Preface

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

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

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 1720.1-1997, Timber Structures and other references will still be required.

Design methodology for the solution of a range of structural problems is presented in a step-by-step format. A worked example is then done which includes Code references. The methodology presented will provide an adequate solution. However, there is no doubt, through the availability of modern technology other more efficient and economical solutions may be implemented. Until complete familiarity with the idiosyncrasies of the material has been attained and the design concepts have been fully digested, the contents of the Manual 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 considered to be of prime importance, and therefore, is the ‘centre of gravity’ of the Manual. If the designer cannot get member connectivity right, irrespective of how well individual elements and components are designed, the structure will be ‘doomed to failure’.

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

Happy and fruitful designing,
Mick McDowall
January 2007
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Acknowledgements

The Engineered Wood Products Association of Australasia wishes to express its thanks and appreciation to –


Ms Leigh M Punton, M.TimberTech, BEng(Civil), Dip. Law, MIE(Aust), some time Construction Engineer, Research Engineer, Market Development Engineer and Consulting Engineer

for their considerable contributions in the preparation of the text and diagrams of this publication and for bringing this publication to its completion.

The writers wish to acknowledge the following:

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

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

- Thanks to Matthew Bird, EWPAA for his expert knowledge, guidance and contribution to the electronic formatting of this document.

- Last, but certainly by no means least from the viewpoint of the first named writer, is his heartfelt thanks to Ms Vicki Roberts, EWPAA. Thanks are offered not only for her expert typing but also for her punctuality in producing draft chapters, patience in correcting them and inputs concerning layout. Vicki’s allegiance to the project ensured it would be completed and within a reasonable timeframe.

This Manual has been produced for the design and construction industry by the Plywood Association of Australasia Ltd t/a Engineered Wood Products Association of Australasia. 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 Plywood 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.

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Part One

Product Production & Properties

Plywood and LVL – The Manufacturing Process

Structural Plywood

Structural Laminated Veneer Lumber (LVL)

Plywood & LVL Physical and Mechanical Properties
1 Plywood & LVL – The Manufacturing Process

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

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

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

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

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

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

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 plywood range from 1 mm to 3.2 mm.

Drying

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

Grading

Plywood

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

LVL

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

Lay Up & Bonding

Plywood

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

LVL

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

Pressing

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

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

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

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

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

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

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

After trimming, sanding and branding, plywood panels undergo a final inspection for face quality.
2 Structural Plywood

2.1 Introduction

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

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

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

2.2 Bond Type

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

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

2.3 Timber Species Used

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

2.4 Stress Grades

A stress grade defines a codified suite of strength and stiffness properties. There are eight possible stress grades for structural plywood listed in AS 1720.1 Timber Structures Code. The stress grades are: F7, F8, F11, F14, F17, F22, F27 and F34. The characteristic strength and stiffness properties for each stress grade are tabulated in the Timber Structures Code AS1720.1-1997 and reproduced in Table 5.1A, Error! Reference source not found. of this manual. The most commonly available stress grades are F8, F11 and F14, higher stress grades F17, F22, F27 and F34 are also available. However availability should be checked before specifying.

2.5 Veneer Quality

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

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

2.6 Specifying Structural Plywood Grades

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

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

TABLE 2.1: Grade Use Guide
Veneer Quality A

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

Veneer Quality S

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

Veneer Quality B

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

Veneer Quality C

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

Veneer Quality D

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

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

2.8 Panel Dimensions

Length and Width

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

(Length x Width)
2400 mm x 1200 mm
2700 mm x 1200 mm

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

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

Thickness

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

Standard Tolerances

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

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

Length and Width: +1.5 mm
Squareness:

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

Straightness of edges:

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

Flatness:

- **Maximum distance between the underside of the sheet and a flat horizontal surface:**
  - **Unloaded sheets:**
    - Sheets up to 7.5 mm thick: 50 mm
    - Sheets over 7.5 mm thick: 30 mm
  - Sheets up to 7.5 mm thick loaded with a 10 kg weight: 0 mm
  - Sheets over 7.5 mm thick loaded with a 15 kg 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:

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

### 2.9 Other Plywood Types

#### Non-Standard Structural Plywoods

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

#### Marine Plywood

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

Interior Plywood (manufactured to AS/NZS 2270 : 2006 Plywood and Blockboard for Interior Use and Exterior Plywood (manufactured to AS/NZS 2271: 2004 Plywood and Blockboard for Exterior Use) are non structural plywoods used in applications where a high quality aesthetic finish is required. Even when bonded with phenolic adhesive, they are not suitable for use in structural applications and must not be used 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. This Standard sets out the minimum requirements for the manufacture, mechanical property characterisation and verification of the structural properties of LVL intended for structural applications, and for which, structural design is performed in accordance with AS 1720.1 Timber Structures Code, Part 1 - Design Methods or NZS 3603 Timber Structures Standard - Code of Practice for Timber Design.

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

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

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

3.3 Timber Species Used

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

3.4 Stress Grade or Structural Properties

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

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

3.6 Standard LVL Dimensions

3.6.1 Length

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

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

3.6.2 Cross-section

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

3.7 Standard Tolerances

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

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>+4 mm, -0 mm.</td>
</tr>
<tr>
<td>Width</td>
<td></td>
</tr>
<tr>
<td>up to 400mm</td>
<td>+2 mm, -0 mm.</td>
</tr>
<tr>
<td>over 400 mm</td>
<td>+5 mm, -0 mm.</td>
</tr>
<tr>
<td>Length</td>
<td>-0 mm</td>
</tr>
<tr>
<td>Straightness</td>
<td></td>
</tr>
<tr>
<td>Spring</td>
<td>1 mm in 1000 mm</td>
</tr>
<tr>
<td>Bow</td>
<td>1 mm in 1000 mm</td>
</tr>
<tr>
<td>Twist</td>
<td>Length (mm) x Width (mm)</td>
</tr>
<tr>
<td></td>
<td>3500 x Thickness (mm)</td>
</tr>
<tr>
<td>Squareness of Section</td>
<td>1 mm in 100 mm</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>8 – 15 %</td>
</tr>
</tbody>
</table>

3.8 Specification

Specifications for structural LVL should include the following information:

<table>
<thead>
<tr>
<th>Specification</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam depth (mm) x thickness (mm), number of beams/length (m)</td>
<td>400 x 35, 30/6.4m</td>
</tr>
<tr>
<td>LVL type and Standard</td>
<td>Structural LVL to AS/NZS 4357</td>
</tr>
<tr>
<td>Manufacturers’ identification mark</td>
<td>Manufacturers brand name</td>
</tr>
<tr>
<td>Glue bond type</td>
<td>A Bond</td>
</tr>
<tr>
<td>EWPAA / JAS-ANZ product certification stamp</td>
<td>EWPAA / JAS-ANZ Product</td>
</tr>
</tbody>
</table>
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.

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 plywood's 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 form ply applications where large areas of structural plywood form ply are subjected to high temperatures and moisture contents at the time of the concrete pour.

<table>
<thead>
<tr>
<th>Plywood Thickness (mm)</th>
<th>Number of Plies</th>
<th>Direction* of Movement</th>
<th>Moisture Content Range %</th>
<th>Average. 5% to Saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>5% - 12%</td>
<td>12% - 17%</td>
</tr>
<tr>
<td>12</td>
<td>5</td>
<td>II ↓</td>
<td>0.016</td>
<td>0.021</td>
</tr>
<tr>
<td>15</td>
<td>5</td>
<td>II ↓</td>
<td>0.016</td>
<td>0.022</td>
</tr>
<tr>
<td>17</td>
<td>7</td>
<td>II ↓</td>
<td>0.017</td>
<td>0.022</td>
</tr>
<tr>
<td>22</td>
<td>9</td>
<td>II ↓</td>
<td>0.017</td>
<td>0.018</td>
</tr>
</tbody>
</table>

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

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

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

<table>
<thead>
<tr>
<th>Index</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ignitability index (0 - 20)</td>
<td>14</td>
</tr>
<tr>
<td>Spread of Flame index (0 - 10)</td>
<td>8</td>
</tr>
<tr>
<td>Heat Evolved index (0 - 10)</td>
<td>9</td>
</tr>
<tr>
<td>Smoke Developed index (0 - 10)</td>
<td>2</td>
</tr>
</tbody>
</table>
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 Error! Reference source not found.. Thermal Expansion: Wood, including LVL and plywood expand upon heating as do practically all solids. The thermal expansion of plywood is quite small. The average co-efficient of thermal expansion of plywood is in the range $4.5 \times 10^{-6}$ to $7 \times 10^{-6}$ mm/mm/${}^\circ$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 $k=0.1154 \text{ W.m}/(\text{m}^2.\text{C})$ for softwood timbers is sufficiently accurate for determining the overall co-efficient of heat transmission (U value) of a construction assembly.

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

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 Error! Reference source not found..

4.5 Acoustical Properties

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

Sound waves in air is energy in motion and may be absorbed or reflected by a surface. Plywood, like other materials will absorb some of the sound energy and reflect the remainder. A material which exhibits perfect absorptivity is rated as 1.0; a perfect reflector of sound would have a co-efficient of sound absorption of 0.0. The acoustic properties of plywood will vary with density, moisture content and surface coatings, however for most practical purposes plywood can be considered a reflector of sound. Relative co-efficients of sound absorption are given in TABLE 4.2. For further information see Error! Reference source not found..

<table>
<thead>
<tr>
<th>Material</th>
<th>Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open Window</td>
<td>1.0</td>
</tr>
<tr>
<td>Brick</td>
<td>0.03</td>
</tr>
<tr>
<td>Window glass</td>
<td>0.03</td>
</tr>
<tr>
<td>Plywood</td>
<td>0.04</td>
</tr>
</tbody>
</table>

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

4.6 Electrical Properties

Plywood and LVL are excellent electrical insulators, provided they are in the dry condition. Resistance falls off considerably with an increase of moisture content. The 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, alkalies, organic chemicals, neutral and acid salts, both hot and cold. Provided the chemical reagent has a pH above 2 and below 10, any weakening effect will be minimal at room temperature.

4.8 Workability and Bending Radii

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

<table>
<thead>
<tr>
<th>Nominal Thickness (mm)</th>
<th>Along face (m)</th>
<th>Across face (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>1.1</td>
<td>0.6</td>
</tr>
<tr>
<td>7</td>
<td>1.8</td>
<td>1.0</td>
</tr>
<tr>
<td>9</td>
<td>2.3</td>
<td>1.3</td>
</tr>
<tr>
<td>12</td>
<td>3.6</td>
<td>2.4</td>
</tr>
<tr>
<td>15</td>
<td>4.6</td>
<td>3.0</td>
</tr>
</tbody>
</table>

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.

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/m³. Eucalypt hardwood plywood density can exceed 900 kg/m³ depending on the timber species used.
Part Two

Plywood & LVL Design Principles, Procedures and Application

Structural Plywood – Design Principles and Procedures
Structural LVL – Design Principles and Procedures
Basic Structural Plywood & LVL Building Components
5 Structural Plywood - Design Principles & Procedures

5.1 Introduction – Principles

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

5.2 Characteristic Strengths and Stiffness

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

<table>
<thead>
<tr>
<th>Stress Grade</th>
<th>Characteristic Strength, MPa</th>
<th>Short duration average modulus of elasticity MPa</th>
<th>Short duration average modulus of rigidity MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bending</td>
<td>Tension</td>
<td>Panel Shear</td>
</tr>
<tr>
<td>F34</td>
<td>100</td>
<td>60</td>
<td>6.8</td>
</tr>
<tr>
<td>F27</td>
<td>80</td>
<td>50</td>
<td>6.8</td>
</tr>
<tr>
<td>F22</td>
<td>65</td>
<td>40</td>
<td>6.8</td>
</tr>
<tr>
<td>F17</td>
<td>50</td>
<td>30</td>
<td>6.8</td>
</tr>
<tr>
<td>F14</td>
<td>40</td>
<td>25</td>
<td>6.1</td>
</tr>
<tr>
<td>F11</td>
<td>35</td>
<td>20</td>
<td>5.3</td>
</tr>
<tr>
<td>F8</td>
<td>25</td>
<td>15</td>
<td>4.7</td>
</tr>
<tr>
<td>F7</td>
<td>20</td>
<td>12</td>
<td>4.2</td>
</tr>
</tbody>
</table>

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

5.3 Section Properties

Parallel Ply Theory

Parallel Ply theory is used to calculate the structural plywood section properties, e.g. Second Moment of Area, \((I)\) and Section Modulus, \((Z)\). Parallel Ply theory accounts for the differing strength and stiffness properties in the length and width directions of the plywood panel which results from the alternating grain direction of individual veneers in a plywood sheet. Parallel Ply theory assumes veneers with grain direction parallel to the span, carry all of the bending from the applied load, to the supports, as shown in FIGURE 5.1. Veneers with grain direction perpendicular to the span are assumed to contribute nothing to strength and only a minor amount (3%) to stiffness.

Methods for determination of \(I\) are given in Appendix 0.
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

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

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.
TABLE 5.2: Load Distribution Widths

<table>
<thead>
<tr>
<th>Width</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>12 – 13</td>
<td>400</td>
</tr>
<tr>
<td>15 – 19</td>
<td>450</td>
</tr>
<tr>
<td>20 – 25</td>
<td>520</td>
</tr>
<tr>
<td>26+</td>
<td>600</td>
</tr>
</tbody>
</table>

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

**Bending Strength and Bending Stiffness for Loading Normal to the Face**

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

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

For evaluation of bending strength TABLE 5.4 provides comparative bending strength \( (f'\sigma Z) \) 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:

\[
A_s = \frac{2}{3} bt
\]

where: \( b \) = load distribution width (refer TABLE 5.2);
and: \( t \) = full thickness of the plywood sheet.

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

\[
A_s = \text{perimeter of loaded area} \times \text{full thickness of the panel}
\]

It should be noted that the shear capacity of structural plywood loaded normal to the face is governed by the “rolling” shear tendency of the plywood cross-bands. Rolling shear is a term used to describe shearing forces which tend to roll the wood fibres across the grain. The reduced shear capacity of plywood loaded normal to the face, due to rolling shear, is accounted for in AS1720.1-1997, by the use of an assembly factor \( g_{19} \) 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](image)

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

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

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

Section properties for structural plywood loaded in plane, for bending, tension, and compressive strength and bending deflection, are based on the depth of the plywood panel and the sum of the thicknesses of the veneers with grain direction orientated in the span or stress direction.
TABLE 5.3: Standard Structural Plywood Constructions, Thickness of Parallel Plies (tp), Second Moment of Area (Ip) and Section Modulus (Zp)

<table>
<thead>
<tr>
<th>Nominal Thickness (mm)</th>
<th>Nominal Mass* (kg/m²)</th>
<th>Identification Code</th>
<th>Nominal Thickness of Individual Piles Through Assembly (mm)</th>
<th>Face grain parallel to span</th>
<th>Face grain perpendicular to span</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Thickness of Parallel Piles (tp)</td>
<td>Second Moment of Area (Ip)</td>
<td>Section Modulus (Zp)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(mm)</td>
<td>mm²/mm</td>
<td>mm²/mm</td>
</tr>
<tr>
<td>4.5</td>
<td>2.7</td>
<td>4.5-15-3</td>
<td>1.5/1.5/1.5</td>
<td>3</td>
<td>7.3</td>
</tr>
<tr>
<td>6</td>
<td>3.6</td>
<td>6-15-3</td>
<td>1.5/0.0/1.5</td>
<td>3</td>
<td>16</td>
</tr>
<tr>
<td>7</td>
<td>4.2</td>
<td>7-24-3</td>
<td>2/4/2.4/2.4</td>
<td>4.8</td>
<td>30</td>
</tr>
<tr>
<td>7.5</td>
<td>4.5</td>
<td>7.5-25-3</td>
<td>2/5.5/2.5</td>
<td>5</td>
<td>34</td>
</tr>
<tr>
<td>9</td>
<td>5.4</td>
<td>9-15-3</td>
<td>1.5/1.5/3.0/1.5/1.5 or 1.5/2.4/1.5/2.4/1.5</td>
<td>6 or 4.5</td>
<td>45</td>
</tr>
<tr>
<td>9</td>
<td>5.4</td>
<td>9-30-03</td>
<td>3/0/3.0/3.0</td>
<td>6</td>
<td>60</td>
</tr>
<tr>
<td>12</td>
<td>7.2</td>
<td>12-15-5</td>
<td>1.5/3.0/0.0/3.0</td>
<td>6</td>
<td>85</td>
</tr>
<tr>
<td>12</td>
<td>7.2</td>
<td>12-24-5</td>
<td>2.4/2.4/2.4/2.4/2.4</td>
<td>7.2</td>
<td>115</td>
</tr>
<tr>
<td>12</td>
<td>7.2</td>
<td>12-24-5</td>
<td>2.4/2.4/2.4/2.4/2.4</td>
<td>7.2</td>
<td>115</td>
</tr>
<tr>
<td>12</td>
<td>7.2</td>
<td>12-24-5</td>
<td>2.4/2.4/2.4/2.4/2.4</td>
<td>7.2</td>
<td>115</td>
</tr>
<tr>
<td>13</td>
<td>7.8</td>
<td>13-24-5</td>
<td>2/4/3.0/2.4</td>
<td>7.8</td>
<td>165</td>
</tr>
<tr>
<td>13</td>
<td>7.8</td>
<td>13-30-5</td>
<td>3/0/2.4/2.4/2.4/2.4</td>
<td>7.8</td>
<td>165</td>
</tr>
<tr>
<td>14</td>
<td>8.4</td>
<td>14-24-5</td>
<td>2/4/3.0/2.4</td>
<td>7.8</td>
<td>160</td>
</tr>
<tr>
<td>14</td>
<td>8.4</td>
<td>14-30-5</td>
<td>3/0/2.4/2.4/2.4/2.4</td>
<td>9</td>
<td>185</td>
</tr>
<tr>
<td>15</td>
<td>9</td>
<td>15-15-7</td>
<td>1.5/2/4/2.4/2.4/2.4/2.4</td>
<td>7.8</td>
<td>170</td>
</tr>
<tr>
<td>15</td>
<td>9</td>
<td>15-24-7</td>
<td>2.4/2.4/1.5/2.4/2.4/2.4/2.4</td>
<td>7.8</td>
<td>205</td>
</tr>
<tr>
<td>15</td>
<td>9</td>
<td>15-30-5</td>
<td>3/0/3.0/3.0/3.0/3.0</td>
<td>9</td>
<td>225</td>
</tr>
<tr>
<td>17</td>
<td>10.2</td>
<td>17-15-7</td>
<td>1.5/3.0/2.4/3.0/2.4/3.0/1.5</td>
<td>7.8</td>
<td>220</td>
</tr>
<tr>
<td>17</td>
<td>10.2</td>
<td>17-24-7</td>
<td>2.4/2.4/2.4/2.4/2.4/2.4</td>
<td>9.6</td>
<td>285</td>
</tr>
<tr>
<td>17</td>
<td>10.5</td>
<td>17-25-7</td>
<td>2.5/2.5/2.5/2.5/2.5/2.5</td>
<td>10</td>
<td>320</td>
</tr>
<tr>
<td>18</td>
<td>10.8</td>
<td>18-15-7</td>
<td>1.5/3.0/0.0/3.0/0.0/1.5</td>
<td>9</td>
<td>270</td>
</tr>
<tr>
<td>18</td>
<td>10.8</td>
<td>18-30-7</td>
<td>3/0/2.4/2.4/2.4/2.4/3.0</td>
<td>10.3</td>
<td>375</td>
</tr>
<tr>
<td>19</td>
<td>11.4</td>
<td>19-24-7</td>
<td>2.4/3.0/2.4/3.0/2.4/3.0/2.4</td>
<td>9.5</td>
<td>380</td>
</tr>
<tr>
<td>19</td>
<td>11.4</td>
<td>19-24-7</td>
<td>2.4/3.0/2.4/3.0/2.4/3.0/2.4</td>
<td>9.5</td>
<td>380</td>
</tr>
<tr>
<td>19</td>
<td>11.4</td>
<td>19-24-7</td>
<td>2.4/3.0/2.4/3.0/2.4/3.0/2.4</td>
<td>9.5</td>
<td>380</td>
</tr>
<tr>
<td>20</td>
<td>11.4</td>
<td>19-24-7</td>
<td>2.4/3.0/2.4/3.0/2.4/3.0/2.4</td>
<td>9.5</td>
<td>380</td>
</tr>
<tr>
<td>21</td>
<td>12.6</td>
<td>21-24-9</td>
<td>2.4/2.4/2.4/2.4/2.4/2.4/2.4</td>
<td>12</td>
<td>565</td>
</tr>
<tr>
<td>21</td>
<td>12.6</td>
<td>21-30-7</td>
<td>3/0/3.0/0.0/3.0/0.0/3.0</td>
<td>12</td>
<td>555</td>
</tr>
<tr>
<td>25</td>
<td>15</td>
<td>25-30-9</td>
<td>3/0/3.0/2.4/3.0/2.4/3.0/2.4/0.0/3.0</td>
<td>15</td>
<td>900</td>
</tr>
<tr>
<td>25</td>
<td>15</td>
<td>25-30-9</td>
<td>3/0/3.0/2.4/2.4/2.4/2.4/2.4/3.0</td>
<td>15</td>
<td>900</td>
</tr>
<tr>
<td>26</td>
<td>15.6</td>
<td>26-24-11</td>
<td>2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4</td>
<td>1.4</td>
<td>990</td>
</tr>
<tr>
<td>27</td>
<td>16.2</td>
<td>27-30-9</td>
<td>3/0/3.0/0.0/3.0/0.0/3.0/3.0</td>
<td>15</td>
<td>1110</td>
</tr>
<tr>
<td>28</td>
<td>18.8</td>
<td>28-15-13</td>
<td>1.5/2.4/2.4/2.4/2.4/1.5/2.4/2.4/2.4/2.4/2.4/1.5</td>
<td>14.1</td>
<td>1070</td>
</tr>
<tr>
<td>28</td>
<td>18.8</td>
<td>28-30-11</td>
<td>3/0/2.4/2.4/2.4/2.4/2.4/2.4/2.4/2.4/3.0</td>
<td>15.5</td>
<td>1210</td>
</tr>
</tbody>
</table>

Notes:
1. The subscript “p” in Ip and Zp denotes plywood loaded normal to the plane of the plywood panel.
2. Mass of plywood is based on a density of 600 kg/m³. This will be appropriate for most pine species of plywood. Eucalypt hardwood plywood will usually be denser.
<table>
<thead>
<tr>
<th>Nominal Thickness (mm)</th>
<th>Face Grain Perpendicular to Span = f’l_z</th>
<th>Face Grain Parallel to Span = f’l_z</th>
<th>Bending Strength Capacity: (N/mm width)</th>
<th>Shear Strength Capacity in Bending = 0.4 x f’c_Au (N/mm width)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>F14 F17 F22 F24</td>
<td>F14 F17 F22 F24</td>
<td>F8 F11 F14 F17</td>
<td>F8 F11 F14 F17</td>
</tr>
<tr>
<td>4.5</td>
<td>25 30 45 60</td>
<td>25 30 45 60</td>
<td>20 25 30 35</td>
<td>20 25 30 35</td>
</tr>
<tr>
<td>6.5</td>
<td>35 40 50 65</td>
<td>35 40 50 65</td>
<td>30 35 40 45</td>
<td>30 35 40 45</td>
</tr>
<tr>
<td>7.5</td>
<td>45 50 60 75</td>
<td>45 50 60 75</td>
<td>40 45 50 55</td>
<td>40 45 50 55</td>
</tr>
<tr>
<td>8.5</td>
<td>55 60 70 80</td>
<td>55 60 70 80</td>
<td>50 55 60 65</td>
<td>50 55 60 65</td>
</tr>
<tr>
<td>9.5</td>
<td>65 70 80 90</td>
<td>65 70 80 90</td>
<td>60 65 70 75</td>
<td>60 65 70 75</td>
</tr>
<tr>
<td>10.5</td>
<td>75 80 90 100</td>
<td>75 80 90 100</td>
<td>80 85 90 95</td>
<td>80 85 90 95</td>
</tr>
</tbody>
</table>

**TABLE 5.4: Limit State Bending and Shear Strength Capacity – Loading Normal to the Plane of the Plywood Panel**
<table>
<thead>
<tr>
<th>Nominal Thickness (mm)</th>
<th>I.D. Code</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>4.5-15-3</td>
</tr>
<tr>
<td>6</td>
<td>6-15-3</td>
</tr>
<tr>
<td>7</td>
<td>7-24-3</td>
</tr>
<tr>
<td>7.5</td>
<td>7.5-25-3</td>
</tr>
<tr>
<td>9</td>
<td>9-15-5</td>
</tr>
<tr>
<td>9.3</td>
<td>9.3-30-3</td>
</tr>
<tr>
<td>12</td>
<td>12-15-5</td>
</tr>
<tr>
<td>12.5</td>
<td>12.5-25-5</td>
</tr>
<tr>
<td>13</td>
<td>13-24-5</td>
</tr>
<tr>
<td>13.3</td>
<td>13.3-30-3</td>
</tr>
<tr>
<td>14</td>
<td>14-24-5</td>
</tr>
<tr>
<td>14.3</td>
<td>14.3-30-3</td>
</tr>
<tr>
<td>15</td>
<td>15-15-7</td>
</tr>
<tr>
<td>15.5</td>
<td>15.5-25-7</td>
</tr>
<tr>
<td>16</td>
<td>16-24-5</td>
</tr>
<tr>
<td>16.5</td>
<td>16.5-30-5</td>
</tr>
<tr>
<td>17</td>
<td>17-24-5</td>
</tr>
<tr>
<td>17.5</td>
<td>17.5-25-7</td>
</tr>
<tr>
<td>18</td>
<td>18-24-5</td>
</tr>
<tr>
<td>18.5</td>
<td>18.5-30-5</td>
</tr>
<tr>
<td>19</td>
<td>19-24-9</td>
</tr>
<tr>
<td>19.5</td>
<td>19.5-30-5</td>
</tr>
<tr>
<td>20</td>
<td>20-30-7</td>
</tr>
<tr>
<td>20.5</td>
<td>20.5-35-7</td>
</tr>
<tr>
<td>21</td>
<td>21-30-7</td>
</tr>
<tr>
<td>21.5</td>
<td>21.5-35-7</td>
</tr>
<tr>
<td>22</td>
<td>22-30-7</td>
</tr>
<tr>
<td>22.5</td>
<td>22.5-35-7</td>
</tr>
<tr>
<td>24</td>
<td>24-32-9</td>
</tr>
<tr>
<td>24.5</td>
<td>24.5-35-9</td>
</tr>
<tr>
<td>25</td>
<td>25-30-9</td>
</tr>
<tr>
<td>25.5</td>
<td>25.5-35-9</td>
</tr>
<tr>
<td>26</td>
<td>26-24-11</td>
</tr>
<tr>
<td>27</td>
<td>27-30-9</td>
</tr>
<tr>
<td>28</td>
<td>28-30-9</td>
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<td>30</td>
<td>30-30-10</td>
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<td>33-30-11</td>
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<td>36</td>
<td>36-24-15</td>
</tr>
<tr>
<td>39</td>
<td>39-30-13</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Face Grain Parallel to Span</th>
<th>EI x 10^5 Nmm²/mm width</th>
<th>Face Grain Perpendicular to Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>F8</td>
<td>F11</td>
<td>F14</td>
</tr>
<tr>
<td>F17</td>
<td>F22</td>
<td>F27</td>
</tr>
<tr>
<td>F34</td>
<td>F8</td>
<td>F11</td>
</tr>
<tr>
<td>F14</td>
<td>F17</td>
<td>F22</td>
</tr>
<tr>
<td>F27</td>
<td>F34</td>
<td>F8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nominal Thickness (mm)</th>
<th>I.D. Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>4.5-15-3</td>
</tr>
<tr>
<td>6</td>
<td>6-15-3</td>
</tr>
<tr>
<td>7</td>
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</tr>
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<td>7.5-25-3</td>
</tr>
<tr>
<td>9</td>
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</tr>
<tr>
<td>9.3</td>
<td>9.3-30-3</td>
</tr>
<tr>
<td>12</td>
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<td>12.5</td>
<td>12.5-25-5</td>
</tr>
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<td>13.3-30-3</td>
</tr>
<tr>
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<td>14.3-30-3</td>
</tr>
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<td>15.5-25-7</td>
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<td>16.5-30-5</td>
</tr>
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<td>17-24-5</td>
</tr>
<tr>
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<td>18.5-30-5</td>
</tr>
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<td>24.5-35-9</td>
</tr>
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<td>25</td>
<td>25-30-9</td>
</tr>
<tr>
<td>25.5</td>
<td>25.5-35-9</td>
</tr>
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<tr>
<td>27</td>
<td>27-30-9</td>
</tr>
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<td>33</td>
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<tr>
<td>39</td>
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</tr>
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</table>
5.6 Structural Plywood - Design Procedures

Limit State Design to AS 1720.1-1997

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

Strength Limit State Capacity

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

\[
\Phi R > S^*
\]

where \( \Phi R \) = design capacity of the plywood member

\( S^* \) = design action effect, eg. bending moment, \( M^* \), shear force, \( V^* \), etc.

and \( \Phi R = \Phi k_{\text{mod}} [f_0', X] \)

where \( \Phi \) = capacity factor

\( k_{\text{mod}} \) = product of relevant modification factors (eg. \( k_1, k_6, k_7, k_{12}, k_{19}, g_{19} \)).

\( f_0' \) = appropriate characteristic strength

\( X \) = geometric section property.

Serviceability Limit States Capacity

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

5.7 Strength & Stiffness limit states design capacities

5.7.1 Loading Normal to the Plane of the Plywood Panel

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

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

<table>
<thead>
<tr>
<th>Strength Limit State</th>
<th>Design Action Effect</th>
<th>Design Capacity</th>
<th>Strength Limit State Satisfied When</th>
<th>AS 1720.1-1997 Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>$M^*p$</td>
<td>$\Phi M_p = \Phi_1 k_1 g_{19} [f'_b Z_p]$</td>
<td>$\Phi M_p &gt; M^*p$</td>
<td>Clause 5.4.2</td>
</tr>
<tr>
<td>Shear</td>
<td>$V^*p$</td>
<td>$\Phi V_p = \Phi k_1 g_{19} [f'_s A_s]$</td>
<td>$\Phi V_p &gt; V^*p$</td>
<td>Clause 5.4.3</td>
</tr>
<tr>
<td>Bearing</td>
<td>$N^*p$</td>
<td>$\Phi N_p = \Phi k_1 k_7 g_{19} [f'_p A_p]$</td>
<td>$\Phi N_p &gt; N^*p$</td>
<td>Clause 5.4.4</td>
</tr>
</tbody>
</table>

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

Serviceability Limit State

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

where:
- $j_2$ = Duration of load stiffness modification factor
- $j_6$ = Moisture condition stiffness modification factor
- $g_{19}$ = Plywood assembly modification factor

5.7.2 Loading in Plane of the Plywood Panel

FIGURE 5.4 shows loading in the plane of the plywood.

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

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>M*i</td>
<td>$\Phi M_i = \Phi k_1 k_{12} k_{19}{g}<em>{19}[f'</em>{bZ} i]$</td>
<td>$\Phi M_i &gt; M*i$</td>
<td>Clause 5.5.2</td>
</tr>
<tr>
<td>Shear</td>
<td>V*i</td>
<td>$\Phi V_i = \Phi k_1 k_{12} k_{19}{g}_{19}[f'sA_s]$</td>
<td>$\Phi V_i &gt; V*i$</td>
<td>Clause 5.5.3</td>
</tr>
<tr>
<td>Tension</td>
<td>N*t</td>
<td>$\Phi N_t = \Phi k_1 k_{19}{g}_{19}[f'tA_t]$</td>
<td>$\Phi N_t &gt; N*t$</td>
<td>Clause 5.5.4</td>
</tr>
<tr>
<td>Compression</td>
<td>N*c</td>
<td>$\Phi N_c = \Phi k_1 k_{12} k_{19}{g}_{19}[f'cA_c]$</td>
<td>$\Phi N_c &gt; N*c$</td>
<td>Clause 5.5.5</td>
</tr>
</tbody>
</table>

where:

- $M_i, V_i, N_t, N_c$ = Design action effect in edgewise bending, and shear, tension and compression in the plane of the plywood panel, respectively.
- $\Phi M_i, \Phi V_i, \Phi N_t, \Phi N_c$ = Design capacity in bending, shear, tension and compression respectively.
- $\Phi$ = Capacity factor for plywood
- $k_1$ = Duration of load strength modification factor
- $k_{12}$ = Stability modification factor
- $k_{19}$ = Moisture condition strength modification factor
- $g_{19}$ = Plywood assembly modification factor
- $f'_{b}, f'_{s}, f'_{t}, f'_{c}$ = Characteristic strengths in bending, panel shear, tension and compression respectively
- $Z_i$ = Plywood section modulus = $t_i d^2 / 6$
- $A_s$ = shear plane area = $2/3 (dt)$ for shear in bending
- $A_t, A_c$ = Effective cross sectional area
  - $A_t, A_c$ = Effective cross sectional area
  - $t_i d$ for load applied parallel sum of the thickness of veneers with grain parallel to span
  - $t x d$ for load applied at 45° to plywood grain direction

Serviceability Limit State:

(Calculated bending deflection x $j_2 x j_6 x g_{19}$)+(Calculated shear deflection x $j_2 x j_6 x g_{19}$) ≤ deflection limit

Clause 5.5.6 & 5.5.7

where:

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

5.8 Factors

Capacity Factor, $\Phi$

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

<table>
<thead>
<tr>
<th>Application of Structural member</th>
<th>Plywood Capacity Factor, Φ</th>
</tr>
</thead>
<tbody>
<tr>
<td>All structural elements in houses. All secondary structural elements in structures other than houses</td>
<td>0.9</td>
</tr>
<tr>
<td>Primary structural elements in structures other than houses</td>
<td>0.8</td>
</tr>
<tr>
<td>Primary structural elements in structures intended to fulfil an essential service or post disaster function</td>
<td>0.75</td>
</tr>
</tbody>
</table>

**TABLE 5.6: Capacity Factor, Φ**

### Factor $k_1$ – Load Duration

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 plywood 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.

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

**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 $k_6 = 0.9$. If floods are due to cyclonic winds then temperature modification may not be required.

### 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 for bearing lengths less than 150mm along the grain when the bearing length is 5mm or more from the end of the member.

<table>
<thead>
<tr>
<th>Length of bearing of Member (mm)</th>
<th>12</th>
<th>25</th>
<th>50</th>
<th>75</th>
<th>125</th>
<th>150 or more</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value of $k_7$</td>
<td>1.85</td>
<td>1.60</td>
<td>1.30</td>
<td>1.15</td>
<td>1.05</td>
<td>1.00</td>
</tr>
</tbody>
</table>

**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 J of AS 17201.1-1997

Diaphragms with Lateral Edges Supported & Subjected to Uniform Edge Forces (from App. J2.2)

#### k₁₂ - Bending

**F₁₄ k₁ = 1.15**

<table>
<thead>
<tr>
<th>Nominal Thickness (mm)</th>
<th>ID Code</th>
<th>Face grain direction is horizontal (θ=0°)</th>
<th>Depth of Web (mm)</th>
<th>Face grain direction is vertical (θ=90°)</th>
<th>Depth of Web (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>4.5-15-3</td>
<td>1.00</td>
<td>0.64</td>
<td>0.49</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.33</td>
<td>0.27</td>
<td>0.24</td>
<td>0.23</td>
</tr>
<tr>
<td>7</td>
<td>7-24-3</td>
<td>1.00</td>
<td>1.00</td>
<td>0.59</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.51</td>
<td>0.37</td>
<td>0.30</td>
<td>0.28</td>
</tr>
<tr>
<td>7.5</td>
<td>7.5-25-3</td>
<td>1.00</td>
<td>1.00</td>
<td>0.99</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.65</td>
<td>0.40</td>
<td>0.31</td>
<td>0.29</td>
</tr>
<tr>
<td>9</td>
<td>9-30-3</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.84</td>
<td>0.49</td>
<td>0.36</td>
<td>0.33</td>
</tr>
<tr>
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<td>12-24-5</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
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<td>15-30-5</td>
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<td>1.00</td>
<td>1.00</td>
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<td>1.00</td>
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<td>1.00</td>
</tr>
<tr>
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<td>17-24-7</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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<td>1.00</td>
<td>1.00</td>
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</table>

#### k₁₂ - Compression

**F₁₄ k₁ = 1.15**

<table>
<thead>
<tr>
<th>Nominal Thickness (mm)</th>
<th>ID Code</th>
<th>Face grain direction is horizontal (θ=0°)</th>
<th>Depth of Web (mm)</th>
<th>Face grain direction is vertical (θ=90°)</th>
<th>Depth of Web (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>4.5-15-3</td>
<td>0.38</td>
<td>0.30</td>
<td>0.25</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>0.22</td>
<td>0.21</td>
<td>0.21</td>
</tr>
<tr>
<td>7</td>
<td>7-24-3</td>
<td>0.64</td>
<td>0.45</td>
<td>0.51</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.26</td>
<td>0.25</td>
<td>0.23</td>
<td>0.22</td>
</tr>
<tr>
<td>7.5</td>
<td>7.5-25-3</td>
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<td>0.27</td>
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<td>12-24-5</td>
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<td>1.00</td>
<td>0.72</td>
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<td></td>
<td>0.49</td>
<td>0.43</td>
<td>0.33</td>
<td>0.27</td>
</tr>
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<td>15-30-5</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
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<td></td>
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<td>1.00</td>
<td>0.66</td>
<td>0.56</td>
</tr>
<tr>
<td>17</td>
<td>17-24-7</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.00</td>
<td>0.70</td>
<td>0.67</td>
<td>0.40</td>
</tr>
</tbody>
</table>

#### k₁₂ - Shear

**F₁₁ k₁ = 1.15**

<table>
<thead>
<tr>
<th>Nominal Thickness (mm)</th>
<th>ID Code</th>
<th>Face grain direction is horizontal (θ=0°)</th>
<th>Depth of Web (mm)</th>
<th>Face grain direction is vertical (θ=90°)</th>
<th>Depth of Web (mm)</th>
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</thead>
<tbody>
<tr>
<td>4.5</td>
<td>4.5-15-3</td>
<td>0.79</td>
<td>0.66</td>
<td>0.57</td>
<td>0.54</td>
</tr>
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<td>0.53</td>
<td>0.52</td>
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<td>7</td>
<td>7-24-3</td>
<td>1.00</td>
<td>0.89</td>
<td>0.67</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
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<td>0.58</td>
<td>0.54</td>
<td>0.52</td>
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<td>7.5-25-3</td>
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<td>0.95</td>
<td>0.70</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
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<td>0.61</td>
<td>0.59</td>
<td>0.53</td>
</tr>
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<td>9-30-3</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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<td>15-30-5</td>
<td>1.00</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>17</td>
<td>17-24-7</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
FIGURE 5.5 shows an I-beam defining the relevant design parameters, with respect to the values given in TABLE 5.9, for the edge axial forces, moments and shears.

**FIGURE 5.5: Defines diaphragm buckling parameters**

### k<sub>19</sub> – Moisture content factor

The k<sub>19</sub> 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<sub>19</sub> = 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.

<table>
<thead>
<tr>
<th>Strength Property</th>
<th>Factor k&lt;sub&gt;19&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Moisture Content*</td>
</tr>
<tr>
<td></td>
<td>15% or less</td>
</tr>
<tr>
<td>Bending</td>
<td>1.0</td>
</tr>
<tr>
<td>Tension in plane of sheet</td>
<td>1.0</td>
</tr>
<tr>
<td>Shear</td>
<td>1.0</td>
</tr>
<tr>
<td>Compression in plane of sheet</td>
<td>1.0</td>
</tr>
<tr>
<td>Compression in normal to plane of sheet</td>
<td>1.0</td>
</tr>
</tbody>
</table>

*For moisture contents between 15 and 25%, use linear interpolation to obtain k<sub>19</sub>.

**TABLE 5.10: Moisture Content Factor, k<sub>19</sub>**

### g<sub>19</sub> – Plywood Assembly Factor

The g<sub>19</sub> 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<sub>19</sub> 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<sub>19</sub> 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<sub>19</sub> = 1. The g<sub>19</sub> 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<sub>19</sub> 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.
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 $g_{19} = \text{factor for the reduction in capacity.}$ 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 $g_{19} = 1.5$.

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

<table>
<thead>
<tr>
<th>Property</th>
<th>Direction of Face Plies Relative to Load Direction</th>
<th>Assembly Factor $g_{19}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending strength</td>
<td>3 ply perpendicular to span</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>5 ply or more perpendicular to span</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Parallel to span</td>
<td>1.0</td>
</tr>
<tr>
<td>Shear strength</td>
<td>any orientation</td>
<td>0.4</td>
</tr>
<tr>
<td>Bearing strength</td>
<td>any orientation</td>
<td>1.0</td>
</tr>
<tr>
<td>Bending deflection</td>
<td>parallel or perpendicular to span</td>
<td>1.0</td>
</tr>
<tr>
<td>Shear deformation</td>
<td>parallel or perpendicular to span</td>
<td>1.0</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Property</th>
<th>Direction of Grain of Face Plies Relative to Load Direction</th>
<th>Assembly Factor $g_{19}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression and Bending</td>
<td>parallel or perpendicular (II plies only) ±45°</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>±45°</td>
<td>0.34</td>
</tr>
<tr>
<td>Tension Strength</td>
<td>parallel or perpendicular ±45°</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>±45°</td>
<td>0.17</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>parallel or perpendicular ±45°</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>±45°</td>
<td>1.5</td>
</tr>
<tr>
<td>Shear Deformation</td>
<td>parallel or perpendicular (II plies only)</td>
<td>1.0</td>
</tr>
<tr>
<td>Bending Deflection</td>
<td>parallel or perpendicular (II plies only)</td>
<td>1.0</td>
</tr>
<tr>
<td>Deformation in compression or tension</td>
<td>parallel or perpendicular (II plies only) ±45°</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>±45°</td>
<td>1.5</td>
</tr>
</tbody>
</table>

TABLE 5.12: Assembly Factors $g_{19}$ for Plywood Loaded in the Plane of the Plywood Panel
j₂ – 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.

<table>
<thead>
<tr>
<th>Initial Moisture Content %</th>
<th>Load Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>≤1 day</td>
</tr>
<tr>
<td>≤15</td>
<td>1</td>
</tr>
<tr>
<td>20</td>
<td>1</td>
</tr>
<tr>
<td>≥25</td>
<td>1</td>
</tr>
</tbody>
</table>

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

j₃ – 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.

<table>
<thead>
<tr>
<th>Initial Moisture Content %</th>
<th>Load Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>≤1 day</td>
</tr>
<tr>
<td>≤15</td>
<td>1</td>
</tr>
<tr>
<td>20</td>
<td>1</td>
</tr>
<tr>
<td>≥25</td>
<td>1</td>
</tr>
</tbody>
</table>

Use the logarithm of time for interpolation

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

j₆ – 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.

<table>
<thead>
<tr>
<th>Type of Stiffness</th>
<th>Moisture Content* 15% or less</th>
<th>Moisture Content* 25% or more</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Elasticity</td>
<td>1.0</td>
<td>0.8</td>
</tr>
<tr>
<td>Modulus of Rigidity</td>
<td>1.0</td>
<td>0.6</td>
</tr>
</tbody>
</table>

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

**TABLE 5.15: Plywood in Service Moisture Content Factor $j_6$ for Stiffness**
Chapter 5 Appendix

Method of Calculation of Section Properties

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

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

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

This method satisfies the requirements of AS/NZS 2269.

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

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

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

(c) $\overline{y}$ 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 (AS 1720.1-1997).
Using the theory of parallel axes and parallel ply theory, the calculation is as follows:

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

\[
I(NA) = 2 \left[ \frac{1}{12} bd_1^3 + A_1(y_1^2) \right] + 2 \times 0.03 \left[ \frac{1}{12} bd_2^3 + A_2(y_2^2) \right] + \frac{1}{12} bd_3^3
\]

where

\[
A_1 = d_1b \\
A_2 = d_2b \\
0.03 = \text{factor for plies running at right angles to span for I used in stiffness computations only.}
\]

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

\[
= 2 \left[ \frac{1}{12} bd_1^3 + A_1(y_1^2) \right] + \frac{1}{12} bd_3^3
\]

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

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

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

Face Grain Perpendicular to the Span

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

![FIGURE A5.2: Face grain perpendicular to span](image-url)

The calculation is as follows:

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

\[
I(NA) = 2 \times 0.03 \left[ \frac{1}{12} bd_1^3 + A_1(y_1^2) \right] + 2 \left[ \frac{1}{12} bd_2^3 + A_2(y_2^2) \right] + 0.03 \frac{1}{12} bd_3^3
\]

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

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

\[
= 2 \left[ \frac{1}{12} bd_2^3 + A_2(y_2^2) \right]
\]
Z(NA) perpendicular to face grain \( = \frac{I(NA) \text{ strength perpendicular}}{y_2^2} \)

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

**An Equivalent Alternative**

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

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

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

\[
I_{NA} = \sum \frac{bD^3}{12} - \frac{bd^3}{12}
\]

where:

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

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

\[
I_{NA} = \frac{bD^3}{12} - \frac{bd^3}{12} + \frac{bd^3}{12}
\]

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

\[
I_{NA} = \frac{bD^3}{12} - \frac{bd^3}{12}
\]
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 AS1720.1-1997. Typically, structural LVL is used as a beam, tension or column element and therefore grain direction of all veneers is usually orientated in the longitudinal direction to maximise strength and stiffness in the spanned direction. Section properties for standard LVL containing no cross-banded veneer, is calculated using actual cross-section dimensions. However, where cross-bands have been included, for example to increase resistance to nail splitting or to improve dimensional stability, parallel ply theory as applied to plywood (refer Error! Reference source not found.) 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.
When LVL contains cross-bands, the section properties are calculated based on the parallel ply theory used in plywood design.

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

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

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

(c) the full cross-sectional area is effective when resisting in-plane shear.
6.4 LVL – Design Methodology

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

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

\[
\bar{\Phi} R > S^* \]

where \( \Phi R \) = design capacity of the LVL member

\( S^* \) = design action effect, eg. bending moment, \( M^* \), shear force, \( V^* \), etc.

and \( \Phi R = k_{mod} \left[ f_{o'} \cdot X \right] \)

where \( \Phi \) = capacity factor

\( k_{mod} \) = product of relevant modification factors (eg. \( k_1 \), \( k_6 \), \( k_7 \), \( k_{12} \), \( k_{19} \), \( g_{19} \)).

\( f_{o'} \) = appropriate characteristic strength

\( X \) = geometric section property.

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

6.5 Beam Design

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

**Strength Limit State:**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>$M^*$</td>
<td>$\Phi_M = \Phi k_1 k_4 k_6 k_9 k_{12} [f'_b Z]$</td>
<td>$\Phi M &gt; M^*$</td>
<td>clause 3.2.1.1</td>
</tr>
<tr>
<td>For beams that can bend about both the major and minor axes simultaneously:</td>
<td></td>
<td></td>
<td>$\frac{M_x^<em>}{(\phi M_x)} + \frac{M_y^</em>}{(\phi M_y)} \leq 1.0$</td>
<td>clause 3.2.1.2</td>
</tr>
<tr>
<td>Shear</td>
<td>$V^*$</td>
<td>$\Phi V = 1$</td>
<td>$\Phi V &gt; V^*$</td>
<td>clause 3.2.5</td>
</tr>
<tr>
<td>Bearing perpendicular to grain</td>
<td>$N_p^*$</td>
<td>$\Phi N_p = 1$</td>
<td>$\Phi N_p &gt; N_p^*$</td>
<td>clause 3.2.6.1</td>
</tr>
<tr>
<td>parallel to grain</td>
<td>$N_l^*$</td>
<td>$\Phi N_l = 1$</td>
<td>$\Phi N_l &gt; N_l^*$</td>
<td>clause 3.2.6.2</td>
</tr>
</tbody>
</table>

where:

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

Calculated deflection \( x_j \times j_2 \times j_6 \leq \text{deflection limit} \)  
where:  
\[ j_2 = \text{Duration of load stiffness modification factor} \]
\[ j_6 = \text{Moisture condition stiffness modification factor} \]

clauses 5.4.5

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

6.6 Column Design

Strength Limit State:

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression</td>
<td>( N^*_c )</td>
<td>( \Phi N_c = \Phi k_1 k_4 k_6 k_9 k_{11} k_{12} [f'_c A_c] )</td>
<td>( \Phi N_c &gt; N^*_c )</td>
<td>clause 3.3</td>
</tr>
<tr>
<td>For columns that can buckle about both axes:</td>
<td>( \Phi N_{cx}, \Phi N_{cy} )</td>
<td>( \Phi N_{cx} &gt; N^<em><em>c ) and ( \Phi N</em>{cy} &gt; N^</em>_c )</td>
<td>clause 3.3.1.2</td>
<td></td>
</tr>
</tbody>
</table>

where:

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

6.7 Tension Member Design

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

Strength Limit State:

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension parallel to grain</td>
<td>(N^*<em>{\ell}) (\Phi N</em>{\ell} = \Phi k_1 k_4 k_{611} [f'<em>{\ell} A</em>{\ell}])</td>
<td>(\Phi N_{\ell} &gt; N^*_{\ell})</td>
<td>clause 3.4.1</td>
<td></td>
</tr>
<tr>
<td>Tension perpendicular to grain</td>
<td>(N^*<em>{tp}) (\Phi N</em>{tp} = \Phi k_1 k_{11} [f'<em>{tp} A</em>{tp}])</td>
<td>(\Phi N_{tp} &gt; N^*_{tp})</td>
<td>clause 3.5</td>
<td></td>
</tr>
</tbody>
</table>

where:

- \(N^*_{\ell}, N^*_{tp}\) = Design action effect in tension parallel and perpendicular to grain respectively
- \(\Phi N_{\ell}, \Phi N_{tp}\) = Design capacity in tension parallel and perpendicular to grain respectively
- \(\Phi\) = Capacity factor for LVL
- \(k_1\) = Duration of load strength modification factor
- \(k_4\) = Moisture content modification factor
- \(k_{611}\) = Temperature modification factor
- \(f'_{\ell}, f'_{tp}\) = Characteristic strengths in tension parallel and perpendicular to grain respectively
- \(A_{\ell}\) = 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:

<table>
<thead>
<tr>
<th>Strength Limit State</th>
<th>Design Action Effect</th>
<th>Strength Limit State Satisfied when:</th>
<th>AS1720.1-1997 Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combined Bending and Compression about the x axis</td>
<td>$M^<em>_x$, $N^</em>_c$</td>
<td>$(M^<em>_x / \Phi M_x)2 + (N^</em><em>c / \Phi N</em>{cy}) \leq 1$ and $(M^<em>_x / \Phi M_x) + (N^</em><em>c / \Phi N</em>{cx}) \leq 1$</td>
<td>clause 3.6.1</td>
</tr>
<tr>
<td>Combined Bending and Tension actions</td>
<td>$M^<em>$, $M^</em>_x$, $N^*_t$</td>
<td>$(k_{12}M^* / \Phi M) + (N^<em>_t / \Phi N_t) \leq 1$ and $(M^</em>_x / \Phi M_x) - (Z N^*_t / A \Phi M_k) \leq 1$</td>
<td>clause 3.6.2</td>
</tr>
</tbody>
</table>

where:

- $M^*_x$ = Design action effect produced by the strength limit states design loads acting in bending about a beam’s major principal x-axis.
- $M^*$ = Design action effect produced by the strength limit states design loads acting in bending about a beam’s appropriate axis.
- $N^*_c$, $N^*_t$ = Design action effect produced by the strength limit states design loads acting in compression and tension respectively.
- $\Phi M_x$ = Design capacity in bending about a beam’s major principal x-axis.
- $\Phi M$ = Design capacity in bending about a beam’s appropriate axis.
- $\Phi N_{cy}$, $\Phi N_{cx}$ = Design capacity in compression for buckling about a beam’s major y-axis and x-axis respectively.
- $\Phi N_t$ = 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 $\Phi$ 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, $\Phi$, assigned via AS1720.1-1997, to structural materials, is based on current knowledge of product structural performance, intended structural application and material reliability. The capacity factors applied to structural LVL manufactured to AS/NZS 4357 reflects the high degree of manufacturing process control, the low material variability and high product reliability.

<table>
<thead>
<tr>
<th>Application of Structural Member</th>
<th>LVL Capacity Factor, $\Phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>All structural elements in houses. All secondary structural Elements in structures other than houses</td>
<td>0.9</td>
</tr>
<tr>
<td>Primary structural elements in structures other than houses</td>
<td>0.85</td>
</tr>
<tr>
<td>Primary structural elements in structures intended to fulfil an essential services or post disaster function</td>
<td>0.80</td>
</tr>
</tbody>
</table>

**TABLE 6.1: Capacity Factor**
In Service Modification Factors:
The following in service modification factors are applicable to structural LVL -

<table>
<thead>
<tr>
<th>Modification Factor</th>
<th>AS 1720.1-1997 Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strength modification factors</strong></td>
<td></td>
</tr>
<tr>
<td>$k_1$ = Factor for load duration</td>
<td>clause 8.4.2 &amp; 2.4</td>
</tr>
<tr>
<td>$k_4$ = Factor for in-service moisture content</td>
<td>clause 8.4.3</td>
</tr>
<tr>
<td>$k_6$ = Factor for temperature effects</td>
<td>clause 8.4.4 &amp; 2.4.3</td>
</tr>
<tr>
<td>$k_7$ = Factor for bearing length</td>
<td>clause 8.4.5 &amp; 2.4.4</td>
</tr>
<tr>
<td>$k_9$ = Factor for load sharing in grid systems</td>
<td>clause 8.4.6 &amp; 2.4.5</td>
</tr>
<tr>
<td>$k_{11}$ = Factor for member size</td>
<td>clause 8.4.7</td>
</tr>
<tr>
<td>$k_{12}$ = Factor for instability</td>
<td>clause 8.4.8</td>
</tr>
</tbody>
</table>

| **Stiffness modification factors:** | |
| $J_2$ = Factor for duration of load for bending, compression and shear | clause 8.4.2 & 2.4.1.2 |
| $J_3$ = Factor for duration of load for $k_1$ | clause 8.4.2 & 2.4.1.2 |
| $k_1$ = Duration of load strength modification factor | clause 8.4.3 |
| $J_6$ = Factor for in service moisture content | clause 2.4.1 |

**k$_1$ – Duration of Load Strength Modification Factor**

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.

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>$k_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent loads</td>
<td>0.57</td>
</tr>
<tr>
<td>Live loads on floors due to vehicles or people applied at frequent but irregular intervals</td>
<td>0.80</td>
</tr>
<tr>
<td>Live loads applied for periods of a few days and at infrequent intervals</td>
<td>0.94</td>
</tr>
<tr>
<td>Impact of wind loads</td>
<td>1.0</td>
</tr>
<tr>
<td>Wind gusts</td>
<td>1.15</td>
</tr>
</tbody>
</table>

**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.
<table>
<thead>
<tr>
<th>Property</th>
<th>Equilibrium Moisture Content (EMC)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>15% or less</td>
</tr>
<tr>
<td>Bending and Compression</td>
<td>$k_4 = 1.0$</td>
</tr>
<tr>
<td>Tension and Shear</td>
<td>$k_4 = 1.0$</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>$j_6 = 1.0$</td>
</tr>
</tbody>
</table>

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

### k₆ – Factor for Temperature

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

### FIGURE 6.7: Shows regions $k_6$ applies

### k₇ – 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.
### Table 6.4: Factor for length and position on bearing

<table>
<thead>
<tr>
<th>Length of Bearing of Member (mm)</th>
<th>12</th>
<th>25</th>
<th>50</th>
<th>75</th>
<th>125</th>
<th>150 or more</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value of ( k )</td>
<td>1.85</td>
<td>1.60</td>
<td>1.30</td>
<td>1.15</td>
<td>1.05</td>
<td>1.00</td>
</tr>
</tbody>
</table>

\[ k_9 \] – 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.

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

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

#### FIGURE 6.8: Parallel and grid systems

\( k_{11} \) – Size Factor

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

LVL members are modified for size effects as follows:

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

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

For compression \( k_{11} = 1.0 \)

For tension perpendicular to grain \( k_{11} = \left( \frac{10^7}{V} \right)^{0.2} \) where \( V \) is the volume of timber in \( \text{mm}^3 \), stressed above 80% of the maximum value in tension perpendicular to grain.

### \( k_{12} \) – 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 co-efficient.

**Stability Factor** \( k_{12} \) is calculated from the following –

(a) For: \( pS \leq 10 \), \( k_{12} = 1.0 \)

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

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

**Material constants for LVL are:**

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

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

Where:

\[ \rho = \text{ratio (temporary design action effect) / (total design action effect).} \]

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

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

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

<table>
<thead>
<tr>
<th>Initial Moisture Content %</th>
<th>( \leq 1 \text{ day} )</th>
<th>1 week</th>
<th>1 mth</th>
<th>3 mths</th>
<th>6 mths</th>
<th>9 mths</th>
<th>( \geq 1 \text{ year} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \leq 15 )</td>
<td>1</td>
<td>1.2</td>
<td>1.7</td>
<td>1.9</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>20</td>
<td>1</td>
<td>1.4</td>
<td>2.0</td>
<td>2.4</td>
<td>2.4</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>( \geq 25 )</td>
<td>1</td>
<td>1.5</td>
<td>2.3</td>
<td>2.8</td>
<td>2.9</td>
<td>2.9</td>
<td>3.0</td>
</tr>
</tbody>
</table>

**TABLE 6.5: Duration of load factor \( j_2 \) for creep deformation for bending,**
j₃ – Duration of load factor for creep deformation (tension members)

The j₃ load factor given in TABLE 6.6, allows for the time dependent increase in deformation in LVL members subjected to tension type loads.

<table>
<thead>
<tr>
<th>Initial Moisture Content %</th>
<th>LOAD DURATION</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>≤1 day</td>
<td>≥1 year</td>
<td></td>
</tr>
<tr>
<td>≤15</td>
<td>1</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>1</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td>≥25</td>
<td>1</td>
<td>1.5</td>
<td></td>
</tr>
</tbody>
</table>

Use the logarithm of time for interpolation

TABLE 6.6: Duration of load factor j₃ for creep deformation for tension members
Chapter 6 Appendix

Slenderness Co-Efficient for Lateral Buckling Under Bending

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

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

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

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

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

![FIGURE A6.1: Discrete restraints to the compression and tension edge](image)

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

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

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 –

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

49
For a beam that is loaded along its **compression edge** and has a **continuous lateral restraint** system along the compression edge (see **FIGURE A6.2**), then the slenderness co-efficient, denoted by $S_1$, may be taken to be equal to zero.

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

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

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

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

$$S_1 = \frac{1.5 d/b}{\left( \frac{\pi d}{L_{a\phi}} \right)^2 + 0.4}^{0.5}$$
FIGURE A6.3: Continuous restraint along the tension edge combined with discrete torsional restraints

Beams that bend only about their minor axis
For all cases, the slenderness co-efficient, denoted by $S_2$, may be taken to be –

$$S_2 = 0.0$$

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

Slenderness co-efficient for lateral buckling under compression

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

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

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

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

and

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

where

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

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

For restrain systems that restrain movement in the direction of the y-axis, and are continuous along the length of the column, the slenderness co-efficient may be taken to be:
\[ S_3 = 0.0 \]

**FIGURE A6.4: Notation for column restraints**

where

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

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

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

\[ S_4 = \frac{3.5d}{b} \]

(b) **Columns that can bend about both axes.**

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

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

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

$$k_{12} = 1.0$$

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

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

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

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

where:

$S = S_3$ for buckling about the major axis;

$= S_4$ for buckling about the minor axis

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

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

7.2 Structural Plywood Flooring and Floor Systems

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

7.3 Design Issues of Flooring

The excellent load re-distribution capabilities of plywood means uniformly distributed loads are unlikely to govern the design. Structural plywood flooring design is usually governed by the concentrated imposed loads. For more lightly loaded floors deflection under imposed concentrated loads governs plywood selection. Shear strength may govern under higher concentrated loads with closer support spacings. Concentrated loads on structural plywood flooring are treated as a line load. The distribution width of the concentrated load must therefore be determined. TABLE 5.3 in Error! Reference source not found. of this Manual provides standard load distribution widths for various thicknesses of plywood. Structural plywood flooring should be spanned with the face veneer grain direction parallel to the span to maximise the plywood capacity. Support spacings should be selected to suit the plywood sheet length, such that the ends of the sheet land on a support.

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

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

1. Select a joist spacing to suit standard plywood flooring sheet lengths:
   a. **Standard sheet lengths** are:
      i. 2400 - suitable joist spacings include 400, 600, 800 mm
      ii. 2250 - suitable joist spacings include 375, 450, 750 mm (limited availability)
      iii. 2700 - suitable joist spacings include 450, 540, 675, 900 mm
   b. To optimise structural plywood performance ensure plywood is supported continuously over a minimum of two spans.

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

3. Determine **floor imposed load requirements** from AS/NZS 1170.1 for both concentrated and uniformly distributed loads
   i. Typically, the load contribution of the plywood flooring itself is insignificant when compared to the imposed loads and consequently is ignored in strength and stiffness calculations.

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

The relevant factors are:

<table>
<thead>
<tr>
<th>Factor</th>
<th>Description</th>
<th>Refer AS1720.1-1997</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>((\phi k_1 k_{19} g_{19}))</td>
<td>Error! Reference source</td>
</tr>
<tr>
<td>Shear</td>
<td>((\phi k_{19} g_{19}))</td>
<td></td>
</tr>
<tr>
<td>Deflection</td>
<td>((j_2 j_6 g_{19}))</td>
<td></td>
</tr>
</tbody>
</table>

- \(\phi = 0.9\) for all structural elements in houses
- \(k_1 = 0.94\) For concentrated loads assuming loads are applied at infrequent intervals such as might arise due to a pallet, jack or maintenance type load. Effective duration of peak load = 5 days.
- \(k_1 = 0.80\) For uniformly distributed loads assuming loads are typical floor type loads (crowd or vehicle or stored material. Effective duration of peak load = 5 months.
- \(k_{19} = 1.0\) 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%.
- \(g_{19} = 1.0\) Direction of the face veneers is parallel with the span direction. Therefore, \(g_{19} = 1.0\) for bending, shear strength and deflection.
- \(j_2 = 1.0\) for short term concentrated loads of less than 1 days duration.
- \(j_2 = 2.0\) 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

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

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

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

1. **Loads:**
   - Live
     - 7kN concentrated load;
     - 5kPa uniformly distributed load (UDL)
   - Dead
     - **Self weight:**
       For 25mm plywood (~600kg/m3 x 9.81E-3 x 0.025) = 0.15 kPa (Not usually considered but included in this design example for completeness)

2. **Load Combinations**
   - Strength limit state: 1.25G + 1.5Q
   - Serviceability limit state: 1 x Q (short term) G + 0.4Q (long term)

3. **Capacity Factor and Strength Modification Factors**
   - \( \phi = 0.9 \)  
   - \( k_1 = 0.94 \) concentrated live load
   - \( k_1 = 0.80 \) uniformly distributed live load
   - \( k_{19} = 1.0 \) (MC ≤ 15%)
   - \( g_{19} = 1.0 \) for bending strength
   - \( g_{19} = 0.4 \) for shear strength

4. **Serviceability Modification Factors**
   - \( j_2 = 1.0 \) short term load
   - \( j_2 = 2.0 \) long term load
   - \( j_6 = 1.0 \) (MC ≤ 15%)
   - \( g_{19} = 1.0 \) bending deflection
5. Critical Load Action Effects

**Load Case 1**

**UDL**

\[
\begin{align*}
M_{\text{max}} &= 49WL^2/512 \\
V_{\text{max}} &= 9WL/16 \\
\Delta_{\text{max}} &= WL^4/(72.3El)
\end{align*}
\]

**Concentrated Load**

\[
\begin{align*}
M_{\text{max}} &= 13PL/64 \\
V_{\text{max}} &= -19P/32 \\
\Delta_{\text{max}} &= PL^3/(66.7El)
\end{align*}
\]

**Load Case 2**

\[
\begin{align*}
M_{\text{max}} &= -WL^2/8 \\
V_{\text{max}} &= 5WL/8 \\
\Delta_{\text{max}} &= WL^4/(185El)
\end{align*}
\]

\[
\begin{align*}
M_{\text{max}} &= 6PL/32 \\
V_{\text{max}} &= 11P/16 \\
\Delta_{\text{max}} &= PL^3/(110El)
\end{align*}
\]

**Note:**

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

For example, for a floor consisting of 300 x 40mm F8 joists at 400 mm centres, spanning 2400 mm and 24 mm F11 structural plywood flooring, the ratio of joist stiffness to flooring stiffness is \(\frac{E_jL_j^3}{E_fL^3}\) (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

<table>
<thead>
<tr>
<th></th>
<th>( M_p )</th>
<th>( M'_p )</th>
<th>( M'_p/k_1 )</th>
<th>( V_p )</th>
<th>( V'_p )</th>
<th>( V'_p/k_1 )</th>
<th>( \Delta (mm) )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DL</strong></td>
<td>( 0.15 \times 10^3 \times 400^2/8 = 3 )</td>
<td>( 1.25 \times 3 = 3.75 )</td>
<td>( 3.75 / 0.57 = 6.58 )</td>
<td>( 5 \times 0.1510^3 \times 400/8 = 0.04 )</td>
<td>( 1.25 \times 0.04 = 0.05 )</td>
<td>( 0.05 / 0.57 = 0.09 )</td>
<td>( 0.15 \times 10^{-2} = 0.0015 )</td>
</tr>
<tr>
<td><strong>LL(UDL)</strong></td>
<td>( 5 \times 10^3 \times 400^2/8 = 100 )</td>
<td>( 1.5 \times 100 = 150 )</td>
<td>( 150 / 0.8 = 187.5 )</td>
<td>( 5 \times 5 \times 10^3 \times 400/8 = 1.25 )</td>
<td>( 1.5 \times 1.25 = 1.9 )</td>
<td>( 1.9 / 0.8 = 2.3 )</td>
<td>( 5 \times 400 \times 10^{-2} = 0.2 )</td>
</tr>
<tr>
<td><strong>LL(conc)</strong></td>
<td>( 13 \times 7000 \times 400 = 64 \times 520 = 1094 )</td>
<td>( 1.5 \times 1094 = 1641 )</td>
<td>( 1641 / 0.9 = 1823 )</td>
<td>( 11 \times 7 \times 10^3 / (16 \times 520) = 9.25 )</td>
<td>( 1.5 \times 9.25 = 13.9 )</td>
<td>( 13.9 / 0.9 = 15.4 )</td>
<td>( 7 \times 400 \times 10^{-2} = 7 )</td>
</tr>
</tbody>
</table>

Notes:
1. Units for moment are Nmm/mm width, units for shear are N/mm width.
2. As the maximum moment and shear due to self-weight are very small, and do not occur at the same location as the maximum live load moment and shear, load action effects for strength due to dead load will be ignored.
3. Load distribution width for concentrated loads has been assumed to be 520mm (refer TABLE 5.2)

#### 2. Strength Limit State – Design Load Combinations

- **M**\(_{\text{crit}}\)** = **M***(LLconc) = 1823 Nmm/mm width
- **V**\(_{\text{crit}}\)** = **V***(LLconc) = 15.4 N/mm width

#### 3. Shear criteria – (establish minimum \( f'_{sA_s} \))

\[
\phi V_p \geq V'_p
\]

and \( \phi V_p = \phi k_1 k_{19} g_{19}[f'_{sA_s}] \)

\[
\Rightarrow \phi k_1 k_{19} g_{19}[f'_{sA_s}] \geq V'_p
\]

and \( [f'_{sA_s}] \geq V'_p / (\phi k_1 k_{19} g_{19}) \)

Minimum required \( f'_{sA_s} = 15.4 / (0.9 \times 0.94 \times 1.0 \times 0.4) \)

\( = 45.5 \) N/mm width

From Appendix 6C, Require minimum 12mm 12-15-5, F14 (\( f'_{sA_s} = 49 \))

#### 4. Bending criteria – (establish minimum \( f'_{bZ_p} \))

\[
\phi M_p \geq M'_p
\]

and \( \phi M_p = \phi k_1 k_{19} g_{19}[f'_{bZ_p}] \)

\[
\Rightarrow \phi k_1 k_{19} g_{19}[f'_{bZ_p}] \geq M'_p
\]

And \( [f'_{bZ_p}] \geq M'_p / (\phi k_1 k_{19} g_{19}) \)

Minimum required \( f'_{bZ_p} = 1823 / (0.9 \times 0.94 \times 1.0 \times 1.0) \)

\( = 2155 \) Nmm/mm width

From TABLE 5.4, suitable structural plywoods include:

- F11, 25mm 25-30-9 (\( f'_{bZ_p} = 2468 \)) Nmm/mm width
- F14, 25mm 25-30-9 (\( f'_{bZ_p} = 2820 \)) Nmm/mm width
- F17, 19mm 19-30-7 (\( f'_{bZ_p} = 2325 \)) Nmm/mm width
- F27, 17mm 17-24-7 (\( f'_{bZ_p} = 2680 \)) Nmm/mm width
5. Serviceability limit state – Design Load Combinations

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

6. Deflection criteria – (determine minimum required EI)

Maximum allowable deflection = \( \frac{\text{span}}{200} \)

\( = 2\text{mm} \)

**Under short term load:**

\[ \Delta_{\text{max}} = j_2 \times g_{19} \times \Delta_{\text{LL conc}} \]

\[ 2\text{mm} = 1.0 \times 1.0 \times 13.0 \times 10^6 / EI \]

**Required**

\[ EI_{\text{min}} = 6500 \times 10^3 \text{ Nmm}^2/\text{mm width} \]

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

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

**Under long term load:**

\[ \Delta_{\text{max}} = j_2 \times g_{19} \times \Delta ( DL + LL UDL) \]

\[ 2\text{mm} = 2.0 \times 1.0 \times (53 + 1770) \times 10^3 / EI \]

\[ EI_{\text{min}} = 1824 \times 10^3 \text{ Nmm}^2/\text{mm width} \]

which is less than required \( EI \) under short term load \( \Rightarrow \) **not critical**

7. Select Suitable Structural Plywood Flooring

Subject to availability, suitable structural plywoods would include:

- **F11**, 25mm, 25-30-9 (\( EI = 9450 \times 10^3, f'_{bZp} = 2820 \))
- **F14**, 25mm, 25-30-9 (\( EI = 6780 \times 10^3, f'_{bZp} = 2820 \))
- **F17**, 21mm, 21-30-7 (\( EI = 7770 \times 10^3, f'_{bZp} = 2825 \))
- **F27**, 19mm, 19-24-9 (\( EI = 7030 \times 10^3, f'_{bZp} = 3160 \))

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

7.7 Structural Plywood Flooring

Typical structural plywood thicknesses and stress grades for a range of minimum floor imposed loadings detailed in AS/NZS 1170 are given in TABLE 7.1. Other structural plywood stress grades and thicknesses are available and alternate stress grade/thickness combinations can be designed for and specified. Full design information on using structural plywood flooring, including span tables and fixing details are provided in the EWPAA design manuals T&G Structural Plywood for Residential Flooring and Structural Plywood for Commercial & Industrial Flooring available from the EWPAA.
### Table 7.1: Summary of AS/NZS 1170.1 Floor Live Loads & Suitable Structural Plywood Thickness

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

**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.
3. *To be determined but not less than the given value.

### 7.8 Engineered Flooring System

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

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

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

Structural Plywood Wall Bracing Design Manual

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

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

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 to 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 isde 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, go the the EWPAA Structural Plywood Wall Bracing – Limit States Design manual available for free download from the EWPAA web site at http://www.ewp.asn.au.

Wall Bracing Testing Methodology

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

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

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

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

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

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

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

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

- **Stiff enough** to ensure the serviceability limit state is satisfied. This is attained by setting a deflection limit at T1 of panel height / 300;

- **Strong enough** to satisfy the strength limit state. This situation is taken to be satisfied, even though some connector and material distress may be evident, when the panel is still capable of taking further load.

- **Stable**, i.e. shows no significant signs of buckling at the serviceability limit state.
Because of the obvious difficulty associated with having to attempt an analytical check of the racking deflection of a dwelling it is essential:

“satisfaction of the strength limit state results in automatic satisfaction of the serviceability limit state”.

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

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

To fully quantify the racking load variables requires:

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

The bracing topic will be discussed further in Error! Reference source not found. on Shearwalls and Diaphragms.

### 7.11 Structural Plywood Lightweight Roofing Systems

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

<table>
<thead>
<tr>
<th>Rafter or Truss Spacing (mm)</th>
<th>Minimum Allowable Plywood Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>F8</td>
</tr>
<tr>
<td>800</td>
<td>13</td>
</tr>
<tr>
<td>900</td>
<td>16</td>
</tr>
<tr>
<td>1200</td>
<td>19</td>
</tr>
</tbody>
</table>

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

### 7.12 Structural Laminated Veneer Lumber (LVL) Framing Members

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

**Design Issues for LVL Framing Members**

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

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

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

### 7.13 Design Example – LVL Lintel Beam - Specification

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

<table>
<thead>
<tr>
<th>Characteristic Strengths and Elastic Moduli, MPa for BBF LVL as published by the manufacturer of BBF brand LVL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
</tr>
<tr>
<td>Tension</td>
</tr>
<tr>
<td>Shear in beams</td>
</tr>
<tr>
<td>Compression parallel to grain</td>
</tr>
<tr>
<td>Compression perpendicular to grain</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
</tr>
<tr>
<td>Modulus of Rigidity</td>
</tr>
<tr>
<td>Joint Group</td>
</tr>
</tbody>
</table>

**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 9 mm maximum
     - Imposed Loads: Span/250 to 9 mm maximum
     - Downwards Wind Loads: Span/250 to 9 mm maximum
     - Wind Uplift: Span/100 to 50 mm maximum

2. **Loads**

   **Permanent:**
   
   \[
   \text{Roof & Ceiling load} = 40 \text{kg/m}^2 \\
   (40 \times 9.81 \times 4.8 \times 0.9) \times 10^3 \\
   = 1.7 \text{kN/rafter}
   \]
   
   **Self weight** allow 650kg/m³

   Select Trial size beam 300 x 45 mm

   **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)

\[
0.25 \text{kPa} \times 1.2 \text{m} \times 0.9 \text{m} = 0.27 \text{kN/rafter}
\]

Wind:
(Cyclonic C1 Wind classified Area)

**Strength:**
- Net Uplift = 2.04*\text{kPa} \times 4.8\text{m} \times 0.9\text{m}
- \text{Net Uplift} = 8.8 \text{kN/rafter}

**Down:**
- Net pressure co-efficient (\(C_{p,n}\)) = 1.05
- \(V_{h,u}\) = 50 m/s
- Pressure = 0.6\times V_{h,u}^2 \times 10^{-3} \times C_{p,n}
  = 0.6 \times 50^2 \times 1.05/1000
  = 1.58 \text{kPa}
- 1.58 \text{kPa} \times 4.8\text{m} \times 0.9\text{m} = 6.8 \text{kN/rafter}

**Serviceability:**
- Net Uplift = 0.58 \text{kPa} \times 4.8\text{m} \times 0.9\text{m}
- \text{Net Uplift} = 2.5 \text{kN/rafter}

**Down:**
- Net pressure co-efficient (\(C_{p,n}\)) = 1.05
- \(V_{h,s}\) = 32 m/s
- Pressure = 0.6\times V_{h,s}^2 \times 10^{-3} \times C_{p,n}
  = 0.6 \times 32^2 \times 1.05
  = 0.65 \text{kPa}
- 0.65 \text{kPa} \times 4.8\text{m} \times 0.9\text{m} = 2.8 \text{kN/rafter}

3. Load Combinations

**Strength limit state:**
- \(1.2G + 1.5Q\)  \text{AS/NZS1170.0 Cl.4.2.2}
- \(1.2G + W_u + \omega_s Q\)
- \(Q = 0\) under max. downward wind loading
- \(W_u \geq -0.9 \text{G}\)

**Serviceability limit state:**
- \(G\) \text{AS/NZS1170.0 Cl.4.3}
- \(\omega_s Q\)  \text{(\(\omega_s = 0.7\) for roofs)}
- \(W_s\) \text{AS/NZS1170.0 T4.1}

4. Capacity Factor and Strength Modification Factors

The relevant factors for beam design are:

- **Bending**  \((\phi k_1 k_4 k_6 k_9 k_{11} k_{12})\)
- **Shear**  \((\phi k_1 k_4 k_6 k_{11})\)
- **Deflection**  \((j_2 j_6)\)

\[\phi = 0.9\] 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\° S

\[ k_9 = 1.0 \] for all LVL used in parallel systems

\[ k_{11} = 1.0 \] for bending assuming lintel beam depth \(\leq 300\) mm

\[ k_{11} = 1.0 \] for shear

\[ k_{12} \] based on value of \(\rho_b\)S

\[ \rho_b = 14.71 (E/f'_{b})^{-0.480} r^{-0.061}. \]

\[ (E/f'_{b})^{-0.480} = (12400/45)^{-0.480} = 0.067 \]

\( r = \text{temporary design action effect/total design action effect} \)

\[ S_1 = 1.25 \text{ } d/b \text{ } (\text{Lay/d})^{0.5} \]

(downwards loads, Lay = 0.9 m)

\[ S_1 = (d/b)^{1.35} (\text{Lay/d})^{0.25} \] (wind uplift, Lay =0.9 m)

5. Serviceability Modification Factors

\[ j_2 = 1.0 \] short term load

\[ j_2 = 2.0 \] long term load

\[ j_6 = 1.0 \] (MC \(\leq 15\%\))

6. Critical Load Action Effects

Permanent roof loads (G) & Wind Loads (\(W_u\))

\[ M_{\text{max}} = PL/2 \]

\[ V_{\text{max}} = 3P/2 \]

\[ \Delta_{\text{max}} = 19PL^3/(384EI) \]
### Concentrated Imposed Load (Q)

\[ M_{\text{max}} = \frac{PL}{4} \]
\[ V_{\text{max}} = \frac{P}{2} \]
\[ \Delta_{\text{max}} = \frac{PL^3}{48EI} \]

### Lintel Beam Self Weight (G)

\[ M_{\text{max}} = \frac{wL^2}{8} \]
\[ V_{\text{max}} = \frac{wL}{2} \]
\[ \Delta_{\text{max}} = \frac{5wL^4}{384EI} \]

### 7.14 Structural LVL Lintel Beam: Worked Example

#### 1. Design Action Effects on Member Due to Factored Loads

<table>
<thead>
<tr>
<th>Loading Criteria</th>
<th>( M_p ) (kNm) (unfactored)</th>
<th>( M'_p ) (kNm) (factored)</th>
<th>( M'_p/k_1 ) (kNm)</th>
<th>( V_p ) (kN) (unfactored)</th>
<th>( V'_p ) (kN) (factored)</th>
<th>( V'_p/k_1 ) (kN)</th>
<th>( \Delta ) (m)</th>
</tr>
</thead>
</table>
| **DEAD**
  G = Beam Self Weight + Permanent Roof Load | 0.09 x 3.6\(^2\) / 8 = 0.15 1.7 x 3.6/2 = 3.06 | 1.2 x 3.2 = 3.8 3.8 / 0.57 = 6.6 | Total = 2.7 | 0.09 x 3.6/2 = 0.16 3 x 1.7/2 = 2.55 | 1.2 x 2.7 = 3.2 3.2 / 0.57 = 5.6 | Total = 2.0 x 10\(^{11}\) / EI |
| **IMPOSED**
  Q (conc load) | 1.4 x 3.6/4 = 1.26 | 1.26 x 1.5 = 1.9 0.49 x 1.5 = 0.7 | 6.4 / 0.94 = 6.8 | 1.4/2 = 0.7 0.27 x 3.6/2 = 0.5 | 1.5 x 0.7 = 1.1 1.5 x 0.5 = 0.8 | 1.9 / 0.94 = 2.0 | 5 x 0.09 x 3600\(^4\) / 384EI 19 x 1.7 x 3600\(^3\) / 384EI 0.62 x 10\(^9\) / EI |
| Q (UDL) | 0.27 x 3.6/2 = 0.49 | 3.8 + 2.6 = 6.4 | Total = 1.2 | | | | |
| G + Q | | | Total = 1.9 | | | | |
| **WIND UP**
  \( W_u \uparrow \) | 8.8 x 3.6/4 = 7.9 | 1.0 x 7.9 = 7.9 | 7.9 / 1.0 = 7.9 | 3 x 8.8/2 = 13.2 | 13.2 / 1.0 = 13.2 | 19 x 2.5 x 3600\(^3\) / 384EI |
| 0.9G | 0.9 x 3.2 = 2.9 | 2.9 / 0.57 = 5.1 | Total = 2.8 | | | | 5.8 x 10\(^9\) / EI |
| \( W_u \uparrow \) - 0.9G | | | Total = 8.1 | | | | |
2. Strength Limit State - Design Load Combinations

\[
M^\text{crit} = M^*(W_u + G) = 12.7 \text{ kNm}
\]

\[
V^\text{crit} = V^*(W_u + G) = 15.8 \text{ kN}
\]

Trial Beams:

- 240 x 45, A = 10800 mm², \(I_{xx} = 51.8 \times 10^6 \text{ mm}^4\), \(Z_{xx} = 432.0 \times 10^3 \text{ mm}^3\)
- 300 x 45, A = 13500 mm², \(I_{xx} = 101.3 \times 10^6 \text{ mm}^4\), \(Z_{xx} = 675.0 \times 10^3 \text{ mm}^3\)

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

\[
k_{12} = 0.91
\]

Try 240 x 45 section size \(k_{12} = 0.91\)

Minimum required \(Z_p\):

\[
Z_{xx} = 432.0 \times 10^3 \text{ mm}^3 \text{ for 240 x 45 section size}
\]

\=> OK

Try 300 x 45 section size \(k_{12} = 0.77\)

Minimum required \(Z_p\):

\[
Z_{xx} = 675.0 \times 10^3 \text{ mm}^3 \text{ for 300 x 45 section size}
\]

\=> OK

\=> REQUIRE 240 x 45 BBF LVL FOR BENDING STRENGTH

3. Bending criteria - (establish minimum \(Z_p\))

\[
\phi M_p \geq M^* \hspace{1cm} \phi M_p = \phi k_1 k_4 k_6 k_9 k_{11} k_{12} [f_b Z_p]
\]

Try 240 x 45 section size \(k_{12} = 0.91\)

Minimum required \(Z_p\):

\[
Z_{xx} = 432.0 \times 10^3 \text{ mm}^3 \text{ for 240 x 45 section size}
\]

\=> OK

Try 300 x 45 section size \(k_{12} = 0.77\)

Minimum required \(Z_p\):

\[
Z_{xx} = 675.0 \times 10^3 \text{ mm}^3 \text{ for 300 x 45 section size}
\]

\=> OK

4. Shear criteria - (establish minimum \(A_s\))

\[
\Phi V_p \geq V^*_p \hspace{1cm} \Phi V_p = \Phi k_1 k_4 k_6 k_9 k_{11} k_{12} [f_s A_s]
\]

\=> OK

\[
\Phi k_1 k_4 k_6 k_9 k_{11} k_{12} [f_s A_s] \geq V^*_p
\]
5. Serviceability Limit State

Design Load Combinations:

\[ G \]
\[ \omega_s Q \quad (\omega_s = 0.7 \text{ for roofs}) \]
\[ W_s \]

Deflection criteria:

Maximum allowable deflection:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Load Factor</th>
<th>Maximum Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent Loads</td>
<td>Span/300</td>
<td>to 9 mm maximum</td>
</tr>
<tr>
<td>Imposed Loads &amp; downwards Wind</td>
<td>Span/250</td>
<td>to 9 mm maximum</td>
</tr>
<tr>
<td>Loads</td>
<td>Span/100</td>
<td>to 50 mm maximum</td>
</tr>
</tbody>
</table>

6. Under permanent load:

\[
\Delta_{\text{max}} = j_2 \times j_6 \times \Delta_G \\
\Delta_{\text{max}} = 2.0 \times 1.0 \times 2.0 \times 10^{11} / EI \\
240 \times 45 \text{ BBF LVL, } E = 12400 \text{ MPa} \\
I_{xx} = 51.8 \times 10^{6} \text{ mm}^4 \\
\Delta_{\text{max}} = 2.0 \times 1.0 \times 2 \times 10^{11}/(12400 \times 101.3 \times 10^{6}) \\
\Delta_{\text{max}} = 0.6 \text{ mm} \\
<<< 9 \text{ mm} \\
=> OK
\]

7. Under Imposed load:

\[
\Delta_{\text{max}} = j_2 \times j_6 \times 0.7 \Delta_Q \\
= 1.0 \times 1.0 \times 0.7 \times 1.4 \times 10^{9}/EI \\
\Delta_{\text{max}} = 1.0 \times 1.0 \times 0.7 \times 1.4 \times 10^{9}/(12400 \times 51.8 \times 10^{6}) \\
\Delta_{\text{max}} = 0.002 \text{ mm} \\
<<< 9 \text{ mm} \\
=> OK
\]

8. Under Wind load:

\[
\Delta_{\text{max}} = j_2 \times j_6 \times \Delta_{W_s} \\
= 1.0 \times 1.0 \times 6.5 \times 10^{9}/EI \\
\Delta_{\text{max}} = 1.0 \times 1.0 \times 6.5 \times 10^{9}/(12400 \times 51.8 \times 10^{6}) \\
\Delta_{\text{max}} = 0.01 \text{ mm} \\
<<<9 \text{ mm} \\
=> OK
\]

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, 240 X 45 mm BBF BRAND LVL.
Part Three

Plywood Element & System Design Examples

Structural Plywood Webbed Box Beam Design

Structural Plywood Diaphragms & Shearwalls

Structural Plywood / LVL Gusseted Timber Portal Frames

Plywood Stressed Skin Panels
8 Structural Plywood Webbed Box Beam Design

8.1 Introduction

Structural plywood webbed beams are a composite timber beam fabricated utilizing structural plywood as the web of the beam and a structural timber as the continuous beam flanges. The flange and web components are usually connected with nails and/or glue. In this design chapter, information is given for the design of nailed plywood webbed box beams. The design method is based on the information contained in the EWPAA Design Guide for Plywood Webbed Beams. The design aides included within this section facilitate the design of box beams with 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.
8.2 Beam Components and Materials

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

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

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

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

FIGURE 8.2: Jointing of seasoned flanges
Adhesives

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

Nails

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

8.3 Design of Nailed Plywood Webbed Box Beams - Methodology

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

1. Select an initial beam trial size based on

   (a) Span/Depth \((L/D)\) ratio

   (b) Depth/Breadth \((D/B)\) ratio

   (c) Beam deflection approximated from bending deflection

   Total deflection, \(\Delta_t\) in a nailed box beam is the sum of the bending deflection \(\Delta_b\), shear deflection \(\Delta_s\), and nail slip deflection \(\Delta_{ns}\) :

   \[
   \Delta_t = \Delta_b + \Delta_s + \Delta_{ns}
   \]

   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:

   \(\Delta_t\) is approximately in the range 1.5 \(\times\) \(\Delta_b\) to 2.0 \(\times\) \(\Delta_b\)

   Therefore a trial beam size can be selected from an estimate of total beam deflection based on the expected bending deflection. Bending deflection can be calculated from actual load conditions. Or, as done in the worked example, conservatively estimated from the beam flexibility tables, by determining the deflection of a simply supported, single span beam subjected to a central unit point load.
USING THE BEAM FLEXIBILITY TABLES
A trial beam size can be determined based on the value of \( F \), the flexibility co-efficient, determined from Appendix Tables A8.5 and A8.6. The flexibility co-efficient, \( F \), determined from the tables simply requires multiplication by the actual concentrated load \( P \) (kN) or uniformly distributed load \( w \) (kN/m) to determine the beam deflection.

<table>
<thead>
<tr>
<th>Simply supported beam with a centre point load, ( P ):</th>
<th>Simply supported beam with a uniformly distributed load, ( w ):</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Delta_b = \frac{j}{2} P L^3/48EI )</td>
<td>( \Delta_b = \frac{j}{2} 5wL^3/384EI )</td>
</tr>
<tr>
<td>( F = \frac{L^2}{48EI} )</td>
<td>( F = 5L^2/384EI )</td>
</tr>
<tr>
<td>( \Rightarrow \Delta_b = \frac{j}{2} P F )</td>
<td>( \Rightarrow \Delta_b = \frac{j}{2} w F )</td>
</tr>
<tr>
<td>( \Delta_t \approx (1.5 \text{ to } 2.0) \times \Delta_b )</td>
<td>( \Delta_t \approx (1.5 \text{ to } 2.0) \times \frac{j}{2} w F )</td>
</tr>
<tr>
<td>( \Rightarrow \Delta_t \approx (1.5 \text{ to } 2.0) \times \frac{j}{2} P F )</td>
<td>( \Rightarrow \Delta_t \approx (1.5 \text{ to } 2.0) \times \frac{j}{2} w F )</td>
</tr>
<tr>
<td>( \Rightarrow \text{Select } F(\text{max}) \text{ from Table A8.5 such that } F_{\text{max}} \leq \Delta_t(\text{max. allowable}) / (1.5 \text{ to } 2.0)P )</td>
<td>( \Rightarrow \text{Select } F(\text{max}) \text{ from Table A8.6 such that } F_{\text{max}} \leq \Delta_t(\text{max. allowable}) / (1.5 \text{ to } 2.0)w )</td>
</tr>
</tbody>
</table>

2. Check Flange Bending Capacity:

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

Check tension flange:

\[
\frac{M^*}{\varphi k_4 k_5 k_{11}} \leq \frac{2 f_t'(EI) x n}{E f d}
\]

Check compression flange:

\[
\frac{M^*}{\varphi k_4 k_5 k_9 k_{11} k_{12}} \leq \frac{2 f_c'(EI) x}{E f d}
\]

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

3. Check Panel Shear Capacity:

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

Both webs continuous

\[
\frac{V^*}{\varphi k_{12} k_{19} g_{19}} \leq \frac{f_s'(EI) x n t_w x A_s}{(EQ)_x}
\]

At web splice

\[
\frac{V^* \ (\text{at web splice})}{\varphi k_{12} k_{19} g_{19}} \leq \frac{f_s'(EI)_{x n} n t_w}{(EQ)_x}
\]

(i.e. one web continuous only)

4. Check Flange - Web Capacity:

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

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

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

Nail spacing, \( s = Q^*/q \)
5. Check Beam Stiffness:

Check beam deflection \( \Delta \) is not excessive, where:

\[
\Delta_t = \Delta_b + \Delta_s + \Delta_{ns}
\]

**Box Beam Section Property Formula**

- \( A_f \) = Area of flange = \( 2 \, b_f \, t_f \)
- \( A_w \) = Area of web = \( 2 \, d \, t_w \)
- \( A_s \) = Web shear Area = \( 2 \, d_n \, t_w \)

where

- \( d_n \) = depth between top and bottom flange-web nailing = \( (d - t_f) \), usually

**Plywood Webbed Beam Dimensions**

**First Moment of Area**

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

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

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

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

**Second Moment of Area**

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

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

where \( n \) = number of plywood webs (e.g. 2 for a box beam) and \( k_{34} \) = parallel ply factor (note \( k_{34} \) is not an AS1720.1-1997 factor)

\[
I_{yf} = \text{Second Moment of Area of the Flange about the beam y axis} = 2t_f \, b_f^3/12
\]

\[
I_{yw} = \text{Second Moment of Area of the Web about the beam y axis} = k_{34} t_w \, d(b_f + t_w)^2/2
\]
(using close approximation \(2 A_w x_f^2\) where \(x_f = t_w/2 + b_f/2\)).

<table>
<thead>
<tr>
<th>Number of plywood veneer layers</th>
<th>Plywood face grain parallel to span</th>
<th>Plywood face grain perpendicular to span</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 ply</td>
<td>2/3</td>
<td>1/3</td>
</tr>
<tr>
<td>5 ply</td>
<td>3/5</td>
<td>2/5</td>
</tr>
<tr>
<td>7 ply</td>
<td>4/7</td>
<td>3/7</td>
</tr>
</tbody>
</table>

**TABLE 8.1: Parallel Ply Factor, \(k_{34}\)**

**Rigidity in Bending About x-axis where:**

\[
(EI)_x = E_f l_{xf} + E_w l_{xw}
\]

\(E_f =\) 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:**

\[
(EI)_y = E_f l_{yf} + E_w l_{yw}
\]

### 8.4 Design Example – Nailed Plywood Webbed Box Beam

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

**Given that:**

![Diagram of the box beam](image)

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

**Deflection Limits:**

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

AS 1720.1 Appendix .B
1. Initial beam trial size:

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

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.

\[ \Delta_t = \Delta_b + \Delta_s + \Delta_{ns}. \]

Assume shear and nail slip deflection are 75% of bending deflection.

\[ \Rightarrow \Delta_t = 1.75 \times \Delta_b. \]

For a simply supported beam with a central point load:

\[ \Delta_b = \frac{j_2 \times PL^3}{48(El)x} \]

& \[ \Delta_s \geq 1.75 \frac{j_2 \times PL^3}{48(El)x} \]

\[ F = \frac{L^3}{48} \]  where \( F \) = flexibility co-efficient

\[ \Rightarrow \Delta_s = 1.75 \times j_2 \times P \cdot F \]

\[ \Rightarrow F_{(max)} \leq \Delta_t / 1.75j_2^2 P \]

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Load (kN)</th>
<th>Deflection limit (mm)</th>
<th>( j_2 )</th>
<th>( \Delta_{max}/(1.75 \times j_2 \times P)(\text{mm/kN}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>10.8</td>
<td>30</td>
<td>2</td>
<td>0.79</td>
</tr>
<tr>
<td>LL</td>
<td>16.2</td>
<td>30</td>
<td>1</td>
<td>1.06</td>
</tr>
<tr>
<td>WL</td>
<td>-33.0</td>
<td>50</td>
<td>1</td>
<td>0.87</td>
</tr>
</tbody>
</table>

Rightarrow 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, \( & F = 0.82 \text{ mm/kN} \), gives a 900mm deep trial box beam with 150 x 35 LVL flanges and 7mm F11 structural plywood webs.

Beam Capacities
Box Beam: 900 mm deep x 150 x 35 LVL Flanges x 7 mm F11 structural plywood webs.

Capacities for the various beam actions have been extracted from Table A8.7 and are given in TABLE 8.2.

<table>
<thead>
<tr>
<th>Moment Capacity - Tension Flange</th>
<th>( 2f'<em>t(El)x</em>{an}/E_t \cdot d = 161 \text{ kNm} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment Capacity - Compression Flange</td>
<td>( 2f'<em>c(El)x</em>{an}/E_t \cdot d = 242 \text{ kNm} )</td>
</tr>
<tr>
<td>Web Shear</td>
<td>( f'<em>s(El)x</em>{an}. n.tw/(EQ)x_{an} = 59.3 \text{ kN} )</td>
</tr>
<tr>
<td>Web Shear at splice (only one web continuous)</td>
<td>( f'<em>s(El)x</em>{an}. n.tw/(EQ)x_{an} = 30.7 \text{ kN} )</td>
</tr>
<tr>
<td>Unit Shear Flow for Nail Connection</td>
<td>( (EQ)_{sd}/(El)_s = 0.94 \times 10^{-3} \text{ mm}^{-1} )</td>
</tr>
</tbody>
</table>

TABLE 8.2 : Beam Capacities
Check factored loads and critical load cases:

<table>
<thead>
<tr>
<th>Loads</th>
<th>Factored load combinations for strength limit states</th>
<th>Load combination value (kN)</th>
<th>k₁</th>
<th>V⁺ (kN)</th>
<th>V⁺/k₁ (kN)</th>
<th>M⁺ (kNm)</th>
<th>M⁺/k₁ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>1.25G</td>
<td>15.8</td>
<td>0.57</td>
<td>7.9</td>
<td>13.8</td>
<td>42.6</td>
<td>74.7</td>
</tr>
<tr>
<td>DL + LL</td>
<td>1.25G + 1.5Q</td>
<td>40.1</td>
<td>0.97</td>
<td>20.0</td>
<td>20.7</td>
<td>108.2</td>
<td>111.5</td>
</tr>
<tr>
<td>DL + WL</td>
<td>0.8G + Wu</td>
<td>-47.7</td>
<td>1.15</td>
<td>-23.8</td>
<td>-20.7</td>
<td>-128.8</td>
<td>-112.0</td>
</tr>
</tbody>
</table>

2. Flange Bending Capacity

Check Tension Flange

\[ M^* / \phi k_1 k_4 k_6 k_{11} = 128.8 / (0.85 \times 1.15 \times 1 \times 1) \]
\[ = 131.8 \text{ kNm} \ (< 161 \text{ kNm}) \text{ OK} \]

Check Compression Flange

Calculate \( k_{12} \):

\[ S_1 = \left[ \frac{(5.3 L_\alpha / (E I_s)) / (h_1 D (E I_s))}{(5.5 x 5400 x 31894 x 10^9) / (5.5 x 900 x 803.7 x 10^9)} \right]^{0.5} \]
\[ = 1.12 \]
\[ k_{12} = 1.5 - 0.05 \rho S_1 \]
\[ = 1.5 - 0.05 \times 1.12 \times 15.2 \]
\[ = 0.65 \]

\[ M^*/\phi k_1 k_4 k_6 k_9 k_{11} k_{12} = 128.8 / (0.85 \times 1.15 \times 1 \times 1 \times 1 \times 0.65) \]
\[ = 203 \text{ kNm} \ (< 242 \text{ kNm}) \text{ OK} \]

3. Panel Shear Capacity

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

\[ V^*/\phi k_1 k_{12} k_{19} g_{19} = 23.8 / (0.8 \times 1.15 \times 0.52 \times 1 \times 1) \]
\[ = 49.7 \text{ kN} \ (< 59.3 \text{ kN}) \text{ OK} \]

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

\[ V^*_{\text{(at splice)}} / \phi k_1 k_{12} k_{19} g_{19} = 23.8 / (0.8 \times 1.15 \times 1 \times 1) \]
\[ = 26 \text{ kN} \ (< 30.7 \text{ kN}) \text{ OK} \]

4. Flange - Web Connection

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

Check critical load case for fasteners:
Required nail spacing \( s = \phi N_j / q \). Characteristic capacity, \( Q_k \), of 2.8mm nail in JD4 timber is 665 N

\[
\begin{align*}
\text{Design capacity per nail } \phi N_j &= \phi k_1 k_2 k_3 k_4 Q_k \\
&= 0.8 \times 0.86 \times 1 \times 1 \times 1.1 \times 1 \times 665 \\
\phi N_j &= 503 \text{ N/nail}
\end{align*}
\]

Shear flow at connection:

\[
\begin{align*}
q &= V*(EQ) / (EI)_x, n \\
&= (20 \times 10^3 \times 0.94 \times 10^{-3}) / 2 \\
q &= 9.4 \text{ N/mm}
\end{align*}
\]

\[
\Rightarrow s_{\text{max}} = \frac{503/9.4}{53 \text{ mm}} \\
\Rightarrow \text{Use } 2.8 \phi \text{ nails at 50mm spacing}
\]

5. Beam stiffness

For a single span, simply supported beam:

<table>
<thead>
<tr>
<th>Deflection type</th>
<th>Estimated mid-span deflection due to a centre point load, ( P )</th>
<th>Estimated mid-span deflection due to uniformly distributed load, ( w )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>( j_2 \times PL^2/48(EL)_x )</td>
<td>( j_2 \times 5wL^4/384(EL)_x )</td>
</tr>
<tr>
<td>Shear</td>
<td>( j_2 \times PL/4GA_s )</td>
<td>( j_2 \times wL/8GA_s )</td>
</tr>
<tr>
<td>Nail slip*</td>
<td>( \frac{dL^3}{64} \left[ \frac{E_f A_s \cdot s \cdot P}{(EL)_x A} \right]^2 )</td>
<td>( \frac{dL^3}{192} \left[ \frac{E_f A_s \cdot s \cdot w}{(EL)_x A} \right]^2 )</td>
</tr>
</tbody>
</table>

*Refer Chapter 8 Appendix for nail slip deflection equations

<table>
<thead>
<tr>
<th>Load Type</th>
<th>( j_2 )</th>
<th>Load</th>
<th>Estimated Deflection, mm</th>
<th>Total (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>2</td>
<td>10.8</td>
<td>19.5</td>
<td>31.2</td>
</tr>
<tr>
<td>LL</td>
<td>1</td>
<td>16.2</td>
<td>12.5</td>
<td>20.5</td>
</tr>
<tr>
<td>WL</td>
<td>1</td>
<td>-33</td>
<td>25.4</td>
<td>43.7</td>
</tr>
</tbody>
</table>

So the designer can go with the initial trial beam selection or make some adjustments to the design parameters, e.g. by choosing a 900 deep box beam with 150 x 45 LVL flanges.

8.5 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 stand alone 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 8.3 : Shows the two plywood only dependent joints

FIGURE 8.4 : Shows the bolted and rodded joints
Figure 8.5 shows a discontinuous plywood joint located in the Loading Frame prior to testing.

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.

<table>
<thead>
<tr>
<th>PORTAL JOINTS – TP1 AND TP2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plywood</td>
</tr>
<tr>
<td>Framing</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Nailing</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Tie down Bolts</td>
</tr>
</tbody>
</table>

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.

<table>
<thead>
<tr>
<th>Plywood</th>
<th>– 7mm DD F8 (minimum)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framing</td>
<td>– 90 x 45 MGP10, JD5 (minimum)</td>
</tr>
<tr>
<td></td>
<td>Framing procedures to follow those given in Figure 5 of the Research Proposal in Appendix A1</td>
</tr>
<tr>
<td>Nailing</td>
<td>– 2.8 Ø x 35 long galvanised clouts</td>
</tr>
<tr>
<td></td>
<td>– 3.15 Ø x 75 bullet head nails (1 off into end grain of stiffeners and studs)</td>
</tr>
<tr>
<td></td>
<td>– 50 / 50 nailing pattern</td>
</tr>
<tr>
<td>Tie down Bolts</td>
<td>– M12 with nuts and 50 x 50 x3 washers</td>
</tr>
</tbody>
</table>

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.

<table>
<thead>
<tr>
<th>Plywood</th>
<th>– 7mm DD F8 (minimum)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framing</td>
<td>– Framing procedures to follow those given in Figure 6 of the Research Proposal in Appendix A1</td>
</tr>
<tr>
<td>Nailing</td>
<td>– 2.8 Ø x 35 long galvanised clouts</td>
</tr>
<tr>
<td></td>
<td>– 3.15 Ø x 75 bullet head nails (1 off into end grain of stiffeners and studs)</td>
</tr>
<tr>
<td></td>
<td>– 50 / 50 nailing pattern</td>
</tr>
<tr>
<td>Tie down Bolts</td>
<td>– By means of the rods</td>
</tr>
</tbody>
</table>

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.
Chapter 8 Appendix

**Bending / Compressive Strength Stability Factor $k_{12}$**

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

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

$$S_1 = \left(\frac{5.3 \cdot L_{ay} \cdot (EI)_x}{h_1 \cdot d \cdot (EI)_y}\right)^{0.5}$$  \hspace{1cm} (A8.E1)

where:

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

<table>
<thead>
<tr>
<th>Moment parameter $\beta$ (see diagram below)</th>
<th>Slenderness factor $h_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Free restraint condition*</td>
</tr>
<tr>
<td>1.0</td>
<td>3.1</td>
</tr>
<tr>
<td>0.5</td>
<td>4.1</td>
</tr>
<tr>
<td>0.0</td>
<td>5.5</td>
</tr>
<tr>
<td>-0.5</td>
<td>7.3</td>
</tr>
<tr>
<td>-1.0</td>
<td>8.0</td>
</tr>
</tbody>
</table>

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

**TABLE A8.1 : Reproduced from Table E6, AS 1720.1-1997**
FIGURE A8.1: Lateral Buckling Terminology

Formula A8.E1 is accurate to within approximately 10% and is based on the use of 3 ply webs and the following assumptions:
\[ d = 1.1(d-t_f) \]
\[ b_f + t_w = 1.08b_f \]
\[ (E I)_x = 1.25E_t I_{xf} \]
\[ (E I)_y = 1.6E_w I_{yw} \]

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

Nail Slip Deflection Equations

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

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

For a central concentrated load \(P\): \(\Delta_{ns} = \frac{d_n L}{64} \left[ \frac{E_f A_f s P}{(E I)_x A} \right]^2 \)

For a UDL of \(w/\text{unit length}\): \(\Delta_{ns} = \frac{d_n L^3}{192} \left[ \frac{E_f A_f s W}{(E I)_x A} \right]^2 \)

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
\((E I)_x\) = 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:

\[
\Delta_{ns} = \frac{d_n \cdot L}{64} \left[ \frac{E f A_f \cdot s \cdot P}{(E I) x A} \right]^2
\]

\[
= \frac{865 \times 10800 x 13200 x(150 \times 35) x 50 x 16200}{64 \times 31894 \times 10^9 \times 21669} \ m
\]

\[ \Delta_{ns} = 0.001 \ m \]

where:
\[ A = 4.767 h_{32} D^{1.75} \]
\[ = 4.767 \times 750 \times 2.8^{1.75} \]
\[ = 20574 \ \text{Nm}^{0.5} \]

<table>
<thead>
<tr>
<th>Initial Moisture Condition</th>
<th>Species joint group</th>
<th>Factor $h_{32}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unseasoned</td>
<td>J1</td>
<td>1450</td>
</tr>
<tr>
<td></td>
<td>J2</td>
<td>1050</td>
</tr>
<tr>
<td></td>
<td>J3</td>
<td>750</td>
</tr>
<tr>
<td></td>
<td>J4</td>
<td>550</td>
</tr>
<tr>
<td></td>
<td>J5</td>
<td>410</td>
</tr>
<tr>
<td></td>
<td>J6</td>
<td>300</td>
</tr>
<tr>
<td>Seasoned</td>
<td>JD1</td>
<td>1600</td>
</tr>
<tr>
<td></td>
<td>JD2</td>
<td>1250</td>
</tr>
<tr>
<td></td>
<td>JD3</td>
<td>990</td>
</tr>
<tr>
<td></td>
<td>JD4</td>
<td>750</td>
</tr>
<tr>
<td></td>
<td>JD5</td>
<td>590</td>
</tr>
<tr>
<td></td>
<td>JD6</td>
<td>470</td>
</tr>
</tbody>
</table>

TABLE A8.2 : Stiffness Factor $h_{32}$ for Nailed and Screwed Joints in Solid Timber
### Table A8.3: Duration Factors $j$12 and $j$13

<table>
<thead>
<tr>
<th>Initial moisture condition</th>
<th>Duration of load</th>
<th>Factor $j$12</th>
<th>Factor $j$13</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unseasoned</td>
<td>More than 3 years</td>
<td>9</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>5 months</td>
<td>4</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>Less than 2 weeks</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Seasoned</td>
<td>More than 3 years</td>
<td>4</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Less than 2 years</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

**NOTE:** If required, intermediate values of $j$12 and $j$13 may be obtained by linear interpolation with log-time.

### Panel Shear Slenderness Co-efficient, $S$ and Stability Factor $k_{12}$ for Edge Shear Forces

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

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

<table>
<thead>
<tr>
<th>Beam Depth (mm)</th>
<th>(a) Span/Depth ratio</th>
<th>(b) Depth/breadth ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>25 : 1</td>
<td>10 : 1</td>
</tr>
<tr>
<td></td>
<td>Very lightly loaded beams eg purlins</td>
<td>Optimum range</td>
</tr>
<tr>
<td></td>
<td>25 : 1</td>
<td>10 : 1</td>
</tr>
<tr>
<td></td>
<td>18 : 1</td>
<td>10 : 1</td>
</tr>
<tr>
<td></td>
<td>5 : 1</td>
<td>10 : 1</td>
</tr>
<tr>
<td></td>
<td>10 : 1</td>
<td>4.5 : 1</td>
</tr>
<tr>
<td>SPAN (m)</td>
<td>23</td>
<td>50</td>
</tr>
<tr>
<td>BREADTH (mm)</td>
<td>30</td>
<td>67</td>
</tr>
<tr>
<td>225</td>
<td>5.6</td>
<td>4.1</td>
</tr>
<tr>
<td>300</td>
<td>7.5</td>
<td>5.4</td>
</tr>
<tr>
<td>400</td>
<td>10.0</td>
<td>7.2</td>
</tr>
<tr>
<td>450</td>
<td>11.3</td>
<td>8.1</td>
</tr>
<tr>
<td>600</td>
<td>15.0</td>
<td>10.8</td>
</tr>
<tr>
<td>900</td>
<td>22.5</td>
<td>16.2</td>
</tr>
<tr>
<td>1200</td>
<td>30.0</td>
<td>21.6</td>
</tr>
</tbody>
</table>

**TABLE A8.4:** (a) Span/Depth – (b) Depth/Breadth
FIGURE A8.2: Guide for Selecting Initial Beam Depth
Table A8.5: Unit-Load-Deflection Span Tables for a Simply Supported Box Beam with a unit Centre Point Load, \( P = 1 \text{kN} \)

<table>
<thead>
<tr>
<th>Beam Component</th>
<th>Material</th>
<th>Characteristic Strength (Mpa)</th>
<th>Short Duration Average Moduli (Mpa)</th>
<th>Density (kg/m³)</th>
<th>Strength Group</th>
<th>Nominal web thickness, ( t_w ) (mm)</th>
<th>Number of veneers</th>
<th>Number of webs</th>
<th>K34</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flanges</td>
<td>LVL</td>
<td>40</td>
<td>33</td>
<td>53</td>
<td>45</td>
<td>13200</td>
<td>660</td>
<td>620</td>
<td>JD4</td>
</tr>
<tr>
<td></td>
<td>F11 Plywood</td>
<td>35</td>
<td>20</td>
<td>53</td>
<td>25</td>
<td>10500</td>
<td>525</td>
<td>550</td>
<td>JD4</td>
</tr>
<tr>
<td>Webs</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.30</td>
<td>3.00</td>
<td>2.00</td>
<td>0.87</td>
<td></td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Beam Span (m)</th>
<th>Beam deflection per unit kN load (mm/kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.25</td>
<td>0.06</td>
</tr>
<tr>
<td>2.50</td>
<td>0.06</td>
</tr>
<tr>
<td>3.00</td>
<td>0.10</td>
</tr>
<tr>
<td>3.50</td>
<td>0.09</td>
</tr>
<tr>
<td>4.00</td>
<td>0.08</td>
</tr>
<tr>
<td>4.50</td>
<td>0.08</td>
</tr>
<tr>
<td>5.00</td>
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</tr>
<tr>
<td>5.50</td>
<td>0.06</td>
</tr>
<tr>
<td>6.00</td>
<td>0.06</td>
</tr>
<tr>
<td>6.50</td>
<td>0.05</td>
</tr>
<tr>
<td>7.00</td>
<td>0.05</td>
</tr>
<tr>
<td>7.50</td>
<td>0.05</td>
</tr>
<tr>
<td>8.00</td>
<td>0.05</td>
</tr>
<tr>
<td>8.50</td>
<td>0.05</td>
</tr>
<tr>
<td>9.00</td>
<td>0.05</td>
</tr>
<tr>
<td>9.50</td>
<td>0.05</td>
</tr>
<tr>
<td>10.0</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Bending deflection \( \Delta_b \) per unit load \( P = 1 \text{kN} \) is given by

\[ \Delta_b = \frac{pL^3}{48EI_x} \text{mm/kN} \]
Table A8.6: Unit-Load Deflection Span Tables for a Simply Supported Box Beam with a Uniformly Distributed Load, \( w = 1 \text{ kN/m} \)

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

<table>
<thead>
<tr>
<th>Beam Component</th>
<th>Material</th>
<th>Characteristic Strength (Mpa)</th>
<th>Short Duration Average Moduli (Mpa)</th>
<th>Density (kg/m³)</th>
<th>Strength Group</th>
<th>Nominal web thickness, tw (mm)</th>
<th>Number of veneers</th>
<th>Number of webs</th>
<th>K34</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flanges</td>
<td>LVL</td>
<td>( \gamma' ) 46</td>
<td>( \gamma'' ) 33</td>
<td>( \gamma''' ) 5.3</td>
<td>( \gamma'''' ) 45</td>
<td>13200</td>
<td>860</td>
<td>620</td>
<td>JD4</td>
</tr>
<tr>
<td>Webs</td>
<td>F11 Plywood</td>
<td>35</td>
<td>20</td>
<td>5.3</td>
<td>25</td>
<td>10500</td>
<td>525</td>
<td>550</td>
<td>JD4</td>
</tr>
</tbody>
</table>

**Simply supported beam with a uniformly distributed unit load of \( w = 1 \text{ kN/m} \)**

<table>
<thead>
<tr>
<th>Beam Span (m)</th>
<th>Beam deflection per unit load, ( w = 1 \text{ kN/m} ) (mm/kN/m)</th>
<th>W=1 (kNm)</th>
<th>Bending deflection ( \Delta_b ) per unit load ( w = 1 \text{ kN/m} ) = ( \frac{5L^4}{384EI_L} ) mm/kNm</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Beam Span (m)</th>
<th>Beam deflection per unit load, ( w = 1 \text{ kN/m} ) (mm/kN/m)</th>
<th>W=1 (kNm)</th>
<th>Bending deflection ( \Delta_b ) per unit load ( w = 1 \text{ kN/m} ) = ( \frac{5L^4}{384EI_L} ) mm/kNm</th>
</tr>
</thead>
</table>

**Diagram of Simply Supported Beam**

- L: Beam Span
- W: Uniformly Distributed Load
- \( \Delta_b \): Bending Deflection
Table A8.7: Section Properties and Beam Capacities – Plywood Box Beam with Structural LVL Flanges and 7mm Thick Structural Plywood Webs

<table>
<thead>
<tr>
<th>Box Beam Component</th>
<th>Material</th>
<th>MDES Mpa</th>
<th>MDR Mpa</th>
<th>T'Mpa</th>
<th>F'Mpa</th>
<th>I' Mm4</th>
<th>Section Group</th>
<th>Joint</th>
<th>Plywood Webs:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flanges</td>
<td>LVL</td>
<td>13200</td>
<td>880</td>
<td>40</td>
<td>33</td>
<td>5.3</td>
<td>152</td>
<td>E4</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>F11 Plywood</td>
<td>16526</td>
<td>526</td>
<td>35</td>
<td>20</td>
<td>5.3</td>
<td>152</td>
<td>E4</td>
<td>7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Characteristic Properties</th>
<th>Plywood Web (mm²)</th>
<th>LVL Flanges</th>
<th>Plywood Webs</th>
<th>Section Properties</th>
<th>About X Axis</th>
<th>About Y Axis</th>
<th>Beam Capacities</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Width</td>
<td>Depth</td>
<td>d/B ratio</td>
<td>Area web A_w</td>
<td>Area flange A_f</td>
<td>Volumetric Self Weight kg/m³</td>
<td>Ix x 10⁸</td>
</tr>
<tr>
<td></td>
<td>thickness</td>
<td>height</td>
<td></td>
<td>mm²</td>
<td>mm²</td>
<td>m³</td>
<td>mm²</td>
</tr>
<tr>
<td>125</td>
<td>93</td>
<td>35</td>
<td>4410</td>
<td>5150</td>
<td>2600</td>
<td>2.9</td>
<td>5.1</td>
</tr>
<tr>
<td>125</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>3150</td>
<td>2620</td>
<td>2.8</td>
<td>5.8</td>
</tr>
<tr>
<td>300</td>
<td>93</td>
<td>45</td>
<td>4410</td>
<td>4200</td>
<td>3710</td>
<td>3.9</td>
<td>5.8</td>
</tr>
<tr>
<td>300</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>3150</td>
<td>2620</td>
<td>2.8</td>
<td>5.8</td>
</tr>
<tr>
<td>450</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>5660</td>
<td>5110</td>
<td>2.9</td>
<td>8.7</td>
</tr>
<tr>
<td>450</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>5660</td>
<td>4970</td>
<td>2.5</td>
<td>7.5</td>
</tr>
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<td>93</td>
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<td>5670</td>
<td>5660</td>
<td>4970</td>
<td>2.5</td>
<td>7.5</td>
</tr>
<tr>
<td>450</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>5660</td>
<td>4970</td>
<td>2.5</td>
<td>7.5</td>
</tr>
<tr>
<td>450</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>5660</td>
<td>4970</td>
<td>2.5</td>
<td>7.5</td>
</tr>
<tr>
<td>450</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>5660</td>
<td>4970</td>
<td>2.5</td>
<td>7.5</td>
</tr>
<tr>
<td>450</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>5660</td>
<td>4970</td>
<td>2.5</td>
<td>7.5</td>
</tr>
<tr>
<td>450</td>
<td>93</td>
<td>45</td>
<td>5670</td>
<td>5660</td>
<td>4970</td>
<td>2.5</td>
<td>7.5</td>
</tr>
</tbody>
</table>

**Work Example**

Flange Bending Capacity (tension flange) = \( \frac{2f_f}{E_f} \) \( \frac{M_{y}}{d} \)

Flange Bending Capacity (comp flange) = \( \frac{2f_f}{E_f} \) \( \frac{M_{y}}{d} \)

Panel Shear Capacity (Max) = \( \frac{2f_f}{E_f} \) \( \frac{M_{y}}{d} \)
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.

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

**Shearwalls** may also act as:

- 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:

- 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.

![Diagram showing plywood panel subjected to shear](image)

**FIGURE 9.3:** Shows panel subjected to shear

From statics:

\[
\begin{align*}
\sum F_x &= 0 : S_A = S_R \\
\sum F_y &= 0 : C_s = C_s \text{ (complimentary shears)} \\
\sum M_o &= 0 = S_A \cdot h - C_s \cdot \ell \\
S_A / \ell &= C_s / h
\end{align*}
\]

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

**NOTE:**

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

### 9.3 Diaphragm Design – Diaphragm Action

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

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

Also, the **chord members** are responsible for transfer of **bending moments**, acting in **uniaxial tension** and **compression**. Chord members must, however, be **continuous** over the length of the diaphragm. The advent of LVL, being available in long straight lengths has reduced the need for incorporating **spliced joints** along the chord lengths. Chord members of plywood sheathed, timber framed diaphragms are not restricted to timber members. They could also be the face of a concrete or masonry wall, a reinforced or masonry beam or a steel beam.
The recommended maximum span to depth ratio for plywood systems blocked or unblocked is 4 : 1.

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

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

Figure 9.4: Diaphragm design formula for lateral loading

**9.4 Diaphragm Design – Methodology**

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

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

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

   ![Wind forces on diaphragm](image)

   **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**

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
   
   \[ P = 0.6 \left( V_{des,0} \right)^2 C_{fig} C_{dyn} \]

   Building orthogonal design wind speeds
   
   \[ V_{des,0} = V_{sit,li} \]

   Site wind speeds
   
   \[ V_{sit,li} = V_R M_d (M_z \text{-cat} M_y M_t) \]

   Regional 3 s gust wind speed for annual probability of exceedance of 1/R.
   
   \[ V_{1000} = 60 \text{ m/s} \]

   Regional wind speed for serviceability calculations
   
   \[ V_{20} = 38 \text{ m/s} \]

   Wind directional multiplier
   
   \[ M_d = 0.95 \]

   Terrain and height multiplying factor
   
   \[ M_s(5.4,2.5) = 0.84 \]

   Shielding multiplier, Table 3.2.7, AS 1170.2
   
   \[ M_i = 1 \]

   Topographic multiplier, Table 3.2.8, AS1170.2
   
   \[ M_t = 1 \]

   Dynamic response factor
   
   \[ C_{dyn} = 1 \]

   Aerodynamic shape factor
   
   \[ C_{fig} = \]

   For external pressures
   
   \[ C_{fig} = C_{p,e} K_a K_c K_e K_p \]

   For internal pressures
   
   \[ C_{fig} = C_{p,i} K_c \]

   External pressure co-efficient, windward wall
   
   \[ C_{p,e} = 0.7 \]

   Internal pressure co-efficient, windward wall
   
   \[ C_{p,i} = -0.65 \]

   Area reduction factor for roofs and side walls
   
   \[ K_a = 1 \]

   Combination factor
   
   \[ K_c = 1 \]

   Local pressure factor for cladding
   
   \[ K_l = 1 \]

   Reduction factor for permeable cladding
   
   \[ K_p = 1 \]

   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

\[ V_{des,li} = 60 \times 0.95 \times 0.84 \times 1 \times 1 = 47.9 \text{ m/s} \]

Design wind pressure for ultimate limit states strength

\[ P = 0.6 \times 47.9^2 \times 10^{-3} (0.7 + 0.65) = 1.86 \text{kPa} \]
Wind force, $w$ on diaphragm (half of wind load on 5.4 m wall is transferred directly to foundations)

Total wind force, $W$ on roof diaphragm, in North-South direction

$$ W_{N-S} = 5.0 \text{ kN/m} \times 36 \text{ m} = 180 \text{kN} $$

Total wind force, $W$ on roof diaphragm, in East-West direction

$$ W_{E-W} = 5.0 \text{ kN/m} \times 12 \text{ m} = 60 \text{kN} $$

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

Diaphragm design unit shear from Figure 9.4

Diaphragm design unit shear in North-South direction

$$ v_{N-S} = (5.0 \times 36/2)/12 = 7.5 \text{kN/m} $$

Diaphragm design unit shear in East-West direction

$$ v_{E-W} = (5.0 \times 12/2)/36 = 0.83 \text{kN/m} $$

LIMIT STATES SERVICEABILITY:

Design wind speed for limit states serviceability

$$ v_{\text{des,0}} = 38 \times 0.95 \times 0.84 \times 1 \times 1 = 30.3 \text{ m/s} $$

Design wind pressure for limit states serviceability

$$ P = 0.6 \times 30.3^2 \times 10^{-3} \times (0.7+0.65) = 0.74 \text{kPa} $$

Wind force, $w$ on diaphragm (half of wind load on 5.4 m wall is transferred directly to foundations)

$$ w = 0.74 \times (5.4/2) = 2.0 \text{kN/m} $$

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

Diaphragm design unit shear from Figure 9.4

Diaphragm design unit shear in North-South direction

$$ v_{N-S} = (2.0 \times 36/2)/12 = 3.0 \text{kN/m} $$

Diaphragm design unit shear in East-West direction

$$ v_{E-W} = (2.0 \times 12/2)/36 = 0.33 \text{kN/m} $$

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

<table>
<thead>
<tr>
<th>Plywood Thicknesses (mm)</th>
<th>Flathead Nail Size</th>
<th>Min. Nail Penetration into Framing (mm)</th>
<th>Frame width (mm)</th>
<th>Blocked Diaphragm Nail Spacing (mm) at Boundary/Other Edges</th>
<th>Unblocked</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>3.75mm dia x 75mm long</td>
<td>40</td>
<td>75</td>
<td>150/150 100/150 65/100 50/75</td>
<td>Case 1  Case 2 to 6</td>
</tr>
</tbody>
</table>

Factored Limit State Shear Capacities ($\text{kN/m}$) $\times k_1 = 1.14$
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

Shear force in the roof diaphragm in the E-W direction:
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
0.83 kN/m << 4.7 kN/m => Unblocked Case 3
Converted to Limit States Capacity
Conversion factor used was 1.3 i.e. allowable shear capacities were multiplied up by 1.3

<table>
<thead>
<tr>
<th>Minimum Structural Plywood Thickness (mm)</th>
<th>Flathead Nail Size (mm)</th>
<th>Minimum Nail Penetration into Framing (mm)</th>
<th>Minimum Nominal Width of Framing Member (mm)</th>
<th>Factored Limit States shear capacities (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Blocked Diaphragms</td>
</tr>
<tr>
<td>7</td>
<td>2.87 dia. x 50</td>
<td>32</td>
<td>50</td>
<td>150</td>
</tr>
<tr>
<td>9</td>
<td>3.33 dia. x 65</td>
<td>38</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>12</td>
<td>3.75 dia. x 75</td>
<td>41</td>
<td>50</td>
<td>75</td>
</tr>
</tbody>
</table>

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

TABLE 9.1 : Shear capacities in kN/m for Horizontal Plywood Diaphragms
3. Roof Framing and Structural Plywood Diaphragm Layout

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

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

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

$$
Chord Force = C = 1T = \left(\frac{w_1 x}{2d}(L-x)\right)
$$

from Figure 9.4.

<table>
<thead>
<tr>
<th>Wind Direction</th>
<th>Total Diaphragm Design Load (kN/m)</th>
<th>Location of Chord Splice “x” (m)</th>
<th>$L-x$ (m)</th>
<th>Diaphragm width “d” (m)</th>
<th>Chord Force $C$ or $T$ $(w_1 x / 2d)(L-x)$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-S</td>
<td>5</td>
<td>12</td>
<td>24</td>
<td>12</td>
<td>60</td>
</tr>
<tr>
<td>E-W</td>
<td>not applicable – no join</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Design tensile capacity required**

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

where:

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

and:

$$\phi N_t \geq N^*$$

$$\Rightarrow \phi N_t \geq 60$$

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

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

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

**Chord:**

Effective cross-sectional area of LVL:

$$130 \times 45, 2 \text{ layers:}$$

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

$$>1860 \text{ OK}$$

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

**Splice:**

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

$$\phi N_j \geq N^*$$

$$= \phi k_1 k_{16} k_{17} n Q s k$$

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

$$n = 3.7 \text{ use 4 bolts each side}$$

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

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

\[ \Delta_{(\text{diaphragm})} = 5 v L^3/(96EAd) + v L/(4Gt) + 0.188 e_n L + \sum (\Delta_c X)/(2d) \]

where
\[ v = \text{unit shear kN/m} \]
\[ L = \text{diaphragm length (m)} \]
\[ d = \text{diaphragm width (m)} \]
\[ A = \text{area of chord cross-section (mm}^2\text{)} \]
\[ E = \text{Modulus of Elasticity of the chord material (MPa)} \]
\[ G = \text{Modulus of Rigidity of the diaphragm material (MPa)} \]
\[ t = \text{Effective plywood thickness for shear (mm)} \]
\[ e_n = \text{nail deformation (mm) from Table 9.2 at calculated load per nail on perimeter of interior panels, based on shear per meter divided by number of nails per meter. If the nailing is not the same in both directions, use the greater spacing.} \]

\[ \sum (\Delta c X) = \text{sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by the distance (v) of the splices to the nearest support.} \]

\[ \Delta_c = \text{Half the allowable hole tolerance in excess of the bolt diameter M12 bolts : Permitted hole tolerance of + 10\% of bolt diameter} \]

\[ \text{AS1720 cl 4.4.1} \]

Diaphragm deflection in the N-S direction:
\[ v = 7.5 \text{ kN/m} \]
\[ L = 36 \text{ m} \]
\[ b = 12 \text{ m} \]
\[ A = 11700 \text{ mm}^2 \]
\[ E = 13200 \text{ MPa} \]
\[ G = 660 \text{ MPa} \]
\[ t = 7.2 \text{ mm} \]
\[ V_{(3.75)} = 7.5/8 = 0.94 \text{ kN/nail (8 nails per metre)} \]
\[ e_n = 1.194 \text{ mm} \]
\[ \Delta_c = 0.6 \text{ mm} \]
\[ \text{M12 bolts => 10\% of 12 mm = 1.2mm Half of this = 0.6mm} \]
\[ X = 12 \text{ m for chord splice slip} \]

\[ \Delta_{(\text{diaphragm})} = 5 v L^3/(96EAd) + v L/(4Gt) + 0.188 e_n L + \sum (\Delta_c X)/(2d) \]

\[ \Delta_{(\text{bending})} = 5 v L^3/(96 EAd) \]
\[ = (5 \times 7.5 \times 36000^3)/\left(96 \times 13200 \times 11700 \times 12000\right) \]
\[ = 9.8 \text{ mm} \]

\[ \Delta_{(\text{shear})} = v L/(4Gt) \]
\[ = (7.5 \times 36000)/(4 \times 660 \times 7.2) \]
\[ = 14.2 \text{ mm} \]

\[ \Delta_{(\text{nail slip})} = 0.188 e_n L \]
\[ = 0.188 \times 1.194 \times 36 \]
\[ = 8.1 \text{ mm} \]

\[ \Delta_{(\text{chord splice})} = \sum (\Delta c X)/(2d) \]
\[ = (4 \times 0.6mm \times 12m)/(2 \times 12) \]
\[ = 1.2 \text{ mm} \]

\[ \Delta_{(\text{diaphragm})} = 9.8 + 14.2 + 8.1 + 1.2 \]
\[ = 33.3 \text{ mm} \]
<table>
<thead>
<tr>
<th>Load/Nail (N)</th>
<th>Nail Deformation (mm)</th>
<th>2.87</th>
<th>3.3</th>
<th>3.76</th>
</tr>
</thead>
<tbody>
<tr>
<td>267</td>
<td>0.305</td>
<td>0.203</td>
<td>0.152</td>
<td></td>
</tr>
<tr>
<td>356</td>
<td>0.508</td>
<td>0.305</td>
<td>0.254</td>
<td></td>
</tr>
<tr>
<td>445</td>
<td>0.762</td>
<td>0.457</td>
<td>0.330</td>
<td></td>
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<tr>
<td>534</td>
<td>1.143</td>
<td>0.584</td>
<td>0.457</td>
<td></td>
</tr>
<tr>
<td>623</td>
<td>1.723</td>
<td>0.787</td>
<td>0.584</td>
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</tr>
<tr>
<td>712</td>
<td>2.590</td>
<td>1.041</td>
<td>0.737</td>
<td></td>
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<tr>
<td>800</td>
<td>-</td>
<td>1.422</td>
<td>0.940</td>
<td></td>
</tr>
<tr>
<td>890</td>
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<td>1.194</td>
<td></td>
</tr>
<tr>
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<td>-</td>
<td>2.438</td>
<td>1.524</td>
<td></td>
</tr>
<tr>
<td>1068</td>
<td>-</td>
<td>-</td>
<td>1.778</td>
<td></td>
</tr>
</tbody>
</table>

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

Diaphragm – Worked Example 2

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

\[
\frac{6}{4.8} \times 1.5 = 1.9 \text{ kN/m}
\]

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

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 \( 1.2 \times 1.5 = 1.8 \text{ kN} \) cannot be directly transferred;
- hence, a collector member must extend far enough above and below the opening to introduce 0.9 kN into the sheathing;
- a similar member is required on the left side of the opening.
- due to the fundamental relationship, because of the development of the vertical shears of 1.9 kN/m a horizontal shear along the edge of the opening is required for segment equilibrium.
The horizontal force is $1.9 \times 2.4 = 4.6$ 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 kN/m then:

- for the vertical member the distance it must extend above and below the hole is:
  \[ \frac{0.9}{3.5 - 1.9} = 0.56 \text{ m} \]

- for the horizontal member assuming the lower bound the distance is:
  \[ \frac{4.6 \cdot 1}{2 \cdot 3.5 - 1.5} = 1.15 \text{ m} \]

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.

**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.

### Sub-diaphragm shears:

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

$$\Sigma M_{oL} \ldots = 0$$

$$0 = 25 \times 5 + F_{ch} \times 3 \quad 2 \times 5 \times \frac{5}{2}$$

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

$$F_{ch} = 33.3 \text{ kN}$$

Consider the isolated sub-diaphragm of FIGURE 9.10 and taking moments about A:

$$\Sigma M_A = 0$$

$$= R_B \times 5 \quad 33.3 \times 2$$

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

$$R_B = 13.3 \text{ kN}$$

$$R_A = 20 \text{ kN}$$

The resulting shear flows within the sub-diaphragm become:

$$v_1 = \frac{13.3}{1.25}$$

$$= 10.6 \text{ kN/m;}$$

$$v_2 = \frac{20}{1.25}$$

$$= 16 \text{ kN/m;}$$

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

$$S_{oL} = 25 - 5 \times 2$$

$$S_{oL} = 15 \text{ kN}$$

$$S_{oR} = 15 \text{ kN}$$

$$v_{oL} = 15/3$$
\[ \text{or} \quad v_{\text{or}} = 15/5 = 3 \text{ kN/m} \]
FIGURE 9.11 summarizes the shear flows and shows the result of superposing the two effects.

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:

$$\sum M_{OR} = 25 \times 17 + F_{Lc} \times 5 - 2 \times 17 \times \frac{17}{2}$$

$$= 0$$

$$F_{Lc} = \frac{136}{5}$$

$$= 27.2 \text{kN}$$

Sub-diagram forces: From the elevation shown in FIGURE 9.12:
FIGURE 9.12: Shows shear flows and chord forces

\[ \Sigma M_A = 27.2 \times 3.6 - F_{uc} \times 4.8 = 0 \]

\[ F_{uc} = \frac{27.2 \times 3.6}{4.8} \]

\[ = 20.4 \text{kN} \]

\[ \Sigma F_x = 27.2 \quad 20.4 \quad F_A = 0 \]

\[ F_A = 6.8 \text{kN} \]

Resulting shear flows within sub-diaphragm:

- upper section: \( v_u = \frac{20.4}{1.2} \)
  - \( = 17 \text{kN/m} \)

- lower section: \( v_L = \frac{6.8}{1.2} \)
  - \( = 5.7 \text{kN/m} \)

Although the anchorage force at A, directly under the vertical offset is small in this instance (0.12kN) it will be increased by \((25 - 8 \times 2) \times 1.2/5 = 2.2 \text{kN}\) 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.

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:

\[
 k = \frac{c}{h^2} \left[ \frac{l_x l_y}{l_x + l_y} \right] \quad (9.1)
\]

where:

- \( h \) = height of the bracing panel;
- \( l_x \) = second moment of area of the nail group about the X-axis, see FIGURE 9.13;
- \( l_y \) = second moment of area of the nail group about the Y-axis;
- \( \frac{l_x l_y}{l_x + l_y} \) is equivalent to the second moment of area (I) for the panel and can be seen to be entirely dependent upon nailing density;
- \( c \) = takes into account material aspects and is equivalent to the modulus of elasticity (E).
As previously mentioned the shearwall was described as cantilever like but not as a cantilever per se. To further emphasize this point compare the classical stiffness relationship of Equation 9.1 with the stiffness of a simple cantilever beam loaded in flexure, i.e.

\[ k = \frac{3EI}{h^3} \]

where:
- \( E \) is modulus of elasticity of the plywood sheathing;
- \( I \) is the second moment of area of the sheathing;
- \( h \) is the height of the bracing panel.

Rigid body overturning tendencies contribute significantly to the forces required to be resisted by the first (6) or so nails along the bottom plate at the loaded end of a bracing panel.

Incorporation of anti-rotation rods at panel ends eliminates the need for any nails having to accommodate overturning forces, making their full capacity available for shear transfer. This is evident in viewing the bracing capacities of the EWPAA wall panels given in Tables 6 and 8 of the Structural Plywood Wall Bracing Limit States Design Manual. Nailed only has a capacity of 3.4 kN/m and with anti-rotation rod fitted the capacity is 6.4 kN/m.

It should be noted the EWPAA Racking Test Procedure does not incorporate the application of any simulated gravity load from the roof to the top plate. This is not the case for other test procedures, e.g. the American Society for Testing Materials. Reasoning behind the EWPAA testing protocol was that lightweight roofs offered little resistance to wind uplift.

### 9.11 Shearwall Design - Methodology

Generally the design process is straightforward. The steps involved require:

- determining the diaphragm reactions to be transferred to the shearwalls;
- determining the unit shear to be transferred by the shearwalls;
- choosing a suitable structural plywood panel layout and fastener schedule e.g., as per the EWPAA Structural Plywood Wall Bracing Limit States Design Manual. Panel layouts for single wall heights are usually arranged with plywood face grain parallel to the studs. The alternative, with no penalty in shear capacity is for the face grain to be perpendicular to the studs;
- decide if the structural configuration will allow advantage to be taken of:
  - location of return walls,
  - 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.
Shearwalls Worked Example 1 - Accepted Solution

Considers the 600 and 900 lengths of shearwall to act independently of each other. The racking load per panel being in the ratio of their width, i.e. the 600 panel would take 600/1500 of the 4.5 kN (1.8 kN) and the 900 panel would take 900/1500 of 4.5 kN (2.7 kN).

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

\[
\frac{1.8 \times 2.4}{0.6} = 7.6 \text{kN}; \quad \frac{2.7 \times 2.4}{0.9} = 7.6 \text{kN}
\]

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

For the 600 panel to attain 3 kN/m would require it to be fitted with coach screws and washers at its four corners as per Table 9 EWPAA Wall Bracing Limit State Design Manual.

Shearwalls – Worked Example 1 - Alternative Solution:

An alternative approach takes into account the contribution made by the panel under the window. FIGURE 9.15 shows the free body diagram of the shearwall, neglecting the contribution of the section above the window.
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:

\[ \sum M_0 = 1.8 \times 2.4 - C_1 \times 1.6 \]

\[ = 0 \]

\[ C_1 = \frac{1.8 \times 2.4}{1.6} \]

\[ = 2.7 \text{kN} \]

For the section to the right of the opening:

\[ \sum M_A = 2.7 \times 24 - T_1 \times 1.6 \]

\[ T_1 = \frac{2.7 \times 24}{1.6} \]

\[ = 4.05 \text{kN} \]

The shear flows in the various sections of the shearwall are as shown in FIGURE 9.15.

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

**NOTE:**
Should it be considered the section above the opening to be a significant contributor to panel response the analytical difficulties are increased significantly. The situation becomes analogous to that of the large opening in a diaphragm.

### 9.13 Design Example 2 - Shearwalls

FIGURE 9.16 shows a shearwall subjected to a racking load of 12kN applied at top plate level. The resultant unit shear is 0.84 kN/m.

It is required to assess the distribution of timber framing forces and panel shears due to the inclusion of the door opening in the shearwall.

The method of analysis chosen is the Shear Transfer method due to Dean et al.

**Shearwalls - Worked Example 2**

FIGURE 9.17 shows a free body diagram of the sections of shearwall adjacent to the door opening.
The free body diagram shows there are:

8 unknown reactions

However, there are only 3 equations of statics. The problem becomes solvable because it is possible to distribute the shears due to the well behaved response of the nailed sheathing.

As done previously:

i. when length \( a \approx length \ b \) the shears can be distributed in the ratio of panel lengths.

ii. when length \( a \ll length \ b \) 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 :

\[
\begin{align*}
  v_u &= \frac{12}{1.44} \\
  v_u &= 0.84 \text{kN/m}
\end{align*}
\]

horizontal panel forces \( = p_h \)

\( = (0.84 \times 1.2) \) for 1200 wide panels

\( = 1 \text{kN} \)

distributing shears according to(i)

\[
\begin{align*}
  S_L &= \frac{4}{10} \times 12 \\
  S_L &= 4.8 \text{kN} \\
  S_R &= \frac{6}{10} \times 12 \\
  S_R &= 7.2 \text{kN}
\end{align*}
\]

allocation of panel shears, adjacent to opening such that:

\[
\begin{align*}
  \sum p_h \text{ to left} & \approx 4.8 \text{kN} \text{ (actually } 2 \times 1.5 + 2 \times 1 = 5 \text{kN)} \\
  \sum p_h \text{ to right} & \approx 7.2 \text{kN} \text{ (actually } 8 \text{kN)}
\end{align*}
\]
In this instance let the two panels either side of the opening have:

\[ p_n = 1.5 \text{ kN} \]

**NOTE:** The values of \( p_n \) can be of any magnitude usually greater than 1.

The above choice of horizontal panel force results in the accumulation of nail force in the timber framing members shown in FIGURE 9.18.

**FIGURE 9.18:** Accumulation of nail forces

Accumulation of nail forces in the vertical and horizontal framing members is demonstrated by referring to FIGURE 9.18. To do this consider the two 1.5 kN and one 1 kN panels to the left of the door opening.

**FIGURE 9.19:** Shows panel shear flows and resulting nail force accumulation

Summation of the nail forces along the vertical members allows \( V_2, V_3 \), etc. to be evaluated as shown in FIGURE 9.17.

### 9.14 Photographs

Appendix A9 illustrates some practical examples of diaphragms and shear walls.
REFERENCES CITED:


9. **A Number of Reports on Racking Test Results for the PAA**, From the 1970’s to 2005, C G “Mick” McDowall.


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](image)

The plywood or LVL gusseted timber portal, in its multitude of structural forms, provides an excellent solution to a wide range of building requirements. Plywood or LVL gussets nailed to the framing elements at the eaves and ridge of the portal frame are economical, easy to fabricate and provide an effective method of developing moment resisting joints at these locations. The two main design components for structural plywood or LVL gussets are:

- sizing of the structural plywood / LVL gussets;
- design of the nailed connection to transmit the applied column / rafter forces developed at the portal eaves and the rafter / rafter moment joint at the ridge.

10.2 Materials

Portal frame gussets can be fabricated from structural plywood or structural Laminated Veneer Lumber (LVL).

**Structural plywood** produced in Australia and New Zealand is typically manufactured from a pine species; Radiata, Slash or Hoop Pine in Australia and Radiata Pine in New Zealand, with F8, F11 and F14 being the most readily available stress grades for these species. The most common sheet size for structural plywood is 2400 x 1200 mm, but other lengths (2700, 1800) and widths (900) are also available. Suitable face veneer grades for gussets would be DD, or possibly CD where appearance is also a consideration. Structural plywood is available in a range of thicknesses and constructions. 5 TABLE 5.3 in Chapter 5 of this Manual details standard structural plywood thicknesses and constructions. The construction or lay-up of the plywood must be specified in the gusset design, as AS 1720 utilises parallel ply theory in evaluation of the strength capacity of structural plywood loaded in-plane in bending. Therefore, thickness of cross-band veneers is not included in the overall structural plywood thickness used to evaluate gusset strength capacity.

**Structural Laminated Veneer Lumber** manufactured in Australia and New Zealand is typically manufactured from Radiata Pine. Strength capacities for LVL vary from manufacturer to manufacturer with each individual manufacturer publishing their products specific structural properties. Properties are identified by the manufacturer’s brand name and this brand name needs to be included in any specification for structural LVL gussets. Structural LVL is available in widths up to 1200 mm and lengths up to 25m. Veneer grades for LVL are in accordance with the manufacturer’s specification, and are based on structural properties rather than aesthetic considerations. Face veneer grades for LVL would be comparable to a plywood D or C quality face. Typical thicknesses for LVL are 35, 36, 45, 63 and 75 mm. Structural LVL is usually manufactured with no cross-bands, however when used as gussets, a cross-band immediately below the face veneers will improve
“nailability” by increasing resistance to splitting out at edges, ends and between nails. Structural plywood with cross-bands is not standard stock and would need to be ordered in advance. Where cross-band layers are included in structural LVL, parallel ply theory applies, with the cross-band veneer thickness not included in the overall structural LVL thickness used to evaluate gusset strength capacity.

Nails used to fix plywood gussets must be flat head structural nails or clouts. Nail sizes should be specified to suit installation with a nail gun. Hot dipped galvanised nails should be used in areas of high humidity or mildly corrosive environments or where preservative treated plywood, LVL or timber are used as components.

10.3 Plywood / LVL Gusset Design – Gusset Action

Gussets

Joint configuration, i.e. the intersection of the column / rafter members and whether joints are internal or external significantly influences the stress distribution likely to occur across the critical section of a plywood / LVL gusset. Quantifying such distributions has been the result of considerable research effort worldwide.

Mitred Internal Knee Gusset

Irrespective of whether the internal gusseted joint is opening or closing the actual stress distribution will be of the form shown in Figure 10.2(a). The idealised stress distribution is shown in Figure 10.2 (b).

Assuming a balanced gusset construction, i.e. that the depth of the plywood gusset is twice the depth of the column / rafter member, allows the applied moment to be expressed in terms of the gusset strength and cross-sectional geometry. The centroid of the stress distribution is taken to be a distance (D) from the gusset point, along its centreline (critical stress line).

The moment / bending stress relationship developed for the stress distribution of Figure 10.2 closely approximates the classical linear distribution, resulting in the flexure formula.

To obtain the plywood gusset thickness requires manipulation of the relationship, \( f_b = \frac{M}{Z} \), thus:

\[
\frac{6M_p^*}{\Phi k_1 k_{19} g_{19} f_b D} \geq t_{ll}
\]

where:
- \( M^*_p \) = in-plane design moment on joint;
- \( \Phi \) = capacity factor for plywood / LVL;
- \( k_1 \) = duration of load strength modification factor;
- \( k_{19} \) = moisture condition strength modification factor;
- \( g_{19} \) = plywood assembly modification factor;
- \( f_b' \) = characteristic bending strength;
- \( D \) = depth of column / rafter member.
FIGURE 10.2: Mitred internal knee gusset and stress distributions

Internal and External Haunch (Knee) Gussets

The external haunched gusseted joint shown in FIGURE 10.2 provides the attractive alternative of being able to locate the plywood / LVL gusset external to the building.

Comparison of the stress distributions on the critical stress lines for the external gusset (FIGURE 10.3 (b)) and the internal gusset (FIGURE 10.3(a)) show:

- internal stresses $f_1$ on the external joint equal the external stresses $f_1$ on the internal joint;
- likewise for the stresses $f_2$;
- stresses $f_1$ are 2 to 3 times greater than stress $f_2$. 
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:

\[
\begin{align*}
  f_t & = \frac{24Mk(1-k)}{t_1D^2(4k-1)} \\
  f_t & = f_t \left(1-k\right) \frac{1}{k}
\end{align*}
\]  

(10.2)

where:

- \( f_t \) = fibre stress in MPa;
- \( M \) = total applied moment on the joint N-mm;
- \( D \) = depth of column/rafter member in mm;
- \( K \) = \( \frac{(y + D/2)}{(y + d)} \)
- \( t_II \) = effective thickness of plywood

For an internal gusset:

\[
y = \frac{L - D}{1 + (1 - D/2L)\tan \alpha}
\]

(10.3)

where:

- \( \alpha \) = roof slope

For an external gusset:

\[
y = L - D
\]

(10.4)

Ridge (Apex) Mitre Joint

The design procedure is similar to that employed in the design of the mitred internal knee gusset. FIGURE 10.4 shows a ridge joint.
Nail Joint Action

Nail forces are evaluated through application of the classic torsion relationship, i.e. $\tau = Tp/I_p$. FIGURE 10.5 shows a simple nailed joint subjected to the combined torsional moment ($T$) and shear force ($P$). The $i$th nail of the nail group is subjected to a force $p_i$, at a radius $p$ from the centroid $C$ of the group.

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

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

(10.5)

where: $x_i$ and $y_i$ = nail co-ordinates in mm;
$n$ = number of nails in the group.

The polar moment of the nail group is given by:

$$I_p = I_x + I_y$$

(10.6)

where: $I_x = A \sum_{i=1}^{n} y_i^2$ and $I_y = A \sum_{i=1}^{n} x_i^2$

(10.7)

$A$ = nail cross-sectional area in mm$^2$. 

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