors for beam design are:

Bending	$(\phi\, k_1\, k_4\, k_6\, k_9\, k_x\, k_{12})$
Shear	$(\phi\, k_1\, k_4\, k_6)$
Deflection	$(j_2\, j_6)$

ϕ = 0.95 for LVL in all structural elements in houses

k_1 = 0.57 for permanent loads such as roof self-weight

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

k_1 = 1.0 for wind gust loads

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 %

k_6 = 0.9 Coastal area of Queensland, north of latitude 25^0 S

k_9 = 1.0 for all LVL used in parallel systems

k_x Factor applies to bending and tension and changes based on member size, manufacturers specification may differ from AS 1720.1 and it may differ between manufacturers.
For this example
$$k_x = \left(\frac{95}{d}\right)^{0.167} bending$$
$$k_x = \left(\frac{95}{d}\right)^{0.140} tension$$
has been used.

k_{12} based on value of $\rho_b S$

$$\rho_b = 14.71 \left(\frac{E}{f'_b}\right)^{-0.480} r^{-0.061}$$

$$\left(\frac{E}{f'_b}\right)^{-0.480} = \left(\frac{13200}{48}\right)^{-0.480} = 1.01$$

r = temporary design action effect/total design action effect

$$S_1 = 1.25 \left(\frac{d}{b}\right)\left(\frac{L_{ay}}{d}\right)^{0.5} \text{ (downwards loads, } L_{ay} = 0.9\ m)$$

$$S_1 = \left(\frac{d}{b}\right)^{1.35}\left(\frac{L_{ay}}{d}\right)^{0.25} \text{ (wind uplift, } L_{ay} = 0.9\ m)$$

5. Serviceability Modification Factors

j_2 = 1.0 short term load (MC $\leq$ 15%)

j_2 = 2.0 long term load (MC $\leq$ 15%)

j_6 = 1.0 (MC $\leq$ 15%)

6. Critical Load Action Effects

Permanent roof loads (G)
&
Wind Loads (W$_u$)
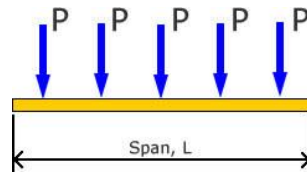
$$M_{max} = \frac{PL}{2}$$
$$V_{max} = \frac{3P}{2}$$
$$\Delta_{max} = \frac{19PL^3}{384EI}$$

Concentrated Imposed Load (Q)
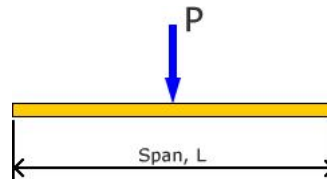
$$M_{max} = \frac{PL}{4}$$
$$V_{max} = \frac{P}{2}$$
$$\Delta_{max} = \frac{PL^3}{48EI}$$

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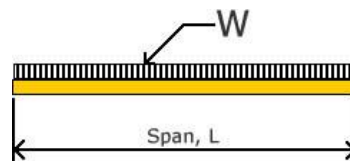
$$M_{max} = \frac{wL^2}{8}$$
$$V_{max} = \frac{wL}{2}$$
$$\Delta_{max} = \frac{5wL^4}{384EI}$$

7.13. Structural LVL Lintel Beam: Worked Example

1. Design Action Effects on Member Due to Factored Loads

Loading Criteria	M_p (kNm) (unfactored)	M_p^* (kNm) (factored)	Load Combination	$\frac{M_p^*}{k_1}$ (kNm)
DEAD G = Beam Self Weight + Permanent Roof Load	$\frac{0.09 \times 3.6^2}{8}$ + $\frac{1.7 \times 3.6}{2}$ $0.15 + 3.06 =$ **3.2**	$1.35 \times 3.2 = \mathbf{4.3}$	$\mathbf{1.35}G$	$\frac{4.3}{0.57} = \mathbf{7.5}$
		$1.2 \times 3.2 = \mathbf{3.8}$	$\mathbf{1.2}G + 1.5Q$	3.8
		$1.2 \times 3.2 = \mathbf{3.8}$	$\mathbf{1.2}G + W_u + \psi_c Q$	3.8
		$-0.9 \times 3.2 = \mathbf{-2.9}$	$-\mathbf{0.9}\,\boldsymbol{G} + W_u$	-2.9
IMPOSED Q (conc load) Q (UDL) G + Q	$\frac{1.4 \times 3.6}{4} = 1.3$ $\frac{0.27 \times 3.6}{2} = 0.5$	$1.5 \times 1.3 = \mathbf{2.0}$ $1.5 \times 0.5 = \mathbf{0.8}$ $0 \times 1.3 = \mathbf{0}$ $0 \times 0.5 = \mathbf{0}$	$1.2G + \mathbf{1.5Q}$ $1.2G + W_u + \boldsymbol{\psi_c Q}$	$3.8 + \mathbf{2.8} = 6.6$ $\frac{6.6}{0.94} = \mathbf{7.0}$ $3.8 + \mathbf{0}$
WIND UP $W_u\uparrow$ 0.9G $W_u\uparrow$-0.9G	$\frac{9.0 \times 3.6}{2} = 16.2$	**16.2**	$-1.2G + \boldsymbol{W_u} + \psi_c Q$ $-0.9\,\boldsymbol{G} + \boldsymbol{W_u}$	$-3.8 + \mathbf{16.2} + 0 = 12.4$ $\frac{12.4}{1.0} = \mathbf{12.4}$ $-2.9 + \mathbf{16.2} = 13.3$ $\frac{13.3}{1.0} = \mathbf{13.3}$
WIND DOWN $W_u\downarrow$ 1.2 G $W_u\downarrow$+G	$\frac{6.8 \times 3.6}{2} = 12.2$	**12.2**	$1.2G + \boldsymbol{W_u} + \psi_c Q$ $0.9\,G + \boldsymbol{W_u}$	$3.8 + \mathbf{12.2} + 0 = 16$ $\frac{16}{1.0} = \mathbf{16.0}$ $2.9 + \mathbf{12.2} = 15.1$ $\frac{15.1}{1.0} = \mathbf{15.1}$

Loading Criteria	V_p (kN) (unfactored)	V_p^* (kN) (factored)	Load Combination	$\frac{V_p^*}{k_1}$ (kN)
DEAD G = Beam Self Weight + Permanent Roof Load	$\frac{0.09 \times 3.6}{2}$ + $\frac{3 \times 1.7}{2}$ $= 0.16 + 2.55 = 2.7$	$1.35 \times 2.7 = \mathbf{4.1}$	$\mathbf{1.35}G$	$\frac{4.1}{0.57} = \mathbf{7.2}$
		$1.2 \times 2.7 = \mathbf{3.2}$	$\mathbf{1.2}G + 1.5Q$	3.2
		$1.2 \times 2.7 = \mathbf{3.2}$	$\mathbf{1.2}G + W_u + \psi_c Q$	3.2
		$-0.9 \times 2.7 = \mathbf{2.4}$	$-\mathbf{0.9}\,\boldsymbol{G} + W_u$	2.4
IMPOSED Q (conc load) Q (UDL) G + Q	$\frac{1.4}{2} = 0.7$ $\frac{3 \times 0.27}{2} = 0.4$	$1.5 \times 0.7 = \mathbf{1.1}$ $1.5 \times 0.4 = \mathbf{0.6}$ $0 \times 0.7 = 0$ $0 \times 0.4 = \mathbf{0}$	$1.2G + \mathbf{1.5Q}$ $1.2G + W_u + \boldsymbol{\psi_c Q}$	$3.2 + \mathbf{1.7} = 4.9$ $\frac{4.9}{0.94} = \mathbf{5.2}$ $3.2 + \mathbf{0}$

WIND UP $W_u\uparrow$	$\frac{3 \times 9.0}{2} = 13.5$	$1.0 \times 13.5 = \mathbf{13.5}$	$-1.2G + \boldsymbol{W_u} + \psi_c Q$	$-3.2 + \mathbf{13.5} + 0 = 10.3$ $\frac{10.3}{1.0} = \mathbf{10.3}$
0.9G				
$W_u\uparrow$-0.9G		$-0.9 \times 3.2 = \mathbf{2.9}$	$-0.9\,G + \boldsymbol{W_u}$	$-2.9 + \mathbf{13.5} = 10.6$ $\frac{10.6}{1.0} = \mathbf{10.6}$
WIND DOWN $W_u\downarrow$	$\frac{3 \times 6.8}{2} = \mathbf{10.2}$	$1.0 \times 10.2 = \mathbf{10.2}$	$1.2G + \boldsymbol{W_u} + \psi_c Q$	$3.2 + \mathbf{10.2} + 0 = 13.4$ $\frac{13.4}{1.0} = \mathbf{13.4}$
1.2 G				
$W_u\downarrow$+G			$0.9\,G + \boldsymbol{W_u}$	$2.9 + 10.2 = 13.1$ $\frac{13.1}{1.0} = \mathbf{13.1}$

Loading Criteria	$\Delta\ (mm)$
DEAD G = Beam Self Weight + Permanent Roof Load	$\frac{5 \times 0.09 \times 3600^{4}}{384EI} = \frac{1.97 \times 10^{11}}{EI}$ $\frac{19 \times 1.7 \times 3600^{3}}{384\,EI} = \frac{0.04 \times 10^{11}}{EI}$ $Total = \frac{\mathbf{2.0 \times 10^{11}}}{EI}$
IMPOSED Q (conc load) Q (UDL) G + Q	$\frac{1.4 \times 3600^{3}}{48EI} = \frac{1.4 \times 10^{9}}{EI}$ $\frac{19 \times 0.27 \times 3600^{3}}{384\,EI} = \frac{\mathbf{0.62 \times 10^{9}}}{EI}$
WIND UP $W_u\uparrow$ 0.9G $W_u\uparrow$-0.9G	$\frac{19 \times 1.2 \times 3600^{3}}{384EI} = \frac{\mathbf{5.8 \times 10^{9}}}{EI}$
WIND DOWN $W_u\downarrow$ 1.2 G $W_u\downarrow$+G	$\frac{19 \times 2.8 \times 3600^{3}}{384EI} = \frac{\mathbf{6.5 \times 10^{9}}}{EI}$

2. Strength Limit State - Design Load Combinations

$M^*_{crit} = M^*(W_u \downarrow + G) = 16.0\ kNm$

$V^*_{crit} = V^*(W_u \downarrow + G) = 13.4\ kN$

Trial Beams:
  240×45, $A = 10800\ mm^2$, $I_{xx} = 51.8 \times 10^6\ mm^4$, $Z_{xx} = 432.0 \times 10^3\ mm^3$
  300×45, $A = 13500\ mm^2$, $I_{xx} = 101.3 \times 10^6\ mm^4$, $Z_{xx} = 675.0 \times 10^3\ mm^3$

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

$$k_{12} : \rho_b = 14.71 \left(\frac{E}{f'_b}\right)^{-0.480} r^{-0.061}$$

AS 1720.1
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$$= 14.71 \left(\frac{12400}{45}\right)^{-0.480} \left(\frac{12.2}{16.0}\right)^{-0.061}$$

$$= 1.01$$

$$S_1 \quad = 1.25 \left(\frac{d}{b}\right)\left(\frac{L_{ay}}{d}\right)^{0.5} \text{(downwards loads, } L_{ay} = 0.9\, m)$$

240 × 45 section size: $\quad = 1.25 \left(\frac{240}{45}\right)\left(\frac{900}{240}\right)^{0.5} = 12.9$

$$\rho_b S_1 \quad = 13.0 \rightarrow k_{12} = 1.5 - (0.05 \times \rho_b S_1)$$

$$k_{12} \quad = 1.5 - (0.05 \times 1.01 \times 12.9)$$

$$\rightarrow k_{12} \quad = 0.85$$

300 × 45 section size: $\quad = 1.25 \left(\frac{300}{45}\right)\left(\frac{900}{300}\right)^{0.5} = 14.4$

$$\rho_b S_1 \quad = 14.6 \rightarrow k_{12} = 1.5 - (0.05 \times \rho_b S_1)$$

$$k_{12} \quad = 1.5 - (0.05 \times 1.01 \times 14.4)$$

$$=> k_{12} \quad = 0.77$$

3. Bending criteria - (establish minimum Z_p)

$$M_d \quad \geq M^*$$

and

$$M_d \quad = \phi\, k_1\, k_4\, k_6\, k_9\, k_x\, k_{12}\, f_b'\, Z$$

$$=> \quad \phi\, k_1\, k_4\, k_6\, k_9\, k_x\, k_{12}\, f_b'\, Z \geq M^*$$

and

$$Z_p \quad \geq \frac{M^*}{\phi\, k_1\, k_4\, k_6\, k_9\, k_x\, k_{12}\, f_b'}$$

Try 240 x 45 section size

$$k_{12} \quad = 0.85$$

$$k_x \quad = \left(\frac{95}{240}\right)^{0.167} = 0.86$$

Minimum required

$$Z_p \quad = \frac{16.0 \times 10^6}{0.95 \times 1.0 \times 1.0 \times 0.9 \times 1.0 \times 0.86 \times 0.85 \times 45}$$

$$= 569 \times 10^3 mm^3 > Z_{xx} = 432 \times 10^3 mm^3$$

for 240 × 45 section size

$\rightarrow$ NOT OK

Try 300 x 45 section size

$$k_{12} \quad = 0.77$$

$$k_x \quad = \left(\frac{95}{300}\right)^{0.167} = 0.83$$

Minimum required

$$Z_p \quad = \frac{16.0 \times 10^6}{0.95 \times 1.0 \times 1.0 \times 0.9 \times 1.0 \times 0.83 \times 0.77 \times 45}$$

$$= 651 \times 10^3 mm^3 < Z_{xx} = 675 \times 10^3 mm^3$$

for 300 × 45 section size

$\rightarrow$ OK

=>REQUIRE 240x 45 BBF LVL FOR BENDING STRENGTH

4. Shear criteria - (establish minimum A_s)

$$V_d \quad \geq V^*$$

and

$$V_d \quad = \phi\, k_1\, k_4\, k_6\, k_9\, f_s'\, A_s$$

$$\rightarrow \quad \phi\, k_1\, k_4\, k_6\, k_9\, f_s'\, A_s \geq V^*$$

and

$$A_s \geq \frac{V^*}{\phi\, k_1\, k_4\, k_6\, k_9\, f_s'}$$

Minimum required

$$A_s = \frac{13.4 \times 10^3}{0.95 \times 1.0 \times 1.0 \times 0.9 \times 1.0 \times 4.6}$$
$$= 3407\ mm^2 < A_s = 13500\ mm^2$$
for 300×45 section size

$\rightarrow$ OK

5. Serviceability Limit State

Design Load Combinations:
$$G$$
$$\psi_s Q \qquad \psi_s = 0.7\ (uniform)\ and\ 1.0\ (point)$$
$$W_s$$

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:

$$\Delta_{max} = j_2 \times j_6 \times \Delta_G$$

$$\Delta_{max} = \frac{2.0 \times 1.0 \times 2.0 \times 10^{11}}{EI}$$

300×45 BBF LVL,

$$E = 13200\ MPa$$
$$I_{xx} = 101.3 \times 10^6\ mm^4$$

$$\Delta_{max} = \frac{2.0 \times 1.0 \times 2 \times 10^{11}}{13200 \times 101.3 \times 10^6}$$
$$\Delta_{max} = 0.3\ mm\ <\ 9mm$$

$\Rightarrow$ OK

7. Under Imposed load:

$$\Delta_{max} = j_2 \times j_6 \times \Delta_Q$$

$$= \frac{1.0 \times 1.0 \times 1.4 \times 10^9}{EI}$$

$$\Delta_{max} = \frac{1.0 \times 1.0 \times 1.4 \times 10^9}{13200 \times 101.3 \times 10^6}$$

$$\Delta_{max} = 0.001\ mm\ <\ 9mm$$

$\Rightarrow$ OK

8. Under Wind load:

$$\Delta_{max} = j_2 \times j_6 \times \Delta_{W_s}$$

$$= \frac{1.0 \times 1.0 \times 6.5 \times 10^9}{EI}$$

$$\Delta_{max} = \frac{1.0 \times 1.0 \times 6.5 \times 10^9}{13200 \times 101.3 \times 10^6}$$

$$\Delta_{max} = 0.005\ mm < 9mm$$

=>   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.

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 (an archived copy of this document is available by contacting the EWPAA). The design aides included within this section facilitate the design of box beams with 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.
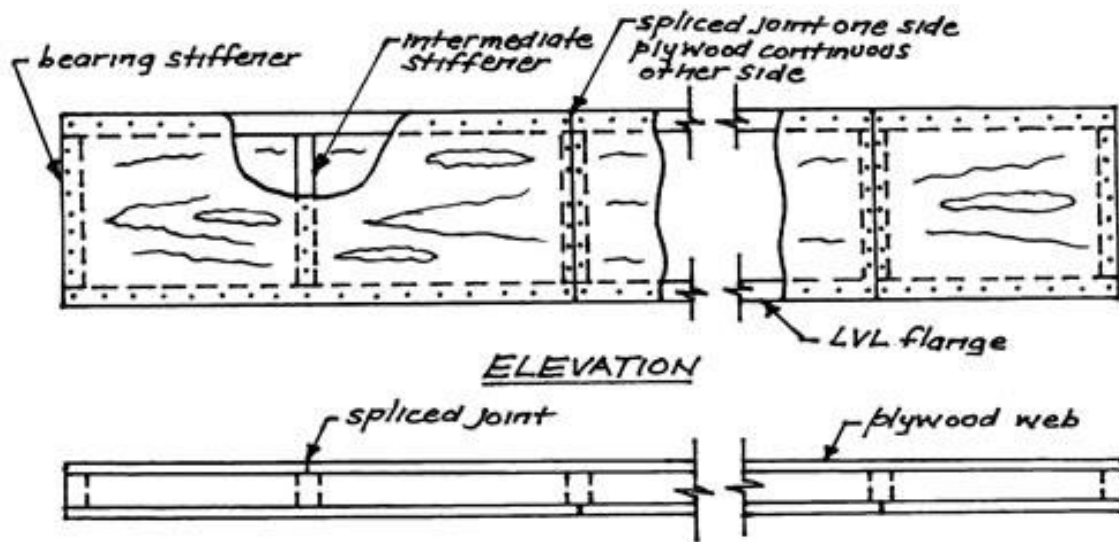
PLAN

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 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.

D = Beam Depth

Minimum slope 1 in 10 glued with rigid permanent adhesive of the resorcinaol or epoxy types

Scarf joint

Splice plate

D

2 x D

Spliced joint

Note: Splice plate must be at least the same size and stress grade as the flange and must be glues with a rigid permanent adhesive of the resorcinol or epoxy type.
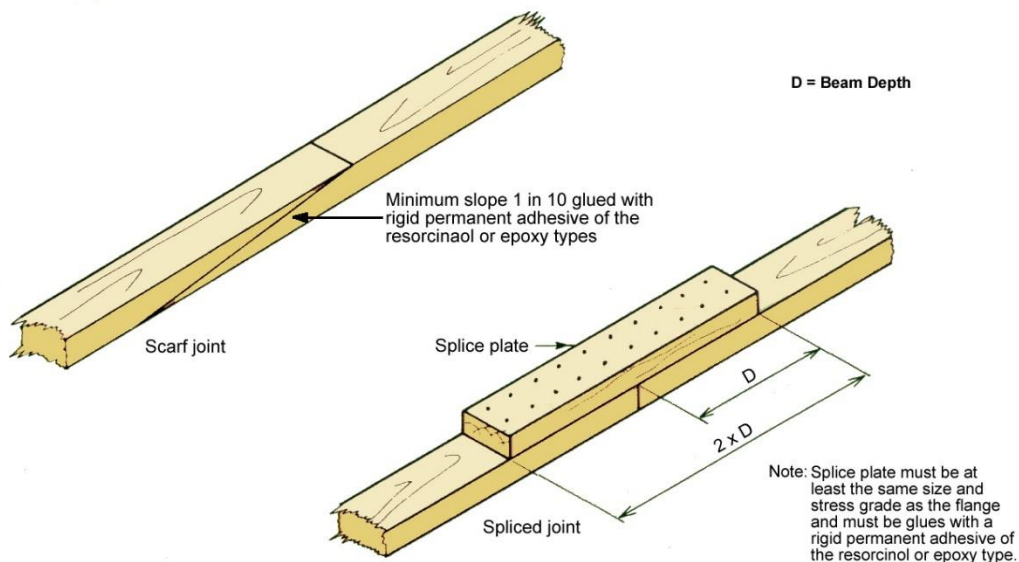
Figure 8-2: Jointing of Seasoned Flanges

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 Appendix I2.5 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 localised 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 (an archived copy of this document is available by contacting the EWPAA) . 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. 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
   (b) Depth/Breadth (D/B) ratio
   (c) Beam deflection approximated from bending deflection

   Appendix Table A8-4(a) & Fig A8-2

   Appendix Table A8-4(b)

   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 $1.5 \times \Delta_b$ to $2.0 \times \Delta_b$

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

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.

Simply supported beam with a centre point load, P:	Simply supported beam with a uniformly distributed load, w:
$\Delta_b = \dfrac{j_2 P L^3}{48\,EI}$	$\Delta_b = \dfrac{j_2\,5wL^4}{384\,EI}$
$F = \dfrac{L^3}{48\,EI}$	$F = \dfrac{5\,L^4}{384\,EI}$
$=> \Delta_b = j_2\,P\,F$	$=> \Delta_b = j_2\,w\,F$
$\Delta_\tau \approx ((1.5\ to\ 2.0) \times \Delta_b$	$\Delta_\tau \approx (1.5\ to\ 2.0) \times \Delta_b$
$=> \Delta_\tau \approx (1.5\ to\ 2.0) \times j_2\,P\,F$	$=> \Delta_\tau \approx (1.5\ to\ 2.0) \times j_2\,w\,F$
=> Select F(max) from Table A8-5 such that $F_{max} \le \dfrac{\Delta_\tau(max.allowable)}{(1.5\ to\ 2.0)P\ j_2}$	=> Select F(max) from Table A8-6 such that $F_{max} \le \dfrac{\Delta_\tau(max.allowable)}{(1.5\ to\ 2.0)w\ j_2}$

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:

$$\frac{M^*}{\phi\,k_1\,k_4\,k_6\,k_x} \le \frac{2\,f_t'\,(EI)_{xn}}{E_f\,d}$$

Check compression flange:

$$\frac{M^*}{\phi\,k_1\,k_4\,k_6\,k_9\,k_{12}} \le \frac{2\,f_c'\,(EI)_x}{E_f\,d}$$

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

k_x applies to tension and bending strength and is specified by the individual manufacturers of the product or as per AS1720.1.

3. **Check Panel Shear Capacity:**

Determine critical load case for shear and check the plywood web capacity for panel shear

Both webs continuous

$$\frac{V^*}{\phi\,k_1\,k_{12}\,k_{19}\,g_{19}} \le \frac{f_s'\,(EI)_x\,n\,t_w\,A_s}{(EQ)_x}$$

At web splice

$$\frac{V^*_{(at\ web\ splice)}}{\phi\,k_1\,k_{12}\,k_{19}\,g_{19}} \le \frac{f_s'\,(EI)_{xn}\,n\,t_w}{(EQ)_x}$$

(i.e. one web continuous only)

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

$$N_{d,j} = \phi\, k_1\, k_{13}\, k_{14}\, k_{16}\, k_{17}\, Q_k$$

Nail spacing

$$s = \frac{N_{d,j}}{q}$$

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

$$A_f = Area\ of\ flange = 2\, b_f\, t_f$$

$$A_w = Area\ of\ web = 2\, d\, tw$$

$$A_s = Web\ shearArea = 2\, d_n\, t_w$$

where

d_n = Depth between top and bottom flange web nailing

$= (d - t_f), usually$

$y_f = \dfrac{(d-t_f)}{2}$

t_w b_f t_f d d_w

On construction drawings plywood face grain direction should be designated thus:

Plywood Webbed Beam Dimensions
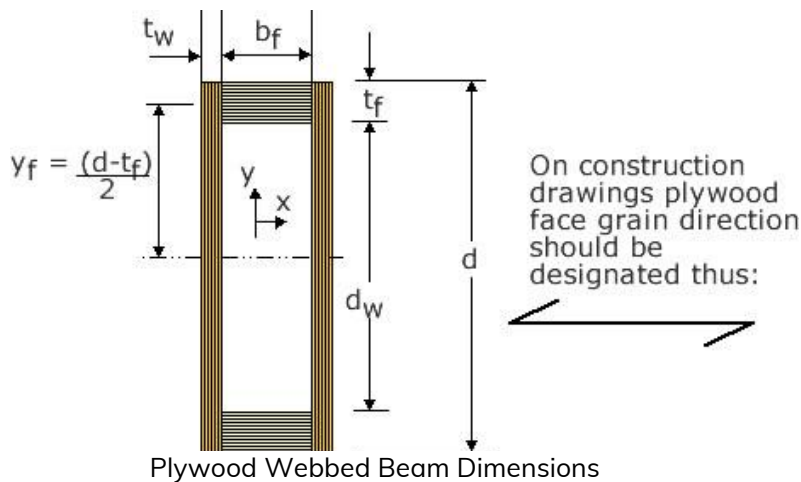
First Moment of Area

Q_{xf} = First Moment of Area of the flange about the beam x axis

$$A_f y_f = b_f\, t_f\, \frac{(d-t_f)}{2}$$

Q_{xw} = First Moment of Area of the webs about the beam x axis

$$= \frac{n\, k_{34}\, t_w d^2}{8}$$

$(EQ)_x$ = 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

$$= \frac{b_f(d^3 - d_w^3)}{12}$$

I_{xw} = Second Moment of Area of the web about the beam x-axis

$$= \frac{n\, k_{34}\, t_w\, d^3}{12}$$

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 factor)

I_{yf} = Second Moment of Area of the Flange about the beam y-axis

$$= \frac{2t_f b_f^3}{12}$$

I_{yw} = Second Moment of Area of the Web about the beam y axis

$$= \frac{k_{34} t_w d(b_f + t_w)^2}{2}$$

(using close approximation $2 A_w x_f^2$ where $x_f = \frac{t_w}{2} + \frac{b_f}{2}$)

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

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

8.2. 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.

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Given that:
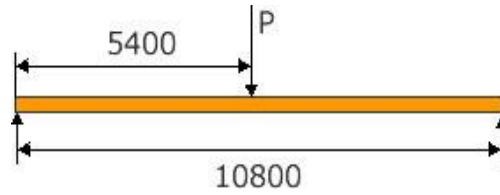
$$P_{DL} = 10.8\ kN$$
$$P_{LL} = 16.2\ kN$$
$$P_{WL} = -57.8\ kN\ (ult)$$
$$= -33.0\ kN\ (serv)$$

Deflection Limits:

$$DL = \frac{Span}{300}\ to\ 10mm\ max$$

$$LL = \frac{Span}{250}\ to\ 10mm\ max$$

$$WL = \frac{Span}{150}\ to\ 10mm\ max$$

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.
   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.
   $$=>\Delta_\tau = 1.75 \times \Delta_b$$
   For a simply supported beam with a central point load:

   $$\Delta_b = \frac{j_2 \times PL^3}{48(EI)_x}$$

   &

   $$\Delta_\tau \sim \frac{1.75\ j_2 \times PL^3}{48(EI)_x}$$

   $$F = \frac{L^3}{48EI} \quad where \quad F = flexibility\ coefficient$$

   $$=> \Delta_\tau = 1.75 \times j_2 \times PF$$

   $$=> F_{(max)} \leq \frac{\Delta_\tau}{1.75\ j_2\ P}$$

Load Type	Load (kN)	Deflection limit (mm)	j_2	$\frac{\Delta max}{1.75 \times j_2 \times P} \left(\frac{kN}{mm}\right)$
DL	10.8	10	2	0.26
LL	16.2	10	1	0.35
WL	-33.0	10	1	0.17

=>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 45 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		
$\dfrac{2f_t'\,(EI)_{xn}}{E_f\,d}$	$= 152\ kNm$	
Moment Capacity - Compression Flange		
$\dfrac{2\,f_c'\,(EI)_x}{E_f\,d}$	$= 247\ kNm$	
Web Shear		
$\dfrac{f_s'\,(EI)_x\,n\,t_w\,A_s}{(EQ)_x}$	$= 51\ kN$	
Web Shear at splice (only one web continuous)		
$\dfrac{f_s'\,(EI)_{xn}\,n\,t_w}{(EQ)_x}$	$= 26 kN$	
Unit Shear Flow for Nail Connection		
$\dfrac{V^*\,(EQ)_{xf}}{(EI)_x\,n}$	$= 0.99 \times 10^{-3}\ mm^{-1}$	

Table 8-2: Beam Capacities

Check factored loads and critical load cases:

Loads	Factored load	Load combination (kN)	k_1	V^* (kN)	$\dfrac{V^*}{k_1}$ (kN)	M^* (kNm)	$\dfrac{M^*}{k_1}$ (kNm)	
DL	$1.2G$	13.0	0.57	6.5	11.4	35.1	61.6	Load Combinations AS1170.1 Cl.4.2.2
$DL + LL$	$1.2G + 1.5Q$	37.3	0.97	18.7	19.3	100.1	103.2	k_1:AS1720.1 Table 2.3 App G, Table G1
$DL + WL$	$0.9G + W_u$	-48.1	1.0	-24.0	-24.0	-129.9	-129.9	

2. Flange Bending Capacity

Check Tension Flange

$$= \frac{M^*}{\phi\,k_1\,k_4\,k_6\,k_x\,k_{12}}$$

$$= \frac{128.8}{(0.9 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0)}$$

$$= 143.1\ kNm\ (< 152\ kNm)\ \ OK$$

Check Compression Flange

Calculate k_{12}:

$$S_1 = \left(\frac{5.3 L_{ay}\,(EI)_x}{h_1\,D\,(EI)_y}\right)^{0.5}$$

$$= \left(\frac{5.3 \times 5400 \times 31894 \times 10^9}{5.5 \times 900 \times 803.7 \times 10^9}\right)^{0.5}$$

$$= 15.2$$

$$\rho_b \quad = 11.39 \left(\frac{E_f}{f_c'}\right)^{-0.408} r^{-0.074}$$

$$= 11.39 \left(\frac{13200}{45}\right)^{-0.408} 1^{-0.074}$$

$$= 1.12$$

$$\rho_b \, S_1 \quad = 15.2 \times 1.12$$

$$= 17.0$$

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

$$= 1.5 - 0.05 \times 1.12 \times 15.2$$

$$= 0.65$$

$$\frac{M^*}{\phi \, k_1 \, k_4 \, k_6 \, k_x \, k_{12}} \quad = \frac{128.8}{(0.9 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 0.65)}$$

$$= 220 \, kNm \ (< 247 \, kNm) \ \ OK$$

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, $k_{12} = 0.56$.

$$\frac{V^*}{\phi \, k_1 \, k_{12} \, k_{19} \, g_{19}} \quad = \frac{24}{(0.85 \times 1.0 \times 0.52 \times 1.0 \times 1.0)}$$

$$= 54.3 \, kN \ (> 51 \, kN) \ \text{not OK}$$

Panel Shear at Web Splice ($k_{12} = 1.0$ at web splice)

$$\frac{V^*_{(at \, web \, splice)}}{\phi \, k_1 \, k_{12} \, k_{19} \, g_{19}} \quad = \frac{23.8}{0.85 \times 1.0 \times 1.0 \times 1.0 \times 1.0}$$

$$= 28 \, kN \ (> 26 \, kN) \ \text{not OK}$$

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)
$1.2G$	6.5	0.57	11.4
$1.2G + 1.5Q$	18.7	0.86	21.7
$0.9G + Wu$	-24.0	1.14	-21.1

Required nail spacing $s = \frac{\phi N_{d,j}}{q}$ Characteristic capacity, Q_k, of 2.8mm nail in JD4 timber is 665 N.

Design capacity per nail

$$\phi N_{d,j} \quad = \phi \, k_1 \, k_{13} \, k_{14} \, k_{16} \, k_{17} \, Q_k$$

$$= 0.8 \times 0.86 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \times 665$$

$$\phi N_{d,j} \quad = 503 \, \frac{N}{nail}$$

Shear flow at connection:

$$q \quad = \frac{V^*(EQ)_{xf}}{(EI)_x \, n}$$

$$= \frac{(18.7 \times 10^3 \times 0.94 \times 10^{-3})}{2}$$

$$q \quad = 8.8 \, \frac{N}{mm}$$

$$=> s_{max} \quad = \frac{503}{9.4}$$

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$$= 57 \, mm$$

$$\rightarrow \text{Use 2.8 } \varnothing \text{ nails at 50mm spacing}$$

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	$\dfrac{j_2\, PL^3}{48(EI)_x}$	$\dfrac{j_2\, 5\, wL^4}{384(EI)_x}$
Shear	$\dfrac{j_2\, PL}{4GA_s}$	$\dfrac{j_2\, wL^2}{8GA_s}$
Nail slip*	$\dfrac{d_n\, L}{64}\left(\dfrac{E_f\, A_f\, s\, P}{(EI)_x\, A}\right)^2$	$\dfrac{d_n\, L^3}{192}\left(\dfrac{E_f\, A_f\, s\, w}{(EI)_x\, A}\right)^2$

*Refer to Appendix-Chapter 8 for nail slip deflection equations.

Load Type	j_2	Load		Estimated Deflection, mm			Total (mm)	
		Point (kN)	UDL (kN/m)	Bending	Shear	Nail Slip		
DL	2	10.8	0.16	19.5	10	1.7	31.2	Not OK
LL	1	16.2		12.5	7	1	20.5	Not OK
WL	1	-33		25.4	14	4.3	43.7	Not Ok

The designer has to make some adjustments to the design parameters.

8.3. Box Beam Portal Joints

There are a number of instances in dwelling construction where a relatively large unsupported span is required within a wall which may have little wall available to transfer the lateral wind or seismic loads.

Such situations arise when large sliding or bi-fold doors are required, where large window openings are incorporated to allow full appreciation of the views and where garages, either attached to a dwelling or standalone require bracing.

To provide at least some answers to a somewhat complex question a Forest & Wood Products Australia (FWPA) funded, EWPAA instigated research program was initiated. This entailed the construction of, and testing of, four box portal joints subjected to gravity type and simulated lateral wind loads at Central Queensland University, Rockhampton and reported in "Results from the Testing of Four Plywood Sheathed, Timber Framed Box Beam Portal Joints" by C G "Mick" McDowall.

Figure 8-3 shows the two plywood only joints tested. Obviously only one joint type was tested per portal.

Figure 8-4 shows the bolted and rodded joints tested.

Figure 8-5 shows a discontinuous plywood joint located in the Loading Frame prior to testing.
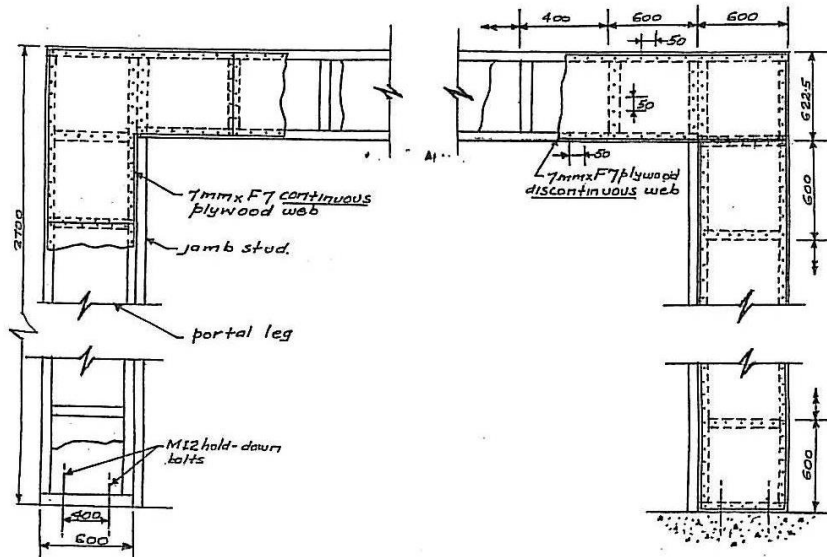
Figure 8-3: Two Plywood Only Dependent Joints
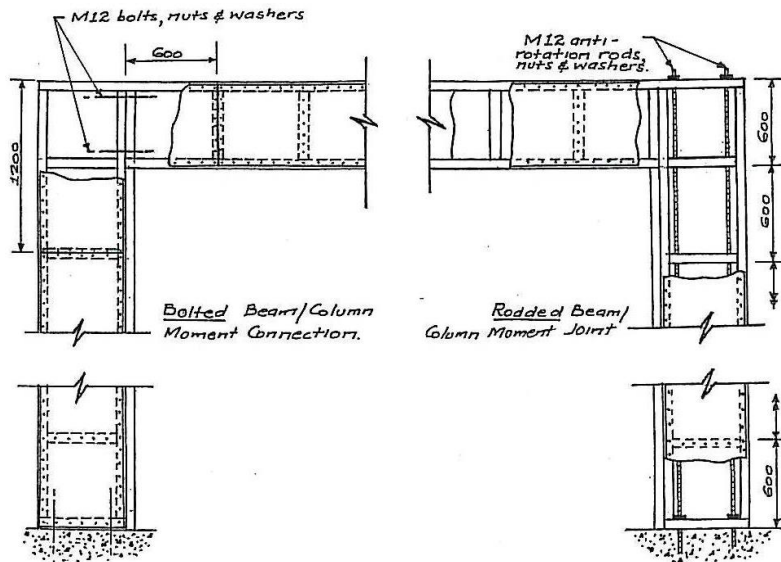
Figure 8-4: Bolted and Rodded Joints

Figure 8-5: Typical Portal Frame with Discontinuous Plywood Joints

Identifying the portal joints thus:

- plywood continuous across the portal leg as shown in Figure 8-5, as TP1;
- plywood continuous around the joint at the portal leg/beam joint as TP2;
- bolted joint as TP3;
- rodded joint as TP4

Portal Joint Racking Resistances

The racking resistances assigned herein to the four portal joints can only be applied in the design situation provided they meet all of the stated criteria.

Portal Joints TP1 and TP2

The racking resistance of these two joints depend entirely upon the plywood and its fixity to the timber framing.

PORTAL JOINTS – TP1 AND TP2	
Plywood	– 7mm DD F8 (minimum)
Framing	– 90 x 45 MGP10, JD5 (minimum)
	Framing procedures to follow those given in Figures 3 and 4 of the Research Proposal in Appendix A1
Nailing	– 2.8 ø x 35 long galvanised clouts
	– 3.15 ø x 75 bullet head nails (1 off into end grain of stiffeners and studs)
	– 50 / 50 nailing pattern
Tie down Bolts	– M12 with nuts and 50 x 50 x3 washers
BRACING CAPACITY: 3.75 kN/m	

Portal Joint TP3

The increase in racking resistance for TP3 can only be attributed to the inclusion of the two bolts used to connect the beam section to the portal legs.

PORTAL JOINT – TP3	
Plywood	– 7mm DD F8 (minimum)
Framing	– 90 x 45 MGP10, JD5 (minimum)
	Framing procedures to follow those given in Figure 5 of the Research Proposal in Appendix A1
Nailing	– 2.8 ø x 35 long galvanised clouts
	– 3.15 ø x 75 bullet head nails (1 off into end grain of stiffeners and studs)
	– 50 / 50 nailing pattern
Tie down Bolts	– M12 with nuts and 50 x 50 x3 washers
BRACING CAPACITY: 5 kN/m	

Portal Joint TP4

TP4 is easily the best candidate if choosing a joint on the basis of racking resistance alone. This could well be the case if constructing a stand-alone garage, shed or similar type structure.

PORTAL JOINT – TP4	
Plywood	− 7mm DD F8 (minimum)
Framing	− Framing procedures to follow those given in Figure 6 of the Research Proposal in Appendix A1
Nailing	− 2.8 ø x 35 long galvanised clouts
	− 3.15 ø x 75 bullet head nails (1 off into end grain of stiffeners and studs)
	− 50 / 50 nailing pattern
Tie down Bolts	− By means of the rods
BRACING CAPACITY :7.5 kN/m	

Closure

The plywood only type joints may provide sufficient racking resistance for a garage having a structural ceiling, interconnected to the house, thus providing a structural diaphragm. For the stand-alone situation, the rodded construction provides the only viable solution. It would be suitable for spans, certainly to 6m, but not exceeding 8m. Cases would have to be viewed individually to ascertain whether there were walls or open under situations. Also, in the stand-alone case this does not solve the bracing requirements in the direction perpendicular to the portal span.

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
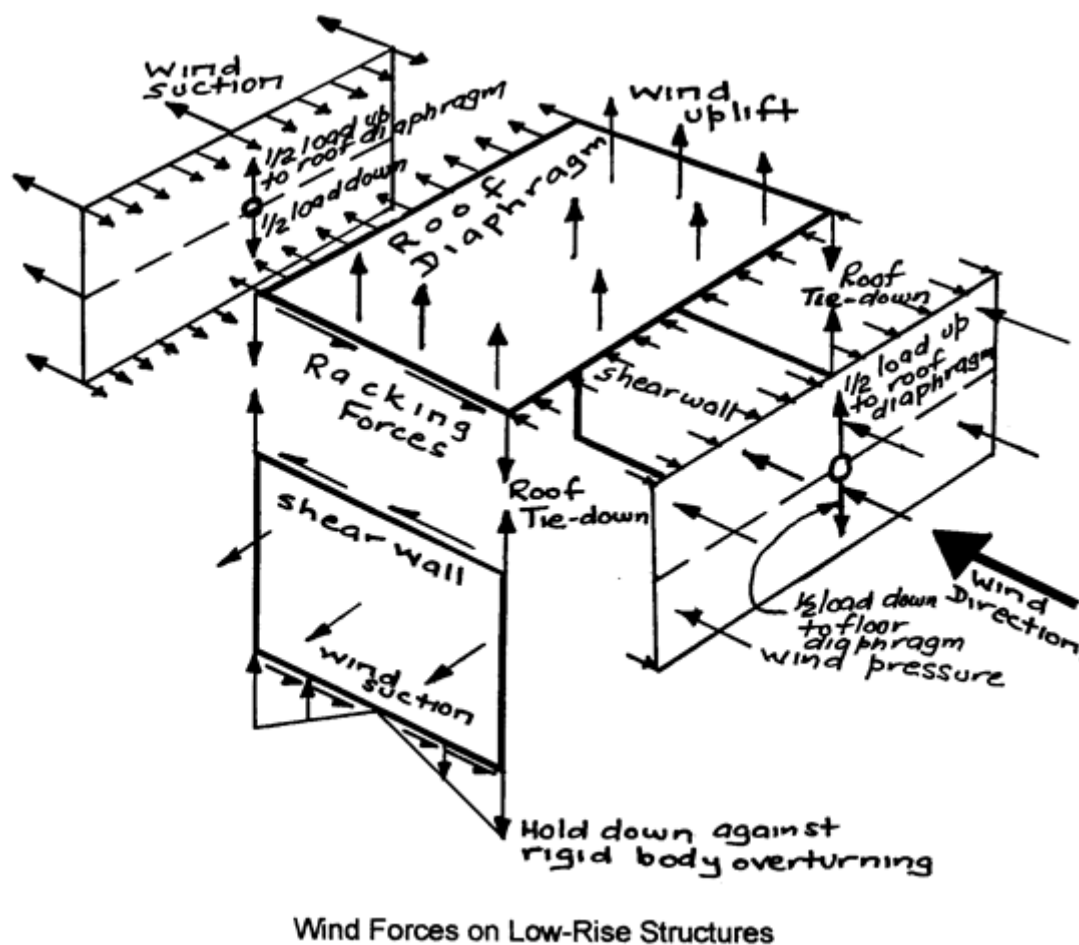
Figure 9-1: Location and Function of the Shearwalls and Diaphragm

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.
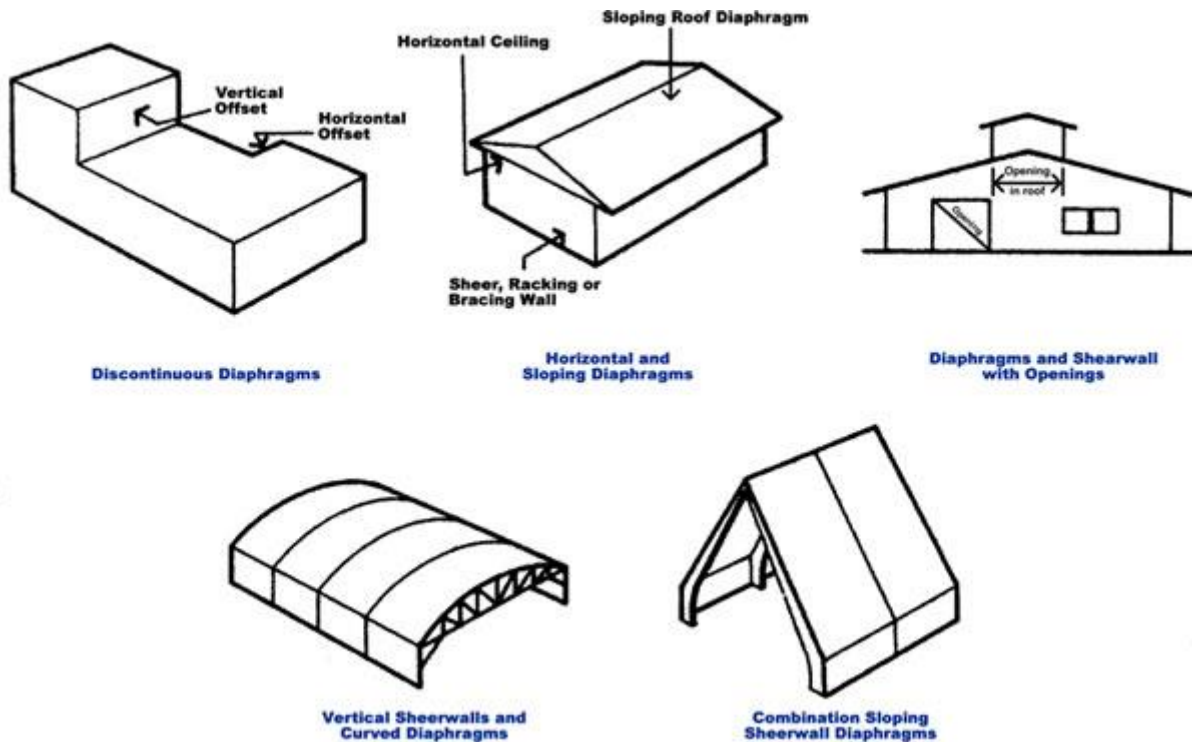
Figure 9-2: Shearwall and Diaphragm Applications

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.
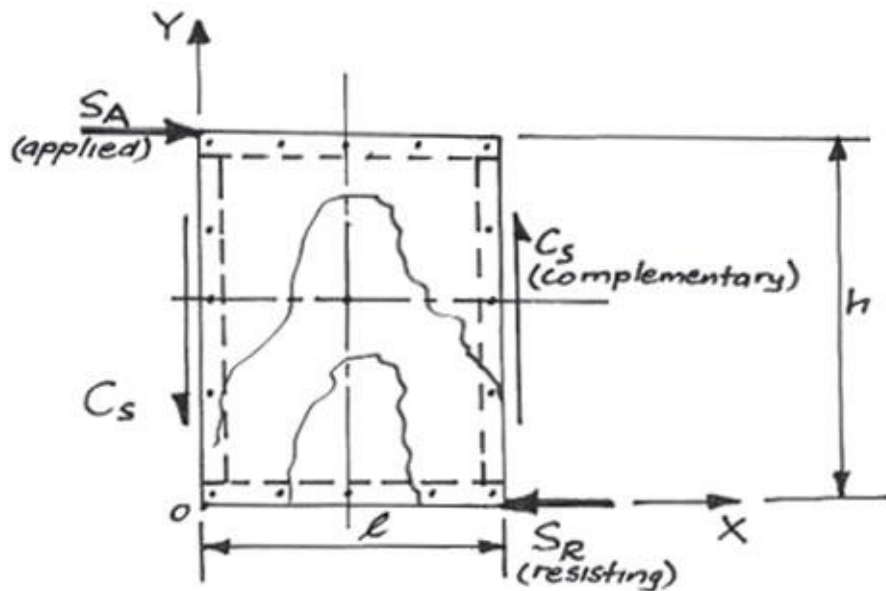
Figure 9-3: Panel Subjected to Shear

From statics:

$$\Sigma F_x \quad = 0 : S_A = S_R$$

$$\Sigma F_y \quad = 0 : C_s = C_s \text{ (complimentary shears)}$$

$$\Sigma M_0 \quad = 0 = S_A h - C_s l$$

$$\frac{S_A}{l} \quad = \frac{C_s}{h}$$

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.

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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
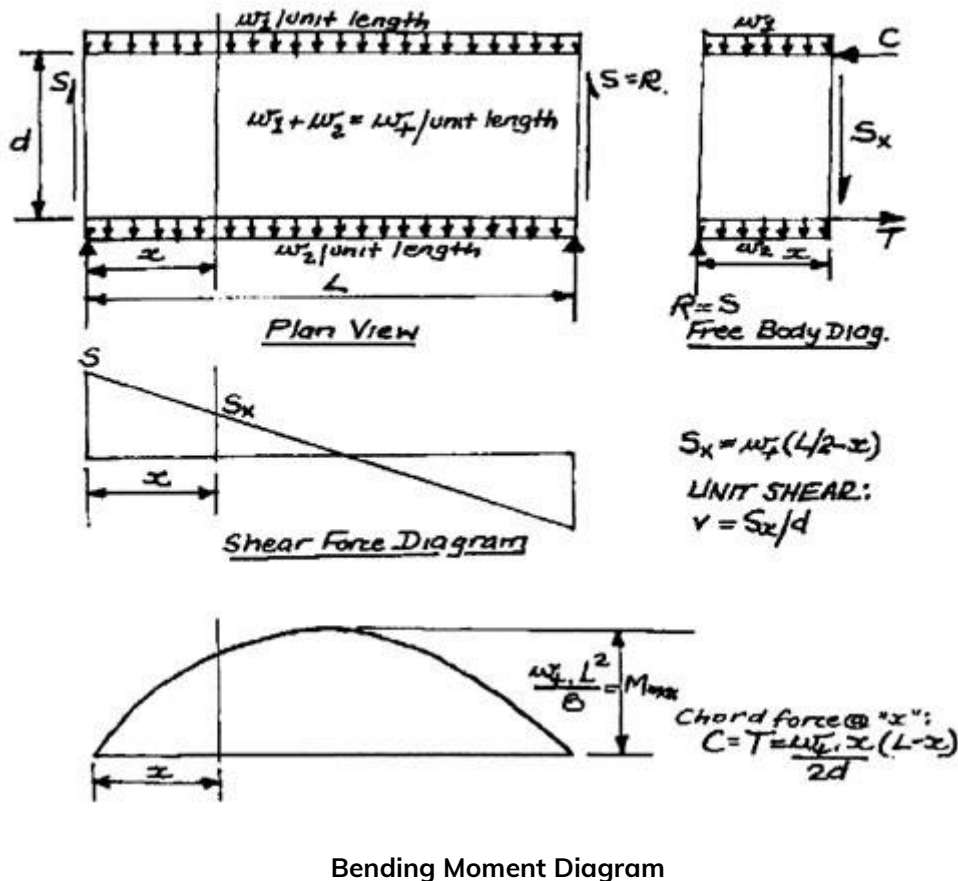
Figure 9-4: Diaphragm Design Formula for Lateral Loading

9.4. Diaphragm Design – Methodology

The design method and values presented in this Guide 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.
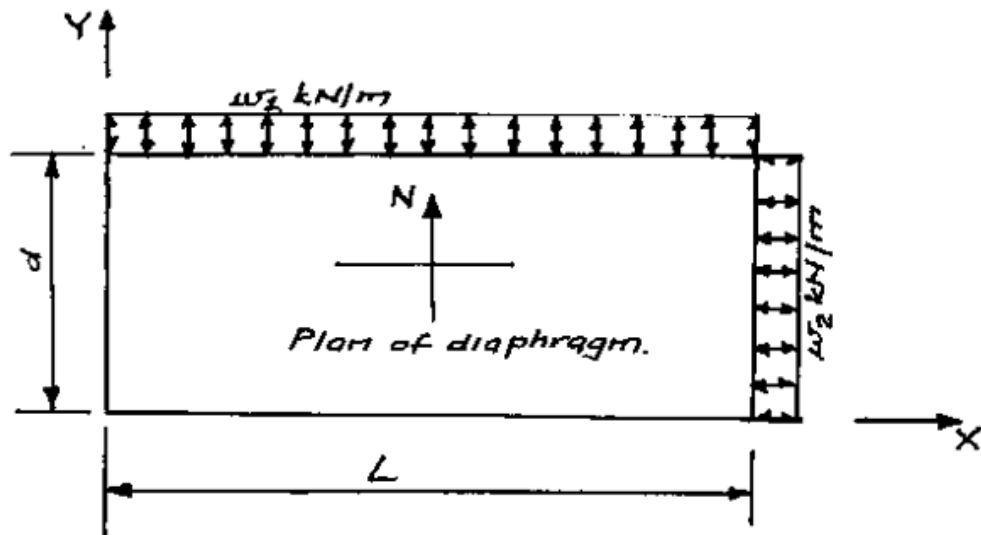
Figure 9-5: Wind Forces on Diaphragm

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: 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
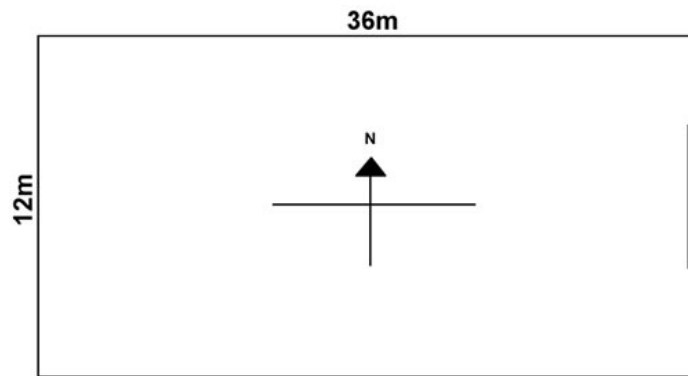
Figure 9-6: Plan of Building

Diaphragm – Worked Example 1

1. Lateral Wind Loads on Roof Diaphragm

Design wind pressure acting normal to a surface, Pa	$P = 0.6 \left(V_{des,\theta}\right)^2 C_{fig}\, C_{dyn}$	1170.2 Cl 2.4.1
Building orthogonal design wind speeds	$V_{des,\theta} = V_{sit,\beta}$	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	$V_{1000} = 60\ \frac{m}{s}$	1170.2 Table 3.1
Regional wind speed for serviceability calculations	$V_{20} = 38\ \frac{m}{s}$	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	$C_{fig} = C_{p,e}\, K_a\, K_c\, K_l\, K_p$	
For internal pressures	$C_{fig} = C_{p,i}\, K_c$	
External pressure coefficient, windward wall	$C_{p,e} = 0.7$	1170.2 Table 5.2(A)
Internal pressure coefficient, windward wall	$C_{p,i} = -0.65$	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	$K_l = 1$	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	$v_{des,\theta} = 60 \times 0.95 \times 0.84 \times 1 \times 1 = 47.9\,\frac{m}{s}$
Design wind pressure for ultimate limit states strength	$P = 0.6 \times 47.9^2 \times 10^{-3}(0.7 + 0.65) = 1.86\,kPa$
Wind force, w on diaphragm	$w = 1.86 \times \left(\frac{5.4}{2}\right)$
(half of wind load on 5.4 m wall is transferred directly to foundations)	$= 5.0\,\frac{kN}{m}$
Total wind force, W on roof diaphragm, in North-South direction	$W_{N-S} = 5.0\,\frac{kN}{m} \times 36\,m = 180\,kN$
Total wind force, W on roof diaphragm, in East-West direction	$W_{E-W} = 5.0\,\frac{kN}{m} \times 12\,m = 60\,kN$

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

Diaphragm design unit shear from Figure 9-4	$v = \frac{(w_t \frac{L}{2})}{d}$
Diaphragm design unit shear in North-South direction	$v_{N-S} = \frac{(5.0 \times \frac{36}{2})}{12} = 7.5\,\frac{kN}{m}$
Diaphragm design unit shear in East-West direction	$v_{E-W} = \frac{(5.0 \times \frac{12}{2})}{36} = 0.83\,\frac{kN}{m}$

LIMIT STATES SERVICEABILITY:

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

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

Diaphragm design unit shear from Figure 9-4	$v = \frac{(w_t \frac{L}{2})}{d}$
Diaphragm design unit shear in North South direction	$v_{N-S} = \frac{(2.0 \times \frac{36}{2})}{12} = 3.0\,\frac{kN}{m}$
Diaphragm design unit shear in East West direction	$v_{E-W} = \frac{(2.0 \times \frac{12}{2})}{36} = 0.33\,\frac{kN}{m}$

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

Factored Limit State Shear Capacities $\left(\frac{kN}{m}\right) \times k_1 = 1.14$

Plywood Thicknesses (mm)	Flathead Nail Size	Min. Nail Penetration into Framing (mm)	Frame width (mm)	Blocked Diaphragm				Unblocked	
				Nail Spacing (mm) at Boundary/Other Edges					
				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 $= 7.0\ kN/m$

≤6.2 kN/m : Unblocked Case 1, Nail spacing 150/300: capacity $= 6.2\ kN/m$
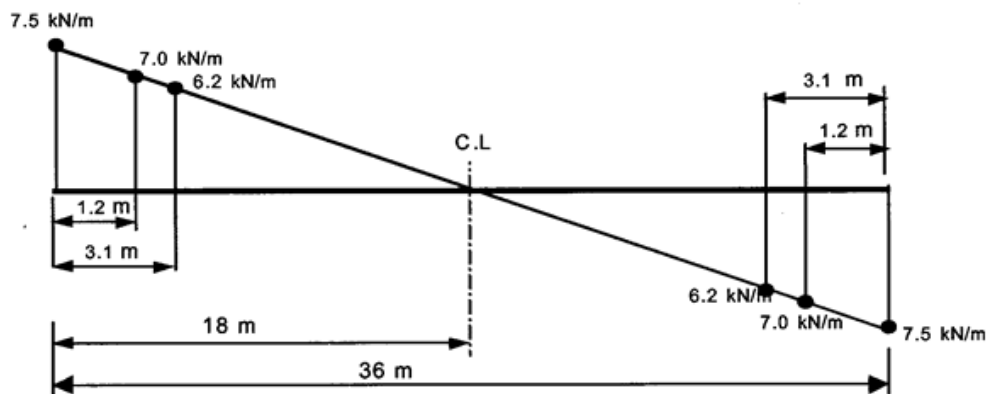
Change in Shear Locations:

$18/7.5 = (18 - x_1)/7: x_1 = 1.2\ m$

$18/7.5 = (18 - x_2)/6.2: x_2 = 3.1\ m$ say 3.6 m

Shear force in the roof diaphragm in the E-W direction:

$0.83\ \frac{kN}{m} << 4.7\ \frac{kN}{m} \rightarrow$ Unblocked

Converted to Limit States Capacity

Conversion factor used was 1.3 i.e. allowable shear capacities were multiplied up by 1.3

				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	
Minimum Structural Plywood Thickness (mm)	Flathead Nail Size (mm)	Minimum Nail Penetration into Framing (mm)	Minimum Nominal Width of Framing Member (mm)	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.
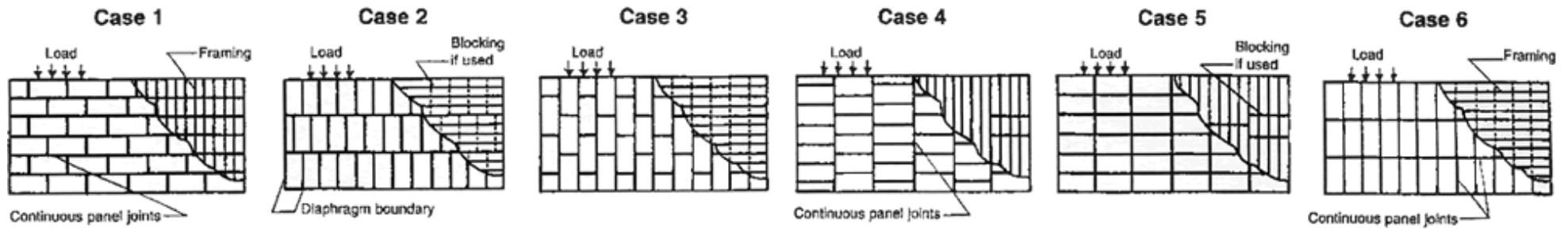
Table 9-1: Shear Capacities in kN/m for Horizontal Plywood Diaphragms

3.   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.

Blocked, 12mm EWPAA branded structural plywood with 3.75mm diameter nails at 100 centers at diaphragm boundry, 150 centers at other panel edges,300 centers on intermediate framing.

Blocked, 12mm EWPAA branded structural plywood with 3.75mm diameter nails at 150 centers at diaphragm boundry, 150 centers at other panel edges,300 centers on intermediate framing.

Unblocked, 12 mm EWPAA branded structural plywood with 3.75 mm diameter nails at 150 centres at all panel edges, 300 centres on intermediate framing.

Nails at 100mm centres

Nails at 150mm centres

Nails at 300mm centres on intermediate supports

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4. Chord Sizes and Splices

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

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

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 $= C = T = \left(\frac{w_t\,x}{2d}\right)(L - x)$ from Figure 9-4.

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

Design tensile capacity required $\quad = N_{d,t}$

where:

$$N_{d,t} = \phi\,k_1\,k_4\,k_6\,k_x\,f_t'\,A_t$$

and: $\quad N_{d,t} \geq N_t^*$

$\Rightarrow \quad N_{d,t} \geq 60\ kN$

Required $\quad A_t \geq \dfrac{N_{d,t}}{\phi\,k_1\,k_4\,k_6\,k_x\,f_t'}$

$$= \frac{60 \times 10^3}{0.9 \times 1.14 \times 1.0 \times 1.0 \times 1.0 \times 33}$$

$A_t = 1772\ mm^2$

Chord:

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

$$2 \times [130 - (2 \times 12)] \times 45 = 9540\ mm^2$$

$$> 1772\ OK$$

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

$N_{d,j} \geq N^*$

$= \phi\,k_1\,k_{16}\,k_{17}\,n\,Q_{sk}$

$\rightarrow n = \dfrac{60}{0.8 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 19.2}$

$n = 3.9$ use 4 bolts each side of centre splice

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ewpaa

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

$$\Delta_{(diaphragm)} = \sum(bending\ deflection, \Delta_b + shear\ deflection, \Delta_s + nail\ slip, \Delta_{ns} + chord\ splice, \Delta_c)$$

$$= \frac{5vL^3}{96EAd} + \frac{vL}{4Gt} + 0.188e_nL + \sum\frac{\Delta_cX}{2d}$$

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.	
$\sum(\Delta c_cX)$	= 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	$= 7.5\ \frac{kN}{m}$	
L	$= 36\ m$	
d	$= 12\ m$	
A	$= 11700\ mm^2$	
E	$= 13200\ MPa$	
G	$= 660\ MPa$	
t	$= 7.2mm$	
$V_{(3.75)}$	$= \frac{7.5}{8} = 0.94\ \frac{kN}{nail}$ (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	$= 12\ m$ for chord splice slip	
$\Delta_{(diaphragm)}$	$= \frac{5vL^3}{96EAd} + \frac{vL}{4Gt} + 0.188e_nL + \sum\frac{\Delta_cX}{2d}$	
$\Delta_{(bending)}$	$= \frac{5vL^3}{96EAd}$	

$$\Delta = \frac{5 \times 7.5 \times 36000^3}{96 \times 13200 \times 11700 \times 12000} = 9.8 \ mm$$

$$\Delta_{(shear)} = \frac{vL}{4Gt}$$

$$= \frac{7.5 \times 36000}{4 \times 660 \times 7.2} = 14.2 \ mm$$

$$\Delta_{(nail \ slip)} = 0.188 \ e_n \ L$$

$$= 0.188 \text{x} \ 1.194 \ \text{x} \ 36 = 8.1 \ mm$$

$$\Delta_{(chord \ splice)} = \sum \frac{\Delta_c X}{2d}$$

$$= \frac{4 \times 0.6 \times 12}{2 \times 12} = 1.2 \ mm$$

$$\Delta_{(diaphragm)} = 9.8 + 14.2 + 8.1 + 1.2 = 33.3 \ mm$$

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.
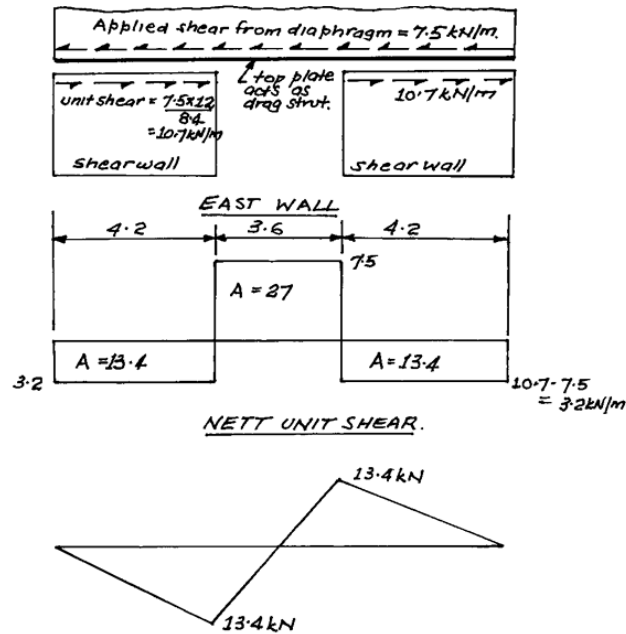
Figure 9-7: Build-up of Axial Force in Drag Strut Due to Opening

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 $3.5\frac{kN}{m}$

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 $6 \times 12\ m$.

Total wind load applied across the opening is $1.5\frac{kN}{m}$.

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:

$$\frac{6}{4.8} \times 1.5 = 1.9\frac{kN}{m}$$

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FIGURE 9-8 shows the distribution of shears around the opening due to the applied wind loading.
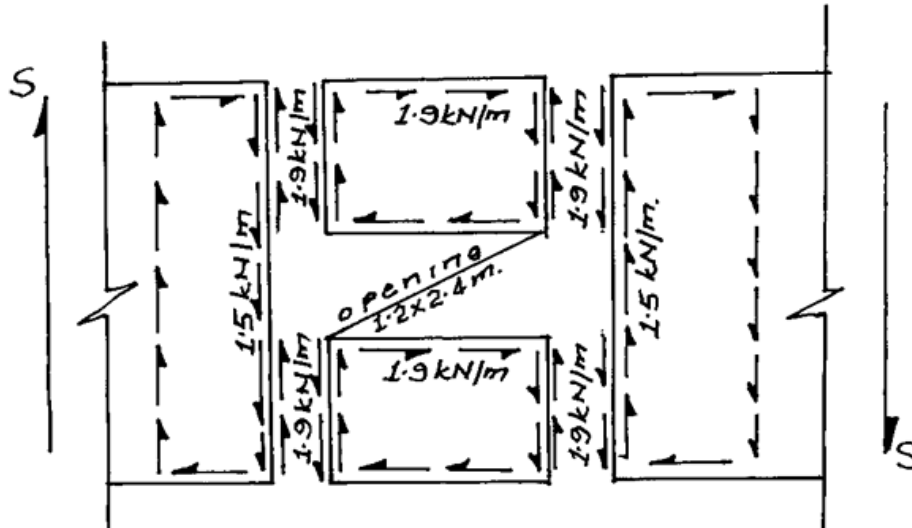
Figure 9-8: Distribution of Shears Around Opening

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 $1.2 \times 1.5 = 1.8\ kN$ 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 $1.9\frac{kN}{m}$ a horizontal shear along the edge of the opening is required for segment equilibrium.
- The horizontal force is $1.9 \times 2.4 = 4.6\ kN$
- This $4.6\ kN$ can be:
  - distributed equally either side of the opening as a lower bound;
  - totally to one side, say the left side as an upper bound.
- Since the shear capacity in the diaphragm is $3.5\frac{kN}{m}$ then:
  - for the vertical member the distance it must extend above and below the hole is:
    $\frac{0.9}{3.5-1.9} = 0.56\ m$
  - for the horizontal member assuming the lower bound the distance is:
    $\frac{4.6}{2}\ \frac{1}{3.5-1.5} = 1.15\ m$

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 $2\ kN/m$.

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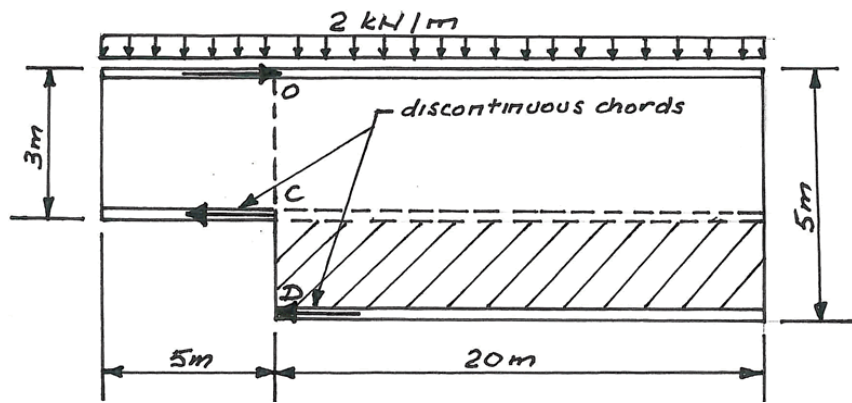
Diaphragm Plan

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 shallower 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.
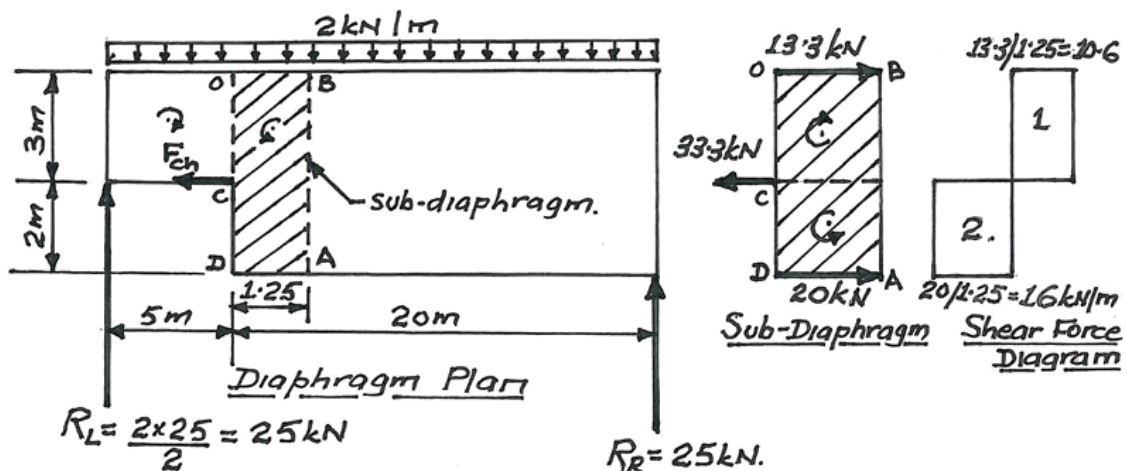
Figure 9-10: Sub-Diaphragm in Offset Diaphragm

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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.

$$\sum M_{0L} = 25 \times 5 + F_{ch} \times 3 - 2 \times 5 \times \frac{5}{2} = 0$$

$$F_{ch} = \frac{(125-25)}{3}$$

$$F_{ch} = 33 \; kN$$

Consider the isolated sub-diaphragm of FIGURE 9-10 and taking moments about A:

$$\sum M_{0L} = R_B \times 5 - 33.3 \times 2 = 0$$

$$R_B = \frac{(33.3 \times 2)}{5}$$

$$R_B = 13.3 \; kN$$

$$R_A = 20 \; kN$$

The resulting shear flows within the sub-diaphragm become:

$$v_1 = \frac{13.3}{1.25} = 10.6 \; \frac{kN}{m}$$

$$v_2 = \frac{20}{1.25} = 16 \frac{kN}{m}$$

Shear flows at the discontinuity due to the actual loading will be:

$$S_{0L} = 25 - 5 \times 2$$

$$S_{0L} = 15 \; kN \qquad\qquad S_{0R} = 15 \; kN$$

$$v_{0L} = \frac{15}{3} = 5 \; \frac{kN}{m} \qquad\qquad v_{0R} = \frac{15}{5} = 3 \; \frac{kN}{m}$$

FIGURE 9-11 summaries the shear flows and shows the result of superposing the two effects.
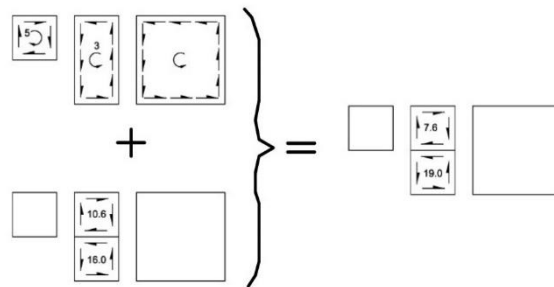
Figure 9-11: The Effect of Superposing Shear Flows

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:

$$\sum M_{0R} = 25 \times 17 + F_{LC} \times 5 - 2 \times 17 \times \frac{17}{2} = 0$$

$$F_{LC} = \frac{136}{5} = 27.2 \; kN$$

Sub-diagram forces: From the elevation shown in FIGURE 9-12:

ELEVATION
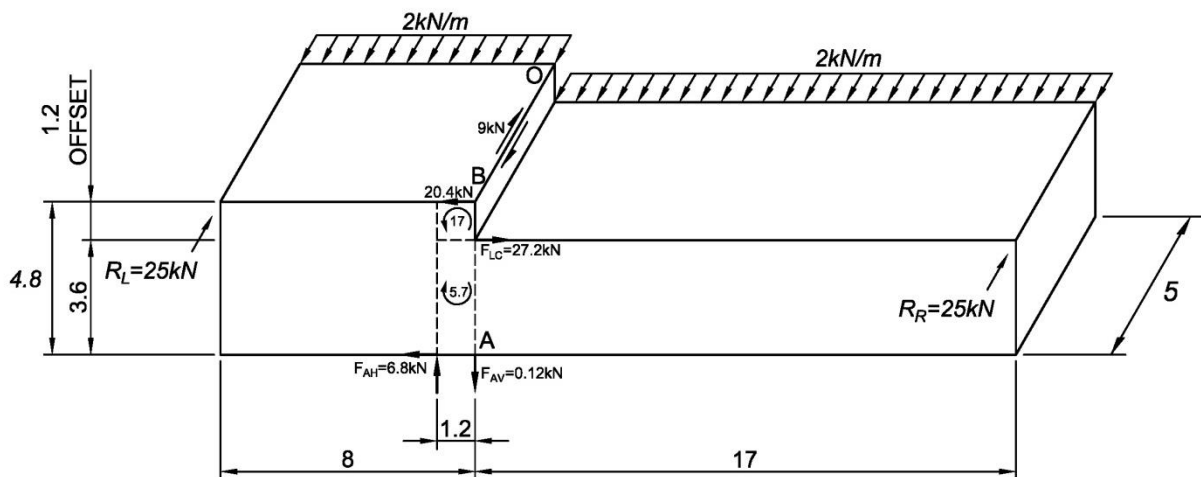
Figure 9-12: Shear Flows and Chord Forces

$$\sum M_A = 27.2 \times 3.6 + F_{uc} \times 4.8 = 0$$

$$F_{uc} = \frac{(27.2 \times 3.6)}{4.8}$$

$$F_{ch} = 20.4 \; kN$$

$$\sum F_x = 27.2 - 20.4 - F_A = 0$$

$$F_A = 6.8 \; kN$$

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Resulting shear flows within sub-diaphragm:

upper section: $v_u \qquad = \frac{20.4}{1.2} = 17 \ \frac{kN}{m}$

lower section: $v_L \qquad = \frac{6.8}{1.2} = 5.7 \ \frac{kN}{m}$

Although the anchorage force at A, directly under the vertical offset is small in this instance ($0.12kN$) it will be increased by: $\frac{(25 - 8 \times 2) \times 1.2}{5} = 2.2 \ kN$ 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.
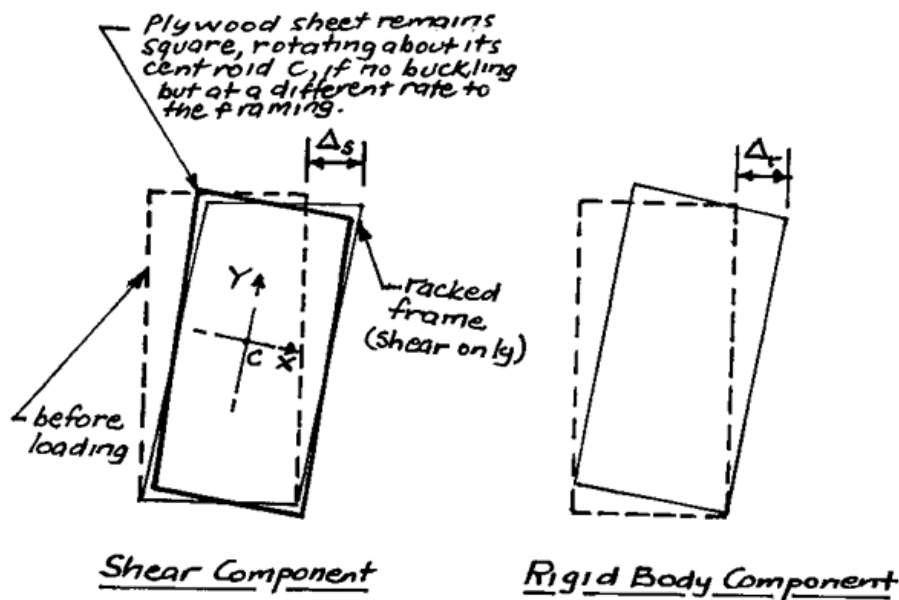
Figure 9-13: Two Major Racking Deflection Components

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:

$$k \quad = \frac{c}{h^2}\left(\frac{I_x I_y}{I_x + I_y}\right) \tag{9.1}$$

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 = considers 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.

$$k = \frac{3\,EI}{h^3}$$

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 $\frac{kN}{m}$ and with anti-rotation rod fitted the capacity is 6.4 $\frac{kN}{m}$.

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. The 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 (this document has been archived and is available by contacting the EWPAA). 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
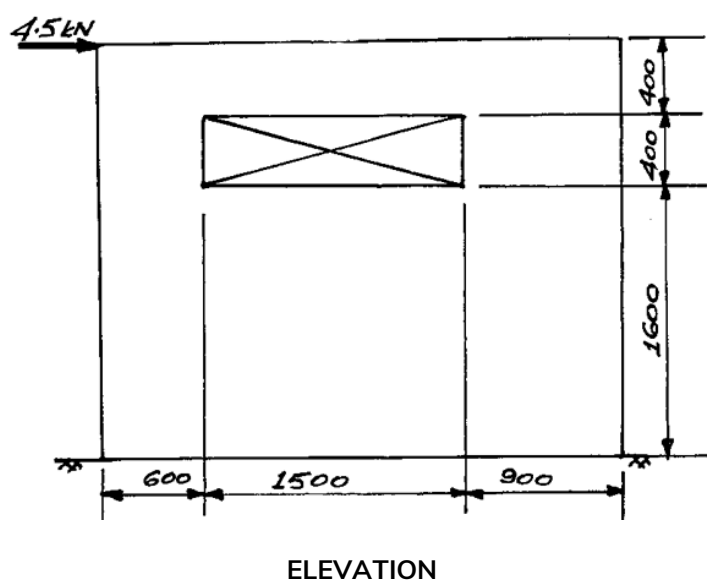
Figure 9-14: Shearwall with Opening

Shearwalls   Worked Example 1 - Accepted Solution

Consider 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 $\frac{600}{1500}$ of the 4.5 kN (1.8kN) and the 900 panel would take $\frac{900}{1500}$ of 4.5 kN (2.7 kN).

Tie-down at the ends of the panels (can be loaded in either direction) is:

$$\frac{1.8 \times 2.4}{0.6} = 7.2 \ kN \ ; \ \frac{2.7 \times 2.4}{0.9} = 7.2 \ kN$$

The unit shear to be resisted in each panel is $\frac{1.8}{0.6} = 3 \ \frac{kN}{m}$ and $\frac{2.7}{0.9} = 3 \ \frac{kN}{m}$. However, the uplift at the panel end is 7.6 kN.

For the 600 panel to attain $3 \ \frac{kN}{m}$ 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 (this document has been archived and is available by contacting the EWPAA).

Shearwalls – Worked Example 1 - Alternative Solution

An alternative approach considers 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.
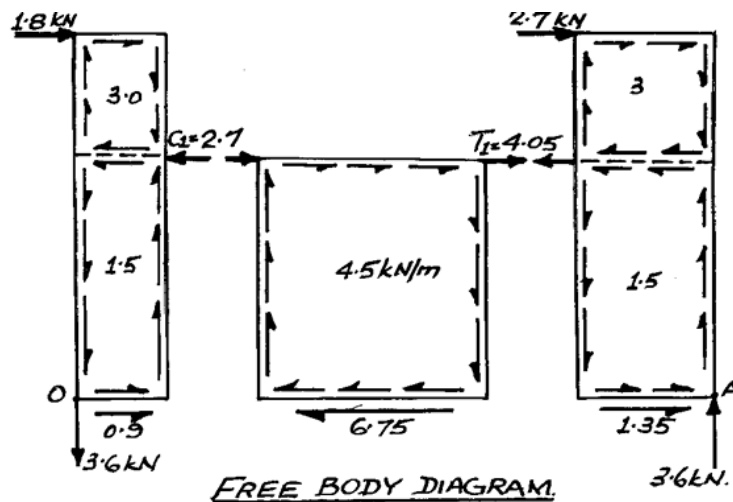
Figure 9-15: Free Body Diagram of Shearwall

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:

$$\sum M_{A0} = 1.8 \times 2.4 - C_1 \times 1.6 = 0$$
$$C_1 = \frac{(1.8 \times 2.4)}{1.6}$$
$$C_1 = 2.7 \ kN$$

For the section to the right of the opening:

$$\sum M_A = 2.7 \times 2.4 - T_1 \times 1.6 = 0$$
$$T_1 = \frac{(2.7 \times 2.4)}{1.6}$$
$$T_1 = 4.05 \ kN$$

The shear flows in the various sections of the shearwall are as shown in FIGURE 9-15.

The unit shears vary, the highest being $4.5 \ \frac{kN}{m}$ which is significantly larger than the $3 \ \frac{kN}{m}$ but uplift at 3.6 kN is significantly less than 7.6 kN.

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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 $0.84 \frac{kN}{m}$.

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.
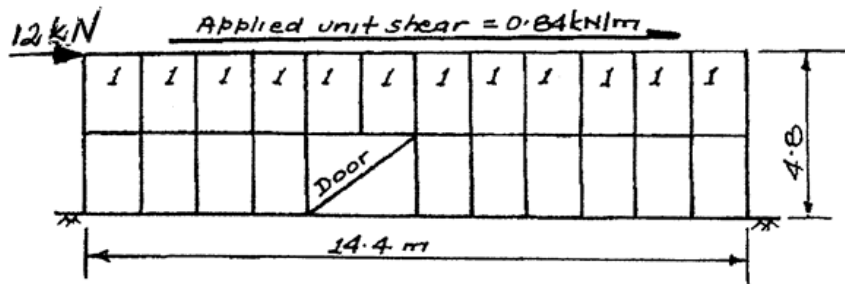
ELEVATION

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.
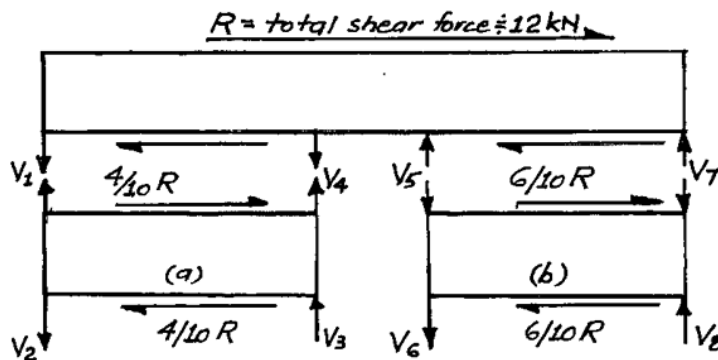
Figure 9-17: Free Body Diagram

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:

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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:

$$v_u \quad = \frac{12}{14.4}$$

$$= 0.84 \; \frac{kN}{m}$$

horizontal panel forces

$$p_h \quad = (0.84 \times 1.2) \text{ for 1200 wide panels}$$

$$= 1 \; kN$$

distributing shears according to (i)

$$S_L \quad = \frac{4}{10} \times 12 = 4.8 \; kN$$

$$S_R \quad = \frac{6}{10} \times 12 = 7.2 \; kN$$

allocation of panel shears, adjacent to opening such that:

$$\sum p_h \text{ to left} \quad \approx 4.8 \; kN \text{ (actually } (2 \; x \; 1.5) + (2 \; x \; 1) = 5 \; kN)$$

$$\sum p_h \text{ to right} \quad \approx 7.2 \; kN \text{ (actually } 8 \; kN)$$

In this instance let the two panels either side of the opening have:

$$p_h \quad = 1.5 \; kN$$

NOTE: The values of p_h 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.
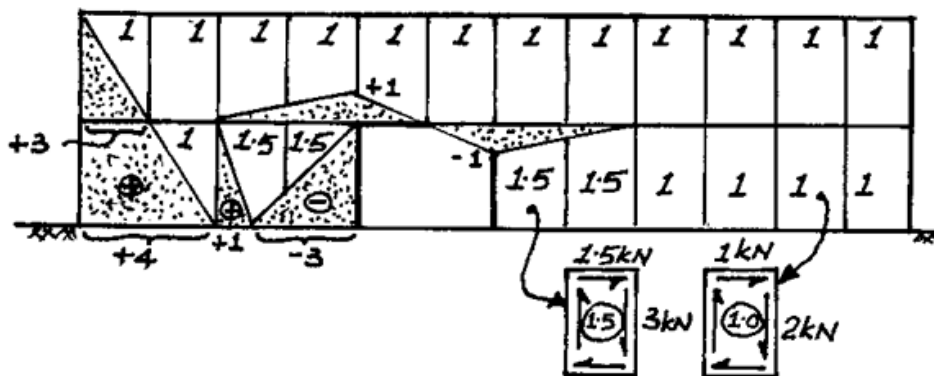
Figure 9-18: Accumulation of Nail Forces

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.
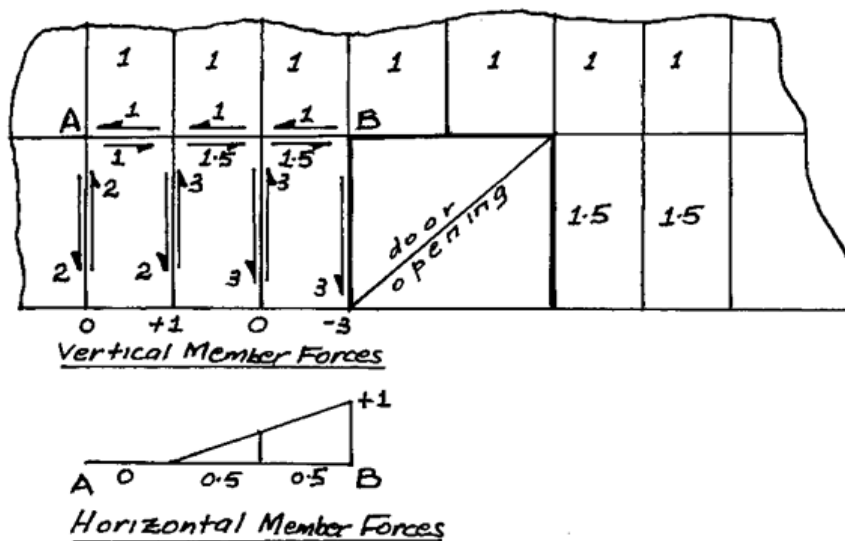
Figure 9-19: Panel Shear Flows and Resulting Nail Force Accumulation

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

The following photographs illustrate some practical examples of diaphragms and shear walls.

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) joint. 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.
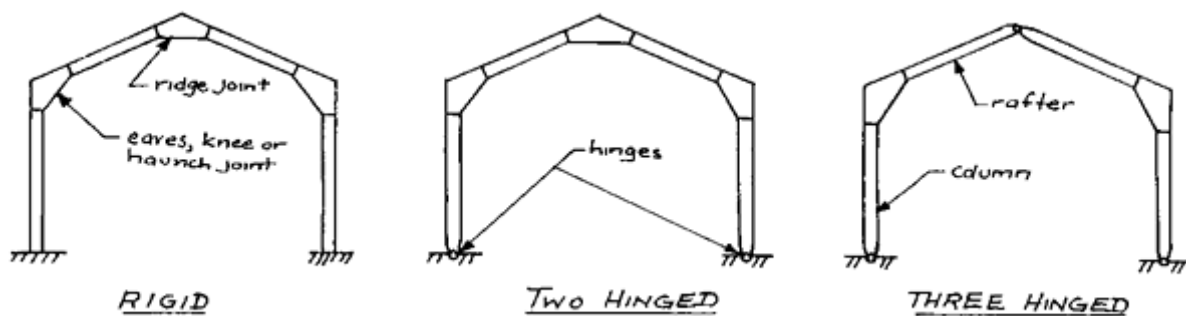
Figure 10-1: Common Single Storey, Single Spanning Portal Frames

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. TABLE 5-3 in Chapter 5 of this Design Guide details standard structural plywood thicknesses and constructions. The construction or lay-up of the plywood must be specified in the gusset design, as AS1720 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.

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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, $f_b = \frac{M}{Z}$, thus:

$$t_\parallel \geq \frac{6\,M_p^*}{\phi\,n\ k_1\,k_{19}\ g_{19}\,f_b^{'}\ D^2}$$

(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;

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D = depth of column / rafter member.

n = number of gussets.

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.

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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:

$$f_1 = \frac{24\,M\,k\,(1-k)}{t_{\parallel}D^2\,(4k-1)}$$

$$f_1 = f_1\frac{(1-k)}{k} \tag{10.2}$$

where:

f_1 = fibre stress in MPa;

M = total applied moment on the joint N-mm;

D = depth of column/rafter member in mm;

$$K = \frac{\left(y+\frac{D}{2}\right)}{(y+d)}$$

$t_{\parallel}$ = effective thickness of plywood

For an internal gusset:

$$y = \frac{L-D}{1+\left(1-\frac{D}{2L}\right)\tan\alpha} \tag{10.3}$$

where:

α = roof slope

For an external gusset:

$$y = L-D \tag{10.4}$$

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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.
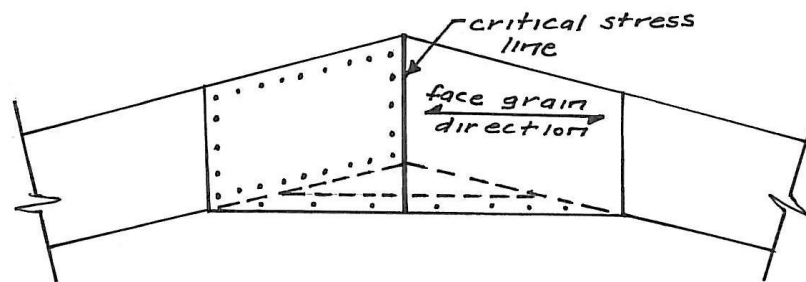
Figure 10-4: Ridge Joint

Nail Joint Action

Nail forces are evaluated through application of the classic torsion relationship, i.e. $\tau = \frac{T\rho}{I_\rho}$. 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.
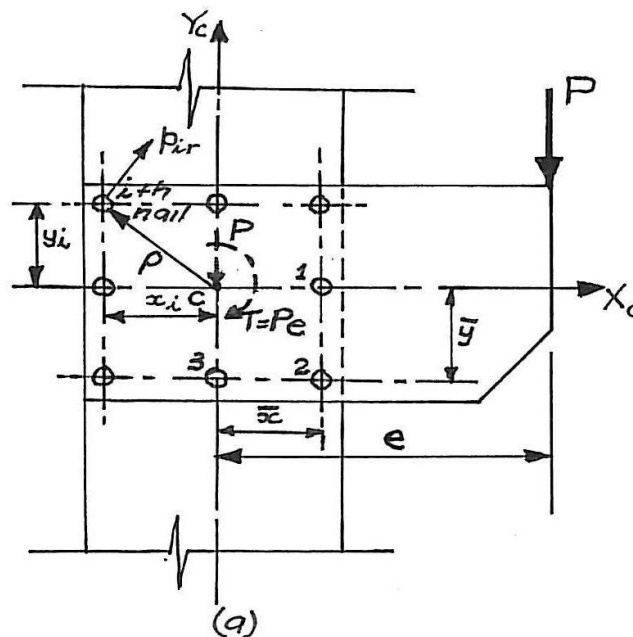
Figure 10-5: Nail Group Subjected to Torsional Moment

The centroid of the nail group can be found from the relationships:

$$\bar{x} = \sum_{i=1}^{n} \frac{x_i}{n}$$

$$\bar{y} = \sum_{i=1}^{n} \frac{y_i}{n}$$

(10.5)

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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:

$$I_p = I_x + I_y \tag{10.6}$$

where:

$$I_x = A \sum_{i=1}^{n} y_i^2 \text{ and } I_y = A \sum_{i=1}^{n} x_i^2 \tag{10.7}$$

$$A = \text{nail cross-sectional area in } mm^2$$

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 $\frac{P}{n}$

Hence, nail 2 in Figure 10-5 will be the worst loaded nail. The components of nail load will be:

where: (10.9)
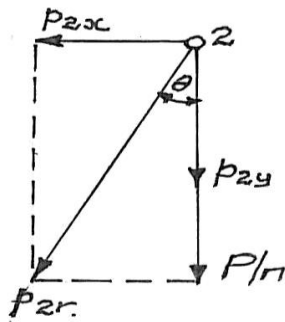
$$p_{2x} = \frac{T\bar{y}}{I_p}$$

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

$$p_{2r} = \sqrt{(p_{2x})^2 + \left(p_{2y} + \frac{P_y}{n}\right)^2}$$

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.
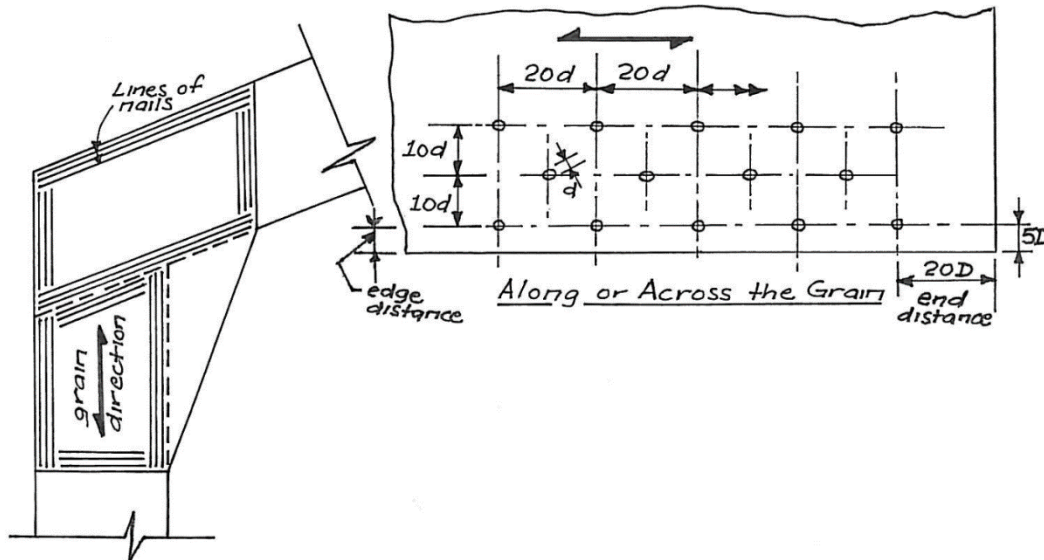
Figure 10-6: Typical Nailing Pattern and an Idealised Line Representation

AS1720.1 (Cl. 4.2.3.3) suggests the application of the relationship:

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

(10.10)

An alternative, simpler but more conservative method of determining the moment capacity of a nail group is that recommended by Hutchings, as described below. This procedure assumes the nails to be smeared as lines whose width (w) is proportional to the nailing density.
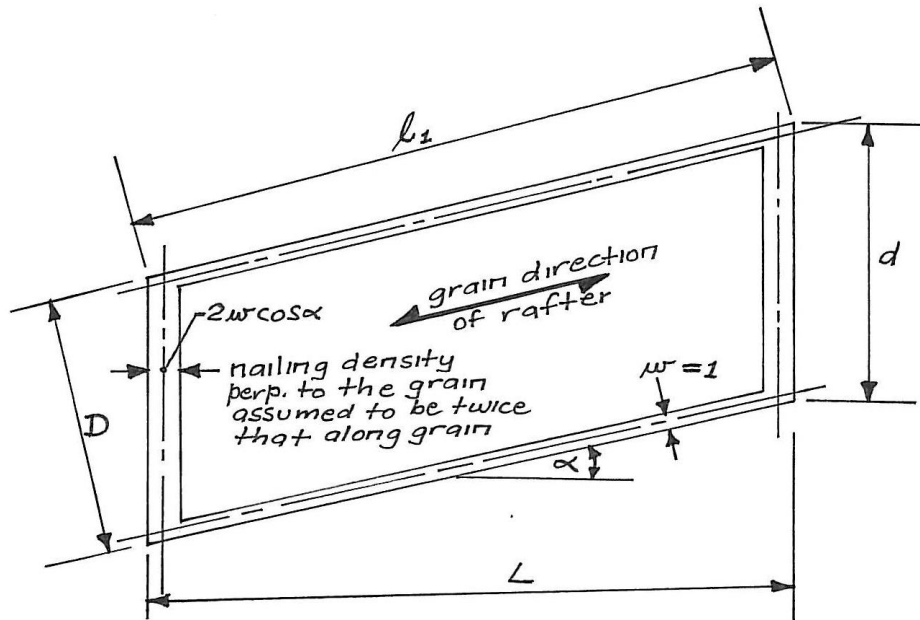
Figure 10-7: Nails Smeared as a Continuous Line

To determine the polar moment of area of a line of width w and length l about a point O as shown in FIGURE 10-8 can be shown by application of the parallel axes' theorem, to be given by:

$$I_{po} = \frac{w\,l^3}{12} + w\,l\,r^2$$
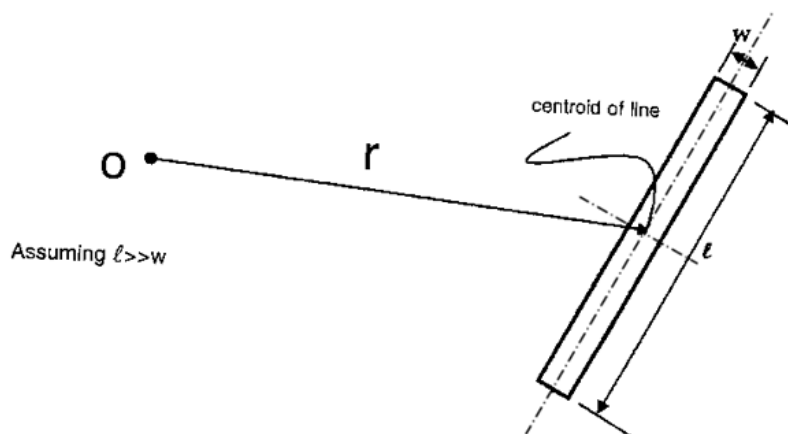
Figure 10-8: Polar Moment of a Line Width w

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

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

where: α = the angle of the roof pitch.

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

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

where:

s = nail spacing along the grain.

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

10.4. Plywood / LVL Gusseted Joints – Methodology

The steps involved in the design of a plywood / LVL gusseted joint, (for the nomenclature refer Figures 10-2 and 10-3) are as follows:

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

$$Z = \frac{M}{f_b}$$

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

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.

$$h = 2D \qquad\qquad\qquad or$$
$$d_e = \frac{L-D}{1+\left(1-\frac{D}{2L}\right)\tan\alpha} + D \qquad or$$
$$d_e = L - y$$

where:

α = 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:

$$t \geq \frac{6\,M_p^*}{2\,(\phi\,k_1\,k_{19}\,g_{19})f_b^{'}\,d_e^2}$$

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:
$$M_d \geq M^*$$

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

The Hutchings 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:

$$M_d = \phi\,k_1\,k_{13}\,k_{14}\,k_{16}\,k_{17}\,Q_k\left(\frac{I_p}{r_m}\right) \tag{10.12}$$

where:

M^* = design action effect on joint (in-plane moment)

ϕ = capacity factor for a nailed joint — AS1720.1 Cl. 2.3

k_1 = the factor for duration of load for joints — AS1720.1, Cl. 2.4.1.1

k_{13} = 1.0 for nails in side grain
= 0.6 for nails in end grain — AS1720.1, Cl. 4.2.3

k_{14} = 1.0 for nails in single shear
= 2.0 for nails in double shear — AS1720.1, Cl. 4.2.3

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 — AS1720.1, Cl. 4.2.3

k_{17} = factor for multiple nailed joints given in AS1720.1 Table 4.3(B) for Type 1 joints to resist in-plane moments — AS1720.1, Cl. 4.2.3

r_m = the maximum value of 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 Table 4.1(A) and 4.1(B)

I_p = polar moment of inertia

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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
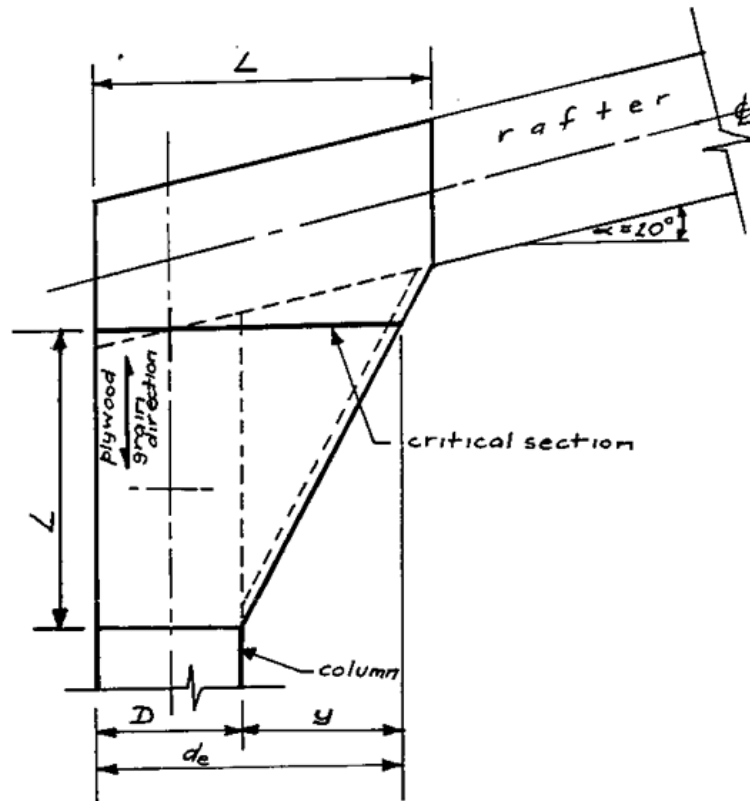
Figure 10-9: Major Dimensions of the Knee Joint

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:

   $$L = 1200mm$$

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

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

$$= \frac{1200 - 600}{1 + \left(1 - \frac{600}{2 \times 1200}\right)\tan 10°} + 600 = 1130 \ mm$$

4. Determine required gusset thickness:

$$t \geq \frac{6 \ M_p^*}{2\left(\phi k_1 k_{19} g_{19}\right)f_b' \ d_e^2}$$

For F11 structural plywood:

f_b'	= 31 MPa (F11 structural plywood)		Table 5.1
M^*	= design action effect		
	= 144 kNm		
ϕ	= capacity factor for plywood = 0.85		Table 2.1
k_1	= 1.14 (wind gust)		Table 2.3 App G, Table G1
k_{19}	= 1.0 (moisture content <15%)		Table 5.2(A)
g_{19}	= 1.0		Table 5.3

Required thickness of parallel plies per side:

$$t = \frac{1}{2}\left(\frac{6 \times 144 \times 10^6}{(0.85 \times 1.14 \times 1.0 \times 1.0)31 \times 1130^2}\right) = 11.3 \ mm$$

Choose 19-30-7 ($t_{II} = 12 \ mm$) F11 DD 2400 x 1200 structural plywood.

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

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

    For convenience of reference, restating Equation 10.11 allows the determination of (I_P) for a smeared single ring (rectangle) of nails as shown in Figure 10-7 and results in:

$$I_p = \frac{2}{s_{\|}}\left[\frac{l_1^3}{12} + \frac{wd^3}{12} + l_1\left(\frac{d}{2}\right)^2 + wd\left(\frac{l_1}{2}\right)^2\right]$$
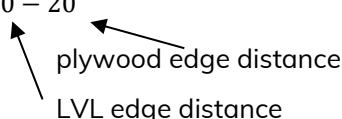
In this example the nail centres for the LVL will be:

- edge distance – 5D
  $= 5 \times 2.9 \cong 15$ say 20 mm.

- parallel to grain – 20D
  $= 20 \times 2.9 \cong 60 \ mm.$

- perpendicular to grain – 10D
  $= 10 \times 2.9 \cong 30 \ mm.$

- assuming width of lines parallel to grain is unity, then width of lines perpendicular to grain will be:
$$w_v = \frac{60}{30}\cos 10° = 1.97$$

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:

$$l_1 = \frac{1200}{\cos 10°} - 60 - 20$$

plywood edge distance

LVL edge distance

$$l_1 = 1140$$

$$d_1 = \frac{D}{\cos 10°} - 2 \times 20$$

$$d_1 = 570$$

Therefore, for the first ring of nails:

$$I_{p1} = \frac{2}{60}\left[\frac{1140^3}{12} + \frac{1.97 \times 570^3}{12} + 1140\left(\frac{570}{2}\right)^2 + 1.97 \times 570\left(\frac{1140}{2}\right)^2\right] = 20.4 \times 10^6 \ mm^2$$

For the second, third and fourth rings of nails:

$$l_2 = 1140 - 60$$
$$= 1080$$

$$d_2 = 570 - 60$$
$$= 510$$

$$l_3 = 1080 - 60$$
$$= 1020$$

$$d_3 = 510 - 60$$
$$= 450$$

$$l_4 = 1020 - 60$$
$$= 960$$

$$d_4 = 450 - 60$$
$$= 390$$

$$I_{p2} = \frac{2}{60}\left[\frac{1140^3}{12} + \frac{1.97 \times 570^3}{12} + 1140\left(\frac{570}{2}\right)^2 + 1.97 \times 570\left(\frac{1140}{2}\right)^2\right] = 16.3 \times 10^6 \ mm^2$$

$$I_{p3} = \frac{2}{60}\left[\frac{1080^3}{12} + \frac{1.97 \times 510^3}{12} + 1080\left(\frac{510}{2}\right)^2 + 1.97 \times 510\left(\frac{1080}{2}\right)^2\right] = 12.9 \times 10^6 \ mm^2$$

$$I_{p4} = \frac{2}{60}\left[\frac{1020^3}{12} + \frac{1.97 \times 450^3}{12} + 1020\left(\frac{450}{2}\right)^2 + 1.97 \times 450\left(\frac{1020}{2}\right)^2\right] = 9.9 \times 10^6 \ mm^2$$

To determine the number of nails per ring:

$$n = \frac{2}{s_\parallel}\left(l_n + \frac{s_\parallel}{s_\perp} d_n\right)$$

where:

$s_\parallel$ = Nail spacing parallel to the grain;

$s_\perp$ = Nail spacing perpendicular to grain;

l_n = Length of the nth nail ring;

d_n = Height of the nth nail ring.

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Nail Ring Number	$I_p(mm^4)$	Nails/ring	Total nails/gusset
1	20.37 x 10⁶	76	76
2	16.33 x 10⁶	70	146
3	12.85 x 10⁶	64	210
4	9.9 x 10⁶	58	268

Co-ordinates of extreme nail from the centroid as defined by x_m and y_m in Figure 10-10.

$$x_m = \frac{L}{2} - 60 = 600 - 60 = 540 \; mm$$

$$y_m = \frac{d_1}{2} + \frac{l_1}{2} \sin 10° = 285 + 99 = 384 \; mm$$

$$\rho = \sqrt{(540)^2 + (384)^2} = 663 \; mm$$

6. Joint Capacity – Moment Joint Design

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

$$M_{d,j} = \phi k_1 \, k_{13} k_{14} \, k_{16} \, k_{17} \, Q_k \left(\frac{I_p}{r_m}\right) \geq M^*$$

where:

ϕ	= capacity factor = 0.8	
$M_{d,j}$	= 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 = 1.14 in this case	App G, Table G1
k_{13}	= 1.0 for nails in side grain = 0.6 for nails in end grain = 1.0 in this case	AS1720.1, Cl. 4.2.3
k_{14}	= 1.0 for nails in single shear = 2.0 for nails in double shear = 1.0 in this case	AS1720.1, Cl. 4.2.3
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	AS1720.1, Cl. 4.2.3
k_{17}	= factor for multiple nailed joints for Type 1 joints resisting in-plane moments = 1.2 in this case	AS1720.1, Cl. 4.2.3

hence:

$$M_{d,j} = 0.8 \times 1.14 \times 1.0 \times 1.0 \times 1.1 \times 1.2 \times 989 \times \left(\frac{I_p}{r_m}\right)$$

$$= 1191 \left[\frac{I_p}{r_m}\right] 2 \; (gusset \; each \; side)$$

where:

$$I_p = 59.45 \times 10^6 \; mm^2$$

$$r_m = 663 \; mm$$

$$M_{d,j} = \left(\frac{1191 \times 59.45}{663}\right) \times 2 \; kN \; m = 214 \; kNm$$

Design Action Effect

From the relationship:

$$M_{d,j} \geq M^*$$

$$214 \, kNm \;>\; 144 \, kNm, \text{ hence the joint is OK}$$

Figure 10-10: 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 x_m and y_m.

   To evaluate the force on the worst loaded nail:

   $$\rho_{ix} = \frac{M \times y_m}{2 \times I_p} + \frac{S}{n}$$

   $$= \frac{144 \times 10^6 \times 384}{2 \times 59.45 \times 10^6} + \frac{20 \times 10^3}{2 \times 268} = 465 + 37 = 502 \, N$$

   $$\rho_{iy} = \frac{M \times x_m}{2 \times I_p} + \frac{S}{n}$$

   $$= \frac{144 \times 10^6 \times 540}{2 \times 59.45 \times 10^6} + \frac{55 \times 10^3}{2 \times 268} = 654 + 103 = 757 \, N$$

   $$\rho_{ir} = \sqrt{(502)^2 + (757)^2} = 908 \, N$$

8. Nail Capacity

The design capacity $(N_{d,j})$ of a 2.9 Ø nail driven into seasoned timber of strength group JD3 is:

$$N_{d,j} = \phi\, k_1\, k_{13} k_{14}\, k_{16}\, k_{17}\, Q_k$$

where:

Q_k from interpolation $= 989\ N$

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:

$$N_{d,j} = (0.8 \times 1.14 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \times 989)N$$

$$N_{d,j} = 992\ N > 908\ N$$

hence the nail capacity is OK.

Capacities: The nail and joint capacity are higher than required. However, for other cases it might be required to repeat the design process until a satisfactory result is achieved. In this instance the choices would have been

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

10.5. Photographs

This section provides some examples of portal frames, moment joints and pin joints.

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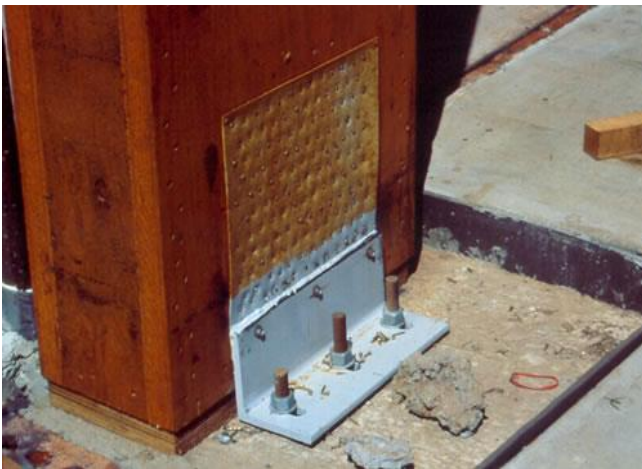
11. Plywood Stressed Skin Panels

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.
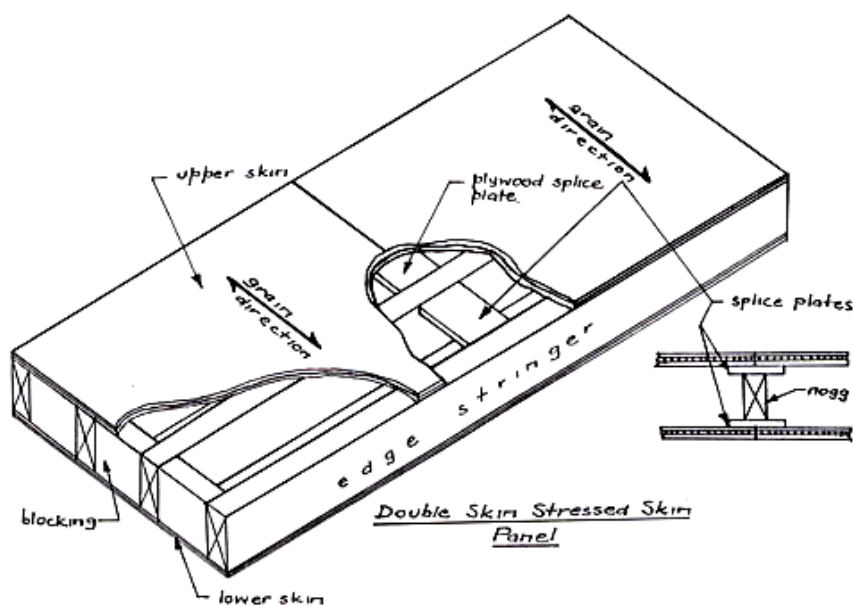
Figure 11-1: Component Parts of a Stressed Skin Panel

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 are generally constrained to the 7 to 9 m range, however with the availability of LVL this range can now be extended.

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

- not greater than 100 mm along the framing members,
- a single row on stringers up to 50mm thick and two rows on stringers greater than 50 mm wide up to 100 mm 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 designer's 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 phenomenon 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.
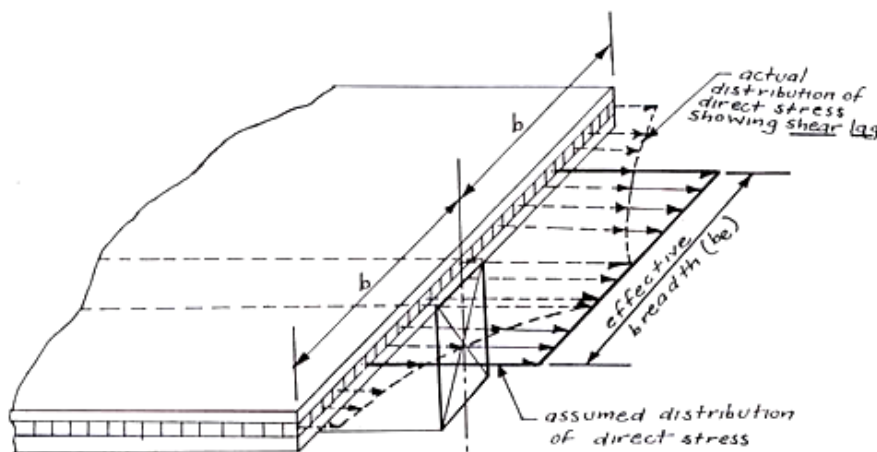
Figure 11-2: Distribution of Flange Stresses

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.

The North American approach is to use the basic spacing (b) shown in FIGURE 11-3 as the design parameter.
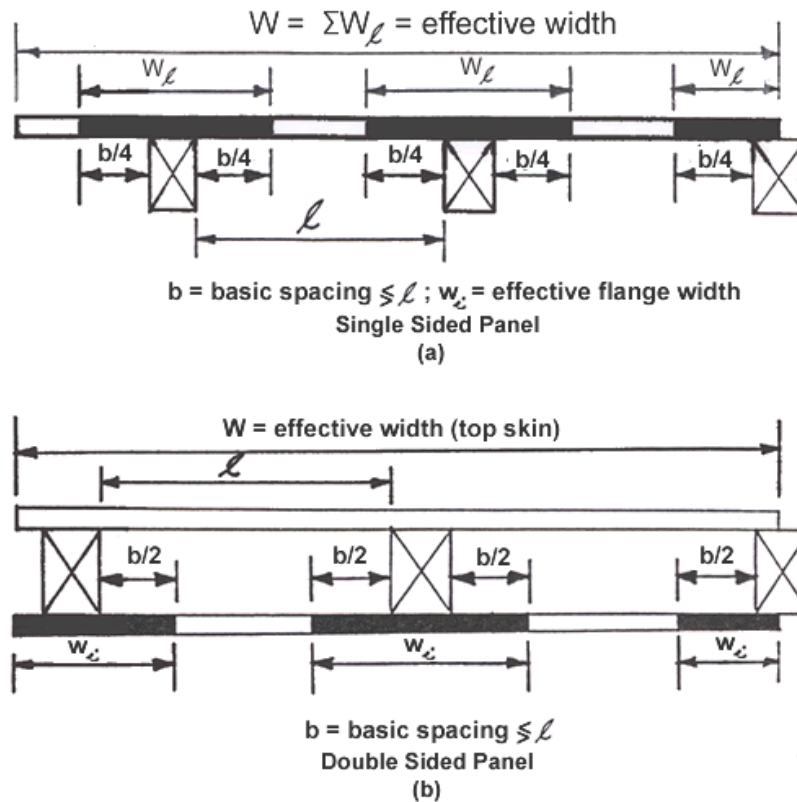
Figure 11-3: Effective Widths of Plywood

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

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

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

If (l) is less than (b) in either skin, then a correspondingly reduced length of skin as shown for the bottom skin in FIGURE 11-3, is effective in resisting the applied bending moment.

NOTE: The full length of both skins is 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 affected by rigid gluing of the interfaces.

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FIGURE 11-4 shows 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.
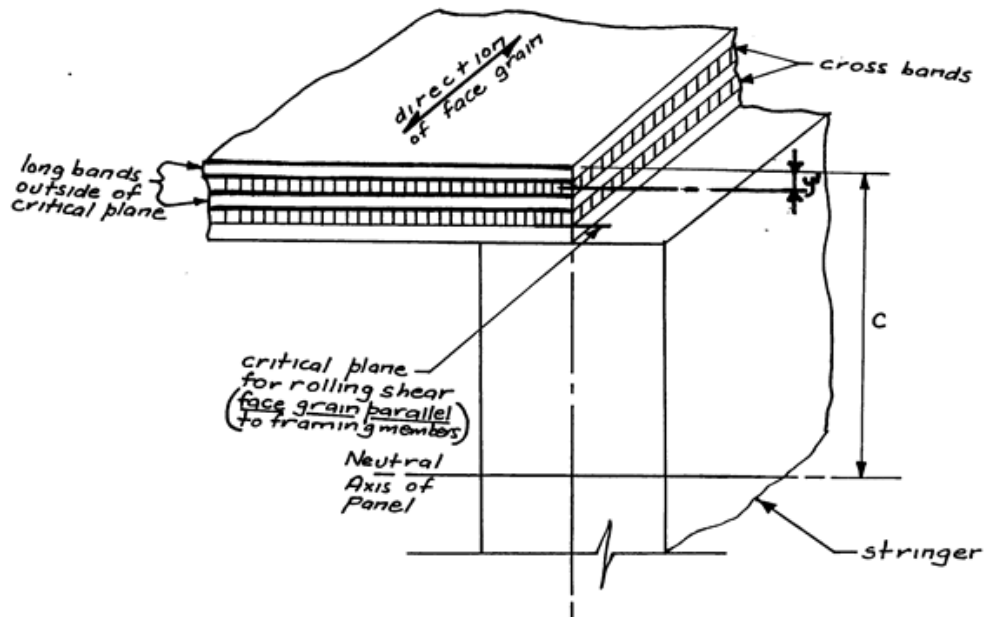
Figure 11-4: Position of Critical Plane for Rolling Shear

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

$$\tau = \frac{S\,Q_s}{I\,b_s} \qquad\qquad (11.1)$$

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 Association of USA.

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 600 mm;
- minimum thickness of tension skin 7 mm;

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- 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) $\geq$ (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:
$\frac{stringer\ MOE}{skin\ MOE} \times stringer\ width$

- 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 $\left(\frac{N}{mm}\right)$	$A_{\parallel}$ (mm^2)	$A_{\parallel}E$ $(N \times 10^6)$	y (mm)	$A_{\parallel}E_y$ $(Nmm \times 10^6)$
Top Skin Stringers Bottom Skin					
			$\sum A_{\parallel}E =$		$\sum A_{\parallel}E_y =$

Table 11-1: Layout to Determine Neutral Axis

$$\bar{y} = \frac{\sum A_{\parallel}E_y}{\sum A_{\parallel}E}$$

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	$A_{\parallel}E$ $(N \times 10^6)$	I_o	EI_o $(Nmm^2 \times 10^6)$	d (mm)	$A_{\parallel}Ed^2$ $(Nmm^2 \times 10^6)$	$EI_o + A_{\parallel}Ed^2$ $(Nmm^2 \times 10^6)$
Top Skin Stringers Bottom Skin						
						$\sum EI_g =$

Table 11-2: Layout to Determine Flexural Rigidity

Flexural deflection can be determined from the familiar relationship:

$$\Delta_b = \frac{5wL^4}{384EI_g} \tag{11.3}$$

where:

w = panel load in kPa

L = panel span in mm

EI_g = flexural rigidity of the panel in Nmm^2

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

$$\Delta_s = \frac{1.8PL}{AG} \tag{11.4}$$

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 $\left(\frac{N}{mm^2}\right)$

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:

$$\Delta = \frac{4wl^4}{384E_t I} \tag{11.5}$$

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.

$$F_{b,a} = \frac{MyE}{EI_g} \tag{11.6}$$

where

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

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

E = MoE of the element being considered;

EI_g = the flexural rigidity of the panel.

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

FIGURE 11-5 shows a typical stress distribution for the plywood skins and stringers of a stressed skin panel.
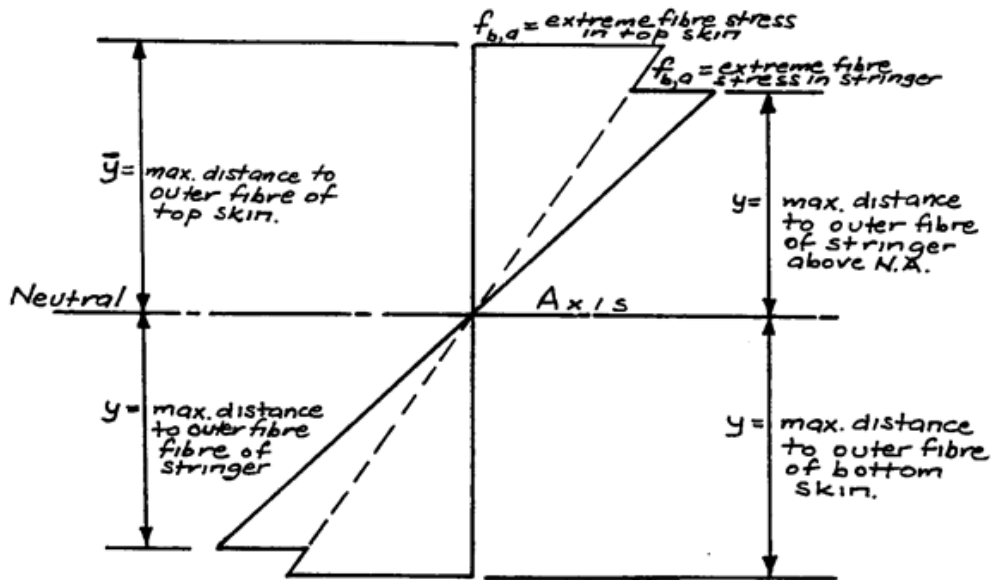
Figure 11-5: Bending Stresses in Stressed Skin Panel

The values of $\bar{y}$ and the y's shown in FIGURE 11-5, when substituted in Equation 11.6 for y, will on solution result in the evaluation of extreme fibre stresses $f_{b,a}$.

Splice Plate Check

From Equation 11.6:

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

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

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

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

where:

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

$$\frac{F_{sp}}{A_p} = f_{sp} = \left[\frac{wl^2}{8} \frac{E_{sp}}{EI_g} y \frac{W_{sp}}{W_p} 10^3\right] MPa \tag{11.7}$$

where

f_{sp} = splice stress

w = uniformly distributed load $\left(\frac{kN}{m}\right)$

E_{sp} = modulus of elasticity of splice material

EI_g = stiffness factor (Nmm^2) from TABLE 11-2

y = distance from neutral axis to the extreme tension or compression fibre (mm)

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

Rolling Shear Stress

From FIGURE 11-4:

$$d_s = c - y'$$
$$Q_s = A d_s$$

where:

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

The rolling shear stress will be, from Equation 11.1:

$$\tau_r = \frac{S Q_s}{I b}$$

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:

$$Q_H = Q_{stringer} + \frac{E_{skin}}{E_{stringers}} Q_{skin} \qquad (11.8)$$

Hence

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

where

$$\tau_r = \frac{S Q_s}{I b}$$

S = total shear force

Q_H = as defined

E_{ST} = modulus of elasticity of stringers $\left(\frac{N}{mm^2}\right)$

EI_g = stiffness factor (Nmm^2)

b = total width of stringers intersected by the neutral axis (mm).

11.6. Design Example – Stressed Skin Panels

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

Uniformly distributed live load	$= 2 \, kPa$
Uniformly distributed dead load	$= 0.5 \, kPa$
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.

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Trial Section

Assume as a trial section the panel having the material specifications and dimensions shown in FIGURE 11-6.
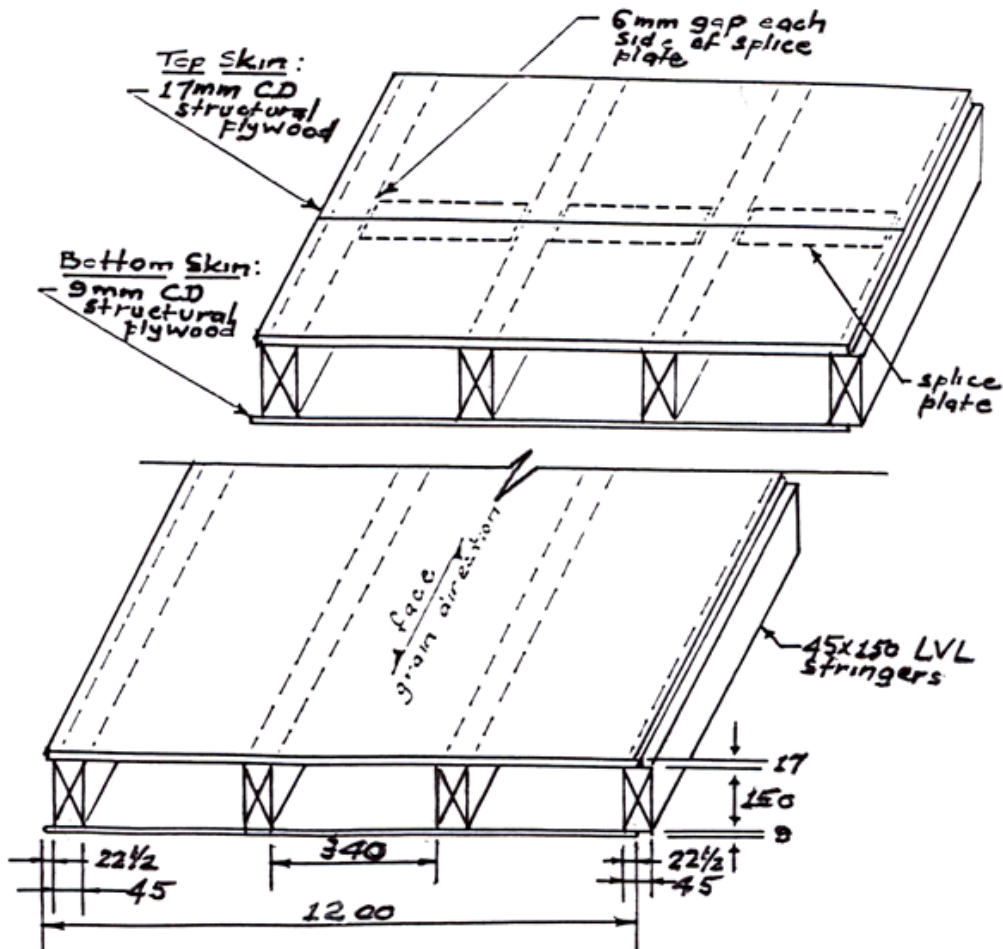
Figure 11-6: Stressed Skin Panel Trial Section

Design data for the structural components is:

F11 x 17 – 24 – 7 plywood $\quad I_{\parallel} \quad = 285 \ \frac{mm^4}{mm}$;

$$I_{\perp} \quad = 120 \ \frac{mm^4}{mm};$$

$$A_{\parallel} \quad = (4 \times 2.4 \times 10^{-3} \times 1)$$

$$= 0.0096 \ \frac{m^2}{m} \ or \ 9600 \frac{mm^2}{m}$$

$$E \quad = 10{,}500 \ MPa$$

F11 x 9 – 30 – 3 plywood $\quad I_{\parallel} \quad = 60 \ \frac{mm^4}{mm}$;

$$I_{\perp} \quad = 4 \ \frac{mm^4}{mm};$$

$$A_{\parallel} \quad = (2 \times 3 \times 10^{-3} \times 1)$$

$$= 0.006 \ \frac{m^2}{m} \ or \ 6000 \frac{mm^2}{m}$$

$$E \quad = 10{,}500 \ MPa$$

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Laminated Veneer Lumber -   45 x 150mm;

$$I \quad = \frac{bd^3}{12} = 12.66 \times 10^6 mm^4;$$

$$E \quad = 13{,}200 \; MPa$$

$$A \quad = 6750 \; mm^2$$

$$G \quad = 660 \; MPa$$

Basic Spacing

Clear distance between stringers (l) $= \frac{1200 - (2 \times 22.5) - (3 \times 45)}{3}$

$$l \quad = 340mm$$

Total splice plate width (S_w) $= 3(340 - 12)$

$$S_w \quad = 984 \; mm$$

For 17mm thick plywood (b) $= (17 \times 45) \; mm$

$$b = 765mm > l = 340mm$$

For 9mm thick plywood (b) $= (9 \times 45) \; mm$

$$b = 405mm \; > l = 340mm$$

Deflection - Section Properties

Element	MoE	$A_\parallel$	$A_\parallel E$ $(\times \; 10^6)$	y	$A_\parallel E_y$ $(\times \; 10^9)$
Top Skin	10500	$1.2 \times 9600 = 11520$	121	167.5	20.3
Stringer	13200	$4 \times 6750 \times 1.26 = 34020$	449	84	37.7
Bottom Skin	10500	$1.2 \times 6000 = 7200$	75.6	4.5	0.34
		Σ	645.6	Σ	58.34

Table 11-3: Procedure for Determining Section Centroid

$$\bar{y} = \frac{\Sigma A_D \, E_{xy}}{\Sigma A_D \; E} = \frac{58.34 \times 10^9}{645.6 \times 10^6} = 90.4 \; mm$$

Figure 11-7: Neutral Axis and "b" relative to "l"

Element	$A_\parallel E$ ($\times\ 10^6$)	I_o ($\times\ 10^6$)	EI_o ($\times\ 10^9$)	d	$A_\parallel Ed^2$ ($\times\ 10^{10}$)	$EI_o + A_\parallel Ed^2$ ($\times\ 10^{10}$)
Top Skin	121	$285 \times 1200 = 0.34$	3.57	77.2	72.0	72.4
Stringer	449	$12.66 \times 10^6 \times 4 = 50.6$	667.9	6.3	1.84	68.7
Bottom Skin	75.6	$60 \times 1200 = 0.07$	0.74	85.8	55.8	55.9
					ΣEI_g	196.9

Table 11-4: Procedure for Determining EI_g

Hence EI_g is:

$$EI_g = \frac{196.9 \times 10^{10} Nmm^2}{1200\ width}$$
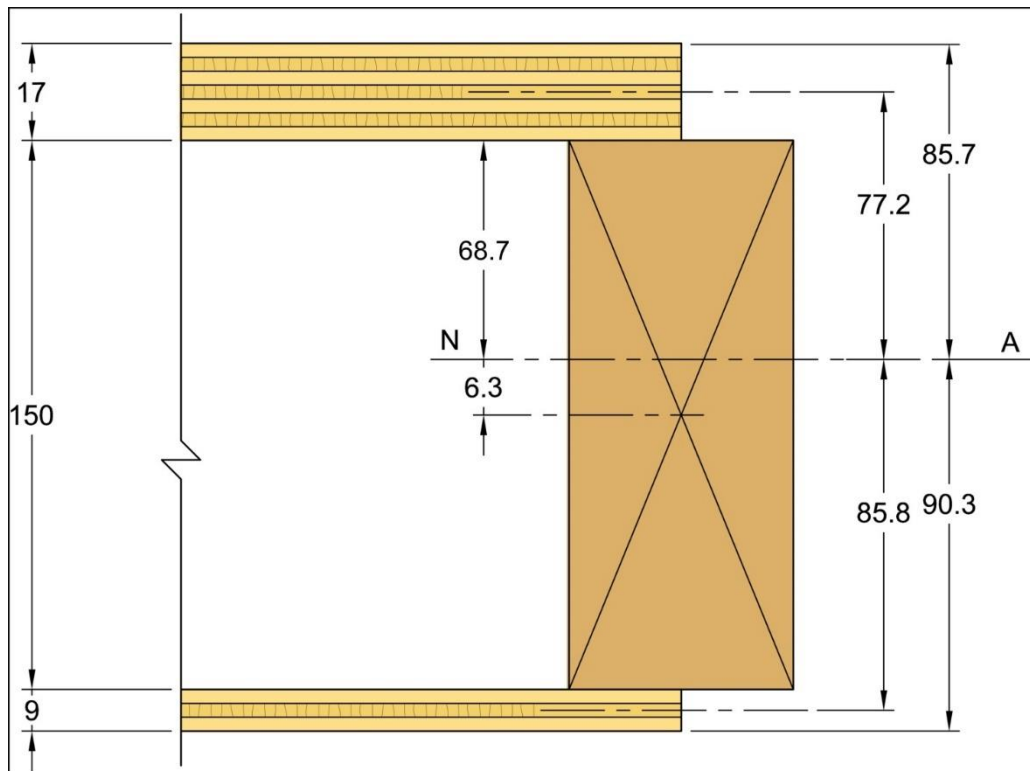
Figure 11-8: Relevant Panel Cross-Section Dimensions

Flexural Deflection Long Term Serviceability Requirements

Flexural Deflection:

$$G + \Psi_l Q \quad G + 0.4Q$$
$$G + 0.4Q \quad = (0.5 + 0.4 \times 2)\ kPa$$
$$G + 0.4Q \quad = 1.3\ kPa$$

$$\Delta_b \quad = \frac{j_2\ 5wL^4}{384\ EI_g}\ where\ j_2 = 2$$

$$\Delta_b \quad = \frac{2 \times 5 \times 1.3 \times 1.2 \times 5^4 \times 10^{12}}{384 \times 196.9 \times 10^{10}} = 12.9\ mm$$

Shear Deflection:

$$\Delta_s \quad = \frac{j_2\ 1.8PL}{AG}\ where\ j_2 = 2$$

$$\Delta_s \quad = \frac{2 \times 1.8 \times 1.3 \times 1.2 \times 5 \times 5 \times 10^6}{4 \times 6750 \times 660}\ 7.9\ mm$$

$$\Delta_b + \Delta_s \quad = 12.9 + 7.9 = 20.8 \; mm$$

$$\Delta_b + \Delta_s \quad = 12.9 + 7.9 = 20.8 \; mm \le 20.8 \; mm \text{ ,i.e. span/240 so OK}$$

Top Skin Deflection:

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

$$\Delta_{ts} \quad = \frac{j_2 \, w \, L^4}{384 \, EI}$$

$$\Delta_{ts} \quad = \frac{2 \times 0.13 \times 340^4 \times 1}{384 \times 10500 \times 120 \times 240} = 0.28 \; mm$$

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

where:

L = clear span between stringers (mm);

E = top skin modulus of elasticity (MPa)

I = $I_\perp$ for top skin of width 240mm;

w = load in $\frac{kN}{m}$

Bending Moment

Member Design Capacity: Strength Limit State

$$M_d = \phi \times k_1 \times k_4 \times k_6 \times k_9 \times k_x \times k_{12}\left(f_b^{'} \, Z\right)$$

Note: Because $\ell < b$, 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 ; $f_b^{'}$ = 31 MPa for F11

k_4 = 1.0 ; $k_6 = 1.0$

k_{11} = 1.0 ; $k_{12} = 1.0$

ϕ = 0.95

M_d = $[0.95 \times 0.8 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \, (31 \times Z)]Nm$

Determination of Z is most conveniently done through TABLE 11-5:

Element	I_o	$A_{\parallel}$	d^2	$I_o + A_{\parallel}d^2$
Top skin	0.34×10^6	11520	77.1^2	68.8×10^6
Stringer	50.6×10^6	34020	6.4^2	52.0×10^6
Bottom skin	0.07×10^6	7200	85.9^2	53.2×10^6
			$\Sigma I_g =$	174.0×10^6

Table 11-5: Layout to Determine the Gross Second Moment of Area

Hence I_g is:

$$I_g = 174.0 \times 10^6 mm^4 \ for \ 1200 \ wide \ panel$$

For the top skin:

$$Z_t = \frac{I}{y} = \frac{I_g}{85.6} = \frac{174 \times 10^6}{85.6} = 2.03 \times 10^6 \ mm^3$$

$$M_d = (0.76 \times 31 \times 2.03 \times 10^6)Nmm$$

$$M_d = 47.8 \ kNm$$

For bottom skin:

$$Z_b = \frac{I_g}{90.4} = \frac{174 \times 10^6}{90.4} = 1.92 \times 10^6 \ mm^3$$

$$M_d = (0.76 \times 31 \times 1.92 \times 10^6)Nmm$$

$$M_d = 45.3 \ kNm$$

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 $1.2G + \Psi_c Q$	$(1.2 \times 0.5) + (0.4 \times 2)$ $= 1.4 \ kPa$	$1.4 \times 1.2 = -1.68$		5.25	9.21
Long term $1.25G + 1.5Q$	$(1.2 \times 0.5) + (1.25 \times 2)$ $= 3.6 \ kPa$	$3.6 \times 1.2 = -4.32$	0.8	13.50	16.87

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 16.87 kNm which is much less than 45.3kNm 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:

$$f_{st} = \left[\frac{wL^2}{8} \frac{E_s}{EI_g} \ y \ \frac{W_s}{W_p} \ 10^6 \right] MPa$$

$$f_{st} = \left[\frac{2 \times 1.2 \times 5^2}{8} \frac{10500}{196.9 \times 10^{10}} \ 90.4 \ \frac{984}{1200} \ 10^6 \right] = 2.96 \ MPa$$

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

$f_t^{'}$ = 18 MPa 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
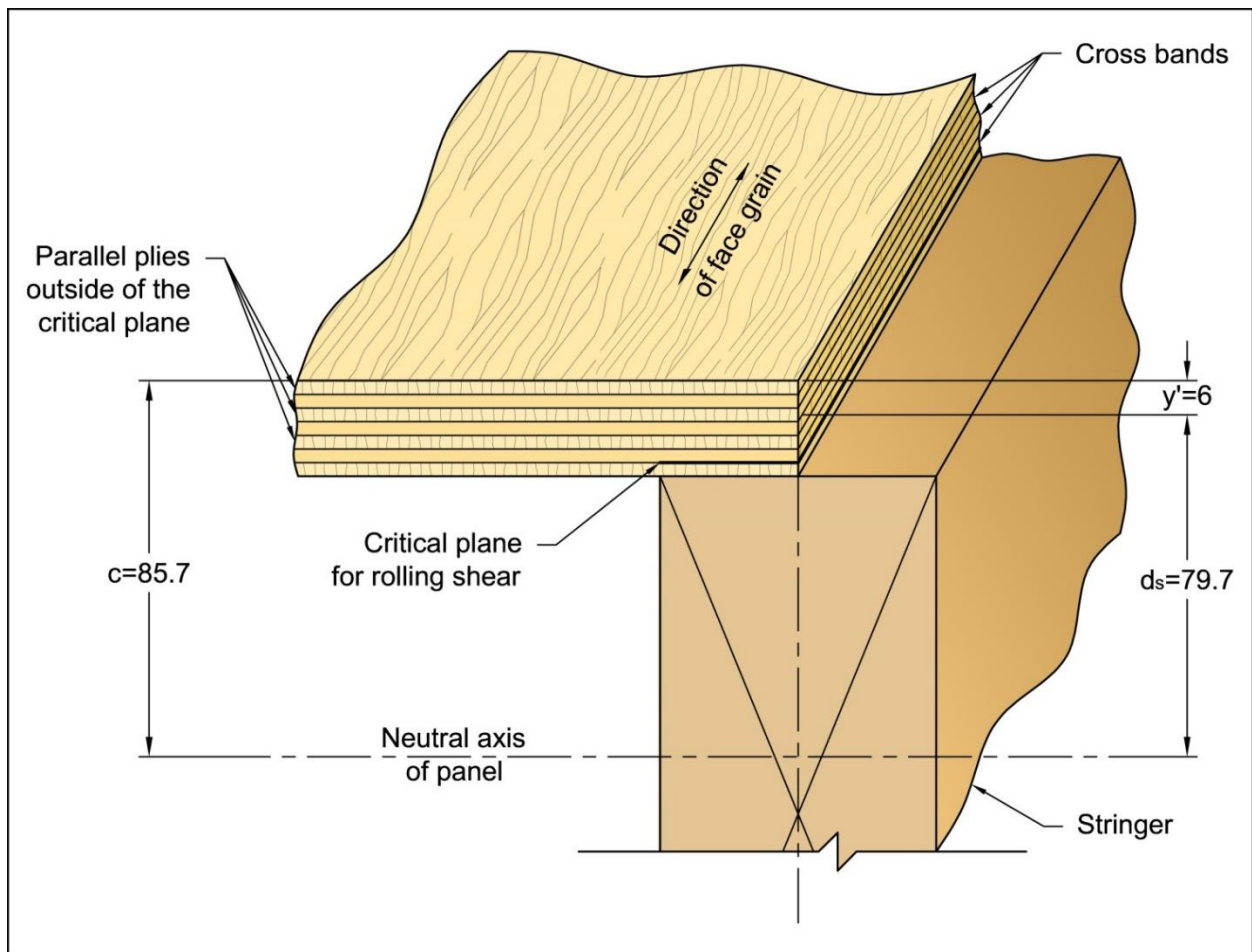
Figure 11-9: Dimensions for Obtaining Q_s

$$\tau_r = \frac{S \times Q_s}{I \times b} = \left[\left(\frac{w \times L \times B}{2} \right) \times Q_s \times \frac{E}{(EI_g)} \times \frac{1}{b} \times 10^3 \right] MPa$$

The static moment Q_s is obtained thus:

$$Q_s \quad = A_{\parallel} \times d_s$$
$$= 3(2.4 \ x \ 1200)79.6$$
$$Q_s \quad = 687744 \ mm^3$$

From FIGURE 11-6:
$$b_s \quad = 3 \times stringer \ width + 0.5 \times stringer \ width$$
$$= (3 \times 45) + (0.5 \times 45)$$
$$b_s \quad = 157.5 \ mm$$

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Writing $(\tau_r \times b_{sf})$ $= q$, a shear flow,

where: b_{sf} $= 45 + (0.5 \times 45)$

$= 67.5 \, mm \ for \ edge \ stringers$

b_{sf} $= (2 \times 45)$

$= 90 \, mm \ for \ internal \ stringers$

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: $\tau_{re} = (0.95 \times 0.8 \times 1.0 \times 0.2 \times 4.5)\frac{N}{mm^2}$

$\tau_{re} = 0.68 \, MPa$

For internal stringers:

$$\tau_{ri} = (0.95 \times 0.8 \times 1.0 \times 0.4 \times 4.5)\frac{N}{mm^2}$$

$\tau_{ri} = 1.37 \, MPa$

Shear Flow: $q = (0.68 \times 67.5) + (1.37 \times 90)$

$q = 169.2 \, \frac{N}{mm}$

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

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

$$1.2 \, w = \left[\frac{169.6 \times 2 \times 196.9 \times 10^{10} \times 1}{1 \times 5 \times 10500 \times 687744} \times \frac{1}{10^3}\right] \frac{kN}{m}$$

$$1.2 \, w = 18.5 \, \frac{kN}{m}$$

or $\quad w = 15.4 \, kPa \ > 2 \, kPa \ \therefore O.K$

Horizontal Shear

From Equations 11.8 and 11.9 and FIGURE 11-8

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

$$Q_H = 4\left(45 \times 68.7 \times \frac{68.7}{2}\right) + \left(\frac{10500}{13200} \times 1200 \times 17 \times 77.2\right) = 1677518 \, mm^3$$

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

$$= \left(\frac{\left(\frac{4.32 \times 5}{2}\right) \times 10^3 \times 1677518 \times 13200}{196.9 \times 10^{10} \times 4 \times 45}\right) MPa$$

$$\tau_H = 0.67 \, MPa < 1.7 \, MPa \ OK$$

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 $\frac{48(EI_g)}{L^3}$ shows $k = 0.7 \, \frac{kN}{mm}$, which in a normal bearer/joist floor system, would be more than adequate to ensure a sufficiently vibration insensitive floor.

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.
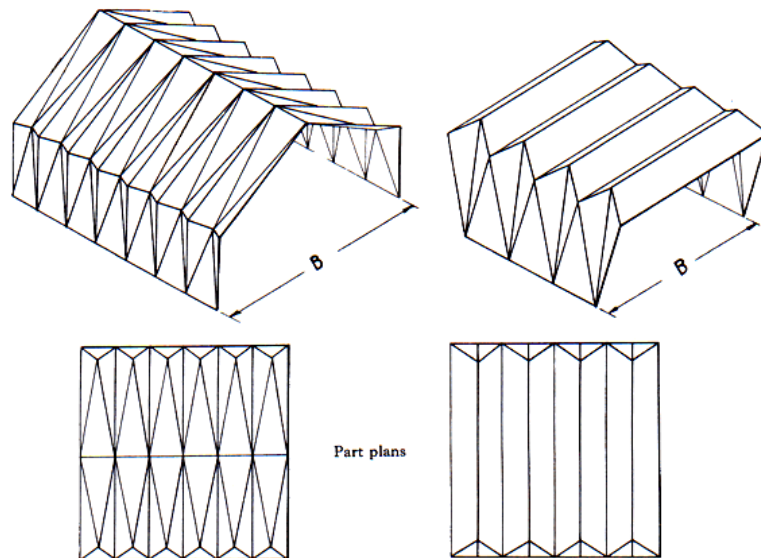
Figure 12-1: Various Types Folded Plate Structures

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.
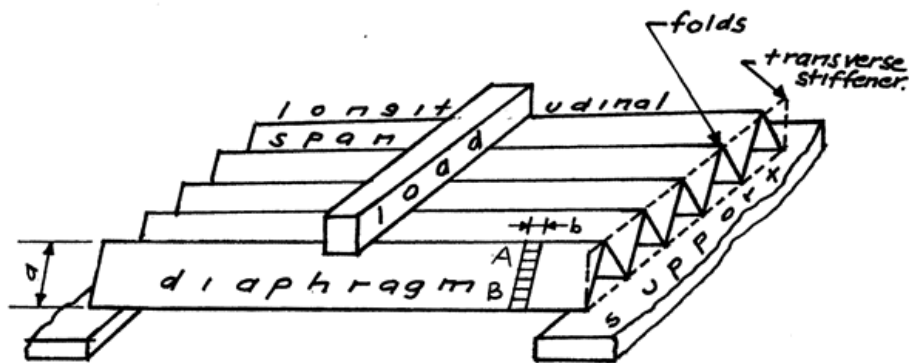
Figure 12-2: Sheet of Paper with Folds Supporting a Load

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.

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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.
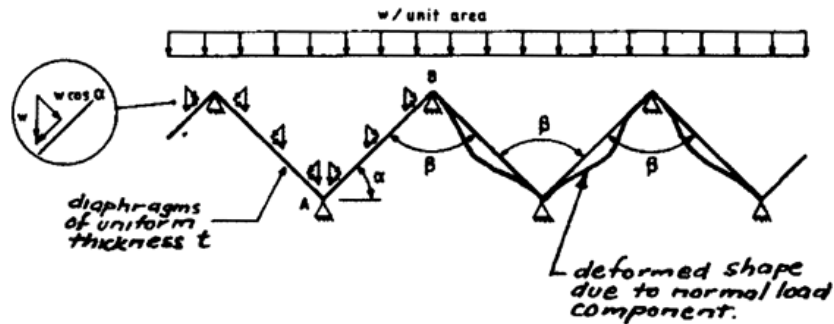
Figure 12-3: Load Components on a Transverse Strip

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:

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

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

The mid-span moment will be:

$$M_{ms} = \frac{(w \cos \alpha)\, a^2}{24}$$
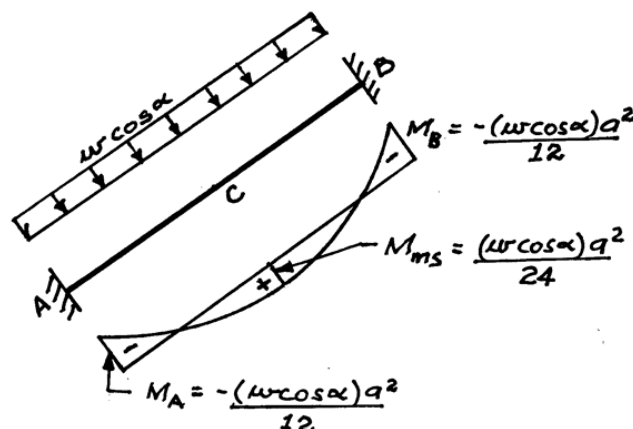
Figure 12-4: Isolated Transverse Element under Load

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

$$Z \;=\; \frac{t^2}{6} \text{ will be:}$$

$$\sigma_{A/B} \;=\; \pm\, \frac{M_{A/B}}{Z} \;=\; \pm\, \frac{(w\cos\alpha)\,a^2}{12\left(\frac{t^2}{6}\right)} \;=\; \pm\, \frac{w\cos\alpha}{2}\left(\frac{a}{t}\right)^2 \tag{12.1}$$

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

Longitudinal action results from the bending action of the diaphragms transferring reactions R due to the normal component $p_n = w \times a \times \cos\alpha$ to the folds. Simultaneously, the tangential component $p_t = w \times a \times \sin\alpha$ is transferred to the folds by direct stress along the diaphragm as shown in FIGURE 12-5 (a).
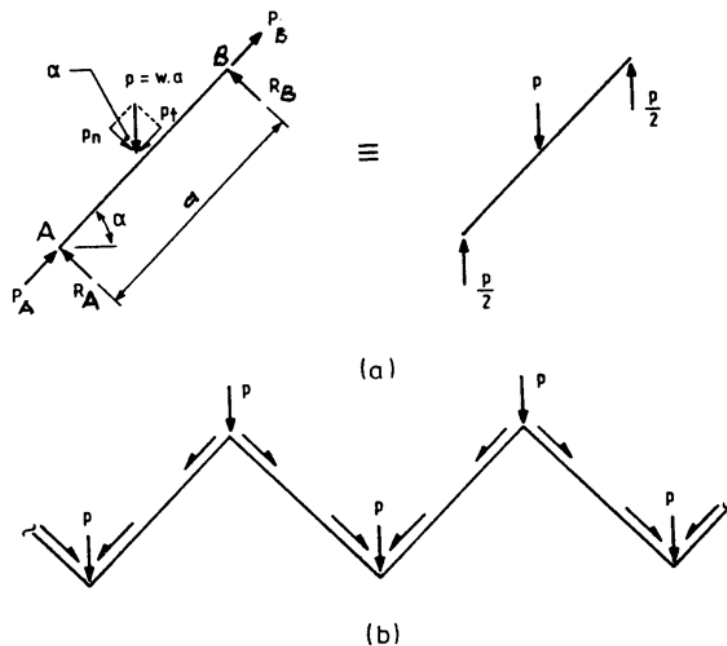
Figure 12-5: Determination of Longitudinal Load on the System

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 $b = \dfrac{t}{\sin\alpha}$

FIGURE 12-6 shows such a sloping section.
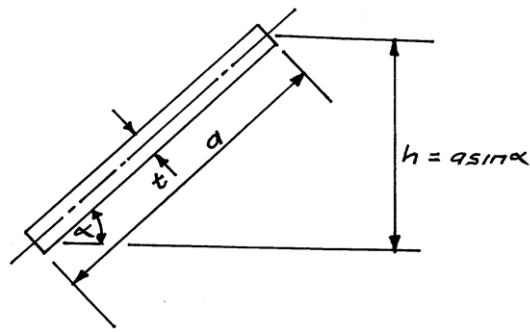
Figure 12-6: Inclined Diaphragm

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

$$I = \frac{bh^3}{12} = \frac{1}{12}\left(\frac{t}{\sin\alpha}\right)(a\sin\alpha)^3 = \frac{1}{12}\,t\,a^3\sin^2\alpha \tag{12.2}$$

The section modulus (Z) is:

$$Z = \frac{I}{y} = \frac{1}{12}\left(\frac{t}{\sin\alpha}\right)(a\,\sin\alpha)^3\,\frac{1}{\frac{h}{2}} = \frac{1}{6}\,t\,a^2\,\sin\alpha \tag{12.3}$$

For a uniform load $p = (wa)\ \frac{kN}{m}$, the maximum bending stress in an isotropic diaphragm is given by:

$$\sigma_{max} = \frac{\frac{pL^2}{8}}{Z} = \frac{\frac{pL^2}{8}}{\frac{ta^2\sin\alpha}{6}} = \frac{6waL^2}{8ta^2\sin\alpha} = \frac{3}{4}\frac{wL^2}{th} \tag{12.4}$$

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.
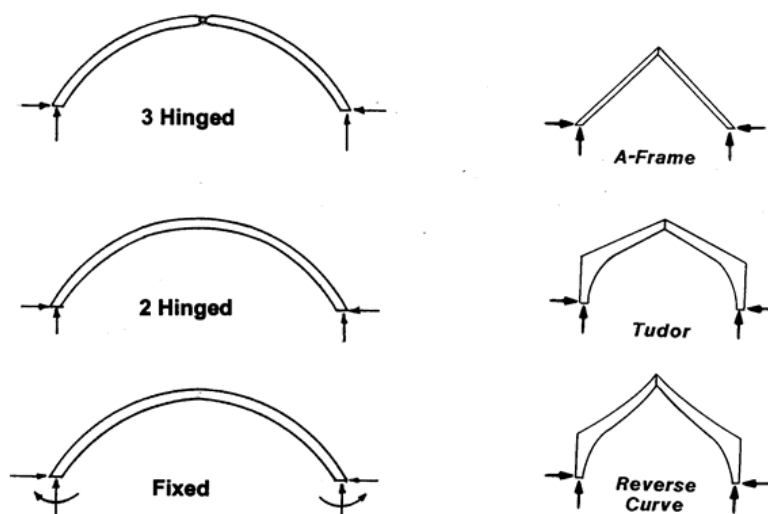
Figure 12-7: Basic Arches and Some Portal Derivatives

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.
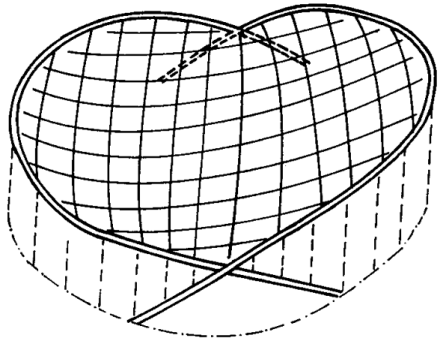
Arches to suspend a roof

(a)
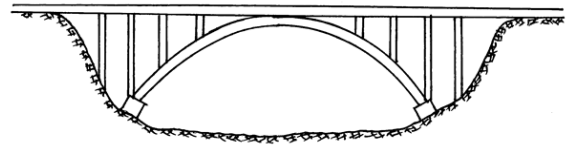
Arches supporting a bridge deck

(b)
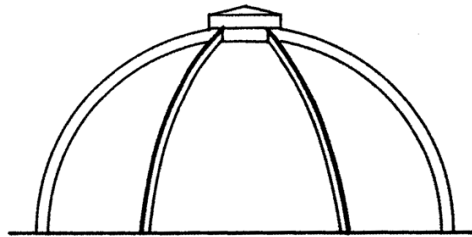
Converging arches

(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 an equivalent beam. However, this increased structural efficiency does not come without cost, i.e. large thrusts are developed at the supports. These can be accommodated by buttresses or a tie between the supports.

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

12.7. Arch Design - Methodology

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

In the case of the parabolic arch shown in FIGURE 12-9 the profile is defined by:

$$y = h\left[1 - \left(\frac{x}{L}\right)^2\right]$$
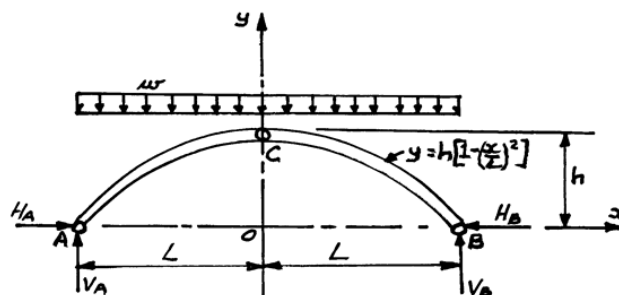
Figure 12-9: Parabolic Arch (Not to Scale)

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

$$\sum F_y \quad = 0: V_A + V_B = 2wl$$

$$\sum F_x \quad = 0: H_A = H_B$$

$$\sum M_{CR} \quad = 0: H_B \times h - V_B \times L + \frac{wL^2}{2} = 0$$

$$\sum M_{CL} \quad = 0: V_A \times L - H \times h - \frac{wL^2}{2} = 0 \tag{12.5}$$

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 $h = 10m$.
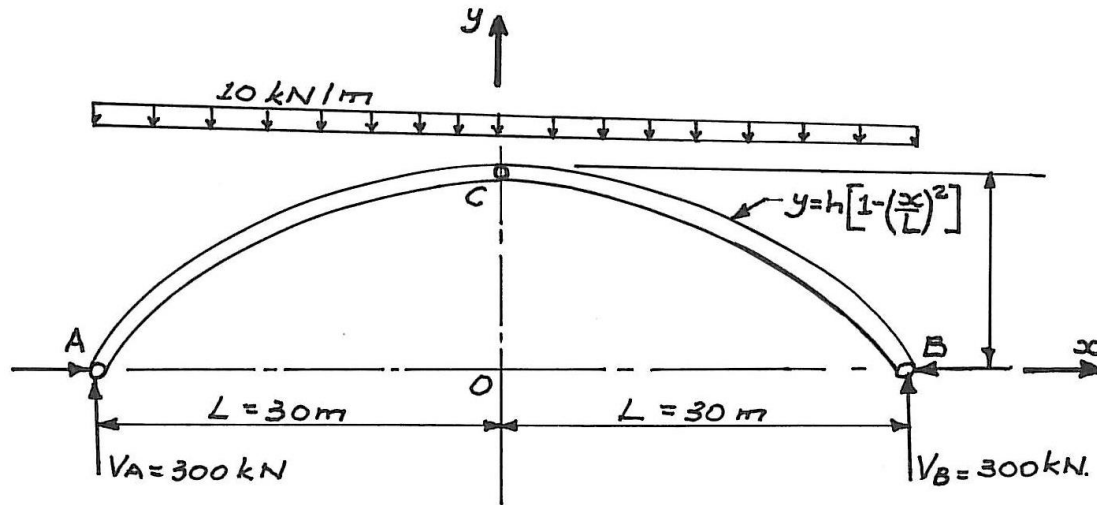
Figure 12-10: Symmetrical Parabolic Arch Symmetrically Loaded

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:

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

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

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

$$V_A = V_B = wl$$
$$V_A = V_B = 300\ kN$$

From:

$$\Sigma M_{CR} = 0$$
$$0 = 10 \times H_B - 300 \times 30 + \frac{10 \times 30^2}{2}$$
$$H_B = \frac{9000 - 4500}{10}\ kN$$
$$H_B = 450kN$$
$$H_A = 450kN$$

From Equation 12.6:

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

when:

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

$$\frac{dy}{dx} = -2\left(\frac{30}{30^2}\right)10 = -\frac{2}{3}$$

FIGURE 12-11 shows the normal (n) and tangential (t) co-ordinates at the base of the arch.
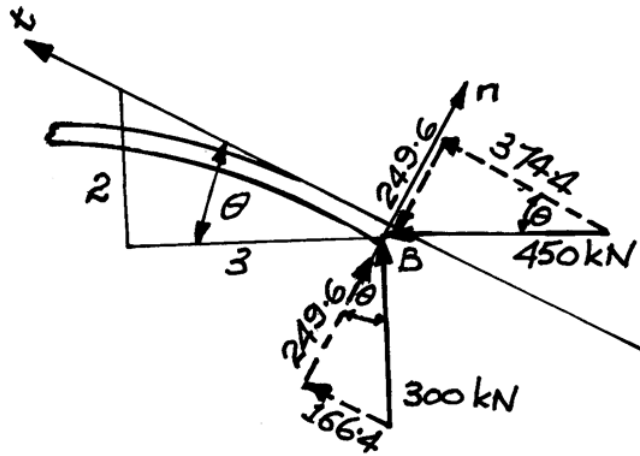
Figure 12-11: Components of Shear and Axial Force

Summing the axial (F_A) and shear (F_s) components at B in FIGURE 12-11 gives:

$$F_A = (374.4 + 166.4)kN$$
$$F_A = 540.8 \ kN \ (compression)$$
$$F_s = (+249.6 - 249.6)kN$$
$$F_s = 0 \ kN$$

Choosing a point mid-way between C and B on the arch as shown in FIGURE 12-12 (a) results in the slope being:

$$\frac{dy}{dx} = -2\left(\frac{x}{L^2}\right)h = \frac{2 \times 15 \times 10}{30^2} = \frac{1}{3}$$
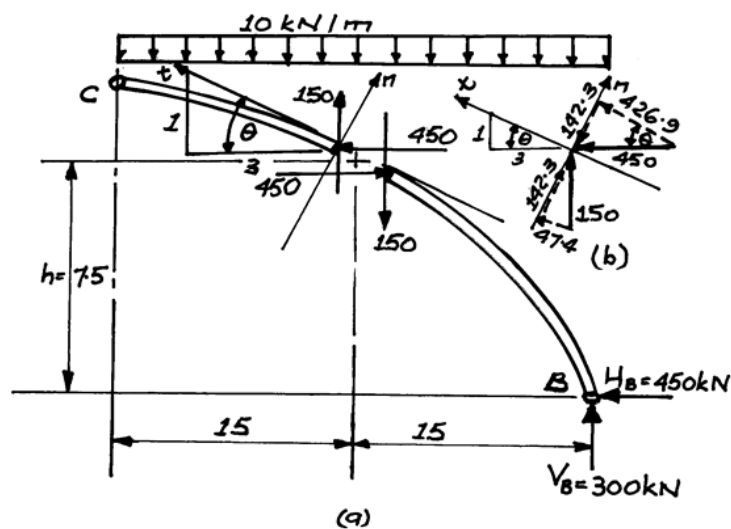
Figure 12-12: Exposed cross-section at mid-length and axial and shear force components

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

$$\sum F : 0 \quad = 300 - F_{VMS} - 10 \times 15$$
$$F_{VMS} \quad = 150 kN$$
$$\sum F_{HR} : 0 \quad = -450 + F_{HMS}$$
$$F_{HMS} \quad = 450 kN$$

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

$$F_A \quad = (426.9 + 47.4) kN$$
$$F_A \quad = 474.3 \ kN \ (compression)$$
$$F_s \quad = (+142.3 - 142.3) kN$$
$$F_s \quad = 0 \ kN$$

To find the moment at any cross-section x from the arch centre, as shown in the free body diagram in FIGURE 12-13.
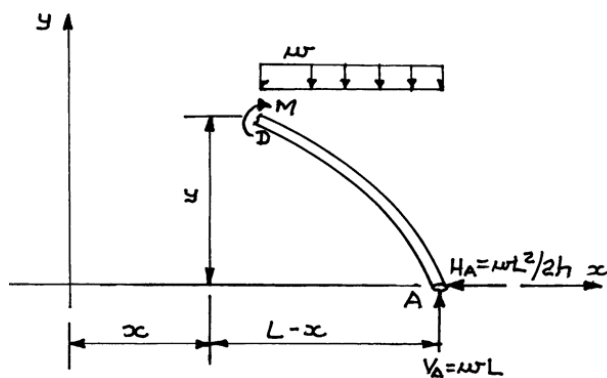
Figure 12-13: Free Body Diagram of Part of Arch

Taking moments about D:

$$\sum M_D \quad = 0 = \frac{wL^2}{2h} + \frac{w}{2}(L-x)^2 \times y - wL(L-x) + M$$
$$-M \quad = \frac{wL^2}{2h} \times h\left[1 - \left(\frac{x}{L}\right)^2\right] + \frac{w}{2}(L^2 - 2xL + x^2) - wL^2 + wxL$$
$$\frac{-2M}{w} \quad = L^2 - x^2 + L^2 - 2xL + x^2 - 2L^2 + 2xL$$
$$\frac{-2M}{w} \quad = 2L^2 - 2L^2 + x^2 - x^2 + 2xL - 2xL$$
$$M \quad = 0$$

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).
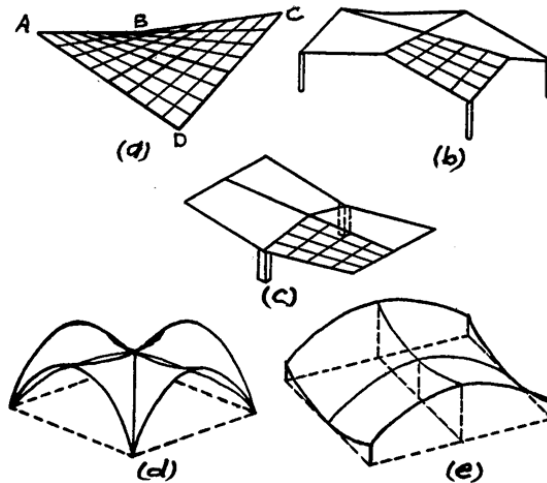
Figure 12-14: Various Hypar Configurations

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 phenomenon 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:

$$z = k\,x\,y \tag{12.7}$$

When k = o the hypar degenerates to a plane surface.
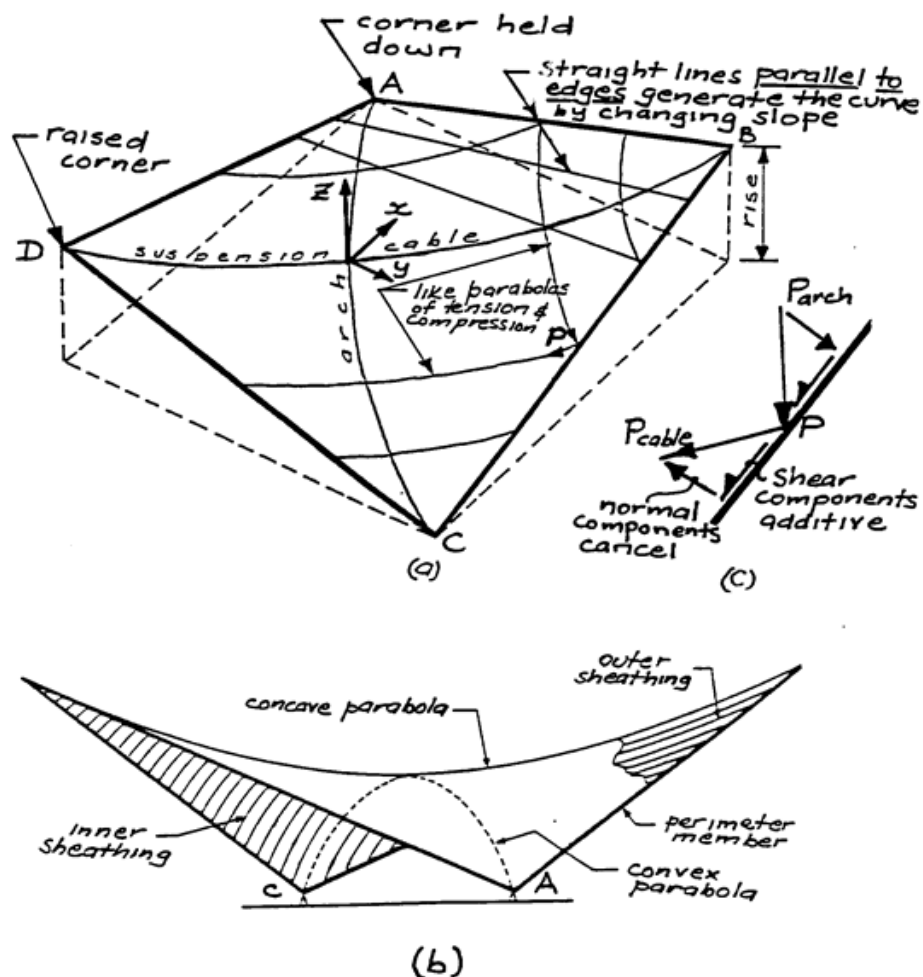
Figure 12-15: Two Views of a Single Hyperbolic-Paraboloid Shell

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 affected 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 $\left(\frac{l}{2}\right)$, 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:

$$\sum M_{OL} = 0 = \frac{Rl}{2} - Hh \tag{12.8}$$

Hence:

$$\frac{R}{h} = \frac{H}{\frac{l}{2}} \tag{12.9}$$

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

$$\frac{R}{h} = \frac{F}{k} \tag{12.10}$$

Giving:

$$F = R\frac{k}{h}$$
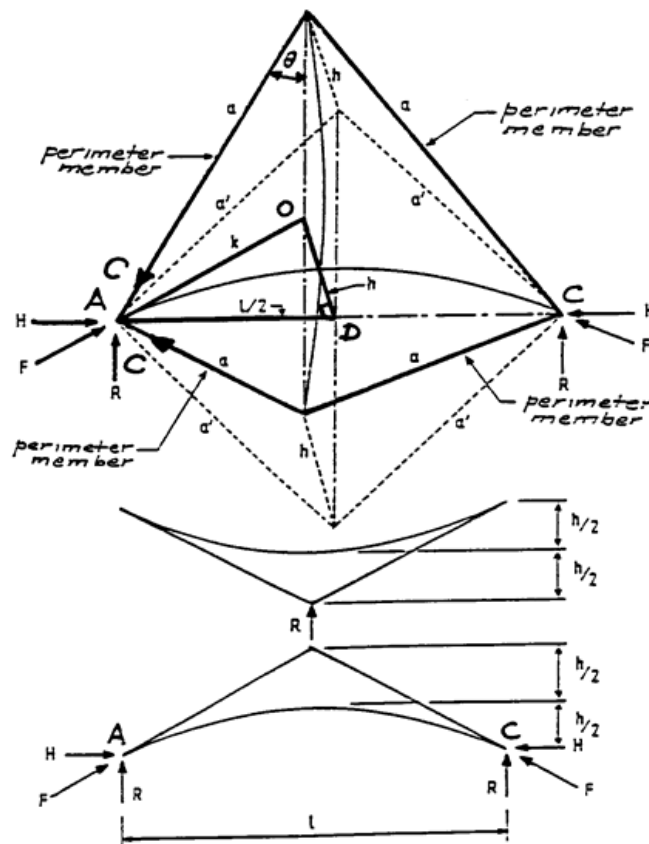
Figure 12-16: Reactive Force Components and Resultant

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:

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

$$\sin\theta = \frac{k}{a}$$

Giving:

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

But:

Hence:

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

Hence:

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

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

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

- transfer all of the accumulated membrane shears to the bearing points;
- resist any bending induced by the sheathing being connected to the top or bottom of these members.

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

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

Since membranal stresses result in boundary shears along the perimeter member these shears can be resolved to determine sheathing stresses. The principal forces in the shell are compressive forces c, parallel to the direction of the convex parabola and tension forces t, parallel to the direction of the concave parabola shown in FIGURE 12-17.
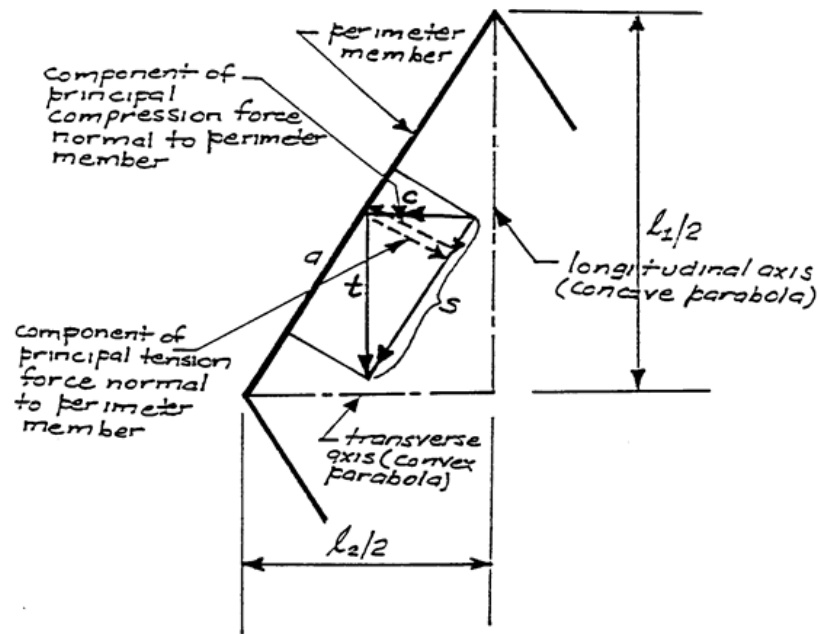
Figure 12-17: Resolved Components of the Tension and Compression Forces

The following lists the nomenclature applicable to FIGURE 12-16 and FIGURE 12-17

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

12.14. Methodology - Principal Membrane Forces

When the projected plan of the hypar is a diamond shape the tension (t) and compression (c) forces shown in FIGURE 12-17 can be resolved by proportion. The principal tensile force $\dfrac{(t)}{metre\ width}$ is:

$$\frac{t}{\frac{l_1}{2}} = \frac{s}{a'}$$

$$t = \frac{l_1\, s}{2\, a'} \tag{12.12}$$

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

$$\frac{c}{\frac{l_2}{2}} = \frac{s}{a'}$$

Hence:

$$c = \frac{l_2\, s}{2a'} \qquad\qquad (12.13)$$

When the projected plan of the hypar is square in shape (t) and $\dfrac{(c)}{metre\ width}$ 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:

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

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.

(a)

(b)
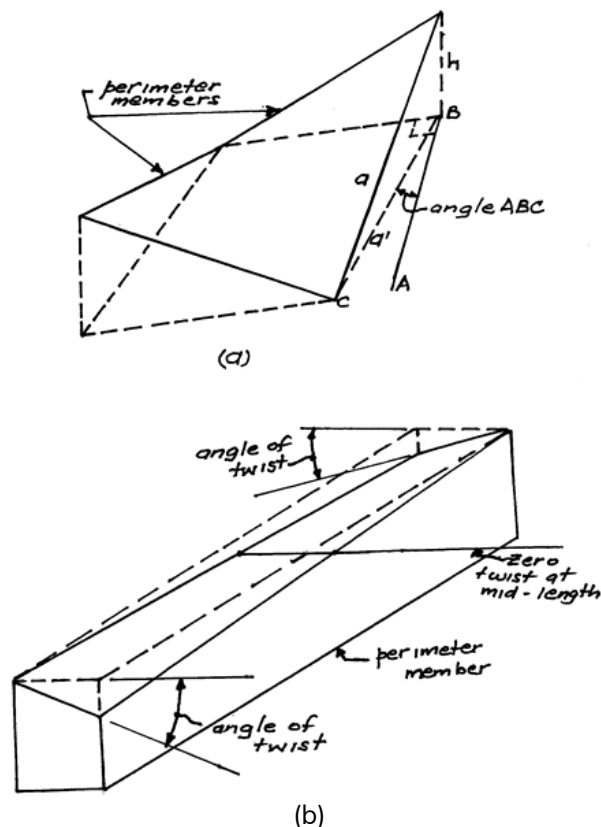
Figure 12-18: Angle of Twist

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 $\frac{(h)}{length\ of\ side\ (a)}$, to $\frac{1}{5}$.

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 constitute 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 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).
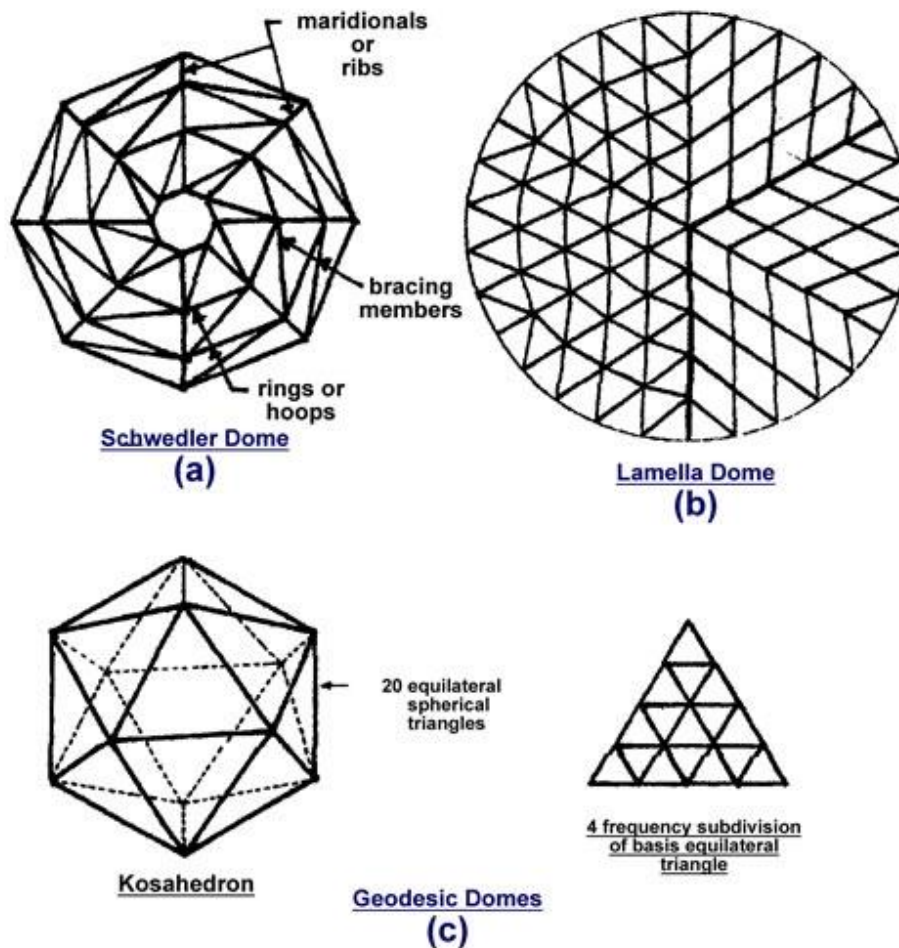
Figure 12-19: Different Dome Geometries

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).
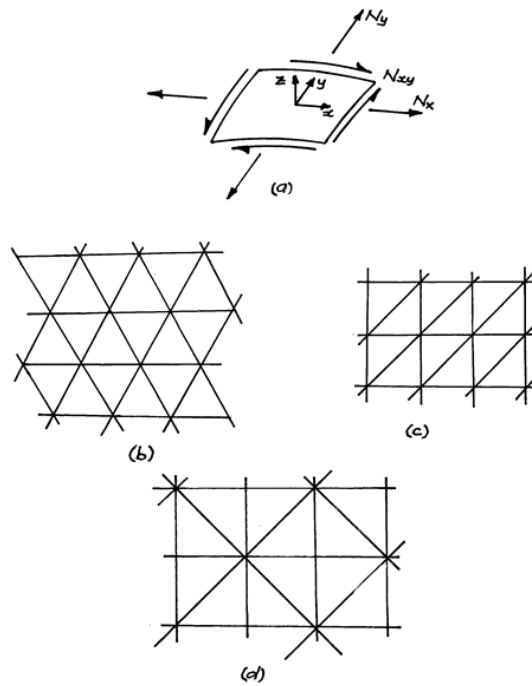
Figure 12-20: Membrane Forces and Reticulated Spatial Systems

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:

$$N_x = wR \left(\frac{1}{1+\cos\theta} - \cos\theta \right) \tag{12.16}$$

and the meridional force:

$$N_y = -wR \left(\frac{1}{1+\cos\theta} \right) \tag{12.17}$$

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.

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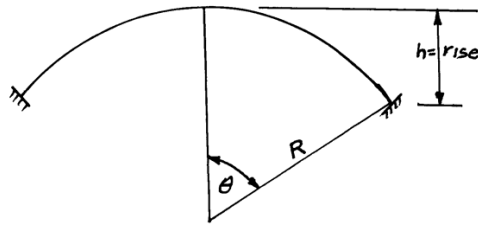
Figure 12-21: Angle to crown = θ and radius of curvature = R

when:

$$\left(\frac{1}{1+\cos\theta}\right) = 0 \text{ from Equation 12.16}$$

Then

$$\theta \approx 52^\circ$$

and

$$N_x = 0$$

With further increase in θ, N_x becomes positive, i.e. from $\theta > 52^\circ$ 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.

Grid System of Fig. 12.20(b)

Membrane Forces

Grid System of Fig. 12.20(c) & (d)
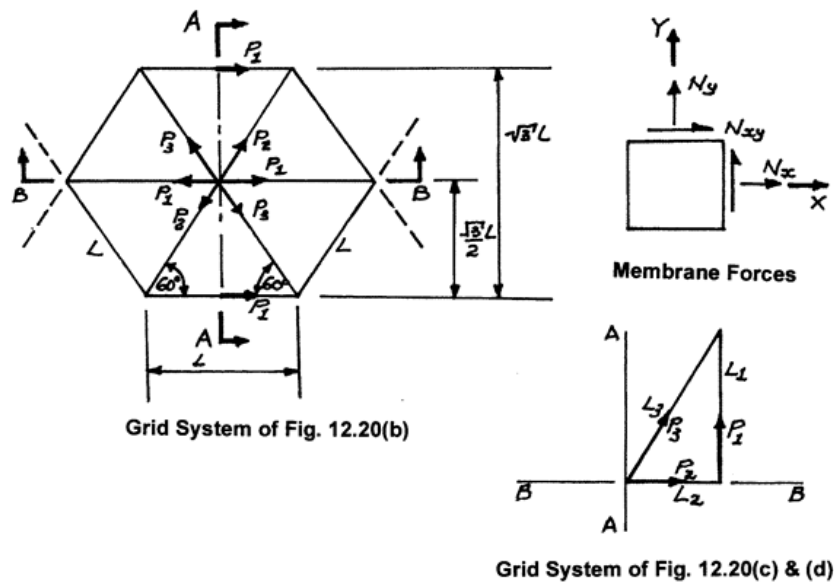
Figure 12-22: Grid Systems and Membrane Forces

Satisfying equilibrium of the forces at the section A-A, of length $\frac{\sqrt{3}}{2\times L\times 2} = \sqrt{3}\,L$, in the x-direction for the grid system of FIGURE 12-20 (b) gives:

$$\sum F_x = 0: \frac{P_1}{2} + \frac{P_2}{2} + P_1 + P_2 \cos 60^\circ + P_3 \cos 60^\circ$$

$$= 2\sqrt{3L \times N_x}$$

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Hence

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

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

In the Y-direction:

$$\sum F_y \quad = 0 : P_2 + P_3 = \tfrac{2}{\sqrt{3}} \times L \times N_y \tag{12.19}$$

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

$$\sum F_x \quad = 0$$
$$P_2 - P_3 \quad = 2LN_{xy} \tag{12.20}$$

Re-arranging the above equations:

$$N_x \quad = \frac{4P_1 + P_2 + P_3}{2\sqrt{3}\,L}$$
$$N_y \quad = \frac{\sqrt{3}\,(P_2 + P_3)}{2\,L} \tag{12.21}$$
$$N_{xy} \quad = \frac{(P_2 - P_3)}{2\,L}$$

Inverting Equations 12.21 gives:

$$P_1 \quad = \frac{L}{2\sqrt{3}}\left(3N_x - N_y\right)$$
$$P_2 \quad = \frac{L}{\sqrt{3}}\left(N_y + \sqrt{3}N_{xy}\right) \tag{12.22}$$
$$P_3 \quad = \frac{L}{\sqrt{3}}\left(N_y - \sqrt{3}N_{xy}\right)$$

At the crown of the dome where $N_{xy} = 0.33$, $N_x = N_y$, all three members forces are equal to:

$$P_1 \quad = P_2 = P_3 = \frac{-LRw}{2\sqrt{3}}$$

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:

$$N_x \quad = \frac{P_2}{L_1} + \frac{L_2 P_3}{L_1 L_3}$$
$$N_y \quad = \frac{P_1}{L_2} + \frac{L_1 P_3}{L_2 L_3} \tag{12.23}$$
$$N_{xy} \quad = \frac{P_3}{L_3}$$

And

$$P_1 \quad = L_2 N_y - L_1 N_{xy}$$
$$P_2 \quad = L_1 N_x - L_2 N_{xy} \tag{12.24}$$
$$P_3 \quad = L_3 N_{xy}$$

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.
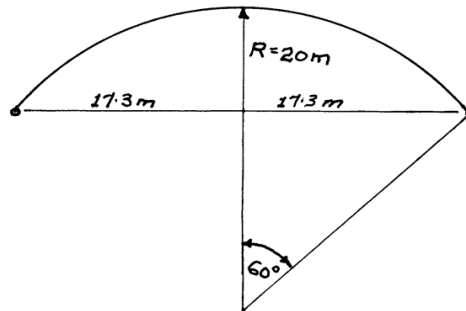
Figure 12-23: Dome Dimensions

12.21. Domes – Worked Example

Span

$$= \quad 2R \sin 60°$$

$$= \quad 34.6m$$

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)* $\frac{kN}{m}$	y (meridonal)~ $\frac{kN}{m}$
0	-15.0	-30.0
30	-9.9	-34.6
45	-3.6	-42.4
60	+5.0	-60.0

$$* \ N_x = wR \left(\frac{1}{1+\cos\theta} - \cos\theta \right); \ \sim N_y = -\frac{wR}{\cos\theta}$$

Table 12-1: Membrane forces, (-) is compression, (+) is tension

From Equations 12.22:

$$P_1 \quad = \frac{L}{2\sqrt{3}} \left(3N_x - N_y \right) = \frac{2}{2\sqrt{3}} (3 \times 5 - 60) = 43.3 \ kN$$

$$P_2 \quad = \frac{L}{\sqrt{3}} \left(N_y + \sqrt{3}N_{xy} \right) = \frac{2}{\sqrt{3}} \left(-60 + \sqrt{3} \times 0 \right)$$

$$= -69.3 \ kN$$

$$P_3 \quad = \frac{L}{\sqrt{3}} \left(N_y - \sqrt{3}N_{xy} \right) = \frac{2}{\sqrt{3}} \left(60 - \sqrt{3} \times 0 \right)$$

$$= -69.3 \ kN$$

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

$$f_c' = \frac{P}{A}$$

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

Assuming an LVL thickness of 45 mm:

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

Try an 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:

  o   individual member buckling due to direct forces on the shell;
  o   snap – through buckling due to local load concentrations;
  o   general shell buckling over a fairly large area;
  o   effects of unsymmetrical loads resulting in membrane shears;
  o   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, this section contains a collection of photographs of actual structures.

Folded Plates
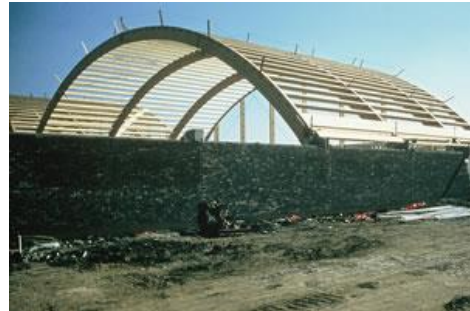
Arches

Hypars
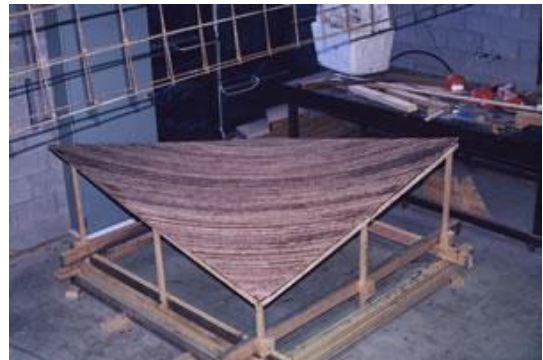
Domes

12.24. Design Aids

This section contains some design aids to assist designers at the preliminary design stage.
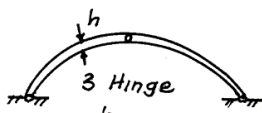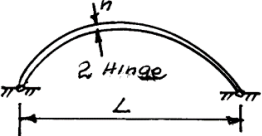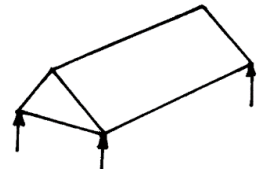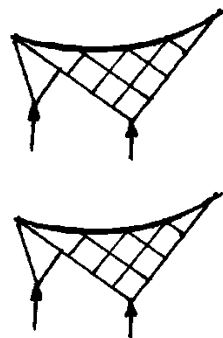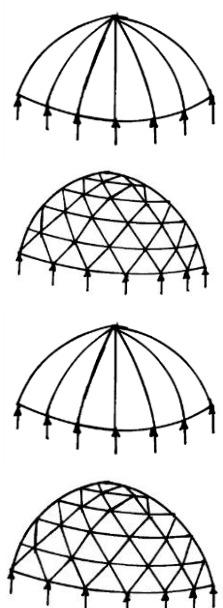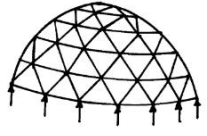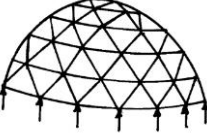
STRUCTURE	SPACING (m)	SPAN (m)	DEPTH
ARCHES			
	$5 - 15$	$20 - 60$	$h = \frac{L}{50}$
	$5 - 20$	$20 - 100$	$h = \frac{L}{55}$
FOLDED PLATE		**Max. Likely (m x m)**	**COMMENTS**
		$20\ x\ 10$	Stiffening ribs may be required
HYPAR			
		$25\ x\ 25$	LVL edge beams with boarded membrane
DOMES			
		$30\emptyset$	Ribbed dome with plywood membrane
		$100 + \emptyset$	Braced dome with LVL members

Table 12-2: Preliminary Design Information

13. Connection Design – Plywood & LVL

13.1. Introduction

In no way is this chapter meant to replace Section 4 of AS 1720.1 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 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 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 result in the fastener being subject to axial loads 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).
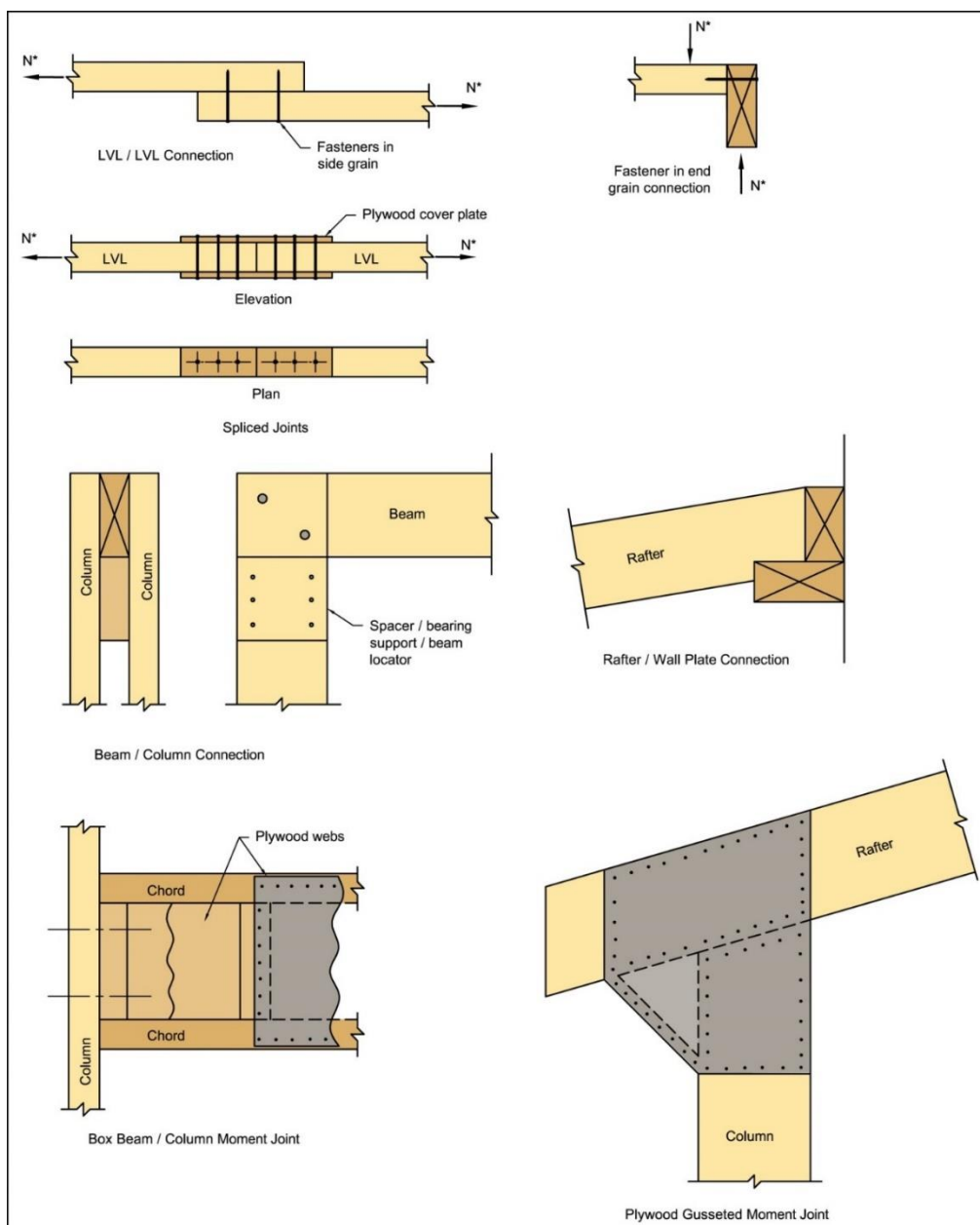
Figure 13-1: Examples of Type 1 Connections
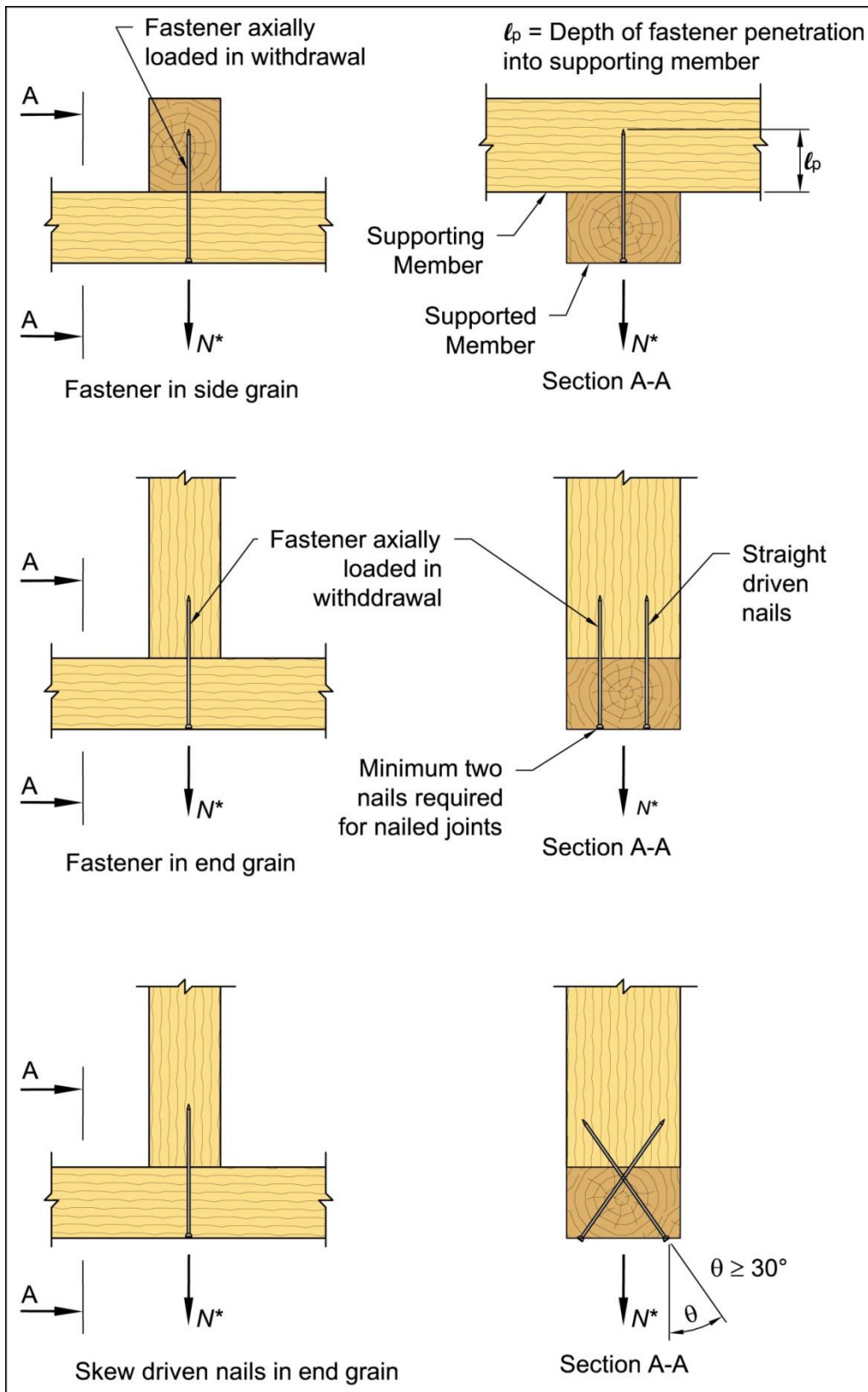
Figure 13-2: Example of Type 2 Connections for Nails and Screws

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 will attain the required capacity.
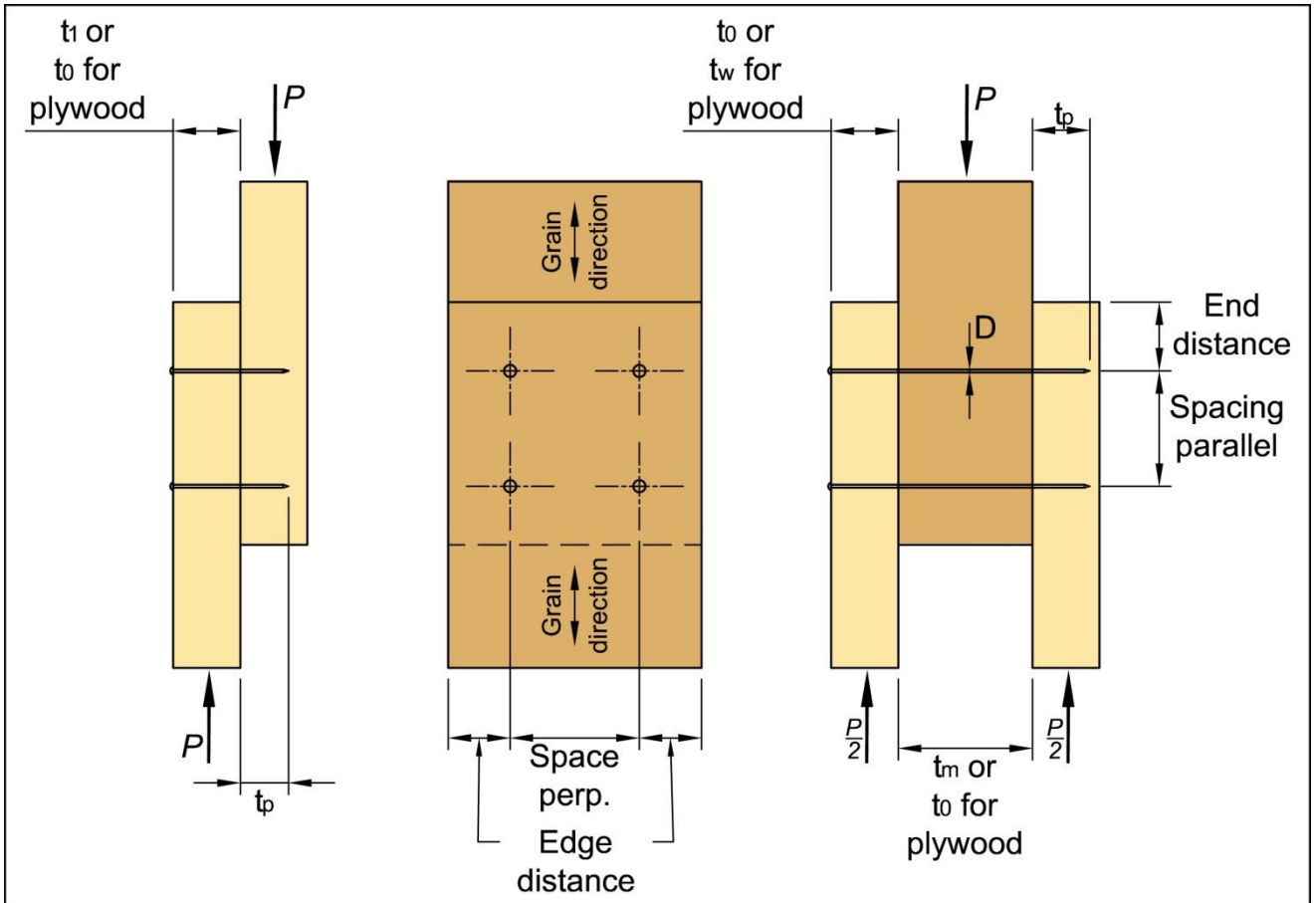
Figure 13-3: Type 1 Nailed and Screwed Connections for 2 and 3 Members

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

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

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	$20D$ $10\,D$	$10D$ $3D$	$10D$ $3D$

Table 13-1: Minimum Distances for Nails and Screws

Other requirements to attain AS 1720.1 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 $t_p < 5D, P = 0.$
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: $t_p < 5D; P = 0.$
Screws	$t_1 > 10D, t_p > 7D$ For: Lesser of t_1 and t_p reduce proportionally until, t_1 or $t_p \leq 4D$ when $P = 0$
Plywood	Fastener capacity 10%>timber to timber joints provided $t_o > 1.5D; t_p > 10D; t_w > 10D$ For: $\frac{t_p}{D}$ or $\frac{t_p}{D} < 5, P = 0$

FIGURE 13-4 defines the critical dimensions for bolts and TABLE 13-2 quantifies them in terms of the bolt diameter D.

NOTE: Bolt characteristic capacities given in Tables 4.9 and 4.10 of AS 1720.1 are for the effective timber thicknesses b_eff for single bolts loaded parallel and perpendicular to the grain. The b referred to in TABLE 13-2 is defined therein.
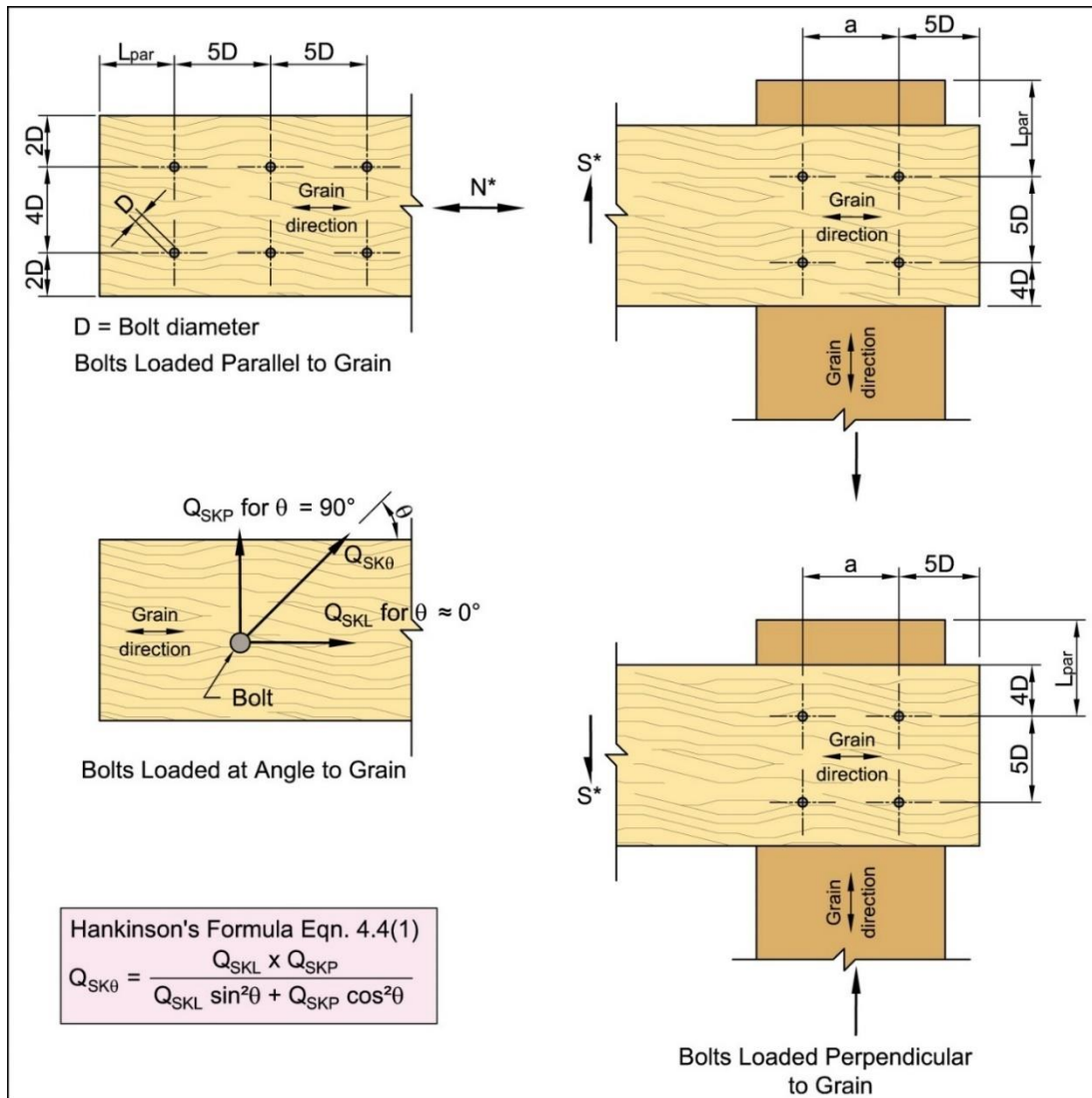
Figure 13-4: Critical Bolt Spacings and Distances

Spacing Type	Distance – bolts loaded parallel to grain ($\theta = 0\,° to\,30\,°$)	Distance – bolts loaded perp. to grain ($\theta = 30\,° to\,90\,°$)
End distance (l_{par})	$8D$ unseasoned tension member $7D$ seasoned tension member $5D$ others	$5D$
Edge distance	$2D$	$4D$
Between Connectors, Along grain (a)	$5D$	$\leq 2.5D$ for $\frac{b}{D} = 2$ and increase proportionally to $\leq 5D$ for $\frac{b}{D} \geq 2.$[*]
Between Connectors, Across grain	$4D$	$5D$

*b = effective thickness of member loaded perpendicular to grain

Table 13-2: Other Requirements for Bolts

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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 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 1720.1.	gust	1.14
Earthquake loads	from AS 1170.4	5 second	1.14
Regular snow loads	alpine regions	5 months	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:

D_L = 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. $k_{14} = 1.0$ (single) and 2 (double) 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).
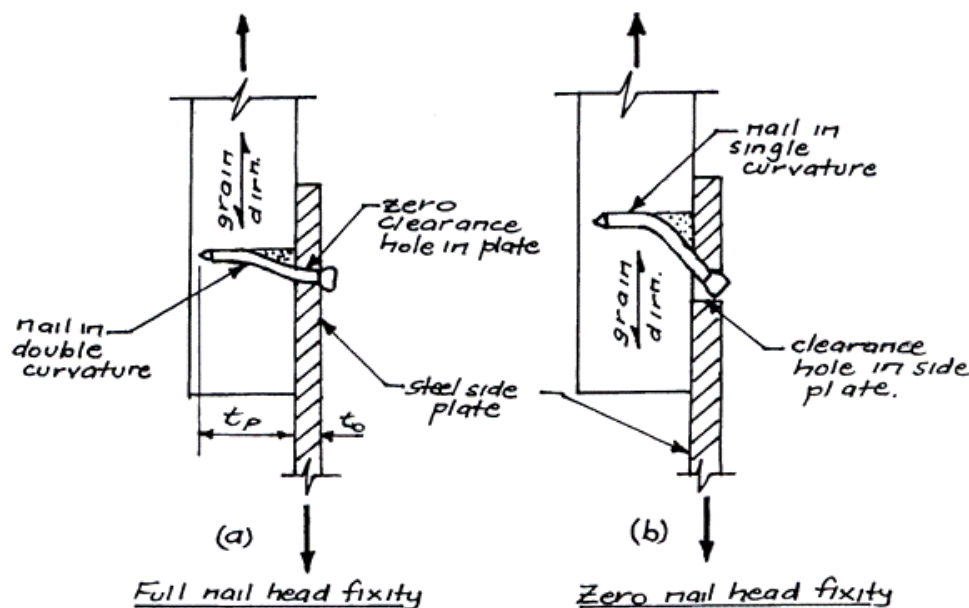
Figure 13-5: Nail Head Fixity

TABLE 13-4 gives values of k_{16} for nailed and screwed joints fixing side plates of various materials.

Side Plate Material	Plate Thickness Guide	Hole diameter	k_{16}
Metal	to $> 1.5D$	tight fitting	1.2
Plywood gusset	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}) considers 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.

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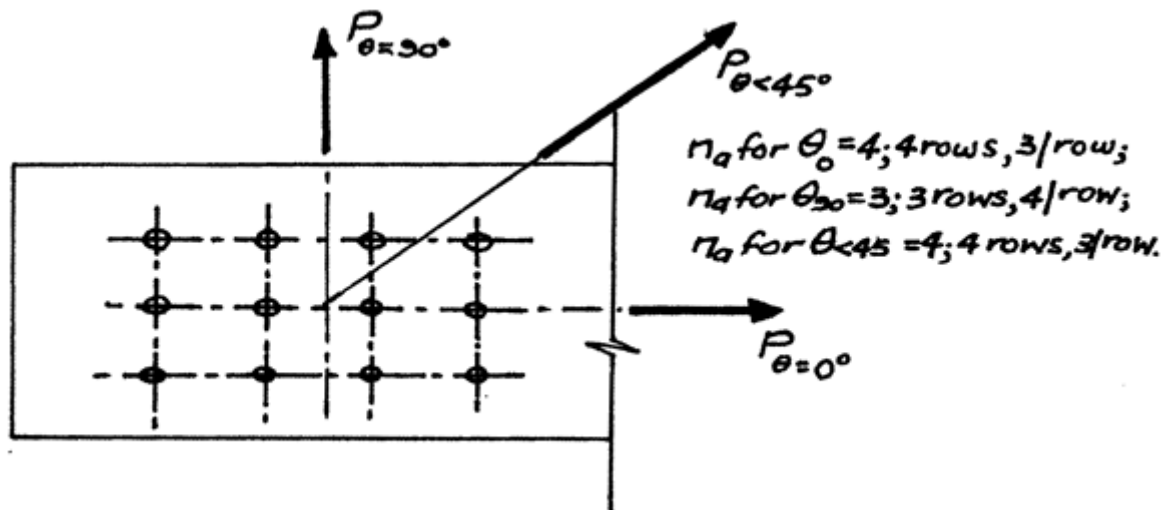
Figure 13-6: Row Definition Relative to Applied Load

TABLE 13-5 gives values for the factor 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.
6. Apply modification factors to the relationship:

$$N_{d,j} = \phi\, k_1\, k_{13}\, k_{14}\, k_{16}\, k_{17}\, n\, Q_k$$

ewpaa

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 $(N_{d,j})$ for a Type 1 joint (containing (n nails)) required to resist direct loads.

For the strength limit state to be satisfied:

$$N_{d,j} \geq N^* \tag{13.2}$$

where:

$$N_{d,j} = \phi k_1 \, k_{13} \, k_{14} \, k_{16} \, k_{17} \, n \, Q_k \tag{13.3}$$

and

N^*	= design action due to the applied factored loads on the connection
ϕ	= capacity factor;
k_1	= the duration of loads factor for joints;
k_{13}	= 1.0 for nails in side grain; = 0.6 for nails in end grain
k_{14}	= 1.0 for nails in single shear; = 2.0 for nails in double shear
k_{16}	= 1.2 for nails driven through close fitting holes in metal side plates; = 1.1 for nails driven through plywood gussets; = 1.0 otherwise.
k_{17}	= factor for multiple nailed joints for Type 1 connections designed to resist direct loads in either tension or compression.
n	= total number of nails in the connection resisting the design action effect in shear.
Q_k	= nail characteristic capacity given in Tables 4.1(A) and 4.1 (B) in AS1720.1.

13.6. Design of Type 2 Nailed Connections

Equation 13.4 gives the design capacity $(N_{d,j})$ 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:

$$N_{d,j} \geq N^*$$

where:

$$N_{d,j} = \phi k_{13} \, l_p \, n \, Q_k \tag{13.4}$$

and

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

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

l_p = depth of penetration (mm) into supporting member.

n = total number of nails in the joint.

Q_k = characteristic nail capacity Table 4.2 A & B

Design of Moment Resisting Nailed Connections

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

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

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

The displacement (Δ), in millimetres, of nailed or screwed joints in single shear for solid wood/solid wood connections may be estimated as follows:

a) For a fastener load (Q^*) where $Q^* \leq Q_a$:

$$\Delta = 0.5 \left(\frac{Q^*}{Q_a}\right)^{2.17} \ where \ Q^* \leq Q_a \tag{13.5}$$

b) For a fastener load (Q^*) where $Q_a \leq Q^* \leq Q_b$

$$\Delta = 2 \left(\frac{Q^* - Q_a}{Q_b - Q_a}\right) + 0.5 \tag{13.6}$$

where:

Q^* = lateral shear force, for which displacement is being calculated, in newtons

Q_a = $0.165 \, D^{1.75} \, j_{12} \, h_{32}$

Q_b = $0.165 \, D^{1.75} \, j_{13} \, h_{32}$

D = diameter of nail or screw, in millimetres;

j_{12}, j_{13} = duration factors given in Table C2, AS 1720.1;

h_{32} = stiffness factor given in Table C1, AS 1720.1;

Note: For metal and plywood side plates Equations 13.5 and 13.6 lead to conservative overestimates of joint 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

ewpaa

Critical Load Combinations

The load combinations normalised for long term application are:

Load Combinations	Factors	Factored Loads (kN)	k_1	$D_L = \frac{N^*}{k_1}$
Dead (permanent)	$1.35G$	$1.35 \times 20 = 27\ kN$	0.57	47.4
Dead + Live (construction)	$1.2G + 1.5Q$	$1.2 \times 20 + 1.5 \times 5.5 = 33.3\ kN$		41.9
Dead + Ultimate Wind Load (compression)	$0.9G - 1.5W_u$	$0.9 \times 20 - 1.5 \times 26.6 = 21.9\ kN$	1.14	19.2
Dead + Ultimate Wind Load (tension)	$1.25G + W_u$	$1.25 \times 20 + 6.6$	1.14	27.7

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

The critical load is the dead load with $D_L = 47.4\ kN$. The connection will be designed for $N^* = 27\ kN$ 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/rows to be determined. In this case:

$$n_r = 5$$
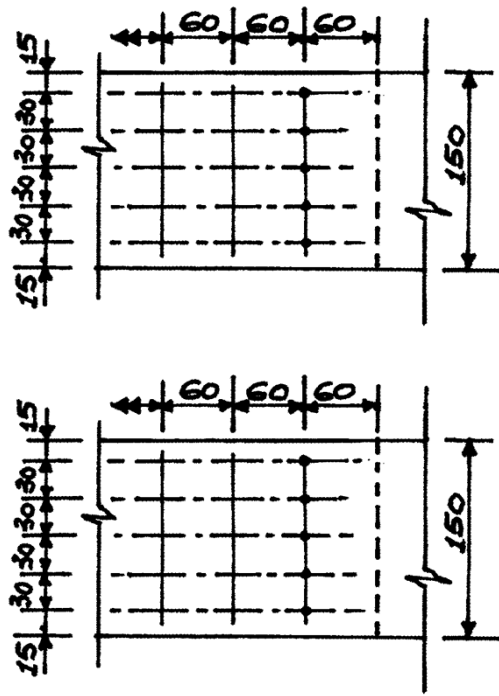
Figure 13-7: Sketch of Joint

Connector Capacity Factors

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

$$\phi \quad = 0.8$$

- Critical load is the dead load:

$$k_1 \quad = 0.57$$

- Grain orientation factor for Type 1 nailed joints:

$$k_{13} \quad = 1.0$$

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

$$k_{14} \quad = 1.0$$

- For nails driven through plywood:

$$k_{16} \quad = 1.1$$

- An assumed value for k_{17} is:

$$k_{17} \quad = 0.94$$

- 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:

$$Q_k \quad = 665\ N$$

Number of Nails

From Equation 13.3:

$$n \quad = \frac{N_{d,j}}{\phi\, k_1\, k_{13}\, k_{14}\, k_{16}\, k_{17}\, Q_k}$$

$$= \frac{25\times10^3}{0.8\times0.57\times1.0\times1.0\times\ 1.1\times0.94\times665}$$

$$n \quad = 42\ nails\ each\ side$$

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Number of Rows

The number of rows of nails can be found from:

$$n_a = \frac{n}{n_r} = \frac{42}{5} = 9 \; rows$$

The assumed value of 0.94 for k_{17} is satisfactory since it applies for up to 10 rows.

FIGURE 13-7 shows the connection with the nailing pattern chosen.
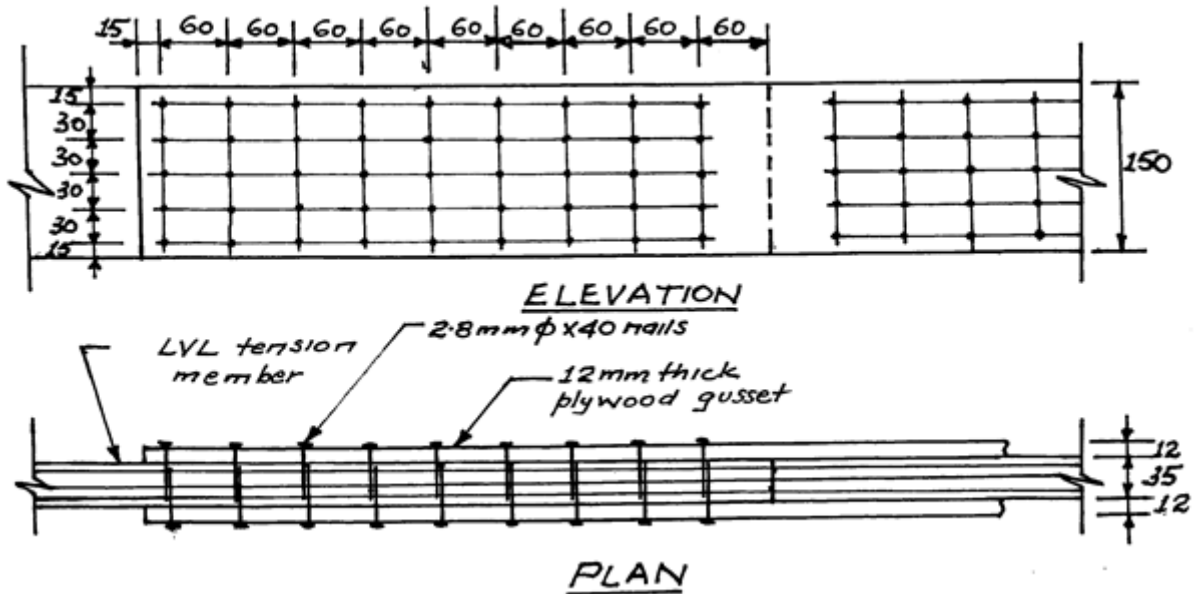
Figure 13-8: LVL/Plywood Spliced Joint

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.

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 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 $(N_{d,j})$ for a Type 1 Joint containing (n) screws required to resist the load.

For the strength limit state to be satisfied:

$$N_{d,j} \geq N^* \tag{13.8}$$

where:

$$\phi Nj = \phi \times k_1 \times k_{13} \times k_{14} \times k_{16} \times k_{17} \times n \times Q_k \tag{13.9}$$

and

$N_{d,j}$ = 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.

N^* = design action effect on joint 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 (ϕNj) for a Type 2 joint containing (n) screws.

For the strength limit state to be satisfied:

$$N_{d,j} \geq N^* \tag{13.10}$$

where $N_{d,j}$ is the lesser of:

$$N_{d,j} = \phi k_{13} \, l_p \, n \, Q_k \tag{13.11}$$

OR

$$N_{d,j} = n \, N_{d,ts} \tag{13.12}$$

where:

$N_{d,j}$ = design joint capacity of Type 2 screwed connection, i.e. along connector axis.

and:

N^* = design action effect in the axial direction of the fastener

ϕ = capacity factor.

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

l_p = depth of screw penetration (mm) into supporting member.

N = total number of screws in joint.

Q_k = characteristic capacity given in Tables 4.6(A) and 4.6(B).

$N_{d,ts}$ = 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 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, 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.

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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 – 25 to 200mm unseasoned and 25 to 120 mm 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 k_{13} and k_{14} are not considered to influence bolted joint response is because:

- 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.
- k_{14}, the shear planes factor for nails and screws, is accounted for in bolted connections by the system capacity quantity Q_{skl} or 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 (k_1) is the same as defined in Section 13.3 for nailed connections.
- Head fixity factor (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 b_{eff} for loads parallel to the grain is $b_{eff} > 5D$ and perpendicular to grain is $b_{eff} > 10D$.

Multiple bolt factor (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 k_{17} for varying number of rows of bolts (n_a).
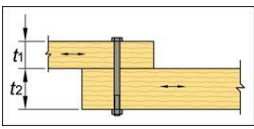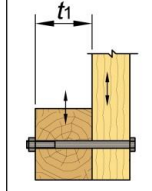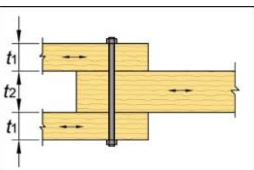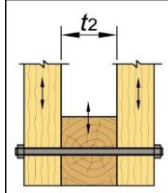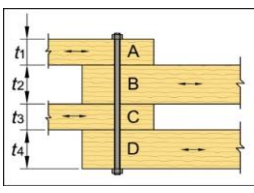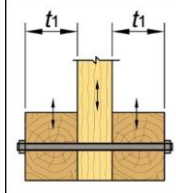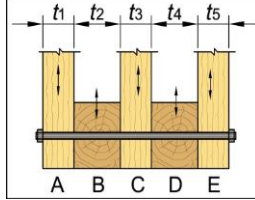
CHARACTERISTIC CAPCITZIES & EFFECTIVE TIMBER THICKNESS FOR SINGLE BOLTS					
Parallel to Grain			Perpendicular to Grain		
Type of Joint	Effective thickness (b_{eff})	System Capacities (Q_{skl})	Type of Joint	Effective thickness (b_{eff})	System Capacities (Q_{skl})
	b_{eff} = smaller of t_1 and t_2	Q_{kl}		$b_{eff} = 2t_1$	Q_{kp}
	b_{eff} = smaller of t_2 and $2t_1$	$2Q_{kl}$		$b_{eff} = t_2$	$2Q_{kp}$
	(1) Between A and B: b_{eff} = smaller of t_1 and t_2; (2) Between B to C: b_{eff} = smaller of t_2 and t_3 (3)etc.	(1) Q_{kl} (2) Q_{kl} (3) etc. Q_{skl} = sum of basic loads (1), (2), (3) etc.		$b_{eff} = 2t_1$	$2Q_{kp}$
				(1) Between A to B: $b_{eff} = t_2$ (2) Between B to C: $b_{eff} = t_2$ (3) Between C to D: $b_{eff} = t_4$	(1) Q_{kp} (2) Q_{kp} (3) etc. Q_{skp} = sum of basic loads (1), (2), (3), etc.

Figure 13-9: System Capacities and Effective Timber Thicknesses

Type of Joint	Values of k_{17}				
	$n_a \le 4$	$n_a = 5$	$n_a = 10$	$n_a = 15$	$n_a \ge 16$
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.
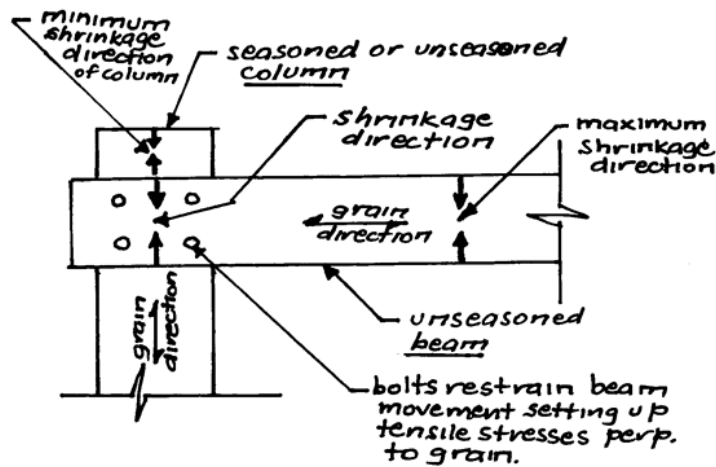
Figure 13-10: Lateral Restraint Stresses

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:

$$N_{d,j} = \phi \, k_1 \, k_{16} \, k_{17} \, n \, Q_{sk}$$

- Find the number of connectors/rows 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 $(N_{d,j})$ for a Type 1 joint containing (n) bolts to resist the applied lateral loads.

For the strength limit state to be satisfied:

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$$N_{d,j} \geq N^* \tag{13.13}$$

where:

$$N_{d,j} = \phi k_1 k_{16} k_{17} n Q_{sk} \tag{13.14}$$

and:

N^* = design action effect in shear

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 plates;
= 1.0 for other cases

k_{17} = Multiple bolt factor

n = total number of bolts resisting design effect in 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 $(N_{d,j})$ for a type 2 joint containing (n) bolts which are loaded in direct tension.

For the strength limit state to be satisfied:

$$N_{d,j} \geq N^* \tag{13.15}$$

where ϕN_j is the lesser of:

$$N_{d,j} = n N_{d,tb} \tag{13.16}$$

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

$$N_{d,j} = \phi k_1 k_7 n f'_{pj} A_w \tag{13.17}$$

and:

N^* = design action due to factored tensile loads

n = number of bolts in the joint

$N_{d,tb}$ = design tensile capacity of bolts   (Table 4.11)

ϕ = capacity factor

k_1 = duration of load factor

k_7 = length of bearing factor of washer   (Table 2.6 & 4.11)

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 1720.1 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 affected by application of the classical mechanics formula $\tau = \frac{T\rho}{J}$. 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 0.

Eccentric joints arise when the centre lines of action of their member forces, for example, those of a truss joint do not intersect at a common point as shown in FIGURE 13-10. This indiscretion can cause fairly high shear and moments to develop and tensile stresses perpendicular to the grain may also be high.
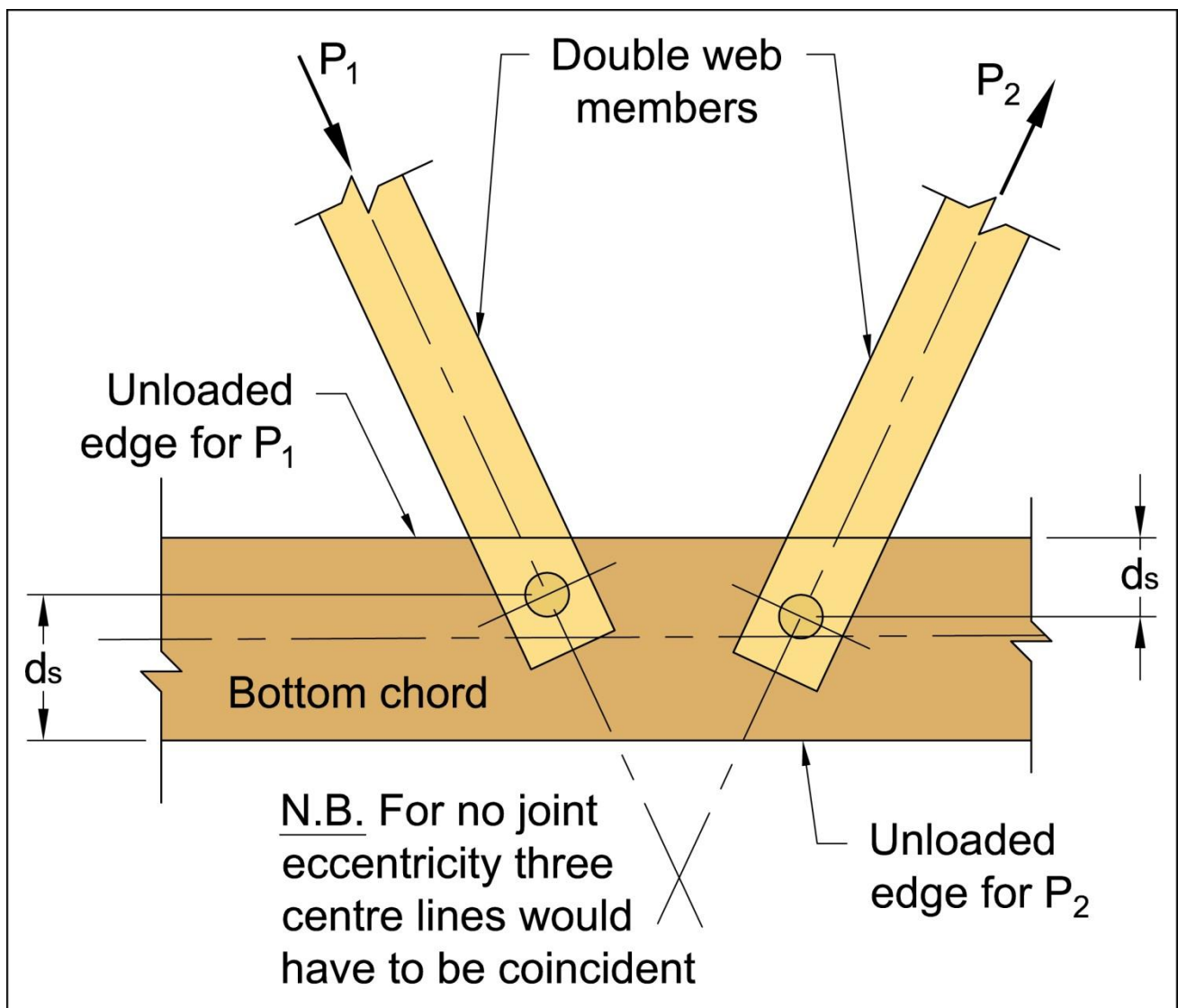
Figure 13-11: Eccentric Joint

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

$$f_s = \frac{VQ}{It}$$

This results in the A_{sj} term containing the $\frac{2}{3}$ rds of shear area, $b \times d_s$.

To accommodate this type of situation AS 1720.1 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 ($V_{d,sj}$) satisfies Equations 13.18 and 13.19.

$$V_{d,sj} \geq V_{sj}^* \tag{13.18}$$

where:

$$V_{d,sj} = \phi\, k_1\, k_4\, k_6\, f_{sj}^{'}\, A_{sj} \tag{13.19}$$

and:

V_{sj}^* = design 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;
    $= \frac{2bd_s}{3}$ where b is thickness of the member.
    See FIGURE 13-10 for d_s.

Washers

AS 1720.1 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 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 $\varDelta$, considering grain direction.

$$\varDelta = \varDelta_i + \left(\frac{j_{14}}{h_{33}}\right)\left(\frac{Q^*}{Q_k}\right) \text{ for loads parallel to grain} \tag{13.20}$$

$$\varDelta = \varDelta_i + \left(\frac{j_{14}}{h_{33}h_{35}}\right)\left(\frac{Q^*}{Q_k}\right) \text{ for loads perp' to grain} \tag{13.21}$$

where:

Δ = total displacement

Δ_i = initial displacement due to oversize holes, in millimetres

    = 0, for a load superimposed on an existing load, otherwise

    = $\dfrac{1}{\sqrt{n_{con}}}$ for bolted joints

    = $\dfrac{1}{2\sqrt{n_{con}}}$ for split-ring fastener or shear-plate fastener

j_{14} = duration of load factor given in Table C3, AS 1720.1;

h_{33} = stiffness factor given in Table C4, AS 1720.1

h_{35} = 1.0 for splitting and shear plate fastener

    = 1.5 for 2 or 3 member bolted joint.

    = 2.5 for multiple member bolted joint

Δ_i = initial displacement of joint due to oversize holes

n_{con} = number of bolts or split-ring or shear-plate fastener sets in the joint

Q^* = force for which displacement is being calculated, per fastener in (N);

Q_k = characteristic capacity of a fastener as defined in section 4, AS 1720.1 in (N)

13.17. Bolted Connection - Design Example

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

A roof truss having the geometry shown in FIGURE 13-11 is to be featured in a commercial building to be constructed on Queensland's Gold Coast. The truss has been designed, but LVL with A faces for appearance, is being considered as an alternative. The joints of the truss are to be bolted using M12 galvanised bolts. The design load is to be taken as the load in the top chord.

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

nominal dead load: G   = 8kN (axial compression top chord)

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

The LVL for the single top chord is 150 x 45mm and for the double bottom chord 150 x 35mm. The joint strength group of the LVL is JD3.
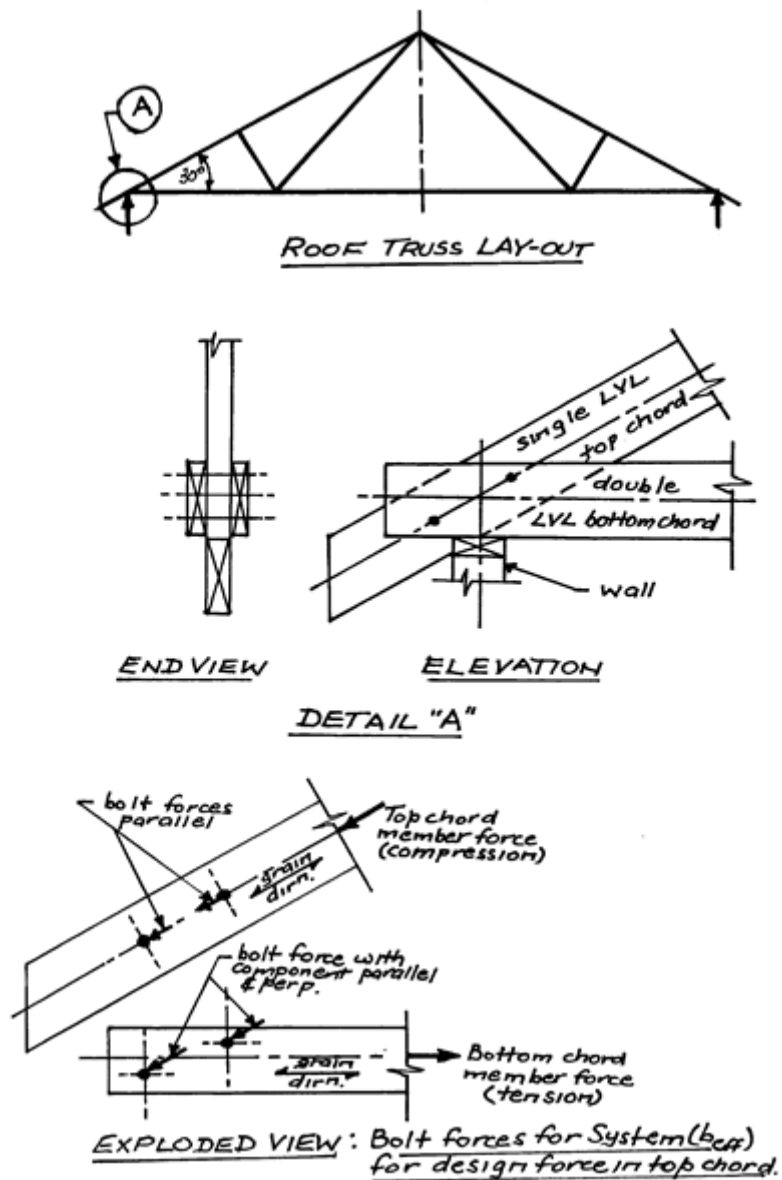
Figure 13-12: Bolted Truss Joint

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.

$$N^* = 1.25 \times 8 + 1.5 \times 6.0$$

$$N^* = 19kN$$

Connector Capacity Factors:

For the live load being applied for 5 days:

$$k_1 \quad = 0.77$$

Because there are no rigid steel side plates:

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

The exploded view in Figure 13-11 shows bolt forces angle to the grain to be:

$$\theta \quad = 0° \text{ for top chord;}$$

$$\theta \quad = 30° \text{ for bottom chord.}$$

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

$$\phi \quad = 0.65$$

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;

$$b_{eff} \quad = 45mm \text{ for top chord;}$$

$$Q_{sk\ell} \quad = 2Q_{kl}$$

For JD3 LVL and 12mm Ø bolts:

$$Q_{kl} \quad = 11900 \, N \qquad\qquad \text{AS 1720.1 Table 4.9(c)}$$

$$Q_{sk\ell} \quad = 2 \times 11.9$$

$$\quad\quad = 23.8 \, kN$$

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

$$b_{eff} \quad = 2 \times 35mm$$

$$\quad\quad = 70mm \text{ for bottom chord}$$

$$Q_{kp} \quad = 7410 \, N \qquad\qquad \text{AS 1720.1 Table 4.10}$$

$$Q_{skp} \quad = 2 \times 7.41$$

$$\quad\quad = 14.28 \, kN$$

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:

$$Q_{sk\theta} = \frac{Q_{skl}\, Q_{skp}}{Q_{skl}\, \sin^2\theta + Q_{skp}\, \cos^2\theta}$$

$$Q_{sk\theta} = \frac{23.8 \times 14.28}{23.8\, \sin^2 30° + 14.28\, \cos^2 30°} = 20.4\ kN$$

Number of Bolts:

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

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

The number (n) of bolts required:

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

Assume $\quad k_{17} = 1.0$

$$n \quad = \frac{18.8}{0.8 \times 0.77 \times 1.0\ \times 1.0\ \times 20.4} = 1.48$$

$$\approx say\ 2\ bolts$$

Number of Rows:

Number of rows of bolts (n_a):

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

For 2 rows of bolts:

$$n_a = 2$$

Number of bolts / row (n_r):

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

i.e. $\qquad\qquad\qquad n_r = 1$, 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. $Q_{sk} = 20.4 kN$

$$\text{Design capacity for joint} \quad = N_j^*$$
$$= \phi\, k_1\, k_{16}\, k_{17}\, n\, Q_{sk}$$
$$= 0.8 \times 0.77 \times 1.0 \times 1.0 \times 2 \times 20.4$$
$$N_j^* \quad = 25.1 kN \geq N_j^* \text{ so OK}$$

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. 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.
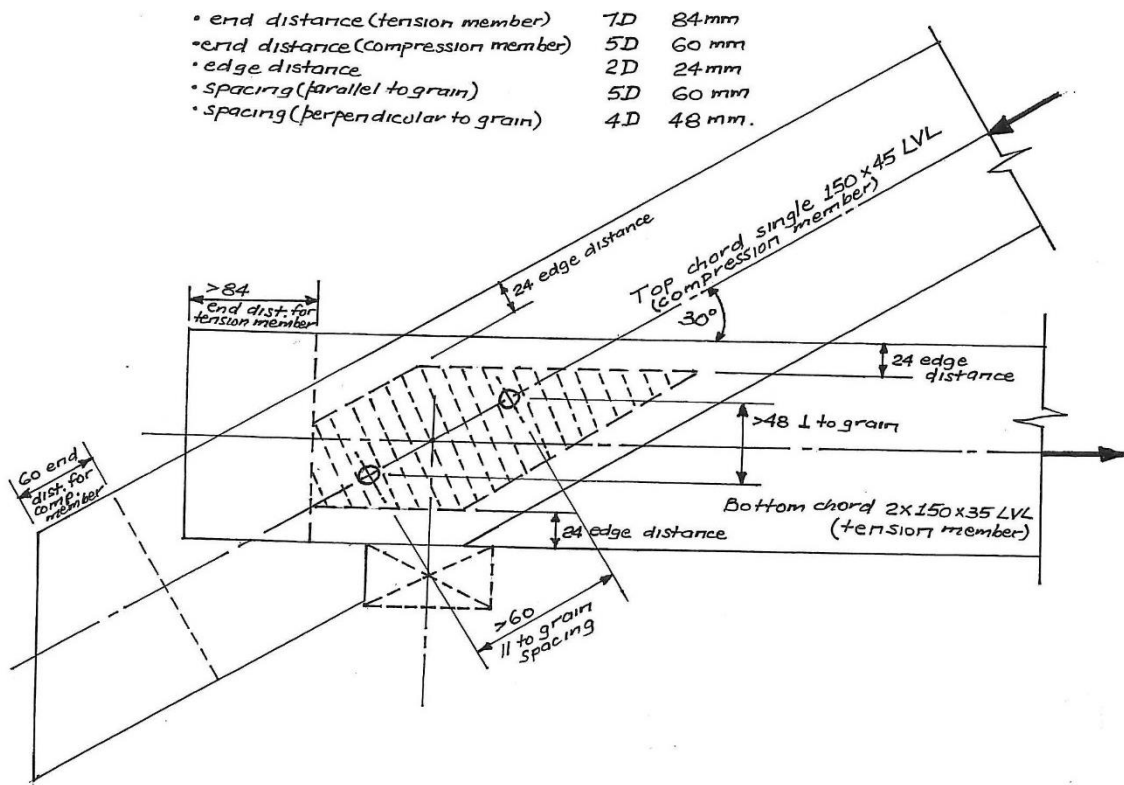
Figure 13-13: Edge, End and Bolt Spacings

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 (Cl. 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;

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- coach screw is fitted with a washer;
- for a two-member joint the thinner member must have a thickness (t_t):

$$t_t \qquad \geq 3D_s$$

where:

$$D_s \qquad = \text{shank diameter } (mm);$$

- hole for shank:

$$= (D_s + 1mm) \text{ or } (D_s + 0.1D_s) \text{ whichever is lesser;}$$

  hole for threaded section: $\quad \leq$ core diameter;

  depth of hole: $\quad \geq$ 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.
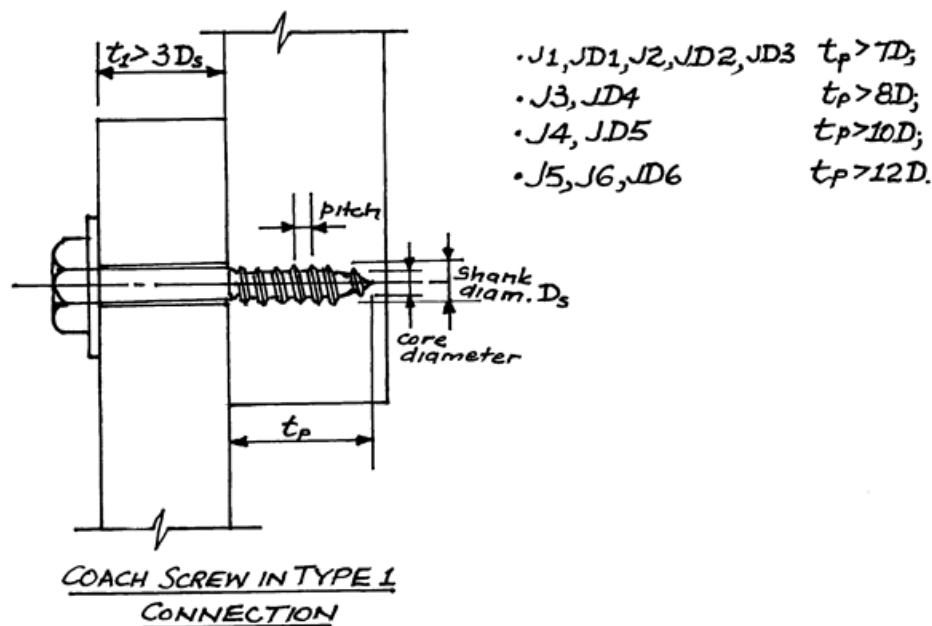
Figure 13-14: Coach Screw Depth of Penetration and Timber Thickness

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 ($N_{d,j}$) for a Type 1 joint containing (n) coach screws to resist the applied load.

For the strength limit state to be satisfied:

$$N_{d,j} \geq N^* \tag{13.22}$$

where:

$$N_{d,j} \quad = \phi k_1 \, k_{13} \, k_{16} \, k_{17} \, n \, Q_{sk} \tag{13.23}$$

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and:

N^* = Design action effect in shear

ϕ = capacity factor

k_1 = duration of load 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 screws resisting applied load

Q_{sk} = 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 ($N_{d,j}$) for a Type 2 joint containing (n) coach screws. For the strength limit state to be satisfied:

$$N_{d,j} \geq N^* \tag{13.24}$$

where: $N_{d,j}$ is the lesser of

$$N_{d,j} = n\,(N_{d,tc}) \tag{13.25}$$

Or

$$N_{d,j} = \phi\,k_{13}\,l_p\,n\,Q_k \tag{13.26}$$

or where crushing under the washer may occur:

$$N_{d,j} = \phi\,k_1\,k_7\,n\,f_{pj}^{'}\,Q_k \tag{13.27}$$

where:

N^* = design action effect due to the application of factored loads causing tension in the joint;

ϕ = capacity factor;

$N_{d,tc}$ = 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)

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$$k_1 \quad = \text{duration of load factor for fasteners;}$$

$$k_7 \quad = \text{length of bearing factor, which for a washer is its diameter or side length;}$$

$$f'_{pj} \quad = \text{characteristic bearing capacity of timber in joints;}$$

$$A_w \quad = \text{effective area of washer for bearing.}$$

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 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

This section 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.
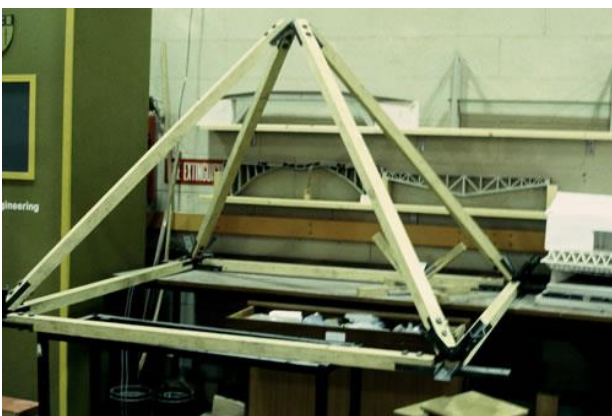
14. Noise Control

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 ($\mu N/m^2$) 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:

$$SPL = 10\ \log_{10}\left(\frac{p}{p_{ref}}\right)^2 \tag{14.1}$$

where:

p = sound pressure;

p_{ref} = reference sound pressure of $20\ \mu Pa$

Sound energy is related to SPL thus:

$$\left(\frac{p}{p_{ref}}\right)^2 = 10^{(spl-10)} \tag{14.2}$$

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.
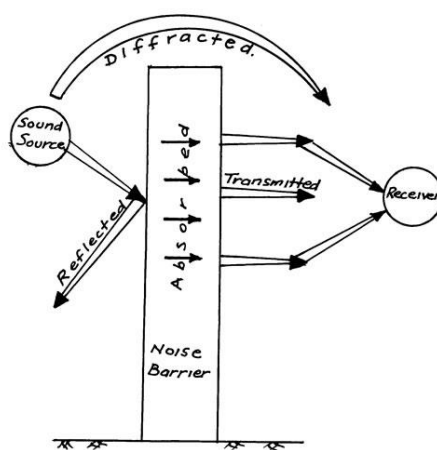
Figure 14-2: Sound Response on Meeting a Barrier

The absorption process of a single leaf panel is a function of:

- panel mass/m²;
- panel bending stiffness (or better its lack thereof);
- frequency of the sound source;

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For a sound source of a given frequency, randomly incident to the panel, the transmission loss is:

$$TL = 20 \, log(Mf) - 47.2 dB \tag{14.3}$$

where:

$$M = \text{mass of panel in } \frac{kg}{m^2};$$
$$f = \text{frequency of source;}$$

The relationship of Equation 14.3 is known as the Mass Law. From Equation 14.3:

$$20 \, log \, 2 = 6 \text{ i.e.}$$

- 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.
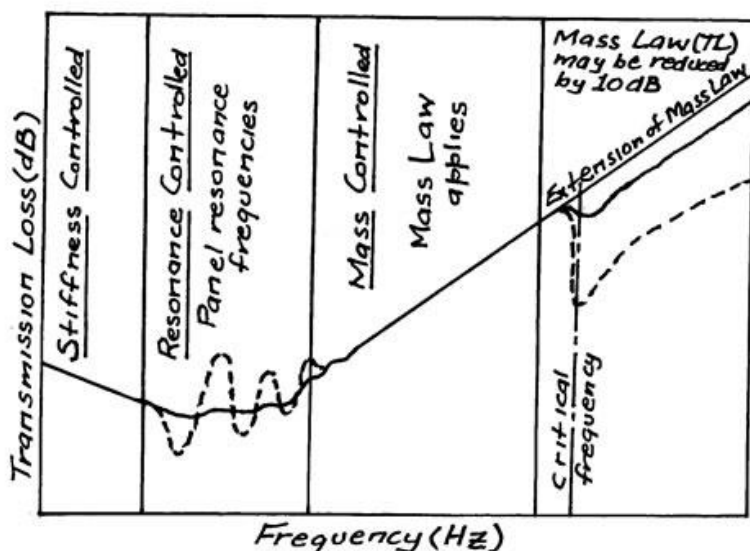
Figure 14-3: Various Regions of Performance for Single Leaf Partition

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_{ntw}). 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.

$$L_{ntw} = 110 - IIC$$

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.

$$90 + 2.1 = 92.1 dB$$

Rounded to the nearest whole dB $\rightarrow 92 dB$

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.
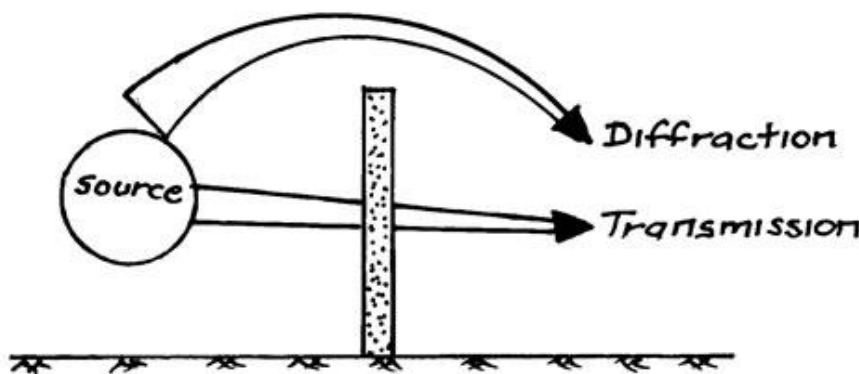
Figure 14-4: Sound Paths

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:

$$70 - 35 = 35 dBa$$

By diffraction:

$$70 - 12 = 58 dBA$$

Adding:

$$58 - 35 = 23 dBA > 10 \ so + 0.$$

Noise received
$$= 58 dBA$$

Timber Wall

Due to transmission

$$70 - 21 = 49 dBA$$

By diffraction:

$$70 - 12 = 58 dBA$$

Adding

$$58 - 49 = 9 dBA \ so + 0.5$$
$$= 58.5 dBA$$

Rounding could go either way. Going down:

Noise received
$$= 58 dBA$$

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 $\frac{mass}{m^2}$ 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
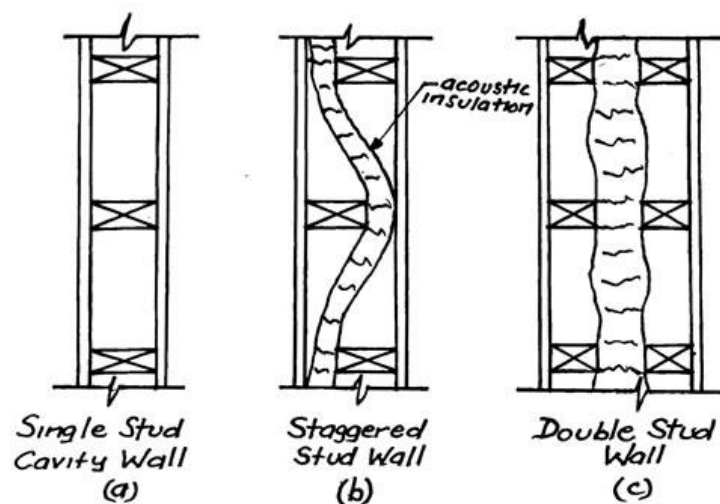
Figure 14-5: Types of Cavity Walls

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.

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.

$$R_h \quad = \frac{p_w}{p_s} \times 100 \tag{15.1}$$

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 considered 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.
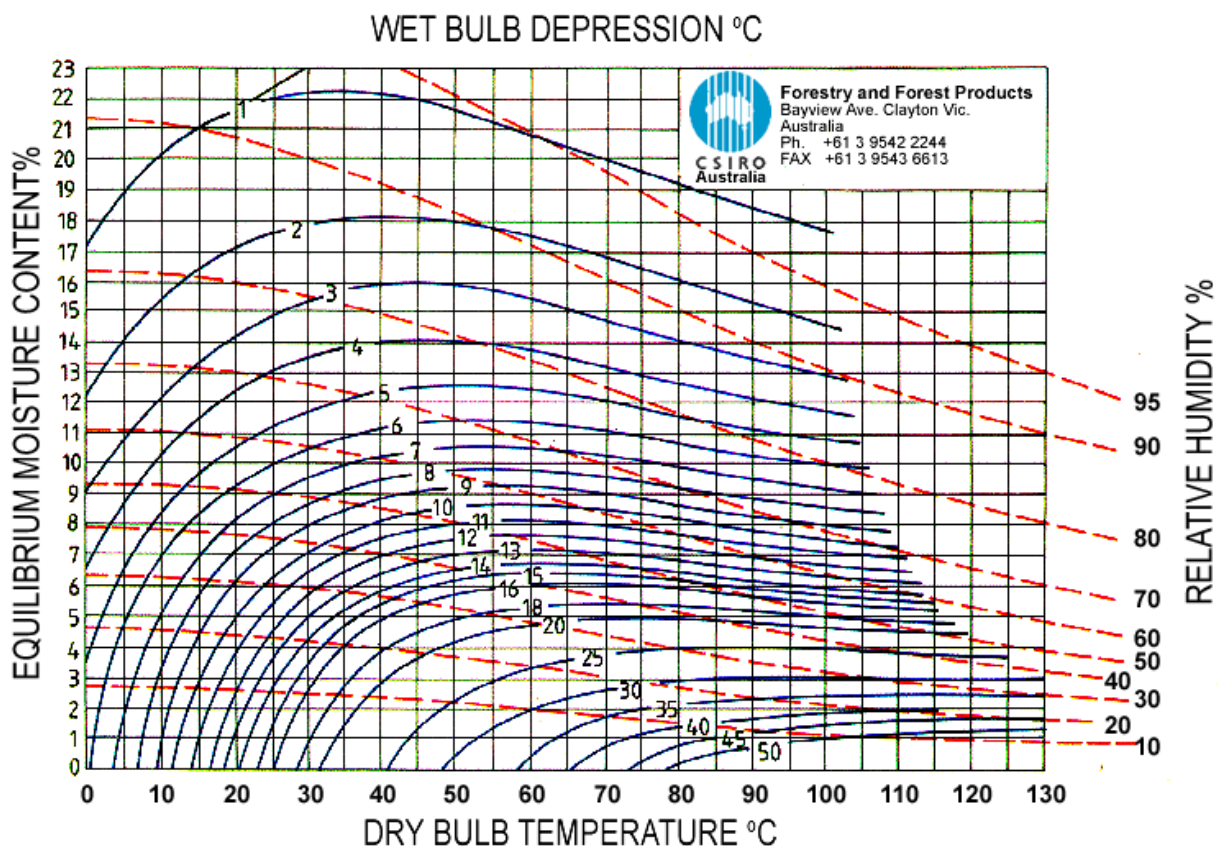
Figure 15-1: Equilibrium Moisture Content of Wood as a function of Dry Bulb Temperature, Wet Bulb Depression and Relative Humidity Thermal Transmission

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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 $\frac{Watt}{m\,°C}$ where:

$$1\ watt = 1\ \frac{Joule}{second}\ \text{or}\ 1\ \frac{Newton\ metre}{second}$$

Unit thermal conductivity (k) of softwood timbers at 12% moisture content is in the range $0.11\ to\ 0.18\ \frac{W}{m°C}$ 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.

$$R\quad = \frac{1}{k}\ \frac{m\,°C}{W} \tag{15.2}$$

Thermal conductivity and thermal resistivity refer to thermal properties of homogeneous materials of uniform composition and specifically relate to a thickness of 1 m 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:

$$r_i\quad = \frac{material\ thickness\ (metres)}{unit\ thermal\ conductivity} = \frac{T_i}{k_i}$$

where:

T_i = thickness of the barrier (m)

k_i = unit (1m thick) thermal conductivity $\left(\frac{W}{m\,°C}\right)$

TABLE 15-1 gives a range of thermal resistances for various thicknesses of softwood plywood having an average density of 550 $\frac{kg}{m^3}$.

Thickness	Density	Unit Thermal Conductivity K	Thermal Resistivity $R = \dfrac{1}{k}$	Thermal Resistance $(r)^*$
(mm)	$\left(\dfrac{kg}{m^3}\right)$	$\left(\dfrac{W}{m\,°C}\right)$	$\left(\dfrac{m\,°C}{W}\right)$	$\left(\dfrac{m^2\,°C}{W}\right)$
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:

$$R_t \quad = r_1 + r_2 + r_3 + \dots \tag{15.3}$$

The overall coefficient of heat transfer (U) is given by:

$$U \quad = \frac{1}{R_t}\frac{W}{m^2\,°C} \tag{15.4}$$

TABLE 15-2 lists the thermal conductivity considered to be achieved by some common construction materials.

This table can be found BCA Vol One, J 1.2., Table 2a.

Material description	Material density $\left(\frac{kg}{m^3}\right)$	Thermal conductivity $\left(\frac{W}{m\,K}\right)$
1. Framing		
(a) Steel	7850	47.5
(b) Timber – kiln dried hardwood (across the grain)	677	0.16
(c) Timber – Radiata pine (across the grain)	506	0.10
2. Roof Cladding		
(a) Aluminium sheeting	2680	210
(b) Concrete or terra cotta tiles	1922	0.81
(c) Steel sheeting	7850	47.5
3. Wall Cladding		
(a) Aluminium sheeting	2680	210
(b) Autoclaved aerated concrete	350	0.10
	900	0.27
(c) Cement render (1 cement: 4 sand)	1570	0.53
(d) Clay bricks		
(i) Clay brick – 2.75 kg	1430	0.55
(ii) Clay brick – 3.25 kg	1690	0.65
(iii) Clay brick – 3.75 kg	1950	0.78
(e) Concrete blocks		
(i) 190 mm dense or 90 mm dense solid	1100/2200	1.1
(ii) 140 mm dense or 190 mm lightweight	1250/910	0.85
(iii) 90 mm dense hollow or 90 mm lightweight solid	1650/1800	0.75
(a) Aluminium sheeting	2680	210

Table 15-2: Thermal conductivity of typical wall roof/ceiling and floor materials

The BCA provides R-Values considered to be achieved by common forms of wall, roof and ceiling construction in Vol One Part J.1.3 and J.1.5.

15.4. Thermal Transmission – Design Example

FIGURE 15-2 shows a wall configuration consisting of 90 x 45mm timber framing and EWPAA branded 12 mm 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 12 mm 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 25 mm with R values ranging from $0.5 \ to \ 1.27 \left(\frac{m\,°C}{W} \right)$.

Reflective Foil Liners can be either single or double sided and result in a reflective air gap, which for walls has a thermal resistance $\left(r = \frac{T}{k} \right)$ of:

$$20mm \text{ reflective air gap} = 0.58 \left(\frac{m^2\,°C}{W} \right)$$

$$> 20 \ mm \text{ reflective air gap}$$

$$= 0.61 \left(\frac{m^2\,°C}{W} \right) \text{ not required – for information only.}$$
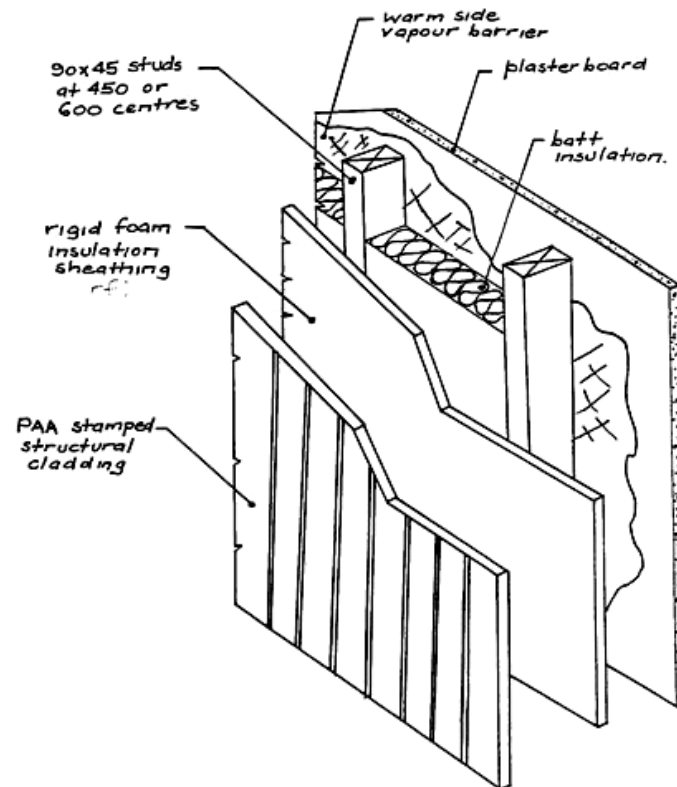
Figure 15-2: Sample Wall

Thermal Transmission – Worked Example

$$R_t = r_{oa} + \frac{T_{ply}}{k_{ply}} + r_{rf} + \frac{T_{batt}}{r_{batt}} + r_{vb} + \frac{T_{pb}}{r_{pb}} + r_{ia}$$

where:

r_{oa} = outdoor surface air resistance;

T_{ply} = thickness of plywood sheathing (m);

r_{rf} = rigid foam resistance;

T_{batt} = batt insulation thickness (m);

r_{vb} = vapour barrier resistance = 0;

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$$T_{pb} \quad = \text{thickness of plasterboard } (m);$$

$$r_{ia} \quad = \text{inside surface air resistance}$$

$$R_t \quad = 0.04 + \frac{0.012}{0.13} + 0.5 + \frac{0.095}{0.05} + 0 + \frac{0.012}{0.22} + 0.12$$

$$= 0.04 + 0.092 + 0.5 + 1.9 + 0 + 0.05 + 0.12 =$$

$$R_t \quad 2.25 \; \frac{m^2 {}^\circ C}{W}$$

$$U \quad = \frac{1}{R_t} = \frac{1}{2.25} = 0.44 \; \frac{W}{m^2 {}^\circ C} \text{ (between studs)} \tag{15.5}$$

If the heat flow through the studs was being considered $r_{stud} = \frac{T_{stud}}{k_{timber}}$, which in this case would be $\frac{0.09}{0.13} =$ 0.69, would have to be included in the calculation for R_t.

The Building Code of Australia (BCA) Vol One – 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 and for more details on availability, the reader is referred to the publication "Insulation Solutions to enhance the Thermal resistance of Suspended Timber Floor System in Australia".

15.5. 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 one's mind.

16.   Resistance to Fire, Decay and Bugs

16.1. Introduction

The main purpose of this Chapter is to provide the reader with an introduction to the design of timber structures to resist the challenges presented by fire in its many variations of occurrence. No doubt the most important aspect of the design process against fire and the possible horrendous aftermath it can impose, is to as near as practicable, design out of the equation its possibility of occurrence.

16.2. Fire & Wood

Three components are required for a fire to occur – fuel, heat and oxygen. Removing any one of these components will prevent fire. This knowledge is essential when considering the prevention and/or containment of fire and can be achieved by 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 at a temperature of approximately 300°C causes decomposition or pyrolysis converting it to gases, tar and charcoal. At temperatures above 300°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.
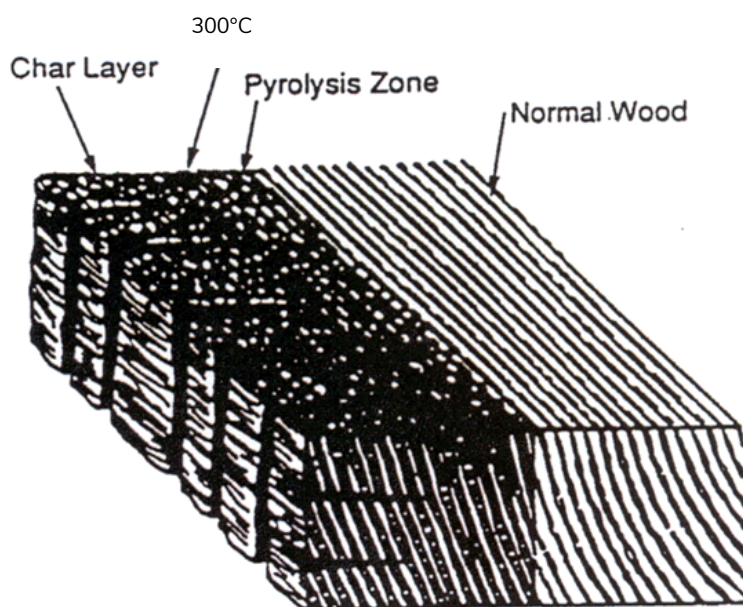
Figure 16-1: Zones of Burning Wood

16.3. 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 NCC 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/NZS 1530 Part 3

Early fire hazard tests to AS/NZS 1530 Part 3 are performed to assess the surface burning characteristics of materials. The data generated through these tests is applicable for sarking type materials, i.e. reflective foil or other flexible membranes for waterproofing, vapour proofing or thermal reflectance in Class (2) to (9) buildings and materials/assemblies used in specific building locations (e.g. fire control rooms). 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/NZS 1530 Part 3 test are:

- tendency to ignite through assigning an ignitability index;
- tendency to propagate flame through assigning a spread of flame index;
- ability to release heat once ignited through assigning a heat evolved index;
- tendency to produce smoke while burning through assigning a smoke developed index

The early fire hazard test indices are scaled according to their performance from best 0 to worst 10. The two most important parameters, i.e. spread of flame index and smoke developed index.

For example, in Class 2 to 9 buildings (NCC Vol. One), sarking type materials must have fire hazard properties thus:

- flammability index = 0 in a fire control room subject to specification E1.8 or a fire isolated exit or fire control room used in the form of an exposed wall or ceiling in other locations < 5.

In the case of other materials:

- spread of flame index $\leq$ 9;
- smoke developed index $\leq$ 8 if spread of flame index is $\leq$ 5.

AS ISO 9239.1

This test applies specifically to floor materials and floor coverings. 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 which must not exceed 750 percent – minutes when a sprinkler system has not been installed.

The critical radiant flux is an indication of the amount of heat flux required to be applied to a material to cause a small flame to ignite it.

TABLE 16-1 outlines the Specification C1.10 of the NCC for Critical Radiant Flux for floor materials and floor coverings.

| Class of building | General | | Fire-isolated exits and fire control rooms |
	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 (i) Class 3 accommodation for the aged; and (ii) class 9b as specified below.	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
Areas other than patient care areas	2.2	1.2	4.5
Class 9b auditorium or audience seating area used mainly for indoor swimming or ice skating; and other sports or multi-purpose functions.	1.2	1.2	2.2
	2.2	1.2	2.2
Class 9c, Resident use areas	-	2.2	4.5
Areas other than resident use areas	-	1.2	4.5

Table 16-1: Critical Radiant Flux (CRF in kW/m^2) of Floor Materials and Floor Coverings

AS 5637.1

AS 5637.1 applies to wall and ceiling linings to assess their fire hazard properties. The main outcome of the Standard 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 growth rate index and an average specific extinction area which are of consequence when no sprinkler system has been installed.

Material Group Number	Description
Group 1	Materials that do not reach flashover when exposed to 100kW for 600 seconds followed by exposure to 300kW for 600 seconds.
Group 2	Materials that do reach flashover following exposure to 300kW within 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 within 600 seconds when exposed to 100kW.
Group 4	Materials that reach flashover within 120 seconds when exposed 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 outlines NCC Specification C1.10 Table 3 in the Deemed-to-Satisfy Provision for wall and ceiling lining materials is given, in terms of Material Group Numbers, for sprinklered and unsprinklered buildings.

Deemed-to-Satisfy

Class of building	Fire-isolated exits and fire control rooms 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 disability, 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; and*
(b) for Class 5 buildings, open plan offices with a minimum floor dimension/floor to ceiling height ratio >5; and
(c) for Class 6 buildings, shops or other building with a minimum floor dimension/floor to ceiling height ratio >5; and
(d) for Class 9a *health-care buildings, patient care areas; and*
(e) for Class 9b theatres and halls, etc. an auditorium; *and*
(f) for Class 9b schools, a classroom; and
(g) for Class 9c *aged care buildings, resident use areas.*

Table 16-3**: Wall and Ceiling Lining Materials (Material Groups Permitted)**

Specific Areas – Example

Consider an open plan office area having floor dimensions of 18m x 20m and a floor to ceiling height of 3m. Since the minimum floor dimension (18m) divided by the ceiling height (3m) is greater than 5 (6 in fact), the area is a specific area. Hence, the ceiling linings in this area would need to comply with the "specific areas" requirements for Class 5 buildings.

NCC Building Classes

The various classes of buildings are described in the Part A3, of the NCC document, and are reproduced here for convenience:

Class	Definition
Class 1a	A single dwelling being – (i) A detached house; or (ii) One of a group of 2 or more attached dwellings, each being a building, separated by a fire -resisting wall, including a row house, terrace house, town house or villa unit.
Class 1b	A boarding house, guest house, hostel or the like – (i) with a total area of all floors not exceeding 300 m^2 measured over the enclosing walls of the Class 1b; and (ii) in which not more than 12 persons would ordinarily be resident; or 4 or more single dwellings located on one allotment and used for short-term holiday accommodation, which are not located above or below another dwelling or another Class of building other than a private garage.
Class 2	A building containing 2 or more sole-occupancy units each being a separate dwelling.
Class 3	A residential building, other than a building of Class 1 or 2, which is a common place of long term or transient living for a number of unrelated persons, including – (a) a boarding house, guest house, hostel, lodging house or backpackers accommodation; or (b) a residential part of a hotel or motel; or (c) a residential part of a school; or (d) accommodation for the aged, children or people with disabilities; or (e) a residential part of a health-care building which accommodates members of staff; or (f) a residential part of a detention centre.
Class 4	A dwelling or building that is class 5, 6, 7,8, or 9 if it is the only dwelling in the building
Class 5	An office building used for professional or commercial purposes, excluding buildings of class 6, 7, 8, or 9.
Class 6	A shop or other public building for the sale of goods by retail or the supply of services direct to the public, including- (a) an eating room, café, restaurant, milk or soft drink bar; or (b) a dining room, bar area that is not an assembly building, shop or kiosk part of a hotel or motel; or (c) a hairdresser's or barber's shop, public laundry, or undertaker's establishment; or (d) market or sale room, show room or service station
Class 7a	A building which is a car park
Class 7b	A building which is for storage, or display of goods or produce for sale by wholesale
Class 8	A laboratory or a building in which a handicraft or process for the production, assembling, altering, repairing, packing, finishing, or cleaning of goods or produce is carried on for trade sale or gain
Class 9a	A building of a public nature that is a health care building, including those parts of a building set aside as a laboratory
Class 9b	An assembly building, including a trade workshop, laboratory or the like in a primary or secondary school, but excluding any other parts of the building that are of another class
Class 9c	An aged care building
Class 10a	A non-habitable building being a private garage, carport, shed, or the like
Class10b	A structure being a fence, mast, antenna, retaining or free-standing wall, swimming pool, or the like
Class10c	A private bushfire shelter

General Information

Design of structures for fire resistance can pose difficulties for the uninitiated, and with the new approach contained in the NCC, initially for the initiated.

It is necessary for the Designer to be familiar with the section on Fire Resistance and Stability given in Part C1 Clause C1.1 of the NCC. It is hoped the contents of this Chapter will aid in the application of the NCC requirements, which in order to satisfy them it is necessary to:

- comply with the Deemed to Satisfy provisions, i.e. sections in the NCC listing ways to satisfy performance criteria;
- formulate an alternative solution that:
  - complies with the performance criteria, or
  - is shown to be at least equivalent to the Deemed to Satisfy provision.

The following lists the relevant sections of the NCC regarding fire resistance.

Reference	Description
Part A3	NCC Building Classes
Section C	Fire Resistance section of the NCC
Section C.1	Deemed to Satisfy provisions regarding fire resistance and stability
Section C1.10	Deemed to Satisfy provisions for Fire Hazard Properties and materials.
Specification C1.10	Deemed to Satisfy requirements for linings, materials and assemblies in Class 2 to 9 buildings
Specification C1.10 - Clause 3 and 4	Deemed to Satisfy requirements for floor linings and floor coverings, walls and ceilings linings respectively

First time designers in the area of fire resistance should also refer to the document Fire Resistance♦

Class 2 and 9 Buildings

Because of the way in which buildings have been categorised, Class 2 to 9 structures demand the most stringent fire resisting characteristics of materials used in their construction.

Such buildings may have a wide variety of unrelated occupants or provide storage for flammable materials of various types.

Therefore, materials used in the construction of floors, walls and ceiling linings for these buildings must have a certain minimum capability regarding resistance to the spread of fire and the development of smoke.

Class 1 and 10 Buildings

Class 1 buildings can generally be classed as detached residential dwellings or two or more attached buildings separated by a fire resisting wall. This is a very different occupancy to that described for Class 2 to 9 buildings and as such also rates a separate volume (Volume Two) by the NCC which cannot be covered in any detail herein.

P2.3.1 from Volume Two of the NCC regarding Protection from Spread of Fire states:

(a)   A Class 1 building must be protected from the spread of fire from:
    (i) another building other than an associated Class 10 building and

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(ii)   the allotment boundary, other than a boundary adjoining a road or public space.

(b)   A Class 10a building must not significantly increase the risk of fire spread between Class 2 and 9 buildings.

Part 3.7.1.5 regarding Construction of External Walls states:

- The intent of the typical construction as shown on NCC Volume Two Fig 3.7.1.3 is to ensure combustible materials (external or internal) are not directly exposed to fire at the junction of the wall and non-combustible roof, eaves lining, guttering and the like.

Bushfire Areas

Since the initial publication of this Manual, Australian Standard AS 3959 Construction of Buildings in Bushfire-prone Areas has been revised and supersedes previous versions. The method of determining the Bushfire Attack Level (BAL) has been revised and provides two methods for its determination. Both procedures are presented in detail and allow the construction details for the various components of a structure to be defined. The BAL is a fundamental parameter, in the event of a bushfire, describing the potential severity of possible exposure to ember attack, radiant heat and direct flame contact.

Having determined the BAL, AS3959 defines the construction requirements for critical external components of the structure and attachments. The NCC also contains "Appendices, Variations and Additions" to construction requirements for the various Classes of Building provided by States and Territories.

The main concern of the NCC is its application to residential buildings i.e. Class 1, Class 10(a), Class 2 and Class 3. Most buildings located in bushfire-prone areas would be single dwelling houses with associated attachments as well as hotel/motel developments. A bushfire-prone area is unlikely to be a sensible choice to site a hospital or aged care facility. Residential buildings constructed in a designated bushfire-prone area must, to the degree necessary, be designed and constructed to reduce the risk of ignition from a bushfire, appropriate to the potential for ignition caused by burning embers, radiant heat or flame generated by a bushfire and intensity of the bushfire attack on the building in accordance with the NCC.

16.4. Plywood and LVL Performance

The most recent data obtained from test results for plywood regarding the NCC specifications for:

- wall and ceiling lining materials;
- flooring materials

are published by Forest and Wood Products Australia (FWPA) – refer to WoodSolutions.com.au (search for fire test reports) for the latest reports. Further information regarding fire performance of timber species can be found in AS1684 series.

LVL is manufactured as a beam type structural element and its performance is governed by its resistance to fire.

16.5. Resistance to Fire

Fire Resistance is the ability of a building component to resist a fully developed fire, while still performing its structural function. Fire resistance levels (FRL) are assigned performance criteria, in minutes, for structural adequacy, integrity and insulation. This important parameter is defined by three numbers, e.g. 60/60/60 for which the:

- first number relates to structural adequacy;
- second number to integrity;

- third number to insulation value

Plywood is quite acceptable as a material used as part of a fire resisting system provided it is combined with other materials so as to meet the fire-resisting requirements. This can be achieved by combining plywood with materials deemed non-combustible such as fibre cement or fire-protective grade plasterboard. The FRL rating is evaluated in a Standard Fire Test as specified in AS 1530.4.

The structural fire resistance level (e.g. 60/-/-) of LVL beam or column components can be calculated in accordance with AS 1720.4 Timber Structures – Fire resistance of structural adequacy of timber members. When establishing the Fire Resistance Level (FRL) of structural untreated wood and wood-based products the charring rate at the surface is very important. As previously described 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.

Glued engineered wood products manufactured using phenol, resorcinol, phenol-resorcinol or poly-phenolic glues are considered equivalent to wood fibre in terms of fire performance (refer AS 1720.4).

16.6. Steps in Establishing the FRL (Structural Adequacy)

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 a reasonably constant rate given in AS1720.4 by:

$$c = 0.4 + \left(\frac{280}{\delta}\right)^2 \qquad (16.1)$$

where:

c = notional charring rate (mm/minute);

δ = timber density $\left(\frac{kg}{m^3}\right)$ at a moisture content of 12%.

The charring rate of a typical softwood having a density of $550 kg/m^3$ is $0.66 mm/minute$.

The effective depth of charring (d_c) for each exposed surface after a period of time (t) is given by:

$$d_c = c\,t + 7.5 \qquad (16.2)$$

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 equation assumes that following a fire a realistic assessment of structural response can be made by neglecting 7.5mm of unburnt wood and assuming the remainder retains its full strength and stiffness. The effective residual section is determined by subtracting d_c from all fire-exposed surfaces of the timber member as shown in FIGURE 16-2.
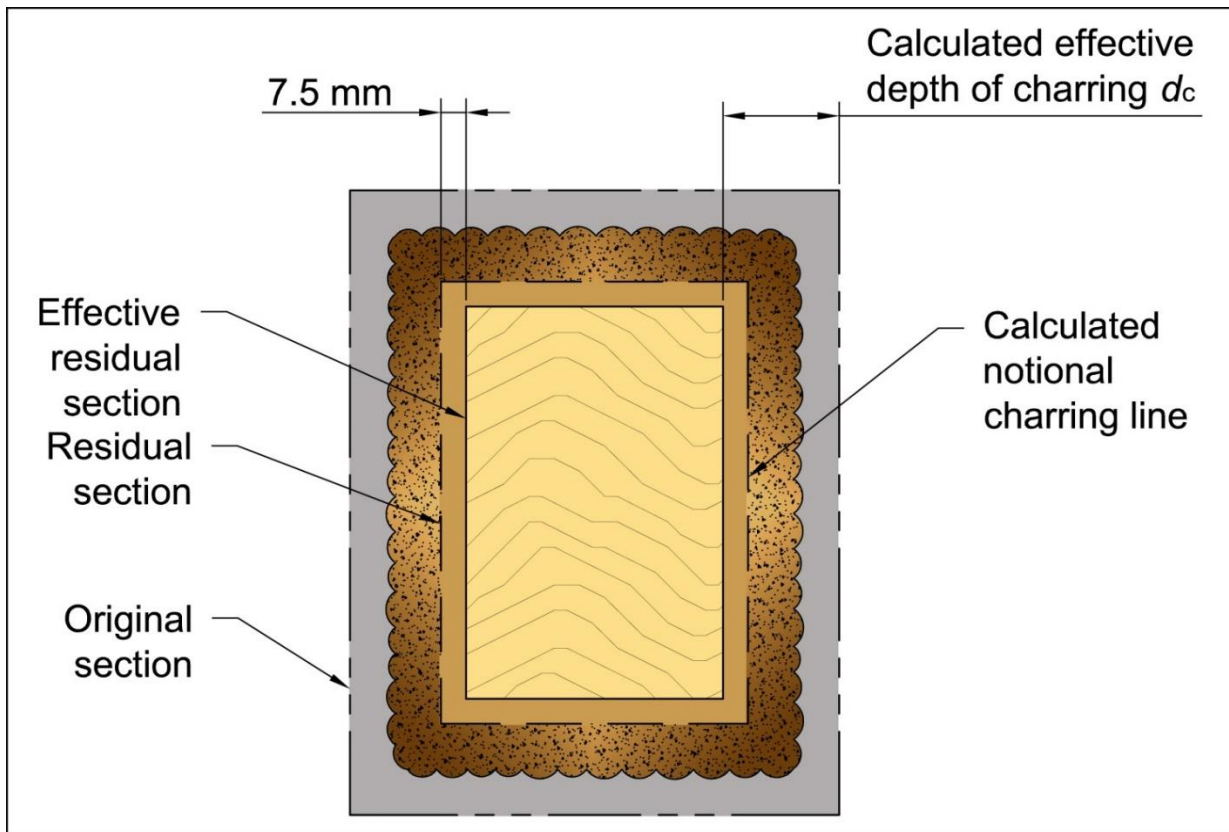
Figure 16-2: Loss of Section Due to Charring

- the fire limit state design loads to be resisted by the residual structural elements/components are determined from the application of AS 1170.0.
- a check of the strength of the effective residual section is done in accordance with the requirements of AS 1720.1. The deflection limits can be;
  o set by the design engineer
  o a maximum of span / 300.

16.7. Other Factors

There are a number of other factors to be considered when assessing the structural adequacy of a member designed to achieve a desired FRL in accordance with AS 1720.4. 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 loads.

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 16-3.
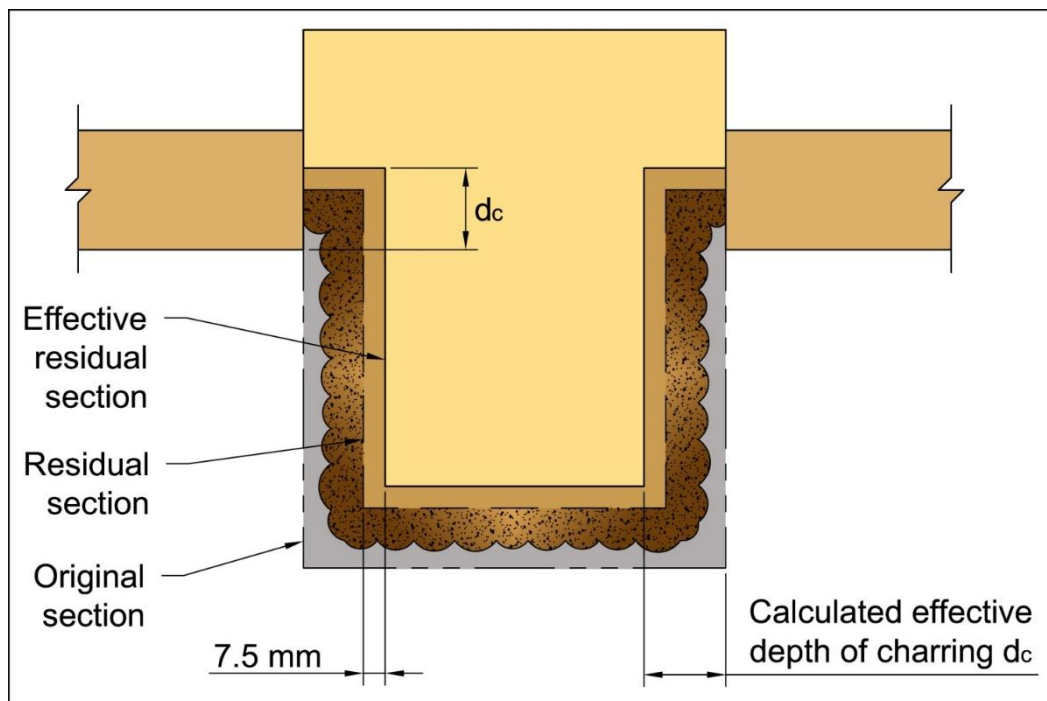
Figure 16-3: Charring at Junction with Fire Proof Barrier

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 fire-resistant insulation, in minutes;

t_i = fire resistance period appropriate to the protective insulating systems, in minutes;

t_M = fire resistance period of the structural timber member

Note: For protected timber c of Equation 16.1 is multiplied by 1.1; T_p of Equation 16.3 is a conservative estimate of the FRP and can be modified if acceptable test data is available, through reference to manufacturers' product catalogues, technical reports and reports on tests performed in accordance with AS 1530.4.

16.8. Fire Protection of Joints with Metal Connectors

There are two possible scenarios in which joints having metal connectors can occur in a fire within a structure. These are as:

1. Unprotected connectors whereby structural adequacy can be established by test or is considered 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 16-4. The resulting holes must be plugged using timber; or
    - cladding which is effective by covering the joint with fire-resistant claddings.
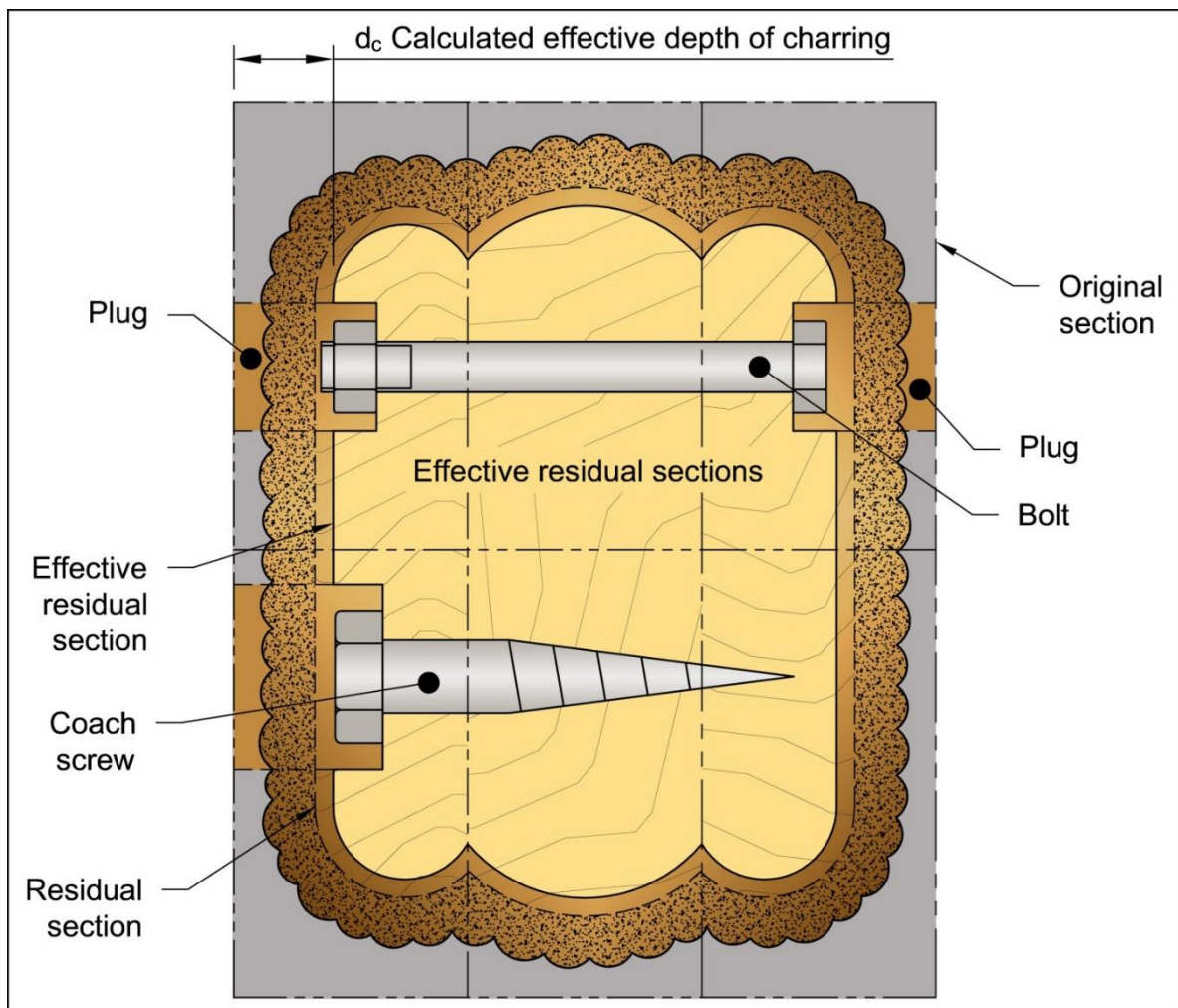
Figure 16-4: Fire Protected Connectors

Closure

The main aim of this revision is to provide the first-time designer of fire-resistant structures with some background information and pointers to aid in the plotting of a path through the process.

Also, it is hoped it provides the experienced practitioner with relevant updated fire-resistant design data regarding plywood and LVL. This should allow their use, with confidence, in a wide range of applications. The ramifications of a fire, be it localised in a kitchen, bedroom, etc. or from a bushfire, can be horrendous. It is, therefore imperative, the designer provides the occupants with every chance of survival.

16.9. Resistance to Decay

The durability of structural laminated wood veneer products is dependent on the durability of the adhesive used to bond the veneers and the durability of the timber veneers themselves.

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.

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 a design 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 design 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 moisture related hazards for which structural plywood and structural LVL durability needs to be considered are:

- decay
- surface moulds
- poor detailing

Decay

Decay or rot is caused by fungi. Decay fungi can cause a significant loss in strength of timber. Decaying of timber will not occur unless conditions are favourable for the fungi to develop and survive. The four required conditions are: a suitable temperature range (ideal 25 to 40°C), moisture content of the timber approximately 20% or higher, the presence of oxygen, and a food source (e.g. starches and sugars in the timber).

Wood which is kept dry with a moisture content below 20% will not be subject to fungal attack. Occasional wetting during the construction phase or while in service, for example due to windblown 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 in this Design Manual (source CSIRO) shows timber moisture content relative to temperature and humidity.

Surface Moulds:

Moulds are a type of fungi whose activities are mainly confined to the wood surface. When exposed to moisture, untreated or unprotected timber surfaces may develop surface moulds. These surface moulds require the moisture content of the timber to be about 20 percent or greater and are more prevalent in warm, humid conditions. Moulds are limited to the surface and can be cleaned off with bleaches or wood cleaners commercially available. Surface moulds have no significant effect on structural performance.

The surface mould becomes inactive when the timber dries out (below 20% moisture content), but will reactivate if the timber is not protected and becomes wet again. Surface moulds can be avoided by keeping the plywood or LVL dry or alternatively surface finishing the plywood with a coating containing mouldicides or fungicides.

Typical example of surface mould

Poor Detailing

If it allows moisture to saturate or become trapped in or on timber will cause untreated timber to decay quickly and will considerably shorten the service life of the timber product. Good detailing includes details that reduce or prevent the timber from and reduce moisture ingress through end grain. Where timber will get wet, good detailing should ensure moisture is shed rapidly and that the timber is able to dry out quickly. If moisture traps exist, preservative treatment to meet the intended service life will usually be required.

16.10. Resistance to Insect Attack

The main insect destroyers of timber are termites and borers.

Termites are not usually a problem with plywood and LVL provided the application does not involve ground contact and good building practices have been implemented in the design and construction stages to restrict termite access. Ongoing inspection and maintenance is essential. Where a termite hazard exists, for example, in applications involving ground contact or termite management systems (e.g. physical barriers) have not been specified, 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 older 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 from H1 to H6 as shown in TABLE 16-4. 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 to be 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 for 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

Note: H6 is only available for Marine Plywood.

Table 16-4: Hazard Class Selection Guide for Preservative Treatments (from AS1604 Specification for preservative treatment, Part 3 & 4)

17. Finishing

17.1. Dry Interior Applications

Structural plywood and LVL used in dry interior applications can be finished in any finishing products suitable for wood surfaces. For plywood, A or B quality faces should be specified as a suitable substrate for high quality interior finishes, stains or paints. An A quality face grade is suitable for clear finishing.

17.2. Exterior Applications

As a general rule all structural plywood and LVL exposed to the weather should be preservative treated against decay and surface finished to prevent surface breakdown due to weathering.

Weathering of unprotected wood surfaces is caused by exposure to sunlight and rain or other moisture sources and is characterised by a change in colour of the exposed wood surface followed by a gradual surface degradation. Rain and sunlight cause wetting and drying of the timber surface resulting in swelling and shrinkage, stressing the wood surface and causing cracks and checks. The leaching and bleaching of the timber surface from weathering eventually results in the timber surface turning grey. In the case of plywood and LVL the small peeler checks produced in the back of the veneer during manufacture become enlarged and break through to the face of the plywood when exposed to continuous wetting and during cycles. This results in surface checking which allows more moisture to penetrate and can eventually cause the surface veneers to breakup. All plywood and LVL surfaces should be protected from weathering to achieve a long service life.

In exterior applications the plywood or LVL surface can be finished by:

- painting
- coating with water repellents
- overlaying with medium density phenolic impregnated papers (plywood only)

Plywoods with an A or B grade face veneer quality are suitable for a high-quality paint or stain finish. Plywood with C or D quality face veneer is not designed to provide a high-quality paint substrate. Plywood cladding products with machined or textured faces are also very suitable for paint or stain finishes.

Where paint systems are required in exterior applications, full acrylic latex paint systems are recommended for structural plywood and LVL. Acrylic latex paint systems are more flexible than oil based or alkyd enamel paint systems and better tolerate any expansion and contraction of the timber substrate due to moisture movement.

Rigid paint systems, including oil based and alkyd enamel paint systems are not recommended for use on plywood or LVL in weather exposed applications. However, they can be used on medium density overlaid plywood because the overlay acts to prevent surface checking of the plywood face veneer.

Edge sealing of plywood and end sealing of LVL is considered good practice to minimise moisture uptake through the end grain and reduce localised swelling and surface checking at the plywood panel edges or LVL ends. The back or unexposed face of plywood should be left unsealed if possible to prevent moisture being trapped within the panel.

Orientation of the plywood or LVL needs to be considered when finishing requirements are being determined. Horizontal surfaces are more exposed to sunlight and moisture ponding than vertical surfaces, and consequently present a greater hazard to paint breakdown and surface checking. The hazard will be increased if the horizontal surface is also subject to traffic.

17.3. Durability and Finishing Applications

Dry interior environments

Structural plywood and LVL used in dry interior environments where the plywood and LVL are installed and kept in the dry condition (moisture content below 15%) will not be subject to the moisture related issues of weathering, surface mould, or decay. No particular finish or treatment will be required for durability provided that in termite susceptible areas, good building practices have been implemented including regular inspection and maintenance.

Exterior exposed above ground

Structural plywood and LVL used in applications exposed to high moisture conditions should be preservative treated to resist decay and insect attack and surface finished to minimise weathering. Good detailing should include sealing of the end grain to minimise moisture ingress. Construction details and installation should allow sufficient space for expansion and contraction of the plywood or LVL due to moisture content changes.

In ground contact with water

Applications in which plywood or LVL are in contact with ground water for extended periods of time provide conditions highly conducive to fungal or insect attack. Preservative treatment appropriate to the hazard level must be specified. Typical applications might include tanks, cooling towers, retaining walls, foundations etc.

Contact with sea water

Salt from sea water will have no adverse effect on plywood or LVL. The water will cause the wood to swell as would exposure to moisture. The main durability issue for plywood or LVL in contact with sea water is marine borers. Preservative treatment to H6 preservative levels will be required where marine borers are present.

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Appendix – Chapter 5

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 computed 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:

1) 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 I5 (AS 1720.1) for standard plywood constructions. In non-standard constructions are the value of (d) shall be taken as the thickness of the dried veneer less 6 percent to allow for compression and sanding losses.

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

3) $\bar{y}$ 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

Face grain parallel to the span

An illustration and section of face grain parallel to the span is shown in FIGURE A5-1
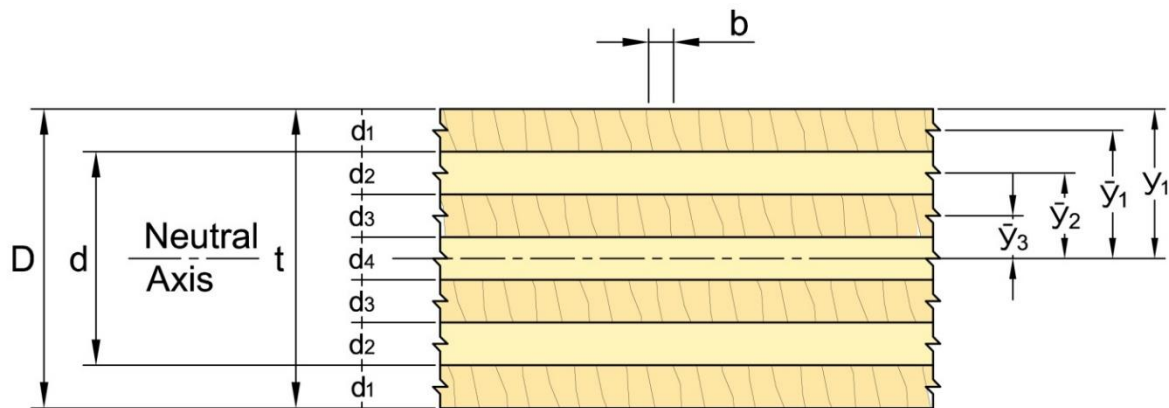
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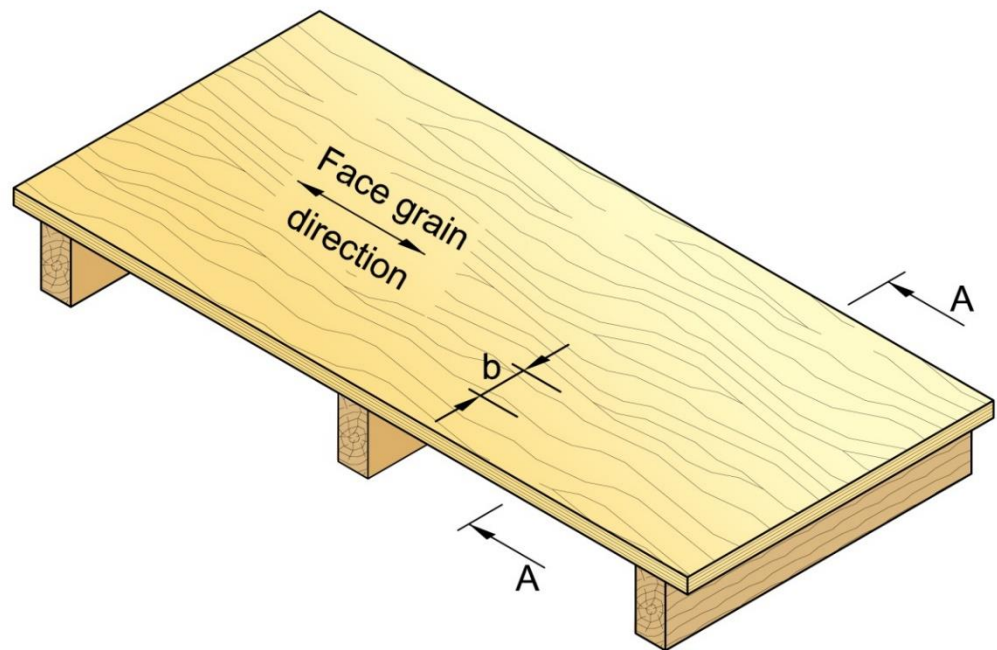
Section properties - face grain parallel to the span

Figure A5-1: Section Properties - Face Grain Parallel to the Span

Using the theory of parallel axes and parallel ply theory, the calculations are as follows:

Second moment of area for calculation of bending rigidity (EI_R) for plywood with face grain parallel to span:

$$I_R = 2\left(\tfrac{1}{12}\, b\, d_1^3 + A_1\, \bar{y}_1^2\right) + 2 \times 0.03 \left(\tfrac{1}{12}\, b\, d_2^3 + A_2\, \bar{y}_2^2\right) + \tfrac{1}{12}\, b\, d_3^3 \qquad \text{A1.5.1}$$

where

$$I_R = \text{second moment of area about the neutral axis for}$$
bending rigidity calculations for a width of $b = 1mm$, in $\frac{mm^4}{mm}$

$$A_1 = d_1 b$$

$$A_2 = d_2 b$$

0.03 = factor applied for plies at right angles to span (for estimation of rigidity only)

a) Section modulus for calculation of bending capacity for plywood with face grain parallel to span:

$$I_C = 2\left(\frac{1}{12} b\, d_1^3 + A_1\, \bar{y}_1^2\right) + \frac{1}{12} b\, d_3^3 \qquad \text{A1.5.2}$$

neglecting cross-directional veneers as required by AS 1720.1

$$Z = \frac{I_C}{y_1} \qquad \text{A1.5.3}$$

where

I_C = second moment of area about the neutral axis for bending capacity calculations for a width $b = 1mm$, in $\frac{mm^4}{mm}$

Z = section modulus for a width $b = 1mm$, in $\frac{mm^3}{mm}$

y_1 = distance from neutral axis (NA) which is at the centre-line of plywood with balanced construction, to the farthest veneer that is parallel to the span (see FIGURE A5-1).

Face grain perpendicular to the span

An illustration and section of face grain perpendicular to the span is shown in FIGURE A5-2

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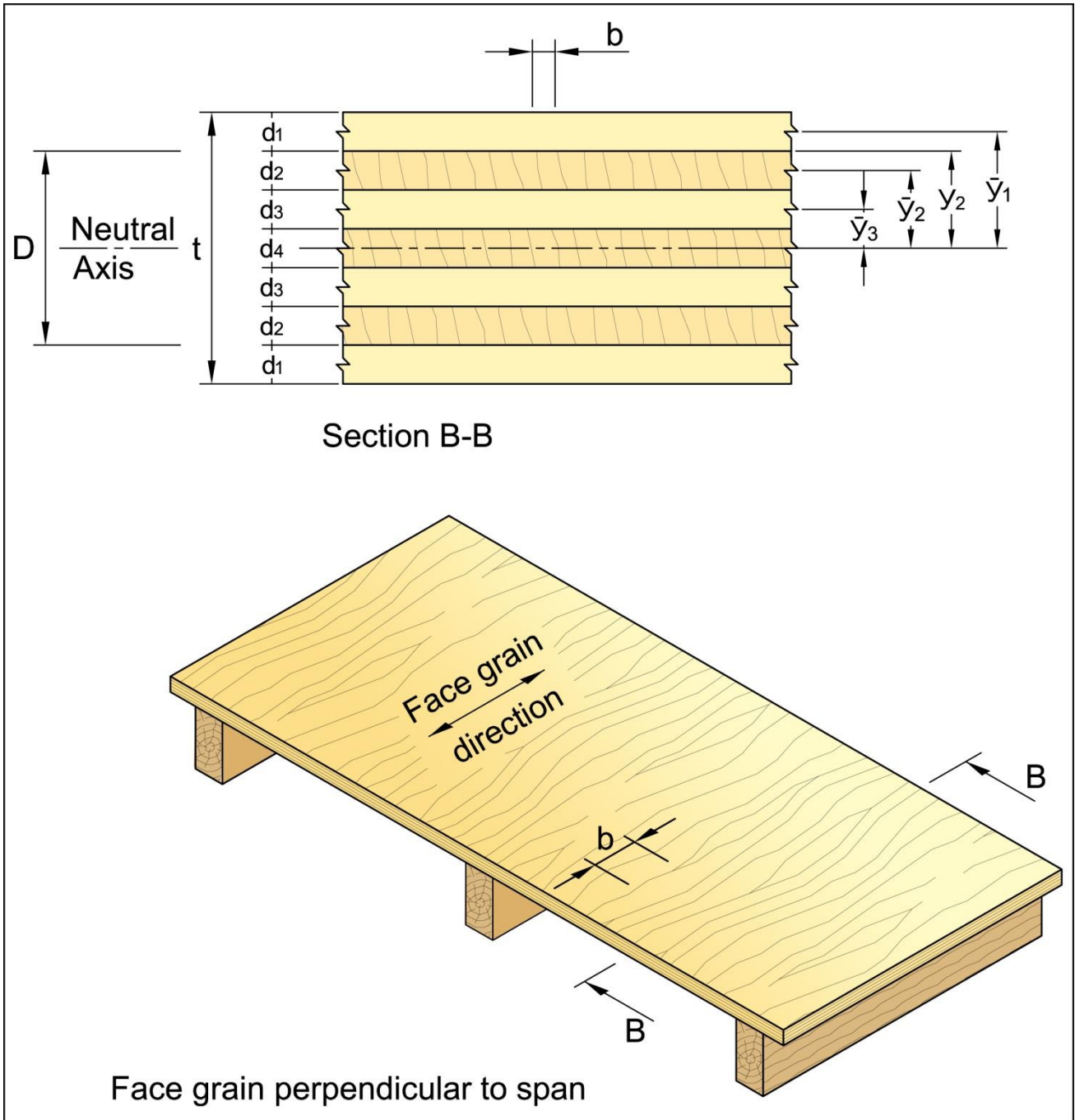
Figure A5-2: Face Grain Perpendicular to Span

The calculations are as follows:

a) Second moment of area for calculation of bending rigidity (EI_R) for plywood with face grain perpendicular to span:

$$I_R = 2 \times 0.03 \left(\frac{1}{12} b \, d_1^3 + A_1 \, \bar{y}_1^2 \right) + 2 \left(\frac{1}{12} b \, d_2^3 + A_2 \, \bar{y}_2^2 \right) + 0.03 \frac{1}{12} b \, d_3^3 \qquad \text{A1.5.4}$$

where:

I_R = second moment of area about the neutral axis for bending rigidity calculations for a width $b = 1mm$, in mm^4/mm

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$$A_1 = d_1 b$$

$$A_2 = d_2 b$$

$$0.03 = \text{factor applied for plies at right angles to span} \\ \text{(for estimation of rigidity only)}$$

b) Section modulus for calculation of bending capacity for plywood with face grain perpendicular to span:

$$I_C = 2\left(\frac{1}{12} b d_2^3 + A_2 \bar{y}_2^2\right) \qquad \text{A1.5.5}$$

$$Z = \frac{I_C}{y_1} \qquad \text{A1.5.3}$$

where:

I_C = second moment of area about the neutral axis for bending capacity calculations for a width $b = 1mm$, in mm^4/mm

Z = section modulus for a width $b = 1mm$, in mm^3/mm

y_2 = distance from neutral to the outside of the farthest veneer parallel to the span (see FIGURE A5-1).

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:

$$I_{NA} = \sum \frac{b D^3}{12} - \frac{b d^3}{12} \qquad \text{A1.5.6}$$

where:

b = width of section (mm);

D = depth of major thickness being considered (mm);

d = depth of section to be removed (mm).

Applying Equation 1.5.6 to the cross-section shown in FIGURE A5-1, for face grain parallel to the span:

$$I_{NA} = \sum \frac{b D^3}{12} - \frac{b d^3}{12} + \frac{b d_3^3}{12}$$

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

$$I_{NA} = \sum \frac{b D^3}{12} - \frac{b d_3^3}{12}$$

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Appendix – Chapter 6

Slenderness Coefficient for Lateral Buckling Under Bending

General

For the general case, and for several useful specific cases, equations for evaluating the slenderness coefficient are given in Appendix E of AS1720.1. 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 coefficients 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 coefficient, denoted by S_1, may be taken to be –

$$S_1 = 1.25 \frac{d}{b} \left(\frac{L_{ay}}{d}\right)^{0.5}$$
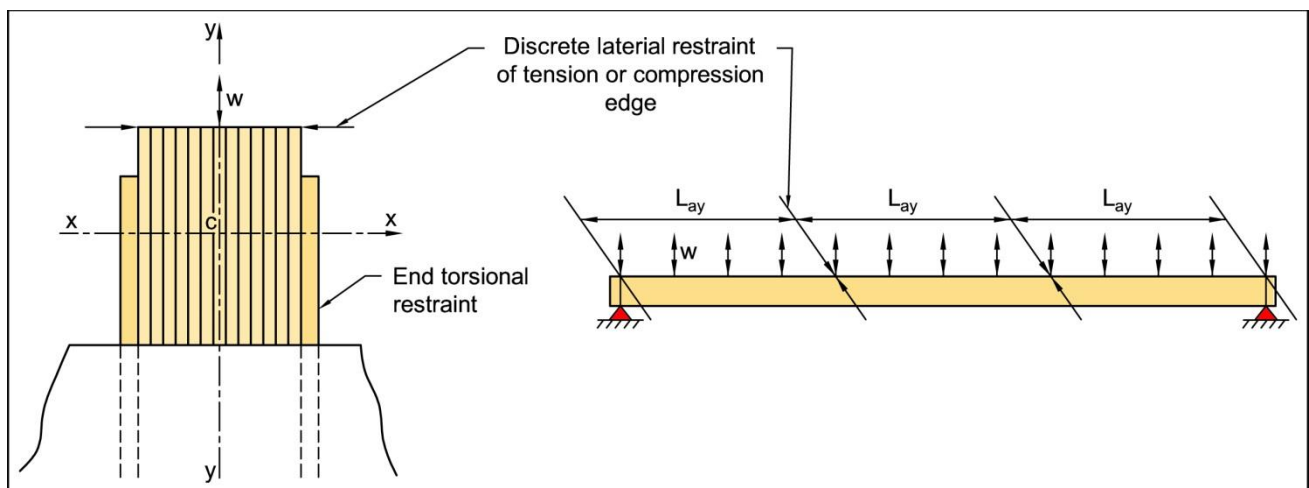
Figure A6-1: Discrete Restraints to the Compression and Tension Edge

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 coefficient, denoted by S_1, may be taken to be –

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

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 –

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

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 coefficient, denoted by S_1, may be taken to be equal to zero.

Continuous restraint along the compression and tension edge
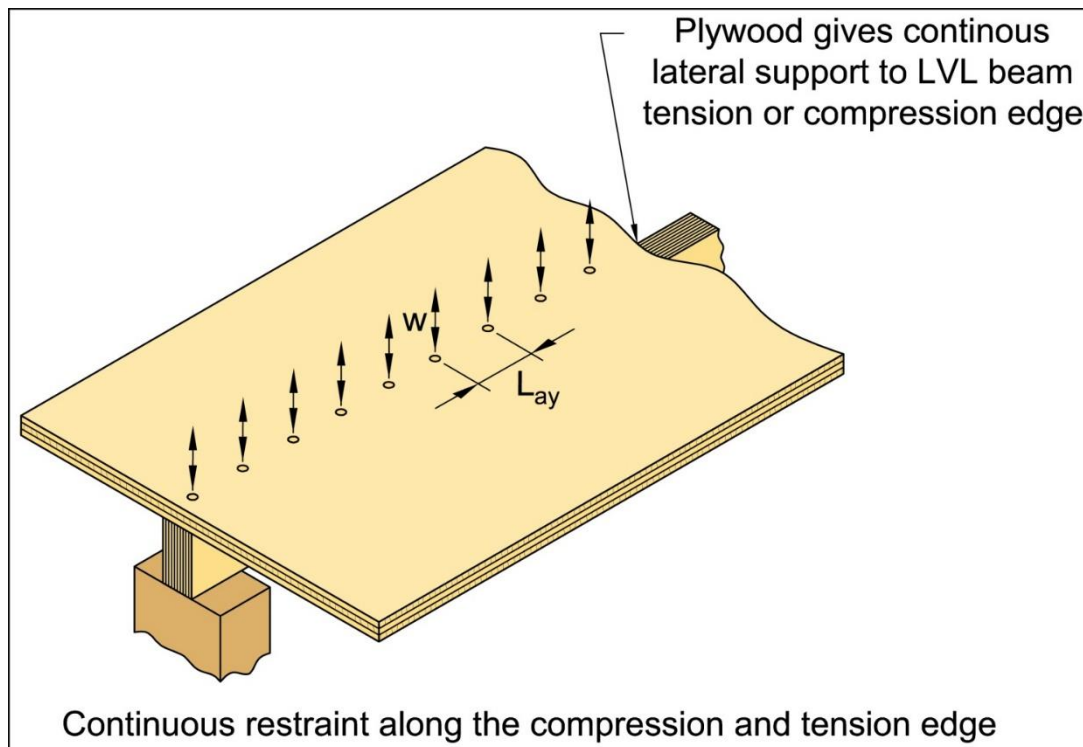
Figure A6-2: Continuous Restraint Along the Compression and Tension Edge

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 coefficient, 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 $L_{a\theta}$ apart, indicated in FIGURE A6-3, to prevent rotation about the beams Z axis, the slenderness coefficient, denoted by S_1, may be taken to be –

$$S_1 = \frac{1.5 \frac{d}{b}}{\left[\left(\frac{\pi d}{L_{a\theta}}\right)^2 + 0.4\right]^{0.5}}$$
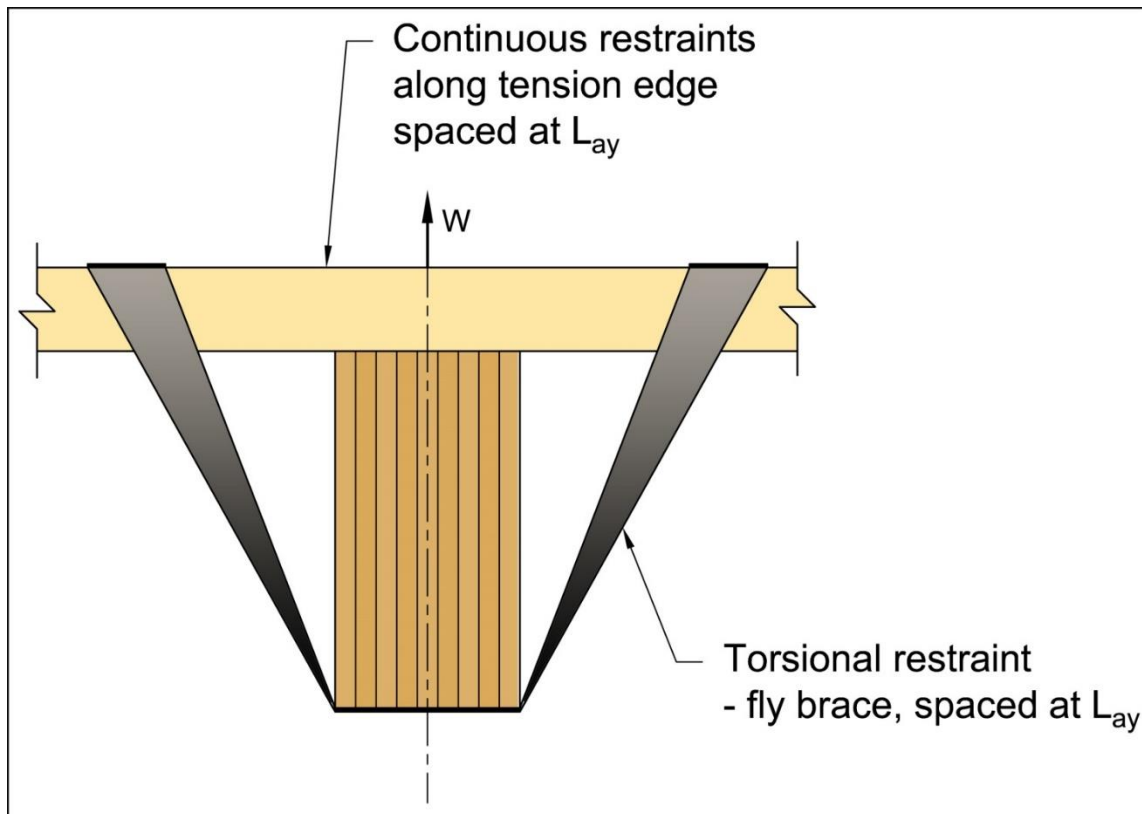
Figure A6-3: Continuous restraint along the tension edge combined with discrete torsional restraints

Beams that bend only about their minor axis

For all cases, the slenderness coefficient, 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 coefficient for lateral buckling under compression

General

For the general case, and for several useful specific cases, equations for evaluating the slenderness coefficient 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 coefficients may be taken as follows:

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

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

and

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

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 coefficient given in Table 3.2, AS1720.1

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

$$S_3 = 0.0$$
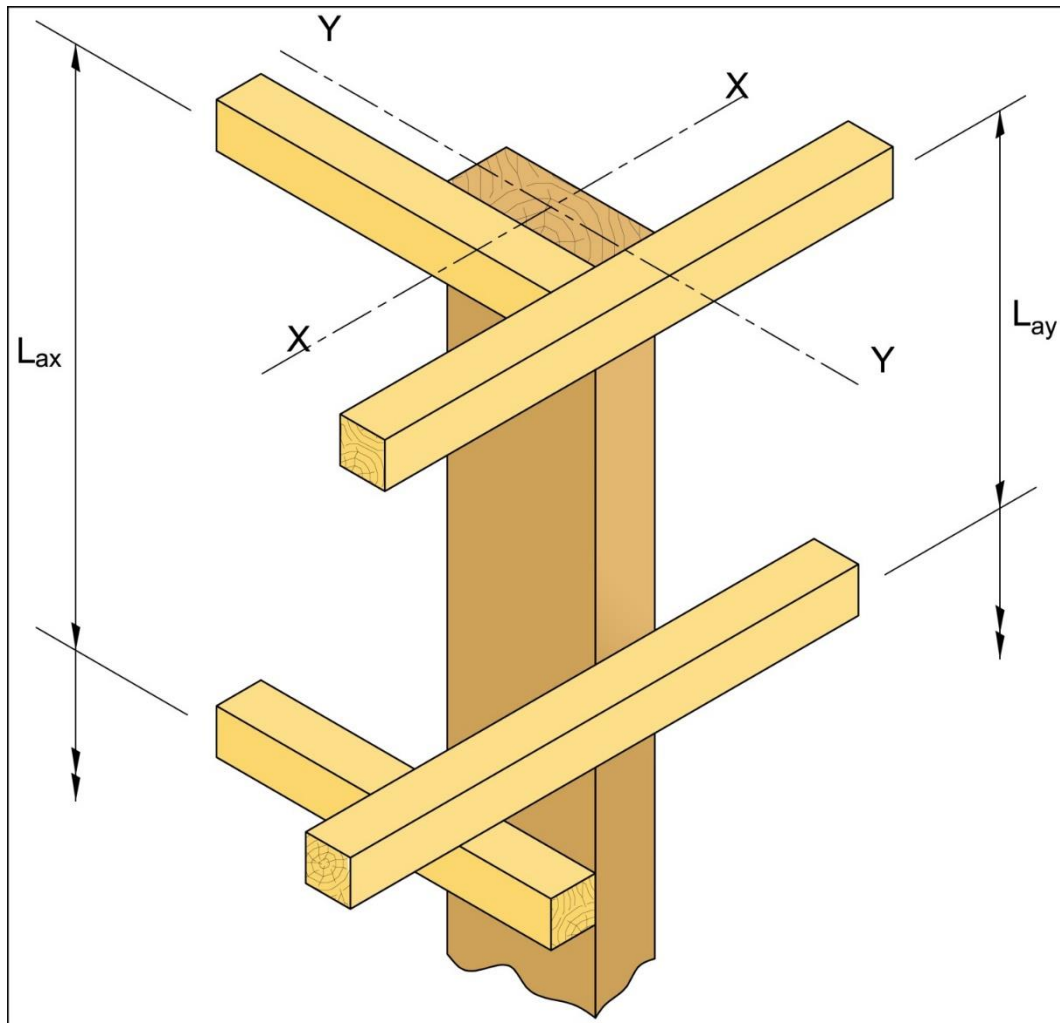
Figure A6-4: Notation for Column Restraints

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 coefficient given in Table 3.2, AS 1720.1 and listed herein in TABLE A6-1.

(b) Slenderness coefficient for buckling about the minor axis. For the case of discrete restraint systems, the slenderness coefficient, denoted by S_4, shall be taken to be the lesser of the following:

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$$S_4 = \frac{L_{ax}}{b}$$

and

$$S_4 = \frac{g_{13}\,L}{b}$$

where

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

g_{13} = the coefficient given in Table 3.2, AS1720.1

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

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

(c) Columns that can bend about both axes.
The design of such columns, described in Clause 3.3.1.2 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 $\rho_b S \le 10$

$k_{12} = 1.0$

(b)   For $10 \le \rho_b S \le 20$

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

(c)   For $\rho_b S \ge 20$

$k_{12} = \dfrac{200}{(\rho_b\,S)^2}$

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	0.85
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 the 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

Appendix – Chapter 8

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. A more thorough examination of lateral torsional buckling, slenderness coefficients and critical elastic buckling moment can be found in Appendix E of AS1720.1 and the EWPAA Design Guide for Plywood Webbed Beams (this archived document is available by contacting the EWPAA). The approach used in this Manual is to approximate the slenderness coefficient for box beams using the formula:

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

where:

L_{ay} = distance between effectively rigid buckling restraints

h_1 = constant from AS1720.1 Table E6

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 shall 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: Slenderness factor h_1

(a) Isometric view of beam

(b) Diagram of bending moment between buckling restraints
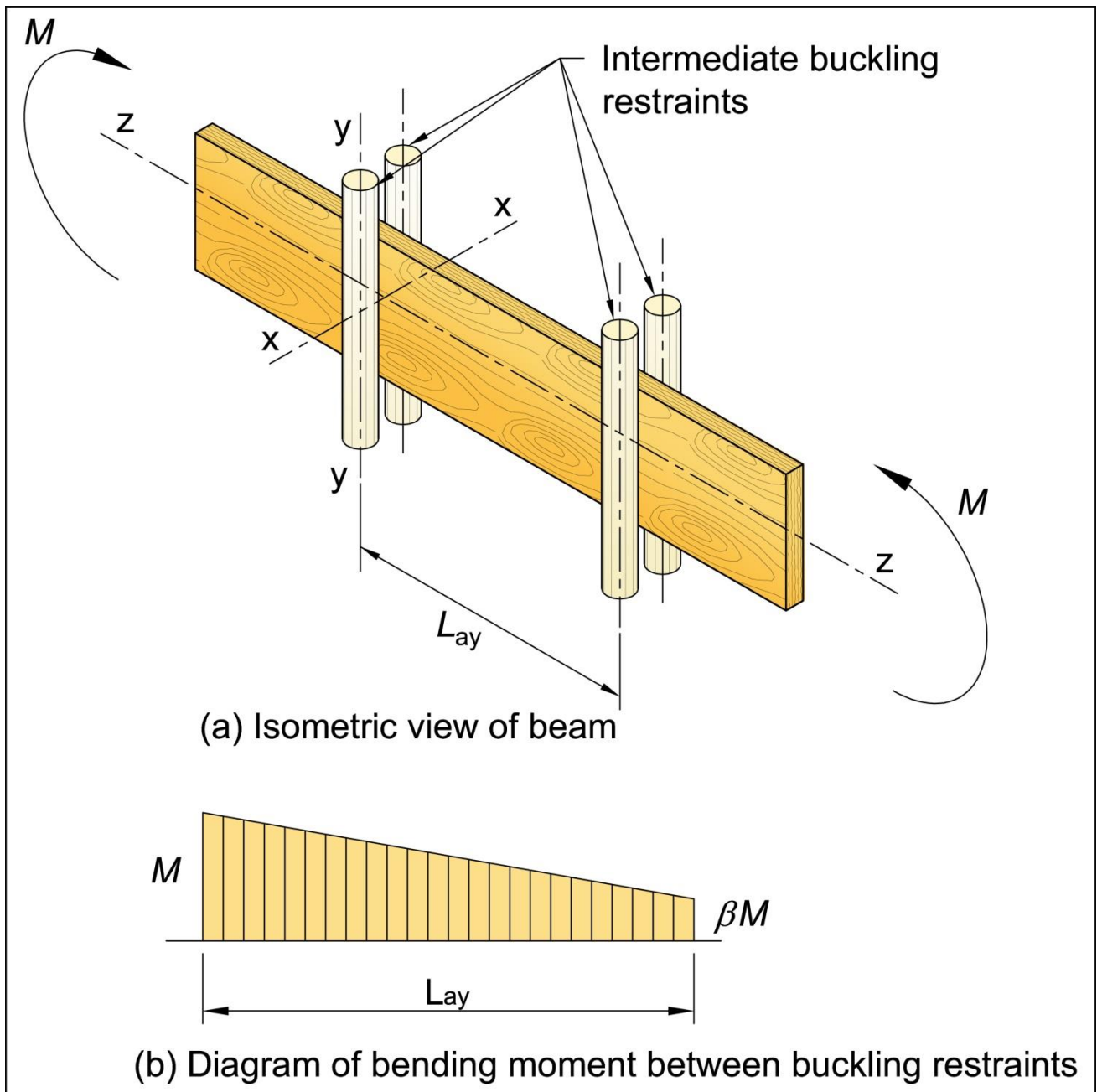
Figure A8-1: Notation for Beams with Intermediate Buckling Restraints

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

$$d \quad = 1.1(d - t_f)$$

$$b_f + t_w = 1.08 \, b_f$$

$$(EI)_x \quad = 1.25 \, E_t \, I_{xf}$$

$$(EI)_y \quad = 1.6 \, E_w \, I_{yw}$$

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

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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 is 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.

Deflections up to 0.5mm can be expressed in the form:

$$P = A \Delta^{0.5}$$

where:

P = lateral load /fastener (N)

$A = h_{32} \sqrt{\dfrac{D^{3.5}\ 10^3}{j_{12}\ 44}}\ Nm^{-0.5}$

h_{32} = stiffness factor from Table C1 of AS 1720.1 – (see TABLE A8-2)

D = Nail diameter in mm

j_{12} = load duration factor from Table C2 of AS 1720.1 – (see TABLE A8-3).

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

For a central concentrated load P:

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

For a UDL of w/unit length:

$$\Delta_{ns} = \frac{d_n L^3}{192} \left(\frac{E_f\ A_f\ s\ w}{(EI)_x\ A}\right)^2$$

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

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:

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

$$= \frac{865 \times 108000}{64} \left(\frac{13200 \times (150 \times 35) \times 50 \times 16200}{31894 \times 10^9 \times 21669} \right)^2$$

$$= 0.01 \, m$$

where:

$$A = 4.767 \, h_{32} D^{1.75}$$

$$= 4.767 \times 750 \times 2.8^{1.75}$$

$$= 20574 \, Nm^{-0.5}$$

For complete coverage of nail slip in plywood webbed box beams the reader is referred to the EWPAA publication *Design Guide for Plywood Webbed Beams*◆. The box beam example presented in the above publication refers to the Sandie Paper presented at the Second Pacific Timber Engineering Conference, IPENZ, Auckland, NZ, August 1989 Vol.1 pp113-118.

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	0.24	0.5
	5 months	0.34	0.7
	Less than 2 weeks	0.65	1.0
Seasoned	More than 3 years	0.24	0.5
	Less than 2 years	0.65	1.0

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 Coefficient, S & Stability Factor k_{12} for Edge Shear Forces

The requirements for strength reductions to allow for buckling of plywood diaphragms is detailed in Appendix I of AS 1720.1. The strength of reductions is stated in terms of a stability factor k₁₂, based on the slenderness coefficient, S of the plywood diaphragm. 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 I, AS 1720.1) 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 e.g. 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 A8-0-4: (a) Span/Depth - (b) Depth/Breadth
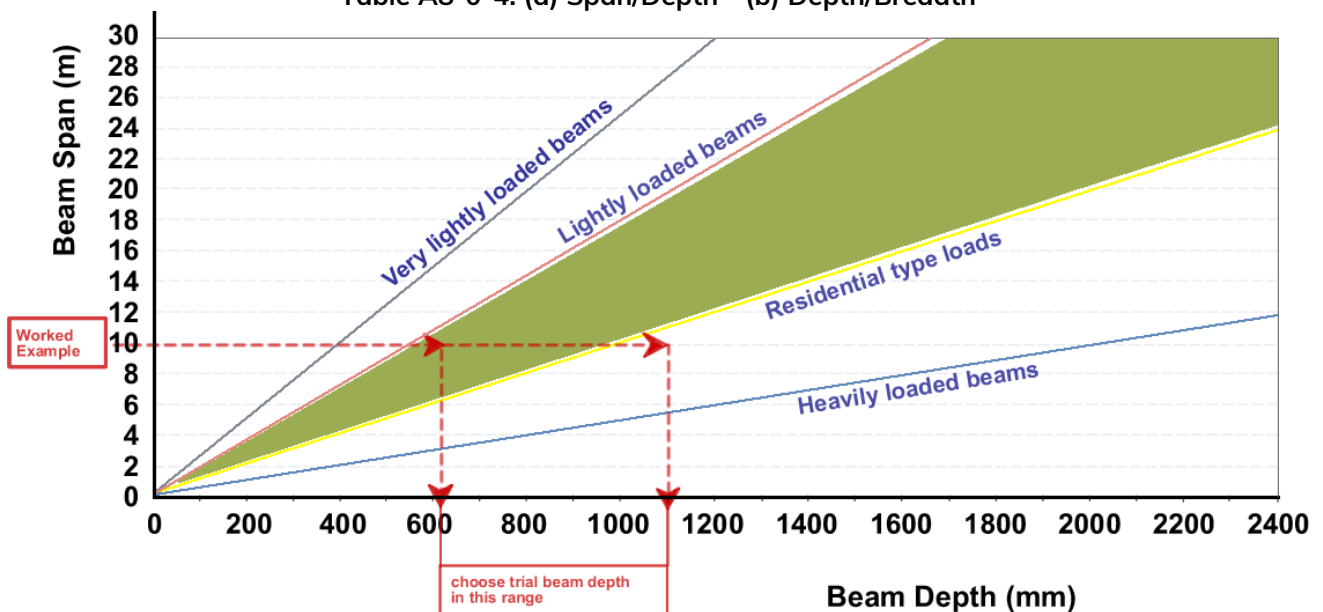
Figure A8-2: Guide for Selecting Initial Beam Depth

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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/m³)	Strength Group	Nominal web thickness, t_w (mm)	Number of veneers	Number of webs	k_{34}
		f'_b	f'_t	f'_s	f'_c	MOE	MOR						
Flanges	LVL	48	25	4.6	38	13200	660	620	JD4	7	3	2	0.67
Webs	F11 Plywood	31	18	5	22	10500	525	550	JD4				

Beam Deflection per unit kN load (mm/kN)

Depth of D (mm)	Flange Width b_f (mm)	Flange Depth t_f (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	11	11	12	13	13	14	14	15	16	16	17	17	18
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	1.36	2.17	3.23	4.61																						
300	63	35		0.10	0.23	0.45	0.78	1.24	1.84	2.63	3.60	4.79	6.22																			
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	0.62	0.93	1.33	1.82	2.42	3.15	4.00	5.00	6.15																
400	63	45		0.04	0.10	0.19	0.34	0.53	0.80	1.13	1.55	2.07	2.69	3.41	4.26	5.24	6.36															
400	83	35		0.04	0.09	0.18	0.31	0.50	0.75	1.06	1.46	1.94	2.52	3.20	4.00	4.92	5.97															
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	3.38	4.16	5.05															
450	63	35			0.09	0.17	0.30	0.47	0.71	1.01	1.38	1.84	2.39	3.03	3.79	4.66	5.65	6.78														
450	63	45			0.08	0.15	0.25	0.40	0.60	0.86	1.18	1.56	2.03	2.58	3.22	3.97	4.81	5.77	6.85													
450	83	35			0.07	0.14	0.24	0.38	0.57	0.81	1.11	1.48	1.92	2.44	3.04	3.74	4.54	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	3.83	4.60	5.46													
600	83	35				0.07	0.12	0.20	0.29	0.42	0.57	0.76	0.98	1.25	1.56	1.92	2.33	2.80	3.32	3.91	4.56	5.27	6.06	6.93								
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	1.96	2.36	2.80	3.29	3.84	4.44	5.11	5.84	6.63	7.49						
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	1.94	2.31	2.71	3.17	3.66	4.21	4.81	5.47	6.18						
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	1.61	1.92	2.25	2.63	3.04	3.50	4.00	4.54	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	1.08	1.26	1.46	1.68	1.92	2.19	2.47	2.78	3.11	3.47	3.86	4.27
900	130	45							0.07	0.10	0.13	0.17	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	2.05	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	2.77	3.09	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	2.84	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	1.17	1.30	1.44
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	0.84	0.95	1.07	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	0.50	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	1.17	1.30
1200	200	63									0.04	0.05	0.06	0.08	0.10	0.12	0.15	0.18	0.22	0.25	0.30	0.34	0.39	0.45	0.51	0.58	0.65	0.73	0.81	0.90	1.00

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	25	4.6	38	13200	660	620	JD4	7	3	2	0.67
Webs	F11 Plywood	31	18	5	22	10500	525	550	JD4				

Depth of section D mm	Flange Width bf mm	Flange Depth tf mm	Beam Deflection per unit load, w = 1kN/m (mm/kN/m) — Beam Span (m)																											
			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	11	11	12	13	13	14	14	15	16	16	17	17	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	15.5	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	26.2	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	16.6	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	35.8														
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	33.9	43.2	54.3												
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	34.7	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	36.8												
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	17.8	22.4	27.8	34.2	41.5	50.0	59.8	70.9	83.4					
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	23.4	28.8	35.0	42.1	50.3	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	19.3	23.7	28.9	34.8	41.5	49.2	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	48.1					
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	36.4	41.9	48.0
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	11.5	13.8	16.4	19.3	22.5	26.2	30.3	34.9	39.9
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	6.1	7.5	9.0	10.8	12.7	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	5.5	6.9	8.5	10.3	12.4	14.8	17.5	20.6	24.2	28.1	32.5	37.4	42.8
900	150	45							0.2	0.3	0.4	0.6	0.9	1.2	1.7	2.2	2.9	3.7	4.6	5.7	7.0	8.5	10.2	12.2	14.5	17.1	20.0	23.2	26.9	30.9	35.4
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	13.2	15.5	18.0	20.8	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	3.3	4.1	5.0	6.0	7.2	8.5	10.0	11.7	13.6	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	16.2
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	6.9	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									0.1	0.2	0.3	0.4	0.5	0.7	0.9	1.2	1.5	1.8	2.2	2.7	3.2	3.9	4.6	5.4	6.3	7.4	8.5	9.8	11.2

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

Box Beam component	Material	MOE MPa	MOR MPa	f'_b MPa	f'_t MPa	f'_s MPa	f'_c MPa	Density kg/m³	Strength group	Nom Thick mm	No of Veneers	No of webs	k_{34}
Flanges	LVL	13200	660	48	25	4.6	38	620	JD4				
Webs	F11 plywood	10500	525	31	18	5	22	550	JD4	7	3	2	0.67

Depth of section D mm	Width b_f mm	Depth t_f mm	A_f (mm²)	Area web A_w (mm²)	Shear Area A_s (mm²)	d/B ratio	Self weight kg/m	Volume m³/m	I_{xf} x10⁶ mm⁴	I_{xw} x10⁶ mm⁴	$(EI)_x$ x10⁹ Nmm²	$(EI)_{x(n=1)}$ x10⁹ Nmm²	Q_{xf} x10³ mm³	E_fQ_{xf} x10⁹ Nmm	Q_{xw} x10³ mm³	E_wQ_{xw} x10⁸ Nmm	$(EQ)_{x(n=1)}$ x10⁸ Nmm	I_{yf} x10⁶ mm⁴	I_{yw} x10⁶ mm⁴	$(EI)_y$ x10⁹ Nmm²	Tension flange kN/m	Compression flange kN/m	Max kN	At web splice* kN	$E_fQ_{xf}/(EI)_x$ x10⁻³ mm⁻¹
225	63	35	4410	3150	2660	2.9	4.5		40	9	625	578	209	2.8	59	3.4	3.1	1.5	2.6	46.4	11	16	12	6	4.43
225	63	45	5670	3150	2520	2.9	5.2		47	9	712	666	255	3.4	59	4.0	3.7	1.9	2.6	51.9	12	18	11	6	4.73
300	63	35	4410	4200	3710	3.9	5.0		78	21	1250	1139	292	3.9	106	5.0	4.4	1.5	3.4	55.4	16	24	16	8	3.09
300	63	45	5670	4200	3570	3.9	5.8		93	21	1451	1340	361	4.8	106	5.3	5.3	1.9	3.4	60.9	18	28	16	8	3.29
400	63	35	4410	5600	5110	5.2	5.8		147	50	2470	2207	402	5.3	188	7.3	6.3	1.5	4.6	67.5	23	36	21	11	2.15
400	63	45	5670	5600	4970	5.2	6.6		180	50	2896	2633	503	6.6	188	8.6	7.6	1.9	4.6	73.0	27	42	21	11	2.29
400	83	35	5810	5600	5110	4.1	6.7		194	50	3087	2825	530	7.0	188	9.0	8.0	3.3	7.6	123.8	29	44	22	11	2.27
400	83	45	7470	5600	4970	4.1	7.7		237	50	3649	3386	663	8.8	188	10.7	9.7	4.3	7.6	136.4	35	53	21	11	2.40
450	63	35	4410	6300	5810	5.8	6.2		190	71	3260	2886	458	6.0	237	8.5	7.3	1.5	5.2	73.5	27	42	24	12	1.85
450	63	45	5670	6300	5670	5.8	7.0		233	71	3830	3456	574	7.6	237	10.1	8.8	1.9	5.2	79.0	32	49	24	12	1.98
450	83	35	5810	6300	5810	4.6	7.1		251	71	4058	3684	603	8.0	237	10.4	9.2	3.3	8.5	133.8	34	52	24	13	1.96
450	83	45	7470	6300	5670	4.6	8.1		308	71	4808	4434	756	10.0	237	12.5	11.2	4.3	8.5	146.4	40	62	24	12	2.08
600	83	35	5810	8400	7910	6.2	8.2		464	169	7901	7015	821	10.8	422	15.3	13.0	3.3	11.4	163.7	50	76	33	17	1.37
600	83	45	7470	8400	7770	6.2	9.3		576	169	9383	8496	1036	13.7	422	18.1	15.9	4.3	11.4	176.3	59	90	33	17	1.46
600	130	35	9100	8400	7910	4.2	10.3		727	169	11371	10485	1285	17.0	422	21.4	19.2	12.8	26.4	446.5	69	109	33	17	1.49
600	130	45	11700	8400	7770	4.2	11.9		903	169	13692	12805	1623	21.4	422	26.5	23.6	16.5	26.4	494.8	83	131	33	17	1.57
900	130	35	9100	12600	12110	6.3	12.6		1703	570	28465	25473	1968	26.0	950	35.9	31.0	12.8	39.6	585.1	115	182	50	26	0.91
900	130	45	11700	12600	11970	6.3	14.2		2140	570	34234	31243	2501	33.0	950	43.0	38.0	16.5	39.6	633.4	138	219	50	26	0.96
900	130	63	16380	12600	11718	6.3	17.1		2874	570	43923	40932	3428	45.2	950	55.2	50.2	23.1	39.6	720.4	177	281	50	26	1.03
900	150	35	10500	12600	12110	5.5	13.4		1965	570	31923	28932	2271	30.0	950	39.9	35.0	19.7	52.0	806.1	126	204	50	26	0.94
900	150	45	13500	12600	11970	5.5	15.3		2469	570	38581	35589	2886	38.1	950	48.1	43.1	25.3	52.0	880.4	152	247	51	26	0.99
900	150	63	18900	12600	11718	5.5	18.6		3316	570	49760	46769	3955	52.2	950	62.2	57.2	35.4	52.0	1014.0	196	318	50	26	1.05
1200	130	45	11700	16800	16170	8.3	16.5		3904	1351	65715	58624	3378	44.6	1688	62.3	53.5	16.5	52.8	772.1	199	315	66	35	0.68
1200	130	63	16380	16800	15918	8.3	19.4		5299	1351	84133	77042	4656	61.5	1688	79.2	70.3	23.1	52.8	859.1	254	404	67	35	0.73
1200	150	45	13500	16800	16170	7.3	17.6		4505	1351	73643	66552	3898	51.5	1688	69.2	60.3	25.3	69.4	1062.4	218	353	67	35	0.70
1200	150	63	18900	16800	15918	7.3	21.0		6115	1351	94895	87804	5372	70.9	1688	88.6	79.8	35.4	69.4	1196.1	281	455	67	35	0.75
1200	200	45	18000	16800	16170	5.6	20.4		6006	1351	93464	86372	5198	68.6	1688	86.3	77.5	60.0	120.6	2058.1	266	448	68	35	0.73
1200	200	63	25200	16800	15918	5.6	24.9		8153	1351	121799	114708	7163	94.6	1688	112.3	103.4	84.0	120.6	2374.9	346	584	68	35	0.78

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