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REINFORCEMENT OF TIMBER MEMBERS

Chapter 12.6 | September 2020

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1 INTRODUCTION

The modification of timber members at connections or where they interact with other aspects of a structure often represents the largest challenge in timber design. Timber, as an anisotropic material, has several characteristics that mean if a designer is not careful premature failure may occur.

Perhaps the most significant of these characteristics is that timber has a very low tension strength perpendicular to the grain. If not considered adequately, structures which are relying on the transfer of force through tension perpendicular to the grain direction may fail long before the design member or connection strength is reached.

Because of the low tension perpendicular to grain strength value, splitting failures might occur, which are sudden and can happen without warning. Non-linearity and ductility, both terms most will be familiar with from seismic design, are also desirable in other design situations. Overloaded members or connections with non-linear force-displacement properties show visible signs of stress or large displacement prior to failure. In order to guarantee this behaviour, reinforcement of critical connections or members is often required.

While not as weak or sudden, timber also has a significantly lower strength in compression perpendicular to the grain when compared to the parallel to grain strength. This can create issues in taller buildings with high gravity loads at the bottom plate or at a beam support.

These peculiarities arise from the nature of timber as a natural material, made up of previously living cells and grown rather than mixed or manufactured. These long, tube-like cells are what is typically referred to as the fibres or the grain in wood. Any tension or compression loads applied parallel to the fibres are resisted by the relatively strong cell walls.

Compression loads applied perpendicular to the fibres results in the hollow cells being squashed, which offers low strength and stiffness. The wood normally undergoes large deformations, resulting in permanent timber crushing, which is a ductile failure mode. Tension loads applied perpendicular to the fibres result in the separation of the fibres and a sudden and very brittle failure mode. It is good design practice to avoid loading timber members perpendicular to grain, if this cannot be avoided, stress levels should be kept to a minimum or, as outlined in this guide the timber member should be reinforced.

It is important to note that perpendicular to grain stresses also regularly occur in timber members due to changes in moisture content, often resulting in shrinkage cracks. Although this phenomenon is inherent to wood and moisture deformation should be taken in account during design, care should be taken if members which are stressed in tension perpendicular to grain under normal loading conditions are exposed to moisture fluctuations. For more information on the nature of timber as a material refer to the NZ Wood Design Guide Chapter 1.2: Trees, Timber, Species & Properties.

This guide discusses common design situations where design strength may be compromised and provides reinforcement techniques and design methods to avoid premature failure. Most of these techniques have been taken from international literature and design codes. In some places, techniques use fracture mechanics, a complicated branch of mechanics concerned with the study of the propagation of cracks in materials. This is a growing area of timber design that is particularly useful in situations where large stress or strain concentrations arise, such as close to penetrations or notches as discussed in this guide.

Where additional information is available regarding the proposed techniques, references will allow the user to read further into the specific topic.

The recommendation around the design of reinforcement in timber beams in this design guide has been written in the same format as the new standard for timber design, NZS AS 1720.1. Where appropriate, information explaining the equations included has been provided as commentary.

Where equations are based on empirical formulations, correct units have been provided in the annotation.

When designing timber reinforcement, the tensile strength perpendicular to the grain of the timber member is neglected, hence assuming that the section is already fully cracked. This also implies that any reinforcement will not avoid the cracking of the timber, but rather limit the crack width while assuring a load path is present.
When using this guide for seismic design, engineers should use judgement as to the application of overstrength design forces. For example, where reinforcement is protecting against a brittle failure in a Potential Ductile Element it is correct to design the reinforcement for an overstrength demand.

2 DESIGN SITUATIONS WHICH CAN REQUIRE REINFORCEMENT

This guide will cover a range of situations where stress and strain concentrations can occur requiring reinforcement. A brief introduction of these situations, with additional photographic evidence of the failure, is described below. Although this guide will provide a designer with the ability to calculate whether or not reinforcement is required, as mentioned in Section 1 the failure being protected against can be sudden and as such it is recommended to always reinforce notches and penetrations.

2.1 NOTCHED BEAMS

Perhaps one of the most common situations that occurs in design requiring reinforcement, the notching of beams creates tension stress concentrations as forces flow around the section of the beam that has been removed. These tension stresses can lead to the type of failure as seen in Figure 1.

(Flaig 2013). *(http://innocrosslam.zag.si).*

Figure 1: Failure of a notched beam and CLT panel.

In addition to the stresses created by the notch, tensile stresses perpendicular to the grain generated will be further increased by moisture induced stresses which can be significant due to the exposed end grain.

2.2 JOINTS LOADED PERPENDICULAR TO GRAIN

Another failure created by tension perpendicular to the grain stresses, failure of connections loaded perpendicular to grain as shown in Figure 2 occurs because the force transfer into the section is not high enough up the member. This location of loading cannot activate the full strength of the member and as such can be prone to premature failure.

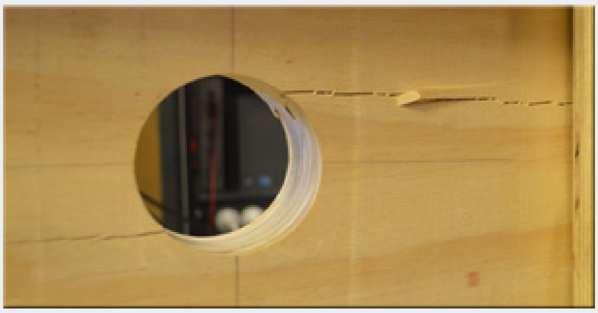
(Photo: Robert Jockwer). *(Photo: Pierre Quenneville, University of Auckland and Pouyan Zarnani, Auckland University of Technology).*

Figure 2: Tension failures at joints loaded perpendicular to grain.

2.3 PENETRATIONS THROUGH BEAMS

Another common design case that can require reinforcement, the drilling of holes in a timber beam for services disrupts the flow of stresses. As the stress flows around the penetration tension perpendicular to grain is created that can lead to failure as shown in Figure 3.

(Ardalany et al. 2012).



(c/o Gary Raftery and Rhonita Andarini, University of Auckland).

Figure 3: Splitting at penetrations through Laminated Veneer Lumber (LVL) beams.

2.4 BEARING REINFORCEMENT

Timber theoretically has unlimited bearing perpendicular to grain strength with the fibres compressing and densifying until the load points creating the stress are in contact. The mobilisation of this strength however comes at very large displacements as shown in Figure 4.

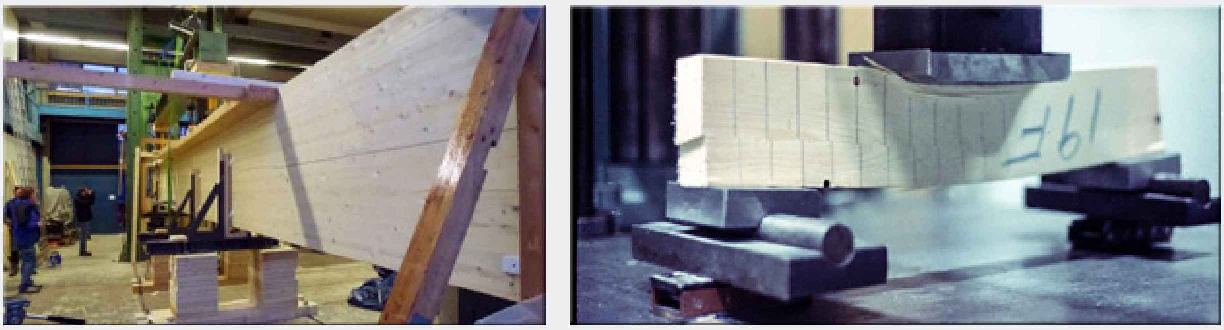
(Holzbau Kompetenzzentren, 2010).

(c/o Andy Buchanan).

Figure 4: Bearing perpendicular to grain.

2.5 SHEAR IN BEAMS

Although not common in timber design, short and deep highly loaded beams can be susceptible to a shear failure as shown in Figure 5.



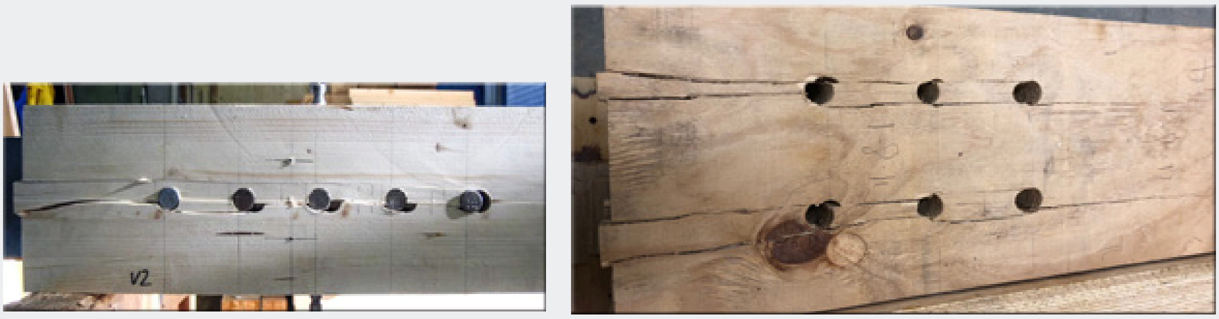
(Bejtka 2014)

(c/o Andy Buchanan).

Figure 5: Shear failure in timber beams.

2.6 CONNECTIONS WITH MULTIPLE LARGE DOWEL TYPE FASTENERS

Under loading all dowel groups will eventually fail in either block tear out or row shear failure. Ideally a large amount of non-linear behaviour will have occurred prior to this point through either yielding of the dowel or timber embedment or both. Figure 6 shows two examples of row shear failure.



(Blaß et al. 2008).

(c/o Pierre Quenneville, University of Auckland and Pouyan Zarnani, Auckland University of Technology).

Figure 6: Splitting and row shear failure in connections with multiple large dowel type fasteners.

2.7 MOMENT RESISTING CONNECTIONS

The tension-compression couple or the direction of the fastener loading in a ring of dowels can create tension perpendicular to grain. This tension can lead to the brittle failure shown in Figures 7 & 8. Protection against this failure mechanism is especially critical if the dowel fasteners are expected to provide in-elastic behaviour as a Potential Ductile Element in a ductile lateral load resisting system.

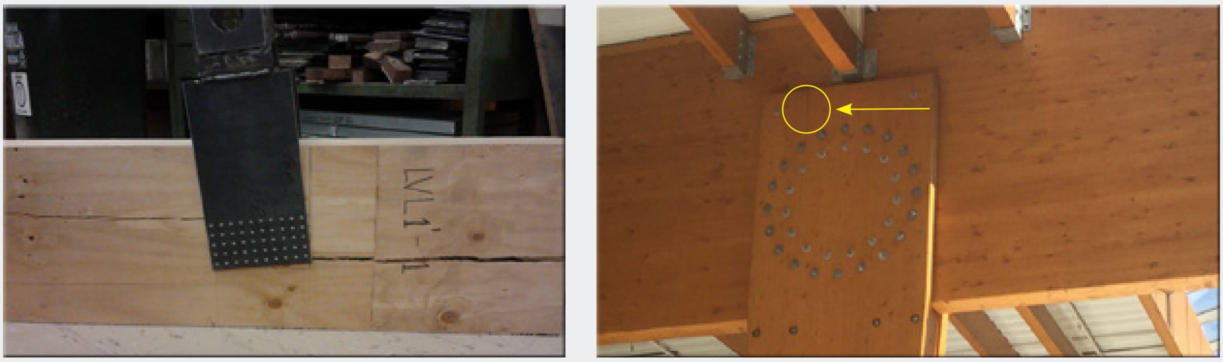


Figure 7: Splitting in moment resisting connections (c/o Pierre Quenneville, University of Auckland and Pouyan Zarnani, Auckland University of Technology).

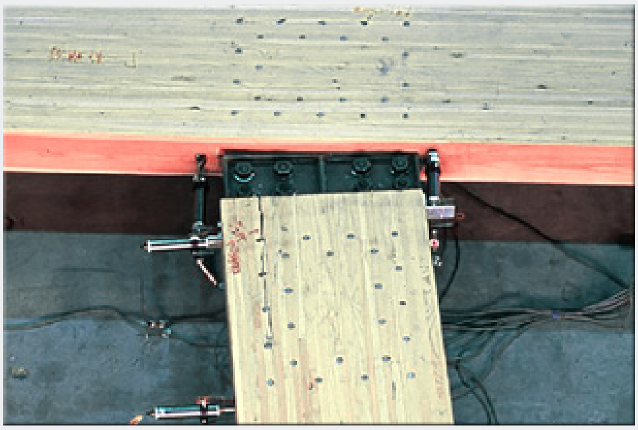


Figure 8: Splitting in moment resisting beam-column joint connection (source unknown).

2.8 MOISTURE DEFORMATION/SPLITTING IN THE PRESENCE OF STEEL PLATES OR GLUED IN RODS

Moisture content changes in timber create swelling and shrinkage (due to the hygroscopic behaviour of wood) that can be especially pronounced perpendicular to the grain. When these movements are blocked by the presence of elements that do not change dimension with moisture change, tensile strain and splitting can occur. Figure 9 and Figure 10 show situations in which this cracking has occurred. While this guide discusses methods for reducing and protecting against this effect, the key to its reduction is adequate moisture management both during manufacture and construction.

Figure 9: Splitting due to shrinkage restrained by steel plates (Brunauer, 2017).

(c/o Andy Buchanan).

(c/o Pierre Quenneville, University of Auckland and Pouyan Zarnani, Auckland University of Technology).

Figure 10: Splitting due to shrinkage restrained by concrete foundation (left) and concealed steel plate (right).

2.9 CURVED, DOUBLE-TAPERED AND PITCHED-CAMBERED BEAMS

While not specifically discussed in this guide, tension forces can also be created by beam geometry. Figure 11 shows cases where failures have been created by beam geometry. Often, this issue is encountered in the retrofit of structures, for more information refer to Sections 13 and 14.

(Winter, 2008).

(Brüninghoff, 2007).

Figure 11: Splitting of curved and pitched-cambered beams.

2.10 GLUED-IN ROD CONNECTIONS

Glue in rods are typically used to transfer high levels of tension and compression force. This concentrated force introduction in a disturbed timber area creates tension stresses that can lead to splitting as shown in Figure 12.

(Deng 1997).

(c/o Gary Raftery and Younes Shirmohammadli, University of Auckland)

Figure 12: Pullout failure of steel rods glued into end grain, caused by splitting .

3 TYPE OF TIMBER REINFORCEMENT

Similarly to reinforcing bars in concrete, timber members can be reinforced in order to provide the required tension and compression strength perpendicular to grain. Due to the difference in stiffness between the timber and the reinforcing (stiffness of the reinforcing element itself or the connection between the reinforcement to the timber), any load in the reinforcement is normally activated only once the tension perpendicular to grain strength in the timber has been exceeded and onset of cracking has occurred. Because of this, the connection between the timber member and the reinforcement should be as stiff as possible, which is normally achieved by gluing or relying on the withdrawal stiffness of a threaded screw. Any reinforcement should also be installed as close as possible to the location where the stress concentration exists and hence onset of cracking will occur.

Traditionally timber has been reinforced with glued-in rods or glued-on wood based panels like plywood. In recent years, the use of fully threaded engineered wood screws has allowed for more cost-efficient reinforcement solutions. These three main types of reinforcement are briefly described in sections 3.1 to 3.3, with section 3.4 mentioning alternative types of reinforcement not further used in this guide.

See Figures 13 to 20 for examples of reinforced timber members.

3.1 FULLY THREADED SCREWS

Fully threaded screws are specifically designed screws used in engineered timber construction and have had extensive use in Europe and North America. Most screws have specific features like self-drilling tips, special coatings for ease of installation, durability and come in a variety of different head types. The biggest difference to traditional wood screws is that the thread extends from the tip all the way to the screw head. Sizes typically vary from diameters ranging from 6mm to 13mm and with lengths up to 1.2 meters or beyond if required.

For applications where longer or larger diameter reinforcement is required, reinforcement rods with a thread specifically designed for use in timber can be used similarly to fully threaded screws. Typically, these rods have diameters in the order of 16 to 20mm and can be more than 2 meters long. Unlike traditional fully threaded rods with washers in oversized holes that transfer load in bearing under the washer head, these bars with wood thread transfer the load through their withdrawal capacity.

These screws and rods are specifically developed to transfer tension loads through their thread withdrawal capacity. When used as reinforcement, they guarantee the tension perpendicular to grain load path across a highly stressed area which is likely to crack during its lifetime due to its low inherent tension perpendicular to grain strength.

Design criteria for these screws can be found in manufacturer's literature and European Technical Assessments (ETA). Typically, only the withdrawal strength and the steel tension strength of the screws needs to be verified. Although differences between the different ETAs exist, all withdrawal strength equations typically have the following form:

$$F_{ax,\alpha,Rk} = \frac{f_{ax,k}\ D\ l_{ef}}{k}\left(\frac{\rho_k}{\rho_a}\right)^{0.8}$$

where

$F_{ax,\alpha,Rk}$	=	characteristic withdrawal capacity of the screw at an angle α to the grain (referred to as Q_k later in this guide)
$f_{ax,k}$	=	characteristic withdrawal parameter
D	=	outer thread diameter
l_{ef}	=	penetration length of the threaded part of the screw (penetration in the member or part of the member to be connected)
k	=	factor accounting for the angle between the grain and the screw axis α (typically 90° when used for reinforcement) typically provided in the manufacturer's literature
ρ_k	=	characteristic density
ρ_a	=	reference density, typically provided in the manufacturer's literature

In cases where head pull through (bearing failure under the head) is of relevance (mostly used for partially threaded screws, but also when there is not sufficient thread penetration), the strength equations typically have the following form:

$$F_{ax,\alpha,Rk} = f_{head,k}\ D_h^2 \left(\frac{\rho_k}{\rho_a}\right)^{0.8}$$

where

$F_{ax,\alpha,Rk}$ = characteristic head-pull through capacity of the screw
$f_{head,k}$ = characteristic head-pull through parameter
D_h = diameter of the screw head or washer

Minimum edge and end distances as called up in this guide vary between the different screw manufacturers and can be found in the screw specific ETAs. Table 1 summarizes key values for the most common screws available in New Zealand at the time of writing and should be used as a guide only. Values provided refer to axially loaded screws only. Always refer to the manufacturer's literature for appropriate edge and end distances for the specific application.

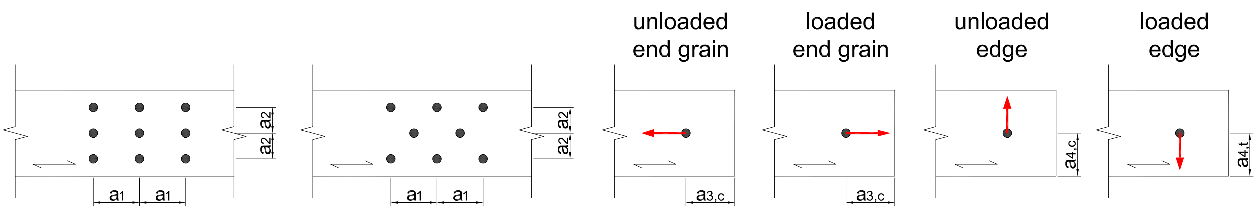
Manufacturer	a_1	a_2	$a_{3,c}$	$a_{4,c}$
Rothoblaas	5D	5D	10D	4D
Spax (with CUT or 4CUT tip)	5D	5D (3D)	5D (2.5D)	3D (2.5D)
Würth	5D	2.5D	5D	3D
Schmidt	5D	5D	5D	4D
Fischer	5D	5D	5D	3D

Table 1: Minimum distances of axially loaded fully threaded screws.

NOTE: Values in parenthesis are for notched supports only

Where

a_1 = spacing parallel to the grain
a_2 = spacing across to the grain
$a_{3,c}$ = distance from centre of the screw-part in timber to the end grain
$a_{4,c}$ = distance from centre of the screw-part in timber to the edge
D = outer thread diameter



For exclusively axially loaded screws, the unloaded end grain distance $a_{3,c}$ and the unloaded edge distance $a_{4,c}$ can be used.

3.2 GLUED-IN ROD CONNECTIONS

Glued-in rods provide a strong and stiff reinforcement solution for timber member and like fully threaded screws have the advantage that they are internal to the timber member and therefore hidden. Installing glued-in rods can be more labour intensive when compared to installing screws, as they require multiple steps for drilling and cleaning the rod hole, drilling the bleeding holes, inserting the bar, sealing off the hole and injecting the glue or epoxy. This operation should only be carried out by experienced timber manufacturers or carpenters in a controlled environment and requires strict adherence to quality assurance protocols.

Glued-in rods are not recommended in structures where timber members will have an in-service moisture content over 20%, which is typically encountered for members which are not covered or otherwise sheltered from the weather.

Guidance on the design of glued-in rods can be found in Chapter 29 of the Timber Design Guideline (Buchanan, 2007) or in the Technical Note 'Design of glued in rods or bars' (Moroder, 2020). If not further specified in this guide, edge and end distances should be taken as per the guidance provided above or as specified by the glue manufacturer's specification.

Additional design information can also be found in the German National Annex to Eurocode 5 (DIN, 2013).

3.3 GLUED-ON WOOD PANELS

Glued-on wood panels are the most effective reinforcement as they provide a very stiff connection and successfully limit the onset of cracking on the outer side of the timber member. The glued-on panels also assure slower moisture transfer between the timber member and the environment, therefore reducing cracks due to moisture deformations. Glued-on wood panels in the form of plywood, standard or cross-banded LVL are common, however Strandboard, Oriented Strand Board (OSB) or particle board are often avoided due to their visual impact on exposed timber members.

The gluing process of the wood panels should only be carried by experienced manufacturers or carpenters in a controlled environment and after all surfaces are adequately prepared. The required clamping pressure for the curing of the glue can be applied by mechanical or hydraulic presses or by screw gluing as described in note 2 in section 4.3.5.

3.4 ALTERNATIVE TYPES OF REINFORCEMENT

Although typically used for retrofit and repair applications, the use of Fibre Reinforced Polymers (FRP) provides very strong and stiff connections. FRP reinforcements tend to be more expensive than other reinforcing solutions and are not visually appealing.

Threaded steel rods with washers in oversized holes in the timber member that transfer the load in bearing under the washer tend to be much more flexible when compared to glued-in rods, due the relaxation of the steel and creep deformation of the timber under the washers. Sufficient stiffness can only be provided if the nuts are retightened regularly during the lifetime of the structure, which is often impracticable or forgotten.

Partially threaded screws can be used instead of fully threaded screws when smaller tension forces need to be transferred. In addition to the withdrawal strength of the thread, the bearing under the screw head needs to be verified.
For this reason, partially threaded screws with larger washer heads are often used. Partially threaded screws can be designed in accordance with NZS AS 1720.1 or the manufacturer's literature and ETAs.

Nailed on metal plates or wooden panels do not provide sufficient connection stiffness to the timber member, resulting in large cracks before they can be engaged. For this reason, it is not recommended to use nailed on metal plates or wooden panels as reinforcement unless for special applications like retrofits or repairs. When using nailed on metal plates or wooden panels, no fastener should be installed in the potential crack position, as this could additionally weaken the section and trigger the onset of cracking.

Although toothed plates provide higher stiffness between the plate and the timber than nailed on plates, these plates are often difficult to install and the full tooth penetration depth often works its way out during the lifetime of the structure due to moisture fluctuations. Toothed plates should not be used in LVL members (due to the high density of the material) and should preferably be mechanically pressed rather than hammered on.

When using nailed on metal plates or wooden panels, or toothed plates, the same design principles as for glued-on plates shall be applied.

(c/o Justin Brown and Minghao Li).

Figure 13: Reinforcement with fully threaded screws.

(c/o Techlam).

Figure 14: Reinforcement with glued in rods.

Figure 15: Reinforcement of penetrations with glued-on wood-based panels (c/o Wiehag).

Reinforcement of a tapered notch with glued-on wood-based panels.

Reinforcement with external Fibre Reinforced Polymer (FRP).

Figure 16: Notch reinforcement (c/o Robert Jockwer).

Bearing reinforcement with fully threaded screws and recess for a steel plate (c/o Robert Jockwer).

Buckling failure of fully threaded screws embedded in timber (Bejtka, 2005).

Figure 17: Bearing reinforcement.

Buckling failure of fully threaded screws embedded in LVL.

Bearing reinforcement of a Pres-Lam column with fully threaded screws.

Figure 18: Bearing reinforcement (c/o Andy Buchanan).

Figure 19: Reinforcement of curved beams with glued-on wood-based panels (c/o Robert Jockwer).

Reinforcement of penetrations with post-installed threaded rods (c/o Robert Jockwer).

Reinforcement with a nailon plate (c/o Andy Buchanan).

Figure 20: Reinforcement to repair existing structures.

The following sections of this Design Guide give recommended design methods for all of the reinforcement options outlined above.

4 NOTCHED BEAMS

4.1 STRENGTH OF NOTCHES

All notches in unseasoned timber or in timber members with an average moisture content over 20% shall be reinforced. For seasoned timber or timber members with an average moisture content below 20% the design strength of notched beams shall be determined with the provisions in section E9 of NZS AS 1720.1.

NOTE: Notches cut on the upper edge opposite that from the support (Figure 22) do not need to be verified with section E9, as both the shear force and the moment are negative in accordance with the sign convention of Figure 21 and Figure 22, creating perpendicular to grain compression stresses, rather than tension stresses. The strength of the beam shall be verified based on the net section properties.

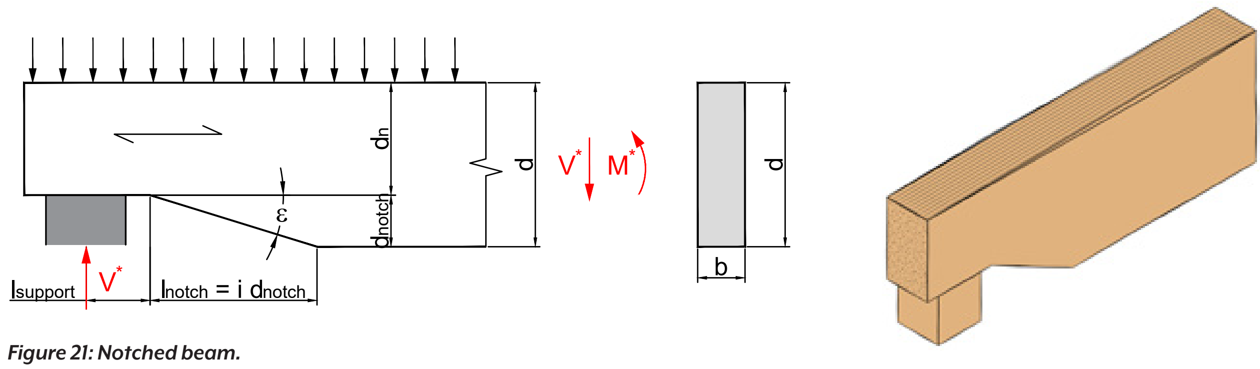


Figure 21: Notched beam.

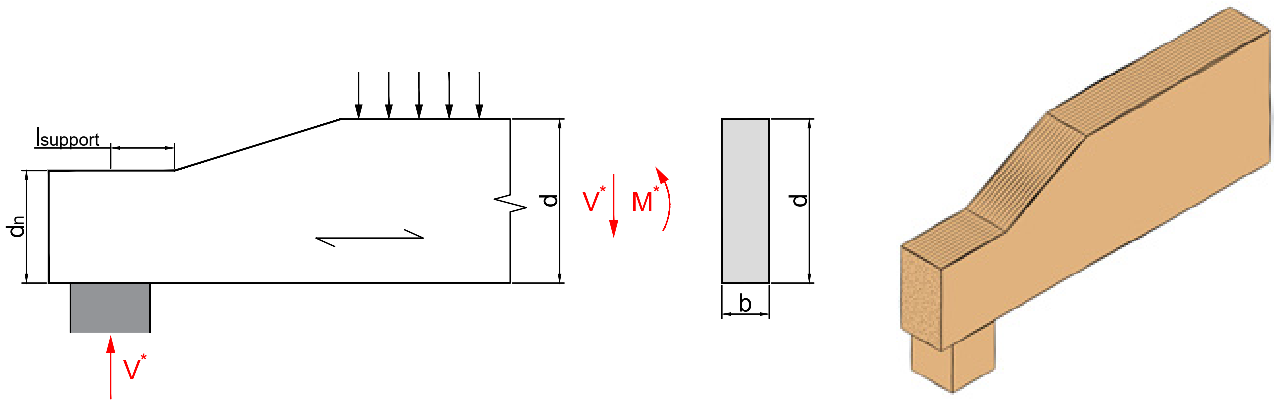


Figure 22: Notched beam cut on the upper edge opposite that from the support.

C4.1

Connections into the end grain of timber members can be verified and reinforced with the same principles as for notched beams with the parameters as per Figure C1.

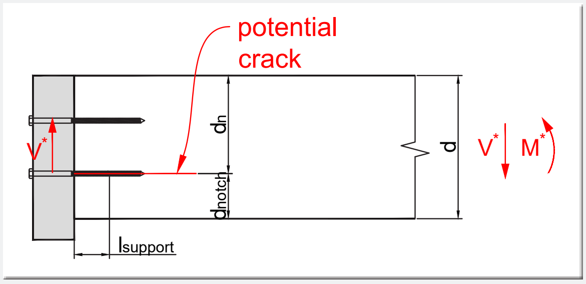


Figure C1: End grain connection verified as a notched beam.

For notches in Cross Laminated Timber (CLT) panels on flat, the effective beam depth d_n used for the verification of notched members needs to consider that transverse laminates (perpendicular to the span direction of the member) do not contribute to the shear strength. Examples of notches with respective effective beam depths d_n are shown in Figure C2. The shear verification at the notch is further reduced because the rolling shear strength, not the shear strength f'_s, needs to be considered. Any opening angle at the notch should be conservatively assumed as 0 (i.e. $l_{notch} = 0$).

Notches in CLT panels can be reinforced as per section 4.3.3, the screw length should be as close as possible to the full panel thickness by still maintaining the minimum edge distance of $a_{3,c}$.

For more information on notches in CLT panels refer to the Commentary to the ETA-06/0138 KLH (2017).

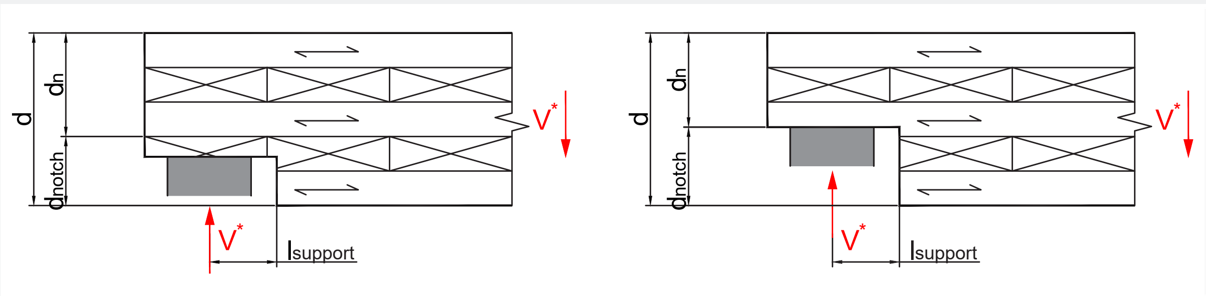


Figure C2: Examples of notched Cross laminated Timber (CLT) members on flat and effective beam depth d_n to be considered in the verification.

4.2 ALTERNATIVE VERIFICATION OF NOTCHED BEAMS

Alternatively, to section E9 of NZS AS 1720.1, the design capacity in shear of notched beams, for the strength limit state, shall satisfy the following:

$$\frac{1.5V^*}{b\,d_n} \leq \phi\, g'_{50}\; k_1\; k_4\; k_6\; k_{12}\; f'_{sj} \qquad \textbf{(1)}$$

where the alternative modification factor for notched beams g'_{50} is calculated as g'_{50}

$$g'_{50} = \frac{k_{50}\left(1 + \frac{1.1\, i^{1.5}}{\sqrt{d}}\right)}{\sqrt{d}\left(\sqrt{\alpha_r - \alpha_r^2} + 0.8\,\frac{l_{support}}{d}\sqrt{\frac{1}{\alpha_r} - \alpha_r^2}\right)} \qquad \textbf{(2)}$$

and

ϕ = capacity factor for members

f_{sj} = the characteristic value in shear at joint details appropriate to species strength group (refer to tables H2.2 and H2.3 in NZS AS 1720.1)

k = modification factors as per NZS AS 1720.1

b = the breadth of member

d = depth of the member

d_n = the effective depth of the member

d_{notch} = depth of the notch

i = $\frac{l_{notch}}{d_{notch}}$

ε = arctan $\left(\frac{1}{i}\right)$

= opening angle of the notch

α_r = $\frac{d_n}{d}$

$l_{support}$ = the distance between the support reaction and the onset of the notch

l_{notch} = the length of the notch with an opening angle of the notch ≥ 0

k_{50} = 5 for timber

= 6.5 for glulam

= 4.5 for LVL

Equation (1) does not apply when $\alpha_r \geq 0.5$ or $\frac{l_{support}}{d} \leq 0.4$. These limitations do not apply if:

- the loads applied are of short duration (5 days or less)
- the notch is reinforced
- the notch is on the unloaded side (see Figure 22)

C4.2

Equation (2) has been derived based on fracture mechanics by equating the energy release rate at which the crack at the notch would propagate. For the full derivation refer to Gustafsson (1988). The original equation used to determine the fracture energy is a function of the modulus of elasticity E and the critical fracture energy G_c, not the tensile strength perpendicular to grain $f_{t,90}$. In the final design equations neither of these two values appear as it is assumed that they are proportional to the shear strength by the factor k_{50} which varies for different materials and has been determined through testing.

Figure C3 shows values for the alternative modification factor for notched beams g'_{50} for timber members as a function of the notch depth, beam depth, distance to the support and opening angle of the notch.

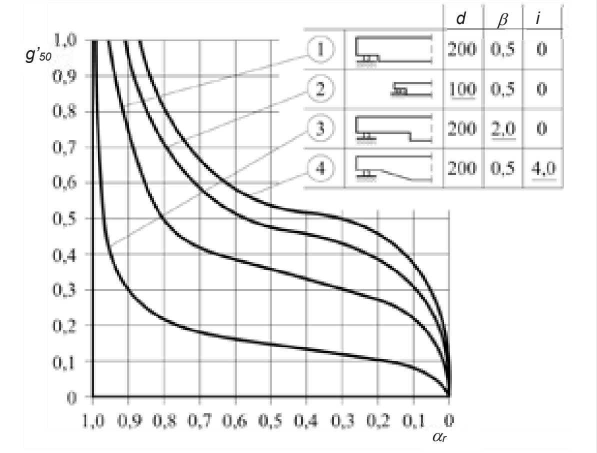


Figure C3: Values for the alternative modification factor for notched beams g'_{50} in function of α_r, the beam depth, the support length ($\beta = l_{support}/d$) and the opening angle of the notch i (adapted from Blaß and Sandhaas, 2017).

C4.2 (continued)

Figure C4 shows the design capacity in shear (in kN) calculated with Equation (1), and the provisions from NZS AS 1720.1 and the former NZS 3603. For shear values of notched beams without opening angle, the design standards provide conservative values. The values from NZS 3603 appear incorrect for very small notches, as the design capacity in shear results are greater than for an unnotched beam.

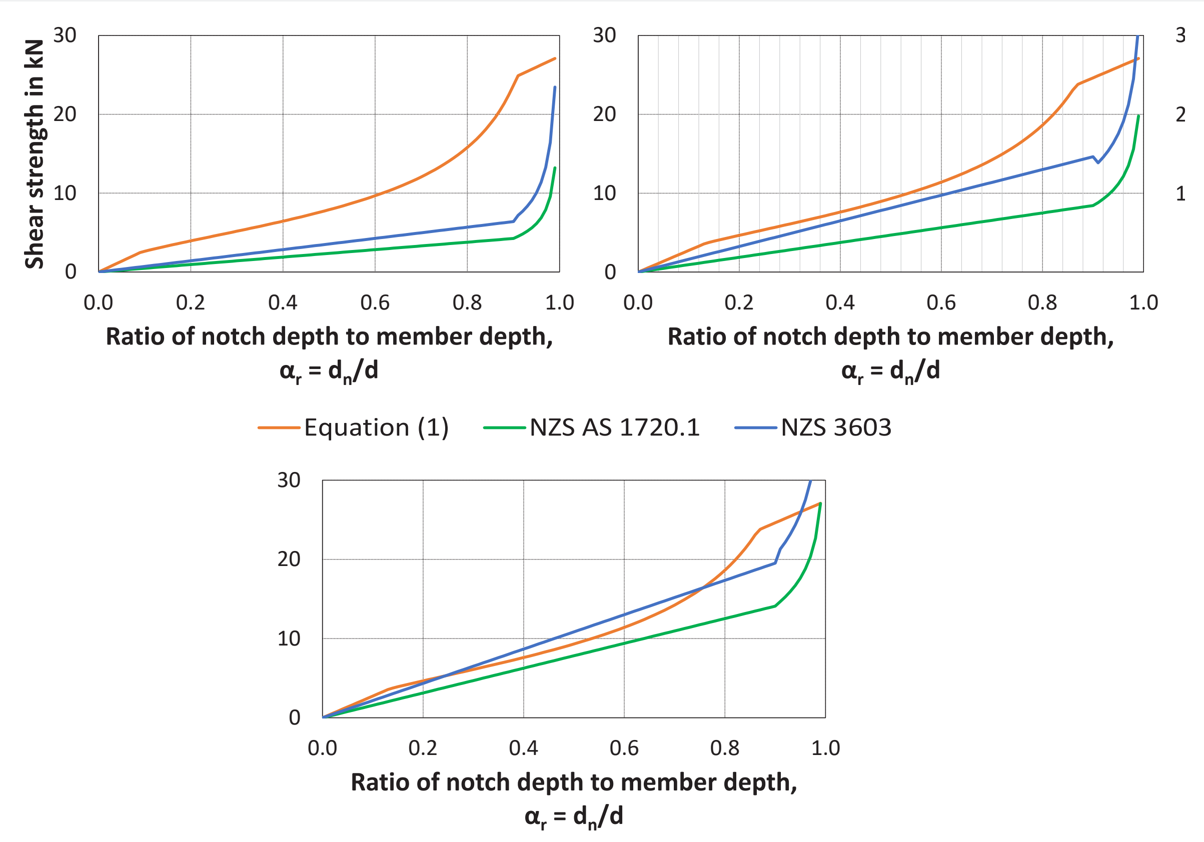


Figure C4 Comparison of the shear strength in kN *of a notched beam with varying ratios* $\alpha_r = d_n/d$ *based on Equation (1), NZS AS 1720.1 and NZS 3603 for a 300 x 45 SG8 beam*
(top: $i = 0$*, top right* $i = 2$*, bottom* $i = 4$*)*

4.3 REINFORCEMENT OF NOTCHES

4.3.1 General

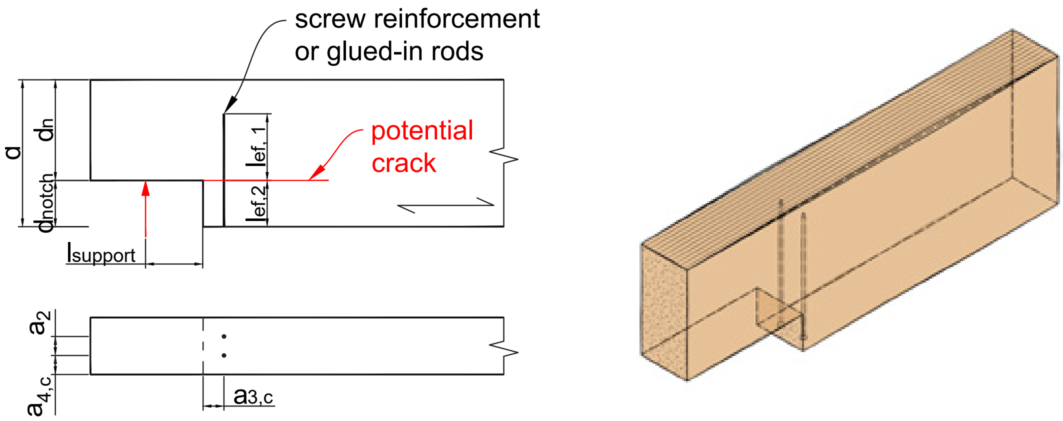


Figure 23: Reinforcement of notched beams with fully threaded screws or glued-in rods.

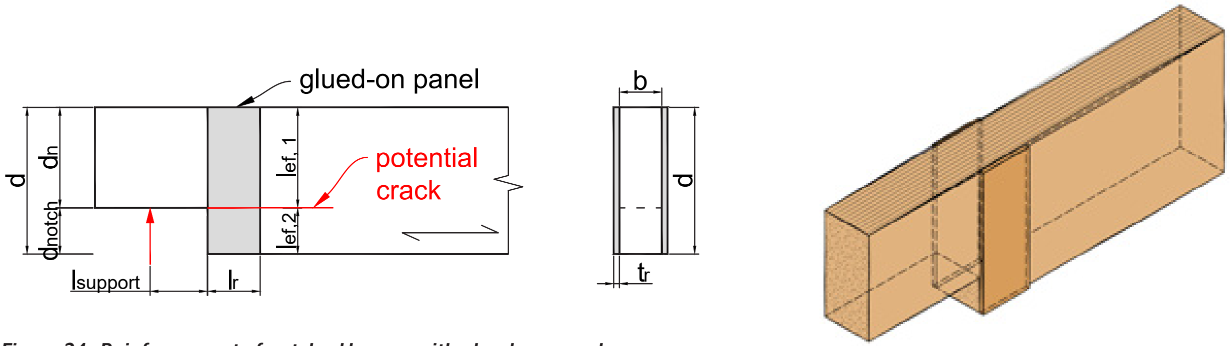


Figure 24: Reinforcement of notched beams with glued-on panels.

4.3.2 Load demand perpendicular to grain in notches

The design action effect in tension perpendicular to grain ($N^*_{90,r}$) of notched beams to be resisted by the reinforcement shall be calculated as:

$$N^*_{90,r} = 1.3k_{51}V^* \qquad \textbf{(3)}$$

where the notch reinforcement factor k_{51} is:

$$k_{51} = 3(1 - \alpha_r)^2 - 2(1 - \alpha_r)^3 \qquad \textbf{(4)}$$

and

V^* = the design action effect in shear at the notch

α_r = $\frac{d_n}{d}$

d = depth of the member

d_n = the effective depth of the member

In addition to the reinforcement, the shear capacity of the member in the notched part needs to be verified as per NZS AS 1720.1.

C4.3.2

The design load effect in tension perpendicular to grain ($N^*_{90,r}$) accounts for the tension perpendicular to grain stresses created by notching the beam. The derivation of factor k_{51} is based on the same principles of fracture mechanics as used for determining the load demand in connections perpendicular to grain.

Since k_{51} has been determined assuming a uniform stress concentration, a force increase of 30% has been introduced to account for the presence of the stress concentration in the corner of the notch (see Figure C5).

This stress concentration reduces rapidly with increasing distance from the notch, this is the reason why only one row of reinforcement in the form of screws or rods perpendicular to the member is effective in maintaining the load path at the onset of cracking.

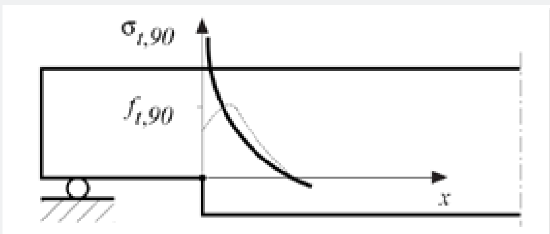


Figure C5: Stress distribution at the notch corner in accordance with the assumption of linear elasticity ($\sigma_{t,90}$, solid line) and assumed actual stress distribution ($f_{t,90}$, dashed line) (Blaß and Sandhaas, 2017).

C4.3.2 (continued)

The minimum allowable value for the distance $a_{3,c}$ to the end grain should be selected when using fully threaded screws or glued-in rods. When using external reinforcement, the panels or plates need to be installed at the corner of the notch.

Screws or glued-in rods can also be inclined on an angle in order to reduce the distance between the area of the stress concentration and the reinforcement as shown in Figure C6. The minimum spacing of the reinforcement can be determined based on the centre of gravity along the effective length $l_{ef,l}$.

Since the reinforcement is supposed to resist tension perpendicular to grain forces, the reinforcement capacity is to be taken as the force component perpendicular to grain (i.e. the projection with cos β of the capacity). When using inclined screws, the withdrawal strength calculation also needs to account for the angle between the screw axis and the grain direction.

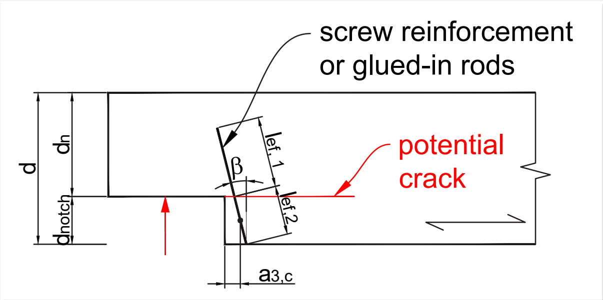


Figure C6: Reinforcement of notched beams with inclined fully threaded screws or glued-in rods.

4.3.3 Reinforcement with fully threaded screws

The design capacity ($N_{d,j}$) for fully threaded screws as reinforcement shall satisfy the following:

$$N_{d,j} \geq N^*_{90,r} \qquad \textbf{(5)}$$

where $N_{d,j}$ is the lesser of

$$N_{d,j} = \phi\, k_1\;\, n\, Q_k \qquad \textbf{(6)}$$

$$N_{d,j} = n\, N_{d,ts} \qquad \textbf{(7)}$$

and

ϕ = capacity factor for screws

k_1 = the factor for load duration for fasteners (to be taken as the minimum between NZS AS 1720.1 and manufacturer's specification)

$N^*_{90,r}$ = design load effect in tension perpendicular to grain

n = is the number of screws in the first row

Q_k = characteristic withdrawal capacity of the screw given by the manufacture's specification calculated for the smaller of $l_{ef,1}$ and $l_{ef,2}$

$N_{d,ts}$ = the design tensile capacity of the screw given by the manufacture's specification

The additional geometric limitation of $l_{ef,1} \geq 1.5\ l_{support}$ needs to be verified. The total length of the screw or glued-in rod shall be at least $2l_{ef.1}$.

The recommended minimum distances for screws as shown in Figure 23 shall be as given in the manufacturer's specification.

NOTE: 1. Only one row of screws in the perpendicular direction of the notched member will be effective and shall be taken into consideration.

2. ETA approved screws sometimes required the use of an effective number of screws n_{eff}, refer to manufacturer's specification.

C4.3.3

The length $l_{ef,2}$ shall be chosen as close as possible to the depth of the notch d_{notch}. The screw should extend at least to a distance equal to $0.7d$ from the edge of the beam (i.e. the beam edge where the notch is cut into).

Instead of fully threaded screws also partially threaded screws can be used. In this case both the withdrawal strength of the screw over the effective thread length $l_{ef,1}$ (which for partially threaded screws is only a fraction of the screw length) and the bearing strength under the screw head need to be larger than the required design load effect in tension perpendicular to grain. The bearing strength under the screw head is not the head pull though strength as defined in NZS AS 1720.1 (the standard considers the head of the screw being pulled through the full member thickness) and can be determined with Section 4.4.3.3 to NZS AS 1720.1 taking the area of washer for bearing, A_w, as the area under the screw head.

The tensile, withdrawal and bearing strength (often referred to as head pull through strength) as well as the minimum distances of most engineered wood screws should be taken as per manufacturer's specification.

4.3.4 Reinforcement with glued-in rods

For reinforcements with glued-in rods, the rod embedment strength and the rod tensile strength shall be verified as described in Chapter 29 of the Timber Design Guideline (Buchanan, 2007). The rod diameter D_r shall not exceed 20mm. The bond strength of the rod shall be for the smaller of $l_{ef,1}$ and $l_{ef,2}$.

The design capacity ($N_{d,j}$) for glued-in rods as reinforcement shall satisfy the following:

$$N_{d,r} \geq N^*_{90,r} \quad \textbf{(8)}$$

where $N_{d,r}$ is the lesser of

$$N_{d,r} = Q_{d,pullout} \quad \textbf{(9)}$$

$$N_{d,r} = \phi_{st}\ n\ N_{tf} \quad \textbf{(10)}$$

where

$N_{t,f}$	=	the nominal tension capacity of the rod as per NZS 3404
$N^*_{90,r}$	=	design load effect in tension perpendicular to grain
$Q_{d,pullout}$	=	the design axial capacity in tension considering bar pullout for n rods as per Chapter 29 of the Timber Design Guide or other relevant literature or testing
ϕ_{st}	=	the strength reduction factor of a bolted connection in tension as per NZS 3404
ϕ	=	capacity factor for other fasteners
n	=	is the number of rods in the first row
k_1	=	the factor for load duration for fasteners

The additional geometric limitation of $l_{ef,1} \geq 1.5\ l_{support}$ needs to be verified.

The minimum distances for glued-in rods as shown in Figure 23 shall not be less than:

$$a_2 \geq 3D_r$$
$$a_{4,c} = a_{3,c} \geq 2.5\ D_r \quad \textbf{(11)}$$

where:

D_r	=	diameter of glued-in rod

4.3.5 Reinforcement with glued-on wood-based panels

The design capacity ($N_{d,j}$) for glued-on wood-based panels as reinforcement shall satisfy the following:

$$N_{d,r} \geq N^*_{90,r} \quad \textbf{(12)}$$

where $N_{d,r}$ is the lesser of

$$N_{d,r} = 2\ V_{d,sj} \quad \textbf{(13)}$$

$$N_{d,r} = 2\ N_{d,t} \quad \textbf{(14)}$$

and

$$V_{d,sj} = \phi_a\ k_1\ k_{19}\ f_a\ A_{sj} \quad \textbf{(15)}$$

$$N_{d,t} = \phi\ k_1\ k_{52}\ t_r\ l_r\ f_t \quad \textbf{(16)}$$

where

$N_{d,t}$ = design action effect in tension of the glued-on panel

$N^*_{90,r}$ = design load effect in tension perpendicular to grain

A_{sj} = $(l_r\ l_{ef})$

= area at glue interface as shown in Figure 24

l_{ef} = minimum $(l_{ef,1}; l_{ef,2})$

$V_{d,sj}$ = design action effect in shear at the glued interface of the glued-on panel and the timber member

k_1 = the load duration factor for members

k_{19} = modification factor for moisture condition

ϕ_a = 0.7 for the capacity factor for adhesives

ϕ = the capacity factor of the glued-on panel

l_r = the width of the glued-on panel

t_r = the thickness of the glued-on panels (effective area in the direction of the force applied)

k_{52} = 0.5

= the reduction factor due to non-uniform stress distribution

f_a = the bond line strength of the adhesive

f_t = the tensile strength of the glued-on panels in the direction of the force applied

The width of the glued-on panels shall satisfy the following:

$$0.25 \leq \frac{l_r}{l_{ef}} \leq 0.5 \quad \textbf{(17)}$$

NOTE: 1. Reinforcement with glued-on panels shall be symmetrical on either side of the notch.

2. To guarantee proper adhesion of the glued-on panels, screw gluing of the panels is recommended. Screw gluing can be used for wood-based panels of up to 50mm with self-drilling screws (i.e. Type 17 tip) with a diameter larger than 4mm. The thread penetration into the timber member needs to be larger than the panel thickness, but not smaller than 40mm. No thread is allowed in the glued-on panel. A screw each 15,000mm^2 of panel area is required, with a maximum screw spacing (in each direction) of 150mm. The moisture content of the panel or timber member shall be less than 15%, and the difference in moisture content between the two elements shall be less than 4%.

This work should only be carried out by a sufficiently competent and certified tradesperson with appropriate quality control measures in place. More guidance on component gluing can be found in Design Guide Australia and New Zealand – Fabrication and Finishing, STIC (2013).

C4.3.5

The bond line strength of the adhesive is the minimum between the rolling shear strength of the member to be reinforced and adhesive shear strength. European standards recommend a value of 0.7 MPa for the adhesive shear strength which also takes the non-uniform stress distribution into account. This value seems to be aligned with the shear strength of glued plywood interfaces in NZS AS 1720.1, which considers a glue strength in the order of 20% to 40% of the plywood shear strength in glued assemblies. For a grade F8 plywood, the glue strength would hence be 0.84 MPa.

The reduction factor k_{52} takes the non-uniform stress distribution at the notch into account, which according to Figure C5 is largest close to the corner of the notch and reduces moving away from the notch.

The geometric limitation of the width of the glued-on plate is to ensure that there is sufficient reinforcement to prevent the onset of cracking and that the reinforcement plate is acting in the area of the highest stress.

The length $l_{ef,2}$ shall be chosen as close as possible to the depth of the notch d_{notch}. The glued-on panel should extend at least to a distance equal to 0.7d from the edge of the beam (i.e. the beam edge where the notch is cut into).

5 JOINTS LOADED PERPENDICULAR TO GRAIN

5.1 STRENGTH OF JOINTS LOADED PERPENDICULAR TO GRAIN

The design strength of joints loaded perpendicular to grain can be determined with the method provided in section 4 of Appendix ZZ in NZS AS 1720.1.

The strength of joints loaded perpendicular to grain does not need to be verified if $d_e/d \geq 0.7$ as per Figure 25.

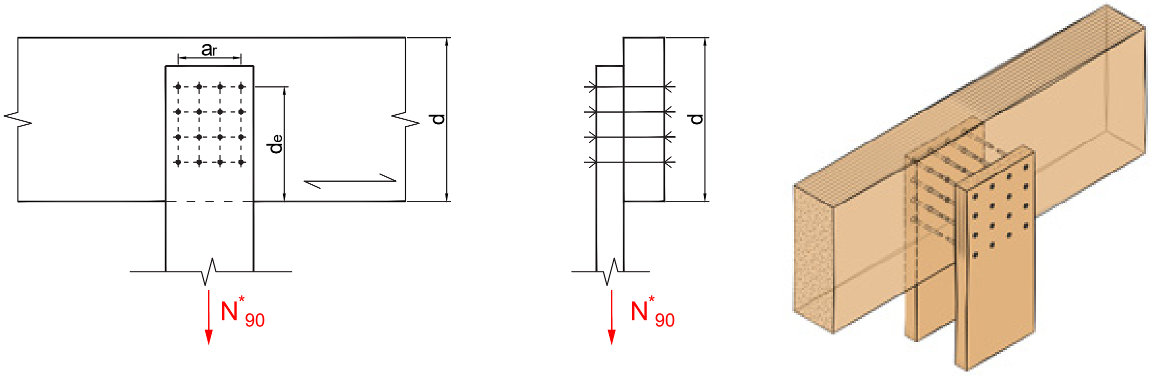


Figure 25: Joints loaded perpendicular to grain.

C5.1

Although stresses in joints loaded perpendicular to grain can be determined using finite element analysis methods, the resulting stresses which are a combination of shear and tension perpendicular to grain stresses cannot be directly verified against the characteristic tension strength perpendicular to grain, which is normally determined on small prismatic test bodies loaded under tension only. The verifications as provided in Appendix ZZ of NZS AS1720.1 are based on fracture mechanics.

An alternative method as shown below is based on a conventional stress criteria using an effective width and is used in the German National Annex to Eurocode 5 (DIN, 2013). This method overcomes some of the limitations and conservatism of the methods based on facture mechanics, which do not all consider that joints often have multiple rows of fasteners or can have fasteners distributed over a longer length, thus reducing perpendicular to grain stresses.

For connections with multiple fastener rows and columns, or any other cases not covered in Appendices ZZ of NZS AS 1720.1, the following strength verification should be satisfied:

$$N_{90,w} \geq N_{90} \qquad \textbf{C(1)}$$

where $N_{90,w}$ is the design strength of the fastener group in a member loaded perpendicular-to-grain:

$$N_{90,w} = \phi \; k_1 \; k_{15} \; k_{53} \; k_{54} \left[6.5+18\left(\frac{d_e}{d}\right)^2\right] (t_{ef} \; d)^{0.8} \; f'_{t,90} \qquad \textbf{C(2)}$$

where

$$k_{53} = \max \begin{cases} 1 \\ 0.7 + 1.4\,\dfrac{a_r}{d} \end{cases} \qquad \textbf{C(3)}$$

$$k_{54} = \frac{n_r}{\sum_{i=1}^{n}\left(\frac{h_1}{h_i}\right)^2} \qquad \textbf{C(4)}$$

C5.1 (continued)

and

N^*_{90} = the perpendicular-to-grain component of the load applied to the member

ϕ = capacity factor for members

k_{15} = the service condition factor

k_{53} = factor to account for the increased tension strength perpendicular to grain with multiple adjacent fasteners

k_{54} = factor to account for joints with multiple fastener rows

n_r = number of fastener rows in the member

a_r = width of fastener group

h_i = distance of the i^{th} fastener row from the unloaded edge

d = the member depth (in mm)

d_e = $(d - a_{3,c})$ the distance from the loaded edge to the fastener furthest away from the loaded edge (in mm)

t_{ef} = the effective joint penetration depth (in mm)

$f'_{t,90}$ = the characteristic tensile strength perpendicular to grain (in MPa)

For joints with two shear planes or with direct force introduction into the member (i.e. see case (d) in Figure C8) the effective joint penetration depth:

- for timber-to-timber and timber-to-wood panels connections with nails and screws:
  $t_{ef} = \min(b;2\,t_{pen};24\,D)$
- for timber-to-steel connections with nails:
  $t_{ef} = \min(b;2\,t_{pen};30\,D)$
- for large diameter dowel connections:
  $t_{ef} = \min(b;2\,t_{pen};12\,D)$
- for glued-in rods or fully threaded screws:
  $t_{ef} = \min(b;6\,D)$

For joints with a single shear plane the effective joint penetration depth

- for timber-to-timber and timber-to-wood panels connections with nails and screws:
  $t_{ef} = \min(b;t_{pen};12\,D)$
- for timber-to-steel connections with nails:
  $t_{ef} = \min(b;t_{pen};15\,D)$
- for large diameter dowel connections:
  $t_{ef} = \min(b;t_{pen};6\,D)$

where

b = the breadth of the member

D = the diameter of the fastener

t_{pen} = the fastener penetration

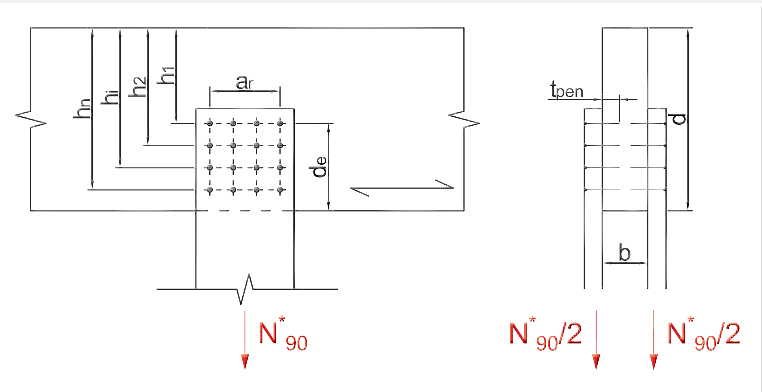


Figure C7: Joints loaded perpendicular to grain (joint with two shear planes).

C5.1 (continued)

For connections with glued-in rods, d_e equals the embedment length of the glued-in rod measured from the loaded edge.

Joints loaded perpendicular to grain with $d_e/d \leq 0.2$ should only be used for load demands of short duration (5 days or less).

Joints with $a_r/d > 1$ and $N^*_{90} \geq N_{90,w}$ are always to be reinforced.

For multiple joint groups with a clear distance to the adjacent joint $a_5 > d$ the design strength of the fastener group in a member loaded perpendicular-to-grain $N_{90,w}$ as calculated with equation C(2) still applies.

For multiple joints with a clear distance to the adjacent joint $a_5 < 0.5\ d$ the fasteners of both connections need to be considered as a single joint when calculating $N_{90,w}$.

For all other cases with $0.5d \leq a_5 \leq 2h$, $N_{90,w}$ for each joint group shall be reduced by a factor k_{55}

$$k_{55} = \frac{a_5}{4\ d} + 0.5 \qquad \textbf{C(5)}$$

where

a_5 = the clear distance between adjacent joints

d = the member depth

In cases with more than two joint groups with $a_5 < 2\ d$ and $N^*_{90} \geq 0.5\ k_{55}\ N_{90,w}$, all joints shall be reinforced.

As a general note, the following should be considered when designing joints loaded perpendicular to grain:

- the ratio d_e/d should be maximised by moving the fasteners closer to the unloaded edge;
- multiple fasteners in a row spread the load over a larger area, thus reducing the tension perpendicular to grain stresses;
- increasing the beam breadth b or depth d leads to a larger volume being stressed and hence reduces the stresses;
- adding additional rows of fasteners reduces the stresses in the upper most row.

Several typical joints loaded perpendicular to grain can be seen in Figure C8.

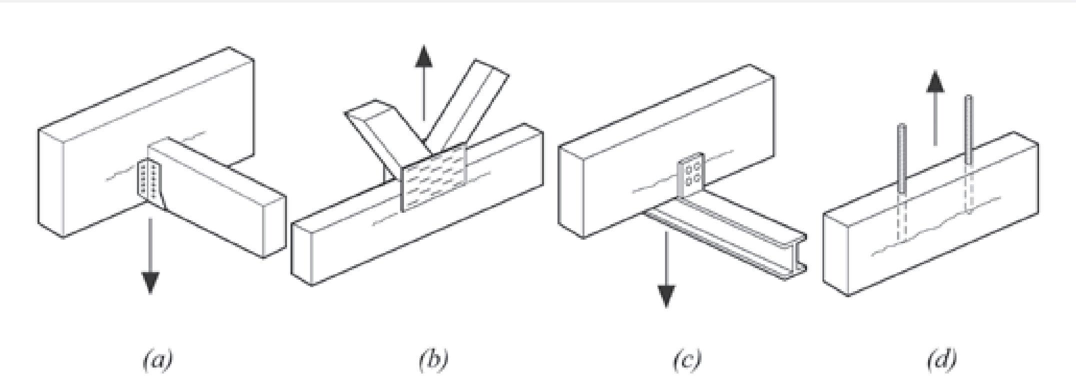


Figure C8: Examples of typical joints loaded perpendicular to grain with possible locations of cracks: a) joist hanger, b) toothed metal plate connector, c) large diameter dowel type connectors, d) glued-in rods (Blaß and Sandhaas, 2017).

5.2 REINFORCEMENT OF JOINTS LOADED PERPENDICULAR TO GRAIN

5.2.1 General

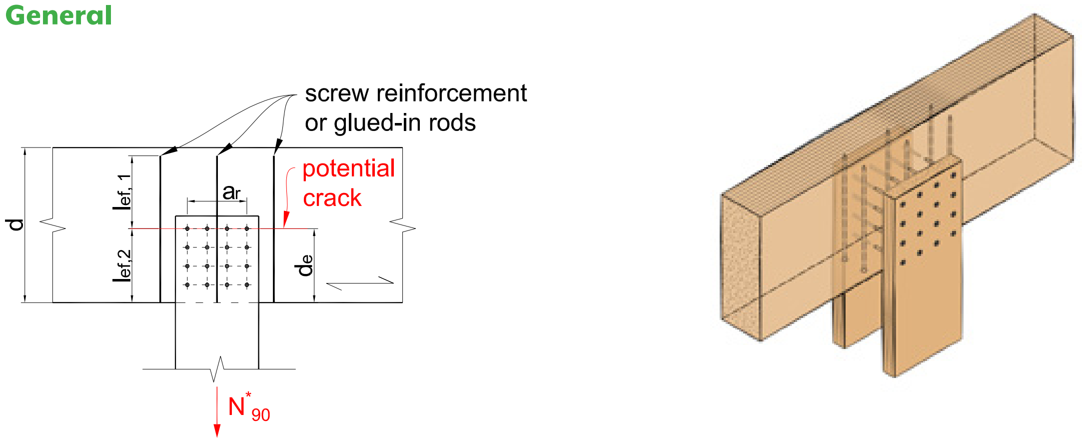


Figure 26: Reinforcement with fully threaded screws or glued-in rods in joints loaded perpendicular to grain.

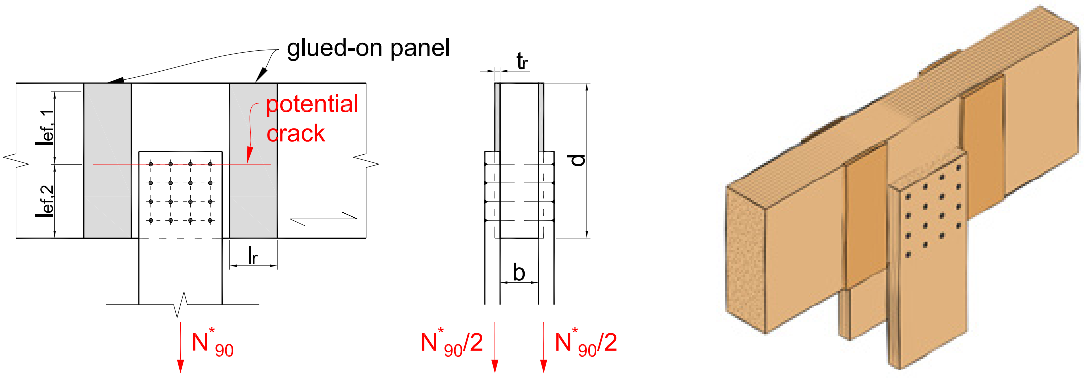


Figure 27: Reinforcement with glued-on panels in joints loaded perpendicular to grain.

5.2.2 Load demand perpendicular to grain in joints loaded perpendicular to grain

The design action effect in tension perpendicular to grain ($N^*_{90,r}$) to be resisted by the reinforcement shall be calculated as

$$N^*_{90,r} = k_{56} \; N^*_{90} \qquad \textbf{(20)}$$

where the reinforcement factor for joints loaded perpendicular to grain k_{56} is

$$k_{56} = 1 - 3\alpha_r^2 + 2\alpha_r^3 \qquad \textbf{(21)}$$

and

N^*_{90} = the design load effect perpendicular-to-grain applied to the member

$\alpha_r = \dfrac{d_e}{d}$

d = the member depth

d_e = the distance from the loaded edge to the fastener furthest away from the loaded edge

5.2.3 Reinforcement with fully threaded screws or glued-in rods

The design capacity ($N_{d,j}$) for the reinforcement shall satisfy the following:

$$N_{d,j} \geq N^{*}_{90,r} \tag{22}$$

where $N_{d,j}$ is determined for fully threaded screws or glued-in rods as per section 4.3.3 and section 4.3.4, respectively, with the lengths $l_{ef,1}$ and $l_{ef,2}$ as per Figure 26.
The number of reinforcing screws or glued-in rods n shall be taken as the number of screws or rods installed in the joint (placed over the length denominated with a_r) plus the number of screws or rods in the first row either side of the joint.

C5.2.3
The screw or glued-in rods should extend at least to a distance equal to $0.7d$ from the loaded edge of the beam.

5.2.4 Reinforcement with glued-on wood-based panels

The design capacity ($N_{d,j}$) for glued-on wood-based panels as reinforcement shall satisfy the following:

$$N_{d,r} \geq N^{*}_{90,r} \tag{23}$$

where $N_{d,r}$ is the lesser of

$$N_{d,r} = n_p \ V_{d,sj} \tag{24}$$

$$N_{d,r} = n_p \ N_{d,t} \tag{25}$$

and $V_{d,s,j}$ and $N_{d,t}$ are determined as per section 4.3.5 with n_p being the number of glued-on panels.
The width of the glued-on panels shall satisfy the requirements of equation (17).

NOTE: 1. Reinforcement with glued-on panels shall be symmetrical on either side of the notch.
2. To guarantee proper adhesion of the glued-on panels, screw gluing of the panels is recommended, refer to section 4.3.5.

C5.2.4
The glued-on wood-based panels should extend at least to a distance equal to $0.7d$ from the loaded edge of the beam. If the distance is less than $0.7d$, the joint needs to be verified for the perpendicular to grain demand assuming d_e equals the depth of the reinforcement.

6 PENETRATIONS

Penetrations are circular or rectangular holes in beams with a diameter or depth, respectively, of $d_d > 50$mm which require the verification of perpendicular to grain tension stresses.

Beams with holes with a diameter or depth smaller than or equal to 50mm do not require verification of perpendicular to grain tension stresses, but still need to provide enough flexural and shear strength with net section properties as per sections 6.1.2 and 6.1.3. For beams with a depth smaller than 300mm, also holes with a diameter or depth of dd ≤ 50mm need to follow all requirements as outlined in this section.

All penetrations in beams in unseasoned timber or in timber members with an average moisture content over 20% shall always be reinforced as per section 6.2. For seasoned timber or timber members with an average moisture content below 20% the design strength of beams with penetrations shall be determined with the provisions in section 6.1.

It is important to note that perpendicular to grain stresses regularly occur in timber members due to changes in moisture content. Due to the exposed end grain at penetrations, moisture content variations are more likely to cause shrinkage cracks in this area. It is hence recommended to always reinforce penetrations when moisture content variations (during construction and/or in-service) are expected.

C6

Although penetrations in beams with a diameter or depth larger than 50mm do not need to be verified for tension perpendicular to grain stresses, the strength of the beam with net section properties still needs to be verified. Small holes should not be placed in the proximity of tension or compression fibres at the edges of the beam or close to supports. Refer to NZS 3604 for holes in floor joists for buildings falling within the scope of that standard. It is not recommended to use the guidance available in NZS 3604, for structures requiring specifying engineering design as per NZS AS 1720.1.

Figure C9 shows the potential position of cracks forming around round and rectangular penetrations depending on the contributions to the tension perpendicular to grain forces from the shear demand or moment demand in the beam as determined in section 6.2. For circular holes centred at the neutral axis of the beam, cracks tend to occur at locations at approximately 45 degrees from the direction of the grain measured from the origin of the hole.

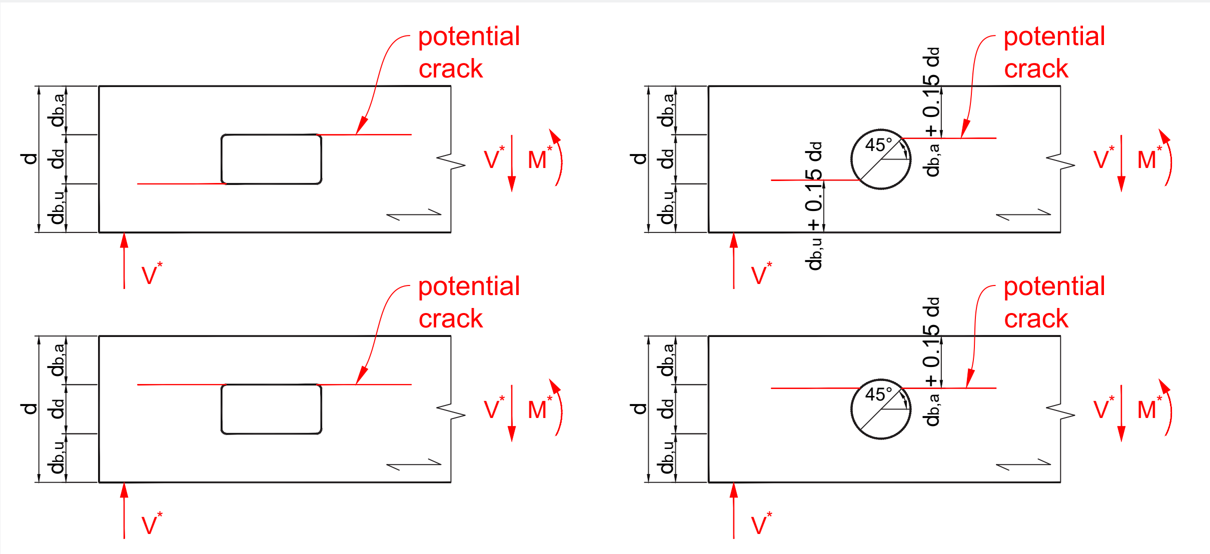


Figure C9: Potential crack position depending on shear and moment contributions to the total tension perpendicular to grain demand.
Top: $N^*_{90,r,M} \leq N^*_{90,r,V}$ *(common in holes close to the supports where shear demand is high).*
Bottom: $N^*_{90,r,M} > N^*_{90,r,V}$ *(common at midspan of the beams where moment demand is high).*

6.1 STRENGTH OF BEAMS WITH PENETRATIONS

6.1.1 Geometrical limitations of beams with unreinforced penetrations

The geometric limitations as shown in Figure 28 apply to all unreinforced penetrations in beams made of glulam or LVL. Sawn timber members should always be reinforced.

The maximum penetration diameter or depth d_d for unreinforced circular and rectangular holes shall be taken as smaller or equal to 0.15 d, respectively.

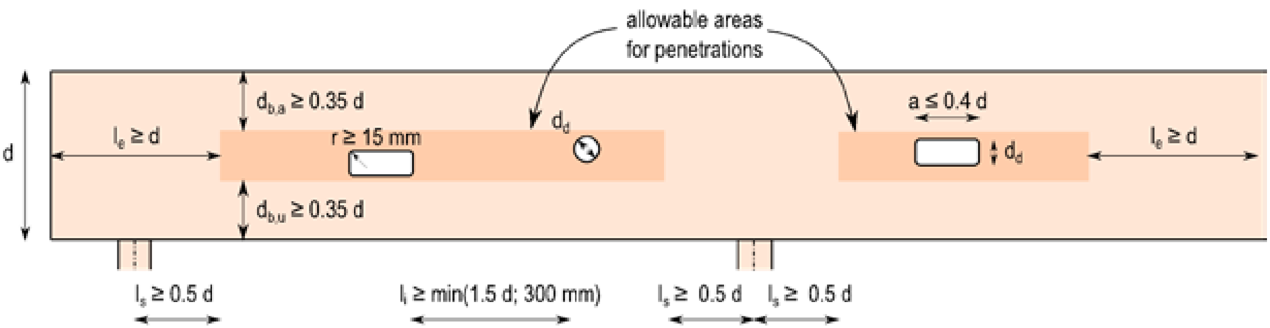


Figure 28: Geometrical limitations for beams with unreinforced penetrations.

The dimensions shown in Figure 28 are defined as:

d = the depth of the beam
d_d = the diameter or depth of penetrations for circular and rectangular holes, respectively
l_e = the distance between the penetration to the end grain of the beam
l_s = the distance between the penetration to the support
l_i = the clear distance between adjacent penetrations
$d_{b,a}$ = the depth of the remaining beam portion above the penetration
$d_{b,u}$ = the depth of the remaining beam portion under the penetration
a = the length of a rectangular penetration
r = the corner radius of a rectangular penetration+

NOTE: More than one penetration is allowed in the highlighted area in Figure 28, as long as the clear distance l_i between adjacent penetrations is guaranteed.

C6.1.1

The requirement of rounded corners in rectangular penetrations is to prevent areas of singularity and hence to reduce stress concentrations at the corner.

6.1.2 Flexural shear strength of beams with penetrations

Beams with reinforced and unreinforced penetrations shall be proportioned so that:

$$\frac{M^*}{M_{d.n}} + \frac{M^*_{b,a}}{M_{d,b,a}} + \frac{N^*}{N_d} \leq 1 \quad \textbf{(26)}$$

$$\frac{M^*}{M_{d.n}} + \frac{M^*_{b,u}}{M_{d,b,u}} + \frac{N^*}{N_d} \leq 1 \quad \textbf{(27)}$$

and

$$M^*_{b,a} = V_{b,a}\frac{a}{2} \quad \textbf{(28)}$$

$$M^*_{b,u} = V_{b,u}\frac{a}{2} \quad \textbf{(29)}$$

where

M^* = design bending action effect at the centre of the penetration
$M^*{}_{b,a}$ = design bending action effect from the frame action in the beam portion above of the penetration
$M^*{}_{b,u}$ = design bending action effect from the frame action in the beam portion under the penetration
N^* = design action effect in tension or compression
$M_{d,n}$ = design capacity in bending of the net section calculated as per NZS AS 1720.1
$M_{d,b,a}$ = design capacity in bending of the beam portion above the penetration calculated as per NZS AS 1720.1
$M_{d,b,u}$ = design capacity in bending of the beam portion under the penetration calculated as per NZS AS 1720.1
N_d = design capacity in tension or compression of the net section calculated as per NZS AS 1720.1
a = the length of the penetration
d = the depth of the beam
d_d = the diameter or depth of the penetrations with circular or rectangular holes, respectively
b = the breadth of the beam

For beams with circular penetrations the terms $M^*_{b,a}$ and $M^*_{b,ua}$ from the design bending action effect from the frame action at the penetration can be ignored.

The shear demand in the portions of the beam above and under the penetration shall be calculated as:

$$V_{b,a} = V^*\frac{d_{b,a}}{d_{b,a} + d_{b,u}} \quad \textbf{(30)}$$

$$V_{b,u} = V^*\frac{d_{b,u}}{d_{b,a} + d_{b,u}} \quad \textbf{(31)}$$

where

V^* = design action effect in shear at the centre of the penetration
$d_{b,a}$ = the depth of the remaining beam portion above the penetration
$d_{b,u}$ = the depth of the remaining beam portion under the penetration

C6.1.2

Equations (26) and (27) consider the bending strength and any tension or compression axial strength at the net section of the penetration. Due to the deviation of shear forces, the bending stresses in the portions of the beam above and under the penetrations are increased. To account for this effect, the additional moment demands $M^*_{b,a}$ and $M^*_{b,u}$ from the 'frame action' at the penetration need to be considered. These additional moment demands can be determined by calculating the proportion of the shear demand in the upper or lower portion of the beam times half the length of the penetration length as show in Figure C10

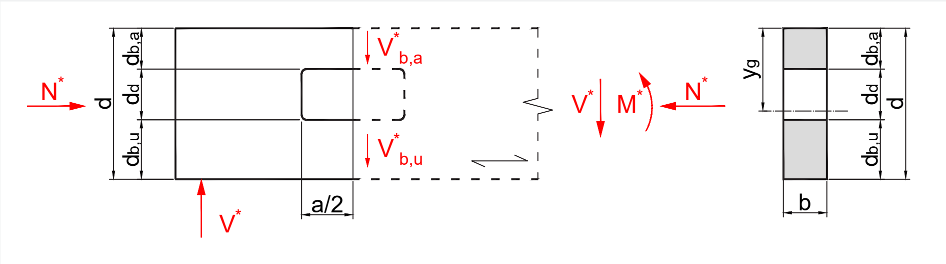


Figure C10: Frame action of a beam with penetrations.

The additional bending demand can be ignored for circular penetrations, as the net beam section properties increase more than the lever arm causing the frame action, hence not creating additional bending stresses.

The design capacities in bending $M_{d,n}$, $M_{d,b,a}$ and $M_{d,b,u}$ shall be calculated in accordance with section 3.2.1 of NZS AS 1720.1 with section modulus determined with the following equations:

$$Z_{b,a} = \frac{b\ d_{b,a}^2}{6} \qquad \textbf{C(6)}$$

$$Z_{b,u} = \frac{b\ d_{b,u}^2}{6} \qquad \textbf{C(7)}$$

$$y_g = \frac{d^2 - 2d_d\ d_{b,a} - d_d^2}{2\ (d - d_d)} \qquad \textbf{C(8)}$$

$$I_n = b \left[\frac{d^3}{12} + d\left(\frac{d}{2} - y_g\right)^2 - \frac{d_d^3}{12} - d_d\left(d_{b,a} + \frac{d_d}{2} - y_g\right)^2 \right] \qquad \textbf{C(9)}$$

$$Z_n = \min\left[\frac{I_{net}}{y_g}\ ;\ \frac{I_{net}}{d - y_g}\right] \qquad \textbf{C(10)}$$

where

$Z_{b,a}$ = section modulus of the remaining beam portion above the penetration
$Z_{b,u}$ = section modulus of the remaining beam portion under the penetration
Z_n = section modulus of the net section at the penetration
I_n = moment of inertia of the net section at the penetration
y_g = the distance of the centre of gravity from the top edge of the beam

6.1.3 Shear strength of beams with penetrations

The design capacity in shear for beams with reinforced or unreinforced penetrations shall satisfy the following:

$$V_{d,n} \geq V^* \quad \textbf{(32)}$$

where

V^* = design action effect in shear at the centre of the penetration

$V_{d,n}$ = design capacity in shear as per NZS AS 1720.1 with the net shear plane area $A_{s,n}$

$A_{s,n}$ = $\frac{2}{3} b (d - d_d)$

= the net shear plane area at the penetration

NOTE: Beams with penetrations reinforced with fully threaded screws and glued-in rods (internal reinforcement) shall additionally satisfy section 6.2.4.

6.1.4 Tension perpendicular to grain strength of glulam beams with unreinforced penetrations

For glulam beams with penetrations the following shall be satisfied:

$$\frac{N^*_{90,r,V}}{N_{90,d,n,V}} + \frac{N^*_{90,r,M}}{N_{90,d,n,M}} \leq 1 \quad \textbf{(33)}$$

and

$$N_{90,d,n,V} = \phi\, k_1\, k_{57}\; 0.5\, l_{t,90,V}\, b\, f'_{t,90} \quad \textbf{(34)}$$

$$N_{90,d,n,M} = \phi\, k_1\, k_{57}\; 0.5\, l_{t,90,M}\, b\, f'_{t,90} \quad \textbf{(35)}$$

where

$N^*_{90,r,V}$ = the design action effect in tension perpendicular to grain from shear action on either side of the penetration as determined in section 6.2.2 for glulam beams

$N^*_{90,r,M}$ = the design action effect in tension perpendicular to grain from moment action on either side of the penetration as determined in sections 6.2.2 for glulam beams

$N_{90,d,n,V}$ = the design capacity in tension perpendicular to grain resisting shear action

$N_{90,d,n,M}$ = the design capacity in tension perpendicular to grain resisting moment action

ϕ = capacity factor for members

k_1 = the factor for load duration for members

k_{57} = $\left(\frac{10^7}{0.225\, b\, d_d^2}\right)^{0.2}$ with b and d_d in mm

= factor to account for the stressed volume in beams with penetrations

$l_{t,90,V}$ = 1.3 d_d

= the length over which the tension perpendicular to grain stresses from shear action are resisted

$l_{t,90,M}$ = 0.5 d_d

= the length over which the tension perpendicular to grain stresses from moment action are resisted

b = the beam breadth

d_d = the diameter or depth for circular or rectangular penetrations respectively

$f'_{t,90}$ = the characteristic tensile strength perpendicular to grain

If equation (33) is not satisfied, reinforcement of the penetration as per section 6.2 is required.

C6.1

In equations (34) and (35) the terms ($l_{t,90,v}$ b) and ($l_{t,90,M}$ b) represent the areas subject to tension perpendicular to grain stresses from shear and moment action, respectively. Due to the stress concentration closer to the penetration a factor of 0.5 has been introduced to account for the non-uniform stress distribution. The factor k_{57} takes the stressed volume of the beam depth into account and refers to a standard reference volume of 0.01 $m^3 = 107mm^3$.
The derivation of the factor can be found in Danzer et al. (2017).

The integration lengths $l_{t,90,V}$ and $l_{t,90,M}$ in equations (34) and (35) for shear and moment demand have been derived by numerical analysis by Danzer et al. (2017) and Tapia and Aicher (2019).

Figure C11 compares three different methods currently available for the verification of unreinforced penetrations. The comparison is based on the following assumptions: $k_1 = 0.8$, $\phi = 0.8$, $E = 10{,}000$ MPa, $G = 750$ MPa, $G_f = 0.7$ N/mm, $f'_{t.90,k} = 0.5$ MPa, $f'_v = 5$ MPa, $b = 120$mm, $h = 200$-1500mm, $V^* = 20\%$ of net section capacity and $M^* = V^*$ x 3m. The formulation by Tapia, which is currently used for glulam members in this guide, is similar to the formulation in the German National Annex to Eurocode 5, but overcomes some of the shortcomings in terms of hole geometry and moment demand contribution. Both Tapia and Eurocode 5 integrate the tension strength $f'_{t.90}$ over a numerically defined integration length to determine the member capacity. In contrast, the formulation by Ardalany uses fracture mechanics and the theory of beams on an elastic foundation, specifically applied to New Zealand grown Radiata Pine LVL. Although Figure C11 shows that the differences in the three formulations are rather small, it has been decided to keep the procedures for glulam separate to the one for LVL, as neither Tapia nor Eurocode have had extensive use on LVL members.

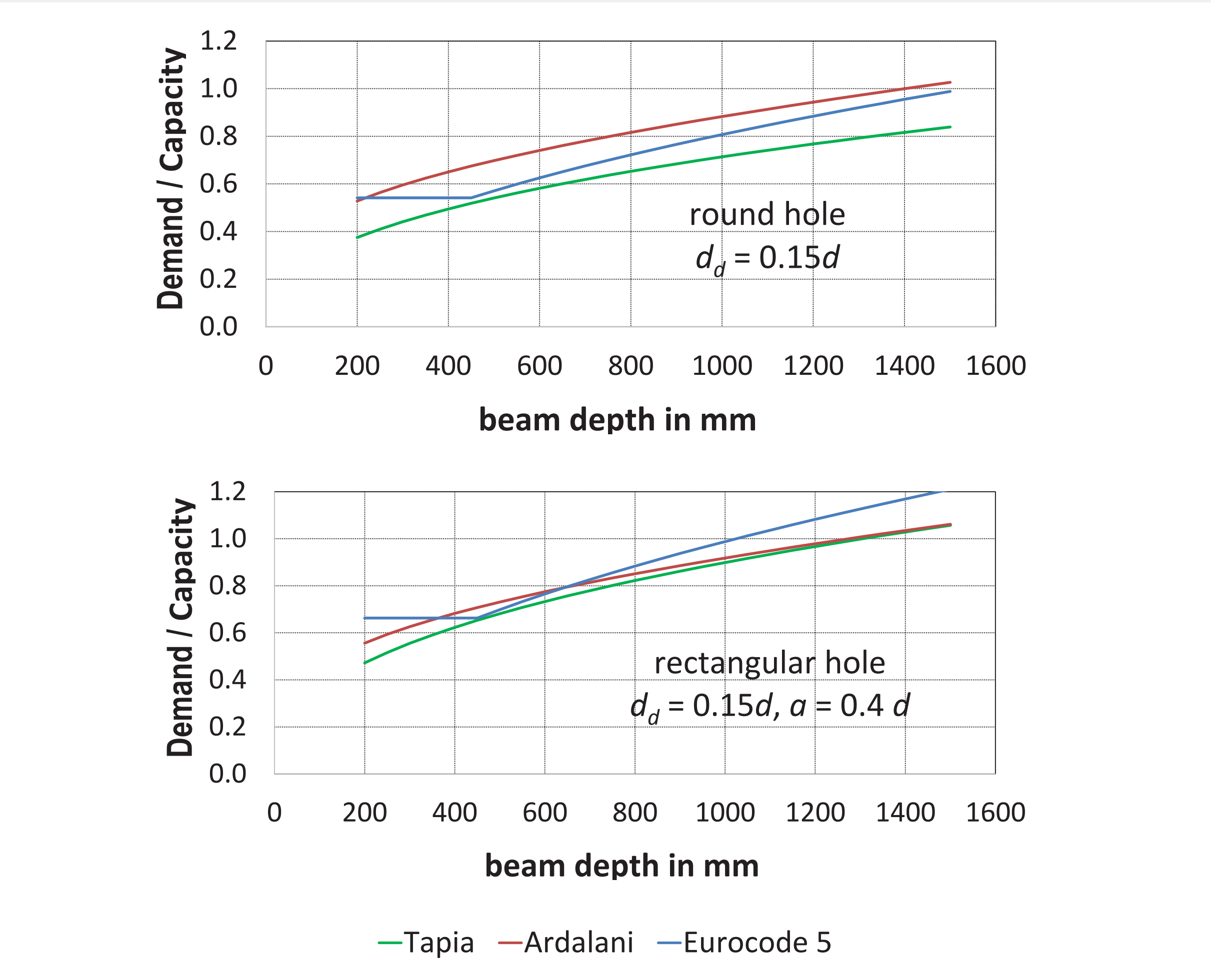
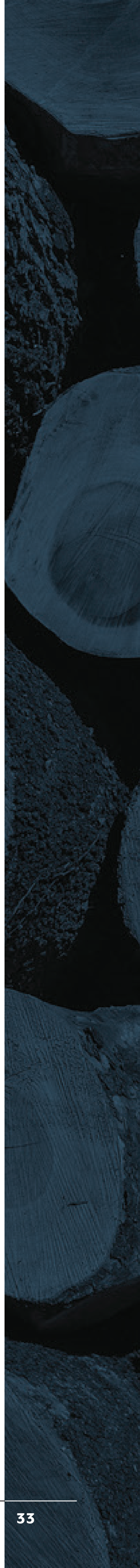


Figure C11: Comparison of different tension perpendicular to grain checks for unreinforced penetrations.



6.1.5 Tension perpendicular to grain strength of LVL beams with unreinforced penetrations

For LVL beams with penetrations the following shall be satisfied:

$$\frac{V^*}{V_{d,t}} + \left(\frac{M^*}{M_{d,t}}\right)^2 \leq 1 \qquad \textbf{(36)}$$

and

$$V_{d,t} = \phi\, k_1\, 2V_{cr} \qquad \textbf{(37)}$$

$$M_{d,t} = \phi k_1\, [2M_{cr} + b\, \sigma_M\, (d_{cr} + k_{64}\, d_d)(d - d_{cr})] \qquad \textbf{(38)}$$

where

ϕ = capacity factor for members

k_1 = the factor for load duration for members

V^* = the design action effect in shear at the penetration

M^* = the design action effect in moment at the penetration

$V_{d,t}$ = the design capacity in shear of the beam at the penetration

$M_{d,t}$ = the design capacity in moment of the beam at the penetration

d_d = the depth of the penetration

d = the depth of the beam

d_{cr} = $(d_b + 0.15\, d_d)$ for circular holes

= d_b for rectangular holes

= the depth of the cracked beam section

d_b = $\min(d_{b,a};\, d_{b,u})$

= the minimum depth of the remaining beam portion above or under the penetration

k_{64} = 0.7 for circular holes

= 1.0 for rectangular holes

σ_M = $\frac{M_{cr}}{Z_{cr}}$

= the bending stress due to the moment M_{cr}

Zcr = $\frac{b\, d_{cr}^2}{6}$

= the section modulus of the cracked beam section

The shear V_{cr} and moment M_{cr} at which onset of cracking occurs shall be calculated as:

$$V_{cr} = \frac{b f'_{t,90}}{\sqrt{4\lambda^2 + \dfrac{6k_{cr}\, b}{5\, GA_{cr}}}} \qquad \textbf{(39)}$$

$$M_{cr} = \frac{f_{t,90}\, b}{2\lambda^2} \qquad \textbf{(40)}$$

and

$$\lambda^4 = \frac{k_{cr}\, b}{4E\, I_{cr}} \qquad \textbf{(41)}$$

$$k_{cr} = \frac{f'^2_{t,90}}{2\, G_f \left(\dfrac{1}{1 - \dfrac{d_{cr}}{d}}\right)^2} \qquad \textbf{(42)}$$

where

$f'_{t,90}$ = the characteristic tensile strength perpendicular to grain

G_f = 0.7 N/mm for Radiata Pine LVL

= the characteristic fracture energy

G = the shear modulus of the beam

E = the modulus of elasticity of the beam

I_{cr} = $\frac{b\, d_{cr}^3}{12}$

= the moment of inertia of the cracked beam section

A_{cr} = $b\, d_{cr}$

= the cross section of the cracked beam section

If equation (36) is not satisfied, reinforcement of the penetration as per section 6.2 is required.

C6.1.5

The design procedure shown is based on work by Ardalany et al. (2012) and is based on fracture mechanics and the analysis of beams on an elastic foundation with a modulus of elasticity of the foundation k_{cr} based on the characteristic fracture energy G_f and the tension perpendicular to grain strength $f'_{t,90}$. The design procedure is based on the assumption that the penetration is centred in the beam and that the remaining beam sections above and under the crack lines with a height h_{cr} provide the cracking shear and moment capacities as shown in equations (37) and (38). For the moment capacity an additional term accounting for the axial load in the beam is added to make up for the full moment resistance. The term in parenthesis in equation (42) only applies to small holes which are typically unreinforced and takes the rotational and transverse stiffness interaction between the beam portions above and under the crack lines into consideration.

The characteristic fracture energy of 0.7 N/mm was specifically measured for Radiata Pine LVL manufactured in New Zealand and should not be used for any other species or timber products unless verified by testing.

According to Ardalany et al. (2012) the design procedure is applicable also to eccentric holes but requires the correct determination of the cracked height.

The proposed procedure assumes that any pipe or other superimposed dead load is not resting on the penetration itself and is supported independently as it otherwise would increase the tensile force demand around the penetration.

Although above formulation provides an estimate of the shear capacity of unreinforced LVL beams with penetrations, it should only be applied to LVL beams with low shear demands (i.e. secondary structural elements), due to potential sudden and brittle failures caused by the tension perpendicular to grain stresses around penetrations. It is generally recommended to reinforce beams with penetrations, especially if they are primary structural elements.

6.2 REINFORCEMENT OF BEAMS WITH PENETRATIONS

6.2.1 Geometrical limitations of beams with reinforced penetrations

The geometric limitations as shown in Figure 29 apply to all reinforced penetrations in beams made of timber, glulam or LVL.

The maximum penetration diameter or depth d_d in timber and glulam beams with circular and rectangular holes, respectively, shall be taken as:

$d_d \leq 0.3\, d$ for penetrations with internal reinforcement (fully threaded screws or glued-in rods)

$d_d \leq 0.4\, d$ for penetrations with external reinforcement (glued-on)

The additional limitation of a $\leq 2.5\, d_d$ applies for rectangular penetrations.

The maximum penetration diameter or depth d_d in LVL beams with circular and rectangular holes respectively, shall be taken as:

$d_d \leq 0.4\, d$ in circular holes with internal reinforcement (fully threaded screws or glued-in rods)

$d_d \leq 0.45\, d$ in circular holes with external reinforcement (glued-on panels)

$d_d \leq 0.35\, d$ in rectangular holes with internal reinforcement (fully threaded screws or glued-in rods)

$d_d \leq 0.4\, d$ in rectangular holes with external reinforcement (glued-on panels)

The additional limitation of a $\leq 3\, d_d$ applies to rectangular penetrations.

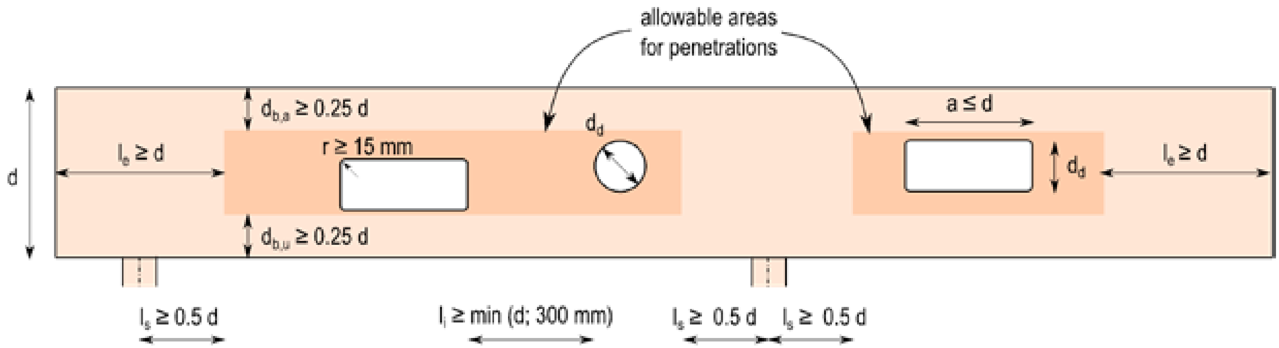


Figure 29: Geometrical limitations for beams with reinforced penetrations.

NOTE: 1. The distances l_e, l_s and l_i in Figure 29 are measured from the outer edge of the penetration.
2. More than one penetration is allowed in the highlighted area in Figure 29, as long as clear distance l_i between adjacent penetrations is guaranteed.

6.2.2 Load demand perpendicular to grain in timber and glulam beams with penetrations

The design action effect in tension perpendicular to grain ($N^*_{90,r}$) for penetrations in glulam and timber beams to be resisted by the reinforcement shall be calculated as:

$$N^*_{90,r} = N^*_{90,rV} + N^*_{90,rM} \quad \textbf{(43)}$$

where

$$N^*_{90,r,V} = \frac{V^*}{4}\left(\frac{k_{59}\ d_d}{d}\right)\left[3-\left(\frac{k_{59}\ d_d}{d}\right)^2\right]\left[1+k_{60}\left(\frac{k_{59}\ d_d}{d}\right)\right] \quad \textbf{(44)}$$

$$N^*_{90,r,M} = \frac{0.1\ M^*}{d}\left(\frac{k_{59}\ d_d}{d}\right)^2\left[1+k_{61}\left(\frac{k_{59}\ d_d}{d}\right)\right] \quad \textbf{(45)}$$

and

V^* = the design action effect in shear at the penetration on either side of the penetration
M^* = the design action effect in moment at the penetration on either side of the penetration
$N^*_{90,r,V}$ = the design action effect in tension perpendicular to grain from shear action
$N^*_{90,r,M}$ = the design action effect in tension perpendicular to grain from moment action
d_d = the depth of the penetration
d = the depth of the beam
$d_{b,a}$ = the depth of the remaining beam portion above the penetration
$d_{b,u}$ = the depth of the remaining beam portion under the penetration

The factors k_{59}, k_{60} and k_{61} are defined in Table 6. 1.

Hole type	k_{59}	k_{60}	k_{61}
Circular	0.81	0.43	0.40
Rectangular	0.83 $+ 0.013\frac{a}{d_d}$	0.57 $+ 0.533\frac{a}{d_d}$	0.05 $+ 0.113\frac{a}{d_d}$

Table 6 1: Factors for the load demand in timber and glulam beams with penetrations.

NOTE: Any additional design load from the service pipe or duct resting on the penetration needs to be added to the demand $N^*_{90,r}$.

C6.2.2

The equations shown here for the determination of the tension perpendicular to grain demand are based on the modifications of the current formulation in the German National Annex to Eurocode 5 (DIN, 2013) by Tapia and Aicher (2019). The modifications address the shortcomings and conservatisms currently found in Eurocode, with the largest change being the new formulation for the tension demand due to the external moment demand.

The factors for the load demand in timber and glulam beams with penetrations with rectangular holes as shown in Table 6 1 written in function of the ratio a/d_d have been found by linear interpolation from the following values as provided by Tapia and Aicher (2019).

Hole type	k_{59}	k_{60}	k_{61}
Square $(a/d_d = 1)$	**0.84**	**1.1**	**0.16**
Rectangular $(a/d_d = 2.5)$	**0.86**	**1.9**	**0.33**

Equations (44) and (45) do not explicitly consider eccentric holes, Tapia and Aicher (2019) however show that the error for eccentricity-to-depth ratios e/d smaller than 0.15 and for moment-to-shear-force ratios M^*/V^* between 1.5 d and 5 d, is negligible. When in doubt, the factor as shown in section 6.2.3 can be used to account for any eccentricities.

Figure C12 shows the comparison of three different methods currently available for the determination of the tension perpendicular to grain forces in reinforced penetrations. The comparison is based on the following assumptions: $k_1 = 0.8$, $\phi = 0.8$, E = 10,000 MPa, $G = 750$ MPa, $f'_v = 5$ MPa, $b = 120$mm, $h = 200$-1500mm, $V^* = 20\%$ of net section capacity and $M^* = V^*$ x 3 m. It can be seen that the three formulations are approximately equal for round holes with a diameter of $0.3d$. Most research has been carried out on this specific penetration configuration, all leading to very similar results. With increasing hole diameter and for rectangular holes, the values obtained with the different formulations show less agreement. Due to the laminate and grain orientation in LVL, and therefore higher likelihood of splitting, it has been decided to use the formulations by Tapia for penetrations in sawn timber and glulam members only and use the formulation by Ardalany, which leads to more conservative results, for penetrations in LVL members.

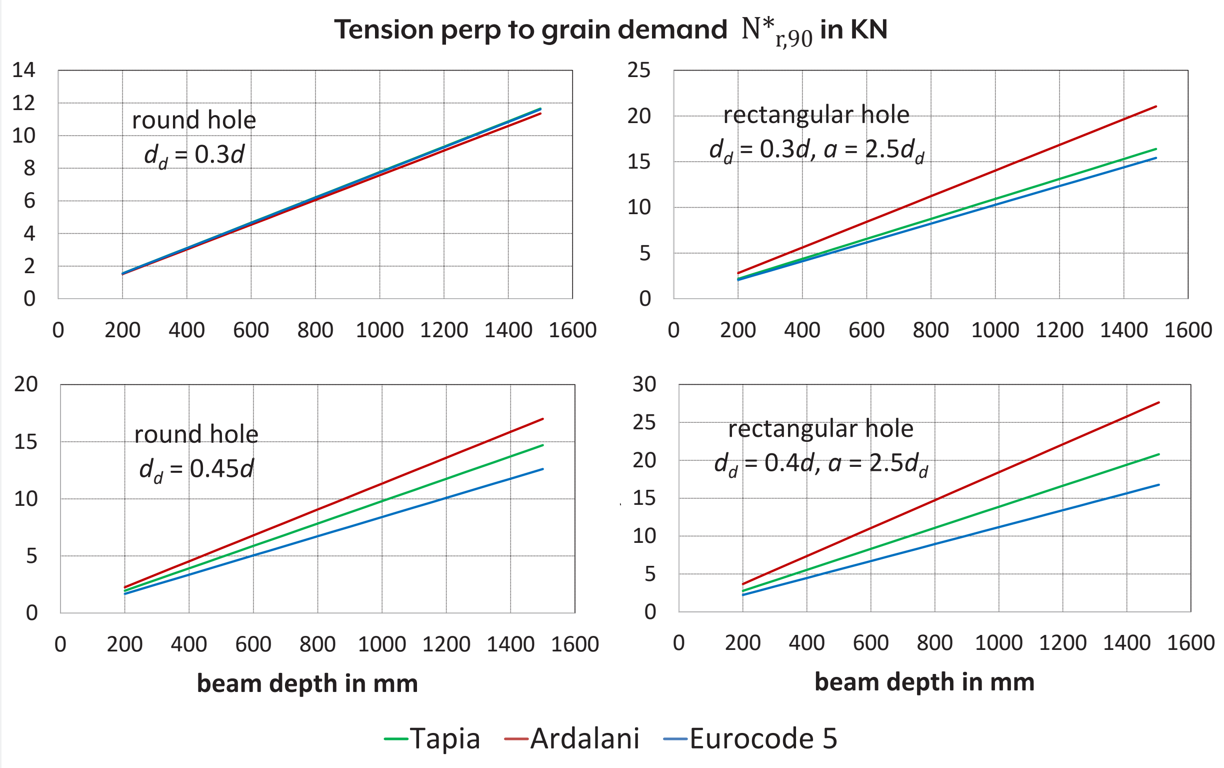


Figure C12: Comparison of different tension perpendicular to grain demand formulations for reinforced penetrations.

6.2.3 Load demand perpendicular to grain in LVL beams with penetrations

The design action effect in tension perpendicular to grain ($N^*_{90,r}$) for penetrations in LVL beams to be resisted by the reinforcement shall be calculated as:

$$N^*_{90,r} = N^*_{90,r,V} + N^*_{90,r,M} \qquad \textbf{(46)}$$

where

$$N^*_{90,r,V} = \frac{V^*k_{62}}{4}\frac{d_d\,(3d^2 - d_d^2)}{d^3} \qquad \textbf{(47)}$$

$$N^*_{90,r,M} = k_{63}\;M^*\frac{d_d^2}{d^3} \qquad \textbf{(48)}$$

and

V^* = the shear force demand at the penetration
M^* = the moment demand at the penetration
d_d = the depth of the penetration
d = the depth of the beam
a = the length of the hole

The factors k_{62} and k_{63} are defined in Table 6 2.

Hole type	k_{62}	k_{63}
Circular	0.7	$\frac{3}{4}\;\frac{d_d\,(d+d_d)}{(d\,d_d + d^2 + d_d^2)}$
Square	0.7	0.7
Rectangular	$\max\left\{\frac{a}{\sqrt{d_d^2 + a^2}};\frac{d_d}{\sqrt{d_d^2 + a^2}}\right\}$	0.7

Table 6 2: Factors for the load demand in LVL beams with penetrations.

The design action effect in tension perpendicular to grain ($N^*_{90,r}$) as determined in Equation (46) shall be increased by the following factors

- $\left(1+\frac{d_d}{d}\right)\frac{e}{0.1d}$ for eccentricities e of up to 10% of the beam depth.
- $\left(\frac{l_i}{d}\right)$ for clear distances between multiple penetrations l_i ranging between 1.25 d and d.

NOTE: Any additional design load from the service pipe or duct resting on the penetration needs to be added to the demand $N^*_{90,r}$.

C6.2.2

The equations presented for the determination of the reinforcement demand has been derived by Ardalany et al. (2012). The equations are based on a strut-and-tie model for circular penetrations and subsequent modification through numerical analysis of LVL beams for square and rectangular penetrations.

6.2.4 Verification of additional shear stresses for internally reinforced rectangular penetrations

In addition to the strength verifications as per section 6.1, the following additional shear strength verification shall be satisfied for rectangular penetrations with internal reinforcement (fully threaded screws and glued-in rods):

$$V_{d,p} \geq V^* \quad \textbf{(49)}$$

where

$$V_{d,p} = \phi\, k_1\, k_4\, k_6\, k_{58} \frac{2}{3} A_{s,n}\, f'_s \quad \textbf{(50)}$$

and

$$k_{58} = \frac{1}{1.84\left(1+\frac{a}{d}\right)\left(\frac{d_d}{d}\right)^{0.2}} \quad \textbf{(51)}$$

$$0.1 \leq \frac{a}{h} \leq 1.0 \text{ and } 0.1 \leq \frac{d_d}{h} \leq 0.4 \quad \textbf{(52)}$$

V^* = the design action effect in shear at the penetration

$V_{d,p}$ = the design shear capacity of beams with internally reinforced penetrations

k_{58} = factor to account for the increased shear demand in internally reinforced penetrations

f'_s = characteristic value in shear

$A_{s,n}$ = $\frac{2}{3}\, b\,(d - d_d)$

= the net shear plane area at the penetration

If equation (49) is not satisfied, external reinforcement shall be used.

C6.2.4

Due to the deviation of shear forces around the corners of the penetrations, the shear stresses are increased when compared to the values obtained with the Euler–Bernoulli beam theory (Blaß and Bejtka, 2003). Equation (51) considers the increased shear stress τ_2 at the edge of the penetration as shown in Figure C13. In the case of externally applied reinforcements like glued-on panels or steel plates these shear stresses are resisted by the panels or plates and no further verification is required.

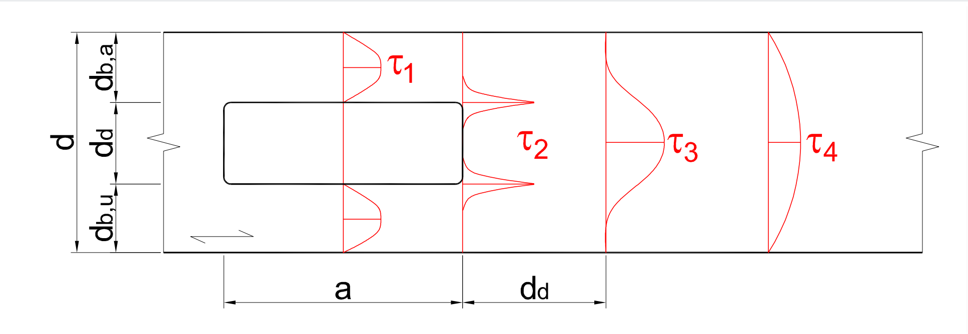


Figure C13: Shear stresses in beams with penetrations (modified from Blaß and Bejtka, 2003).

6.2.5 Reinforcement with fully threaded screws or glued-in rods

The design capacity ($N_{d,j}$) for fully threaded screws or glued-in rods as reinforcement shall satisfy the following:

$$N_{d,j} \geq N^{*}_{90,r} \tag{53}$$

where $N_{d,j}$ is determined for fully threaded screws or glued-in rods with the lengths $l_{ef,1}$ and $l_{ef,2}$ as per Figure 30 as per section 4.3.3 and section 4.3.4, respectively. The number of reinforcing screws or glued-in rods n shall be taken as the number of screws or rods installed in the first row at a distance $a_{3,c}$ from the penetration. The load demand perpendicular to grain $N^{*}_{90,r}$ shall be determined as per section 6.2.2 for timber and glulam beams or 6.2.3 for LVL beams.

The effective length $l_{ef,1}$ shall be taken as the smaller of:

- $l_{ef,1} = d_{b,a} + 0.15d_d$ or $l_{ef,1} = d_{b,u} + 0.15d_d$ for circular holes
- $l_{ef,1} = d_{b,a}$ or $l_{ef,1} = d_{b,u}$ for rectangular holes

where

$d_{b,a}$ = the depth of the remaining beam portion above the penetration

$d_{b,u}$ = the depth of the remaining beam portion under the penetration

The effective length $l_{ef,2}$ shall be equal to or bigger than $l_{ef,1}$.

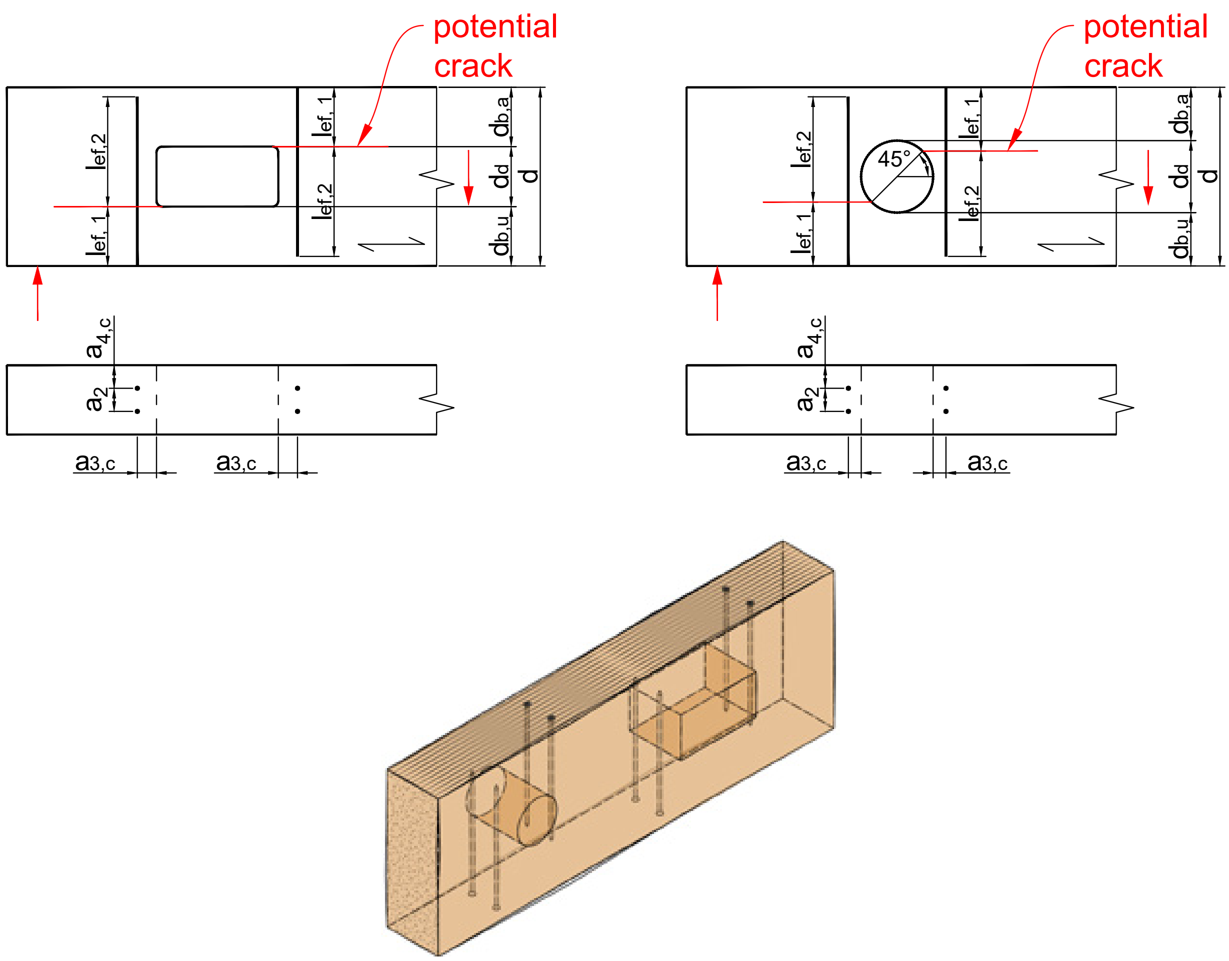


Figure 30: Reinforcement of beams with penetrations with rectangular and round holes reinforced with fully threaded screws or glued-in rods.

The recommended minimum distances for screws for the dimensions (a_2, $a_{3,c}$ and $a_{4,c}$) shown in Figure 30 shall be as given in the manufacturer's specification.

For glued-in rods the following shall be satisfied:

$$2.5\, D_r \leq a_{3,c} \leq 4D_r \qquad \textbf{(54)}$$

where

D_r = is the diameter of the rod.

The requirements of section 6.2.4 shall be satisfied.

6.2.6 Reinforcement with glued-on wood-based panels

The design capacity ($N_{d,r}$) for glued-on wood-based panels as reinforcement shall satisfy the following:

$$N_{d,r} \geq N^*_{90,r} \qquad \textbf{(55)}$$

where $N_{d,r}$ is the lesser of

$$N_{d,r} = 2\, V_{d,sj} \qquad \textbf{(56)}$$

$$N_{d,r} = 2\, N_{d,t} \qquad \textbf{(57)}$$

and

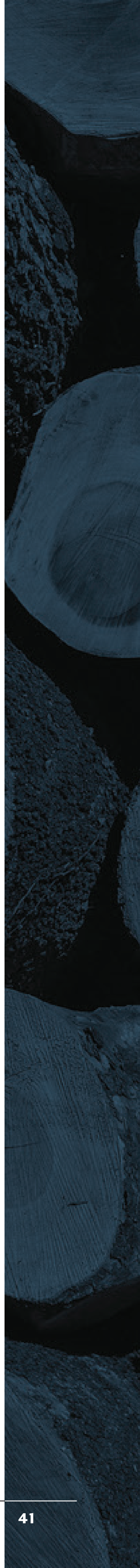
$$V_{d,sj} = \phi_a\ k_1\ k_{19}\, f_a\, A_{sj} \qquad \textbf{(58)}$$

$$N_{d,t} = \phi\, k_1\ k_{52}\, t_r\, l_r\ f'_t \qquad \textbf{(59)}$$

where

$V_{d,sj}$ = design action effect in shear at the glued interface of the glued-on panel and the timber member
$N_{d,t}$ = design action effect in tension of the glued-on panel
ϕ_a = 0.7 for the capacity factor for adhesives
ϕ = the capacity factor of the glued-on panel
f_a = the bond line strength of the adhesive
f'_t = the tensile strength of the glued-on panels in the direction of the force applied
A_{sj} = ($l_r\ d_{sj}$)
= area at glue interface as shown in Figure 31
d_{sj} = d_r + 0.15 d_d for circular holes
= d_r for rectangular holes
k_1 = the load duration factor for members
k_{19} = modification factor for moisture conditions
l_r = the effective width of the glued-on panel
t_r = the thickness of the glued-on panel
k_{52} = 0.5
= reduction factor due to non-uniform stress distribution

The load demand perpendicular to grain $N^*_{90,r}$ shall be determined as per sections 6.2.2 or 6.2.3 for timber and glulam or LVL beams, respectively.



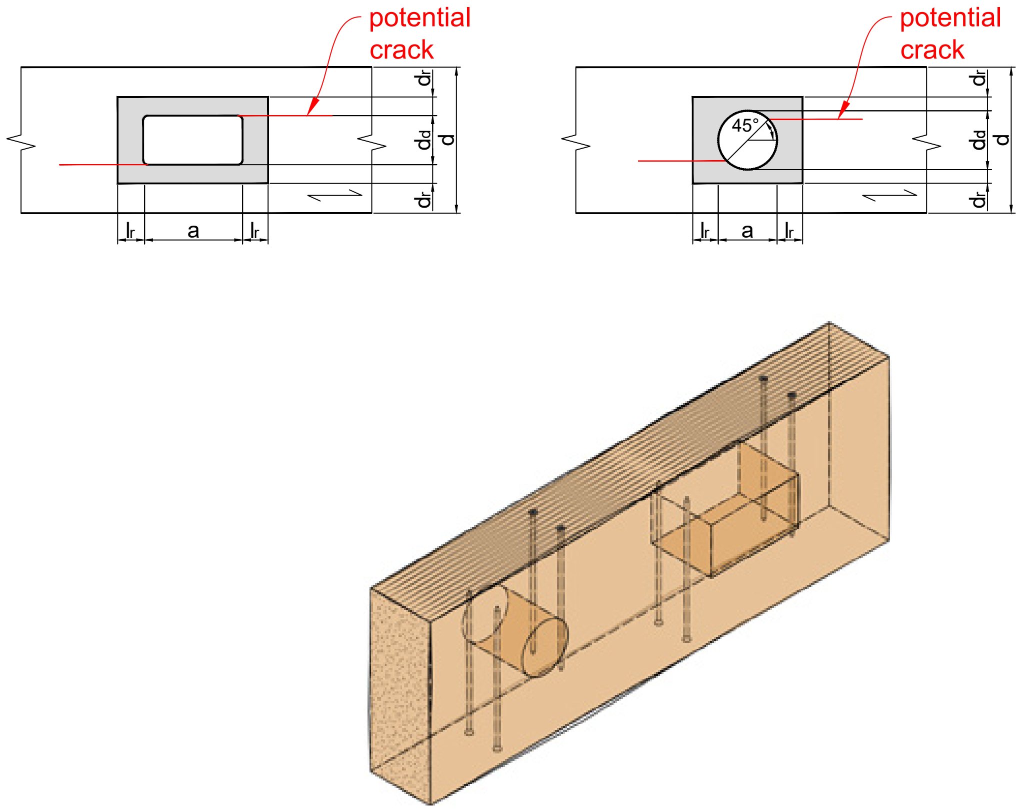


Figure 31: Reinforcement of beams with penetrations with rectangular and round holes reinforced with glued-on panels.

The geometry of the glued-on panels shall satisfy the following:

$$0.25\,a \leq l_r \leq 0.3\,(d_d + d) \qquad \textbf{(60)}$$

$$d_r \geq 0.25\,a \qquad \textbf{(61)}$$

NOTE: 1. Reinforcement with glued-on panels shall be symmetrical on either side of the penetration.
2. To guarantee proper adhesion of the glued-on panels, screw gluing of the panels is recommended, refer to section 4.3.5.

7 BEARING REINFORCEMENT

7.1 BEARING REINFORCEMENT WITH FULLY THREADED SCREWS

The design capacity in bearing perpendicular to grain ($N_{d,p}$) of a structural member reinforced with fully threaded screws (see Figure 33), for the strength limit state, shall satisfy the following:

$$N_{d,p,r} \geq N_p^* \quad \textbf{(62)}$$

where $N_{d,p,r}$ is the lesser of

$$N_{d,p,r} = \phi\, k_1\, k_4\, k_6\, k_7\;\, f'_p\, A_p + n\, N_{d,j,c} \quad \textbf{(63)}$$

$$N_{d,p,r} = \phi\, k_1\, k_4\, k_6\, k_7\;\, f'_p\, A_{p,2} \quad \textbf{(64)}$$

where

ϕ = the capacity factor for members
k_1 = the factor for load duration for members
k_4, k_6, k_7 = the modification factors from NZS AS 1720.1
f'_p = the characteristic value in bearing perpendicular to grain
A_p = $b\, l_p$
= the bearing area for loading perpendicular to grain at the support
l_p = the bearing length at the support (length of steel plate)
b = the breadth of the beam
$A_{p,2}$ = $b\, l_{p,2}$
= the effective bearing area at the tip of the screws
$l_{p,2}$ = $2\, l_{ef} + (n_0 - 1)\;\, a_1$ for intermediate support
= $l_{ef} + (n_0 - 1)\;\, a_1 + \min(l_{ef};\, a_{3,c})$ for end support
= the effective bearing length at the tip of the screws
l_{ef} = the depth of screw penetration
n = the number of screws in the bearing area
n_0 = the number of screws in one row parallel to grain
$N_{d,j,c}$ = the design capacity for a Type 2 joint in compression as per section 7.2

The distances a_1, a_2 and $a_{3,c}$ shall be selected as per manufacturer's specifications.

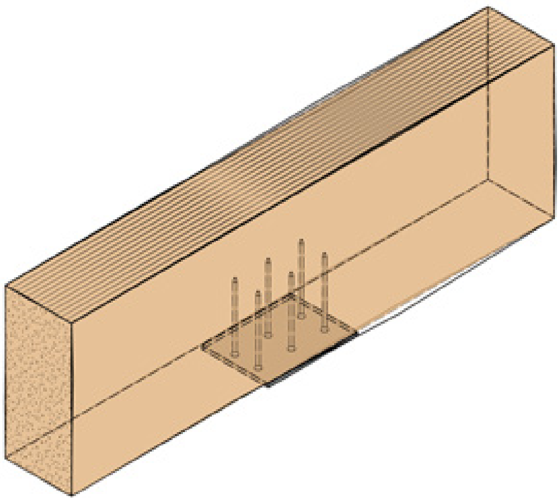


Figure 32: Bearing reinforcement with fully threaded screws and a steel bearing plate.

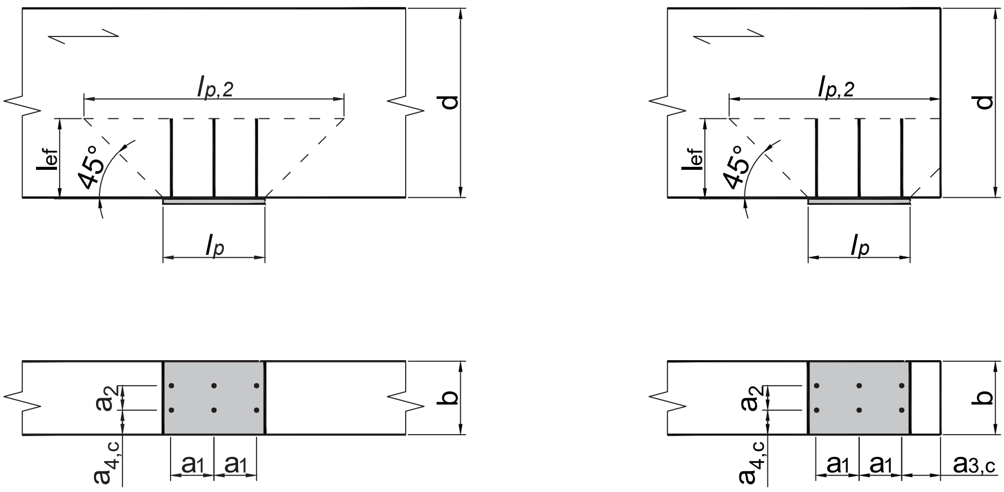


Figure 33: Bearing reinforcement for intermediate supports (left) and end supports (right).

NOTE: Reinforcement with glued-in rods can be designed with the same principles as outlined for fully threaded screws.

C7.1

Bearing reinforcement with fully threaded screws requires a steel plate in contact with the screws which is able to resist the direct bearing stress from the timber and the point loads from the screws. The screws shall be equally distributed over the bearing area. The screw heads shall be flush with the surface of the timber member.
When using screws in compression, the effective number of screws, n_{eff}, can always be taken as the number of screws n.

Differing from reinforcement used to resist tensile perpendicular to grain forces, the capacity of reinforced support is the sum of the perpendicular to grain compression capacity and the screws in compression (i.e. the two contributions are added).

Screws can also be inserted from both sides of the member in order to transfer the forces from the steel bearing plates through the timber. In this case equation (64) does not apply. The screw length and spacing requirements as shown in Figure C14 need to be applied for an effective force transfer. Screws should be arranged symmetrically to the bearing area. Figure C14 shows two possible screw arrangements with the alternative pattern leading to better performance.

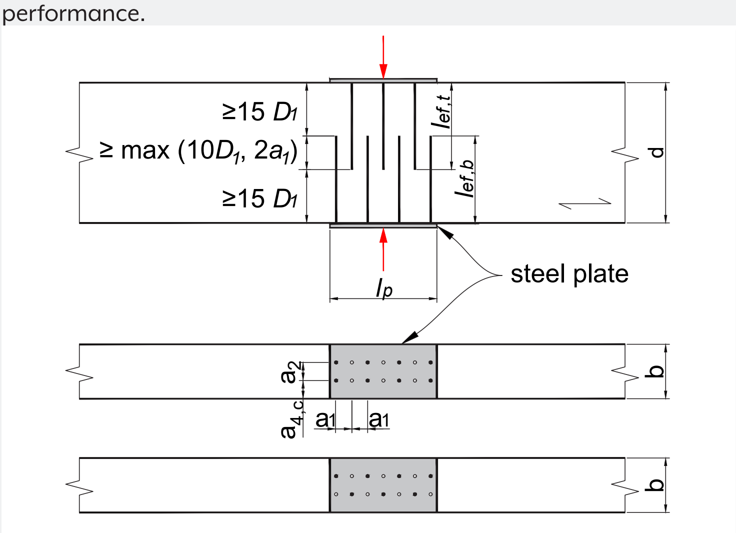


Figure C14: Bearing reinforcement with overlapping screws inserted from both sides.

Refer to Dietsch (2019) for more information on this type of bearing reinforcement.

C7.1 (continued)

To determine the bearing length at the tip of the screws $l_{p,2}$ a linear stress distribution of 45° has been assumed. This has been shown to be correct for direct load introductions (i.e. the load is applied directly opposite to the support, creating a direct load path through the beam), but leads to a small error for indirect load introductions (i.e. the load is applied away from the support, resulting in a more complex load path) as shown in Figure C15. In the case of indirect load introductions, simulations have shown that the stress distribution is exponential, resulting in the following more accurate effective bearing length.

$$l_{p,2} = l_p + 0.58\, l_{ef}\, e^{3.6 \frac{l_{ef}}{d}}$$ for intermediate support

$$l_{p,2} = l_p + 0.25\, l_{ef}\, e^{3.3 \frac{l_{ef}}{d}}$$ for end-supports

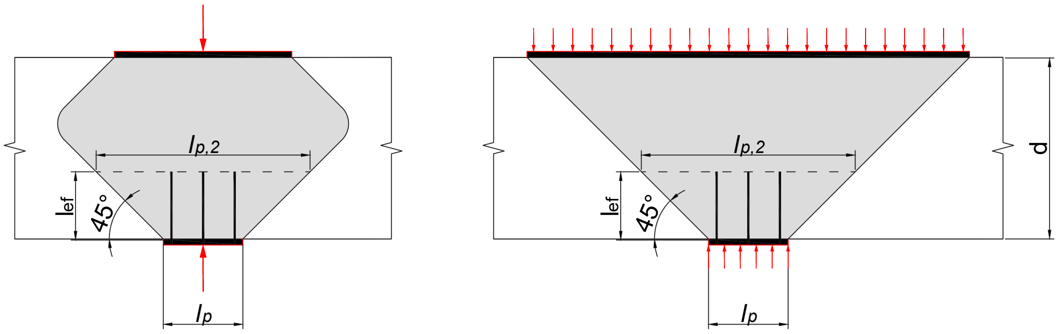


Figure C15: Load distribution with direct load introduction and effective bearing lengths.

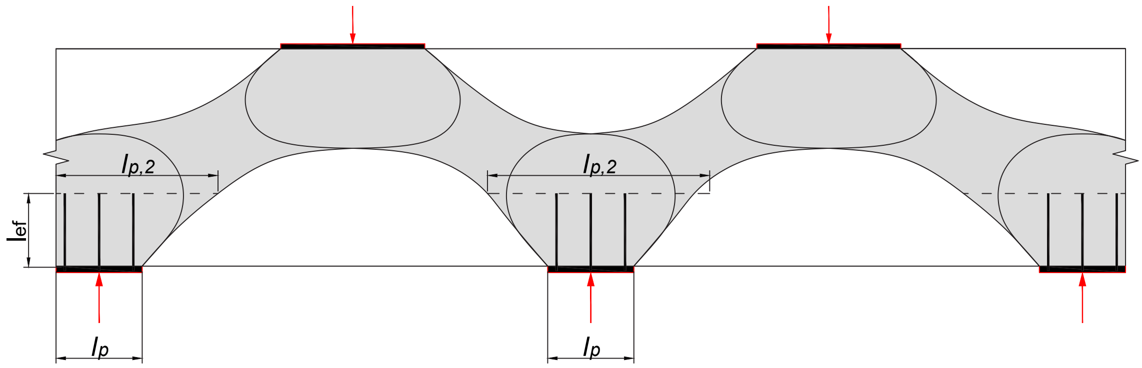


Figure C16: Load distribution with indirect load introduction and effective bearing lengths.

7.2 DESIGN CAPACITY FOR A TYPE 2 JOINT IN COMPRESSION

The design capacity ($N_{d,j,c}$) for a Type 2 joint (i.e. joint loaded in the axial direction, see Figure 4.1 NZS AS 1720.1) designed to resist axial loads in compression, for strength limit states shall be taken as:

$$N_{d,j,c} = \min\left(N_{d,j}, N_{d,cs}\right) \tag{65}$$

and

$$N_{d,j} = \phi\, k_1\, Q_k \tag{66}$$

where

ϕ = capacity factor for screws

k_1 = the factor for load duration for fasteners (to be taken as the minimum between NZS AS 1720.1 and manufacturer's specification)

Q_k = the characteristic withdrawal capacity of the screw given by the manufacturer's specification calculated for the effective thread length l_{ef} (refer to section 3.1).

$N_{d,cs}$ = the design capacity of the screw in compression given by the manufacturer's specification

C7.2

If the design capacity of the screw in compression ($N_{d,cs}$) is not provided by the manufacturer, the following procedure can be used for its determination:

$$N_{d,cs} = \phi_{st}\, \kappa_c\, N_{k,pl} \tag{C(11)}$$

Where the characteristic tensile plastic capacity of the screw ($N_{k,pl}$) can be calculated as:

$$N_{k,pl} = \frac{\pi D_2^2}{4} f_y \tag{C(12)}$$

The buckling factor (κ_c) for a steel member embedded in timber can be calculated as:

$$\kappa_c = \begin{cases} 1 & \text{for } \lambda_k \leq 0.2 \\ \dfrac{1}{k_c + \sqrt{k_c^2 - \lambda_k^2}} & \text{for } \lambda_k > 0.2 \end{cases} \tag{C(13)}$$

where

$$\kappa_c = 0.5\,[1 + 0.49(\lambda_k - 0.2)^2 + \lambda_k] \tag{C(14)}$$

$$\lambda_k = \sqrt{\frac{N_{k,pl}}{N_{k,e}}} \tag{C(15)}$$

and

$$N_{k,e} = \sqrt{(c_h\, E_s\, I_s} \tag{C(16)}$$

$$c_h = (0.19 + 0.012\,D)\,\rho_k \left(\frac{90° + \theta}{180°}\right) \tag{C(17)}$$

C7.2 (continued)

where

ϕ_{st} = the strength reduction factor of a steel member in compression as per NZS 3404

D = the outer diameter of the thread

D_2 = the inner diameter of the thread

θ = the angle between the screw axis and the grain direction

λ_k = the relative slenderness ratio of the screw

$N_{k,e}$ = the characteristic elastic buckling load of the screw embedded in timber

c_h = the elastic foundation modulus from the timber embedment

ρ_k = the characteristic density of the timber

f_y = the characteristic yield tensile strength of the screw

E_s = the modulus of elasticity of the screw

$I_s = \frac{\pi D_2^4}{64}$

= the moment of inertia of the screw at the thread

The modulus of elasticity of the screw can normally be taken as 210,000 MPa, unless specified otherwise by the manufacturer.

The compression capacity of the screw is the minimum value between its resistance from pushing-in (calculated as the withdrawal capacity) and its buckling strength. The buckling capacity presented here is based on the verification in Eurocode 2 and is based on a relative slenderness ratio, taking the elastic foundation stiffness from the embedded timber into account.

8 SHEAR REINFORCEMENT

8.1 STRENGTH AND LOAD DEMAND IN MEMBERS WITH SHEAR REINFORCEMENT

The shear strength in reinforced areas of sawn timber, glue laminated and laminated veneer lumber members shall satisfy the following:

$$V^* \leq \phi\, k_1\, k_\tau \frac{f_s\, A_s}{\eta_H} \tag{67}$$

where:

$$\eta_H = \frac{G\,b}{G\,b + \dfrac{1}{2\sqrt{2}\left(\dfrac{6}{\pi\, D\, d\, k_{ax}} + \dfrac{a_1}{E_s\, A_{st}}\right)}} \tag{68}$$

and:

$$k_\tau = 1 + 0.46\,\sigma_{90}^* - 0.052\,\sigma_{90}^{*\,2} \tag{69}$$

Where:

σ_{90}^* = the design stress perpendicular to the grain (take as negative for compression):

$$\sigma_{90}^* = \frac{N_{90}^*}{\sqrt{2}\, b\, a_1} \tag{70}$$

and:

$$N_{90}^* = \frac{\sqrt{2}\,(1 - \eta_H)\, V^*\, a_1}{d} \tag{71}$$

Where:

V^* = the design action effect in shear at the reinforced area

G = the shear modulus of the timber

b = the breadth of the beam

d = the depth of the beam

D = the outer thread diameter of the screw

k_{ax} = the stiffness between the fastener and the timber given by the manufacturer's specification

a_1 = the spacing of fasteners as per Figure 34

E_s = the modulus of elasticity of the screw

A_s = $\frac{2}{3}\, b\, d$

= the shear plane area

A_{st} = $\frac{\pi\, D_2^2}{4}$

= the net area of the screw at the thread

D_2 = the inner diameter of the thread

f'_s = characteristic value in shear

C8.1

The approaches in design of reinforcement in this guideline generally assume that the full tension stresses are carried entirely by the reinforcement. It should be noted however that the shear strength of timber is generally in the range of 5 times the tension strength and as such it would be uneconomical to completely discard this component. The equation above therefore looks at the relative stiffnesses in a composite section – timber and tensile steel – to assess the stress in the individual components.

Correctly arranged shear reinforcement creates a two-fold improvement on the shear strength of a beam. If the arrangement is chosen so that the shear reinforcement is loaded in axial tension, the resulting stresses in the glulam element are in compression perpendicular to the grain. Although not taken into account in the above formulation, compression stresses perpendicular to the grain have a positive effect on the shear capacity of the member meaning that the shear reinforcement leads not only to a reduction of shear stresses in the timber also but to an increase in the shear capacity of the element.

For additional information regarding the application of shear reinforcement, including application for the repair of an already fractured member or CLT refer to Design of shear reinforcement for timber beams (Dietsch et al. 2013).

8.2 SHEAR REINFORCEMENT WITH FULLY THREADED SCREWS

The design capacity ($N_{d,j}$) of the screw reinforcement shall satisfy the following:

$$N^*_{90} \leq N_{d,j} \quad \textbf{(72)}$$

where:

$N_{d,j}$ = the strength of the screw calculated in accordance with section 4.3.3

N^*_{90} = the design load effect in tension in the screw reinforcement as determined in section 8.1

A minimum of four fasteners along the member length shall be provided in each reinforced area. Reinforced areas shall be defined as shown in Figure 34. The length of the fastener shall extend over the full depth of the beam as shown in Figure 34.

NOTE: in the calculation of the withdrawal capacity of the fastener the effective penetration length l_{ef} shall be taken as 50% of the threaded length.

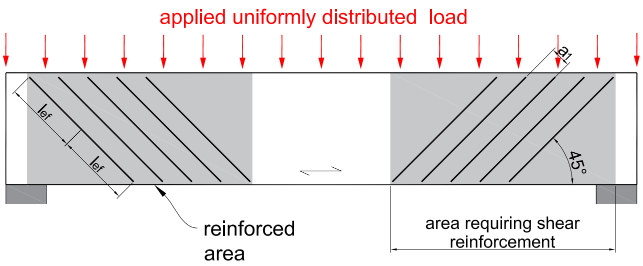


Figure 34: Beam with shear reinforcement.

9 JOINTS WITH MULTIPLE LARGE DOWEL TYPE FASTENERS

9.1 GENERAL

Joints with multiple large diameter dowel type fasteners can be reinforced with fully threaded screws in order to prevent premature splitting and row shear failure as it provides transverse restraint.

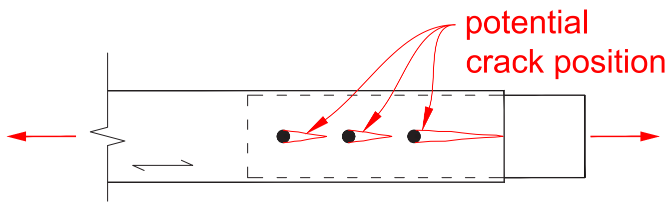


Figure 35: Potential failure of a joint with multiple dowel type fasteners.

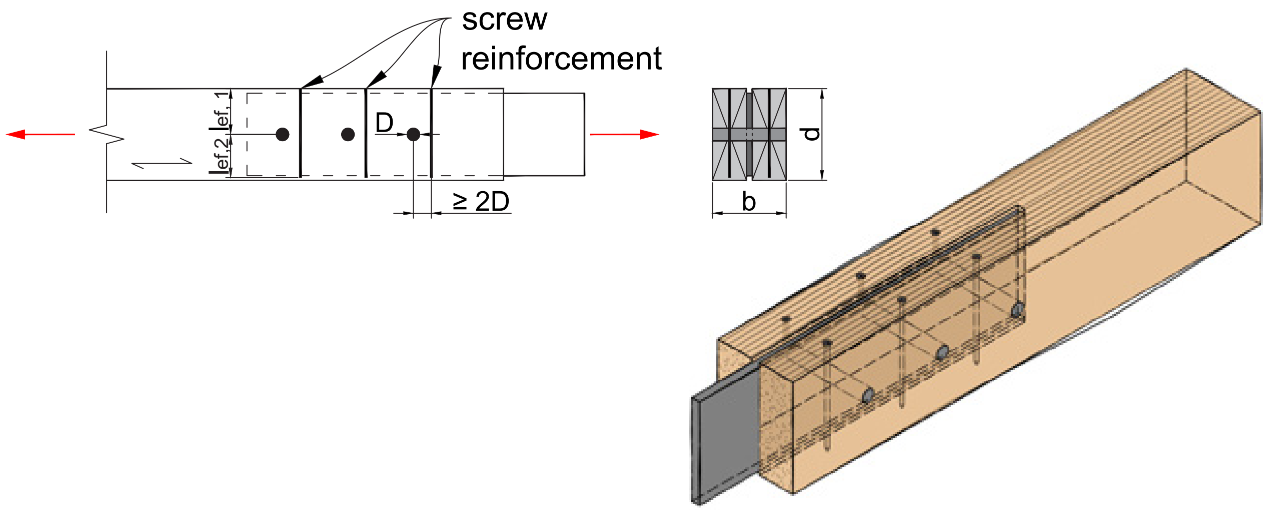


Figure 36: Reinforcement of a connection with multiple dowel type fasteners in one row.

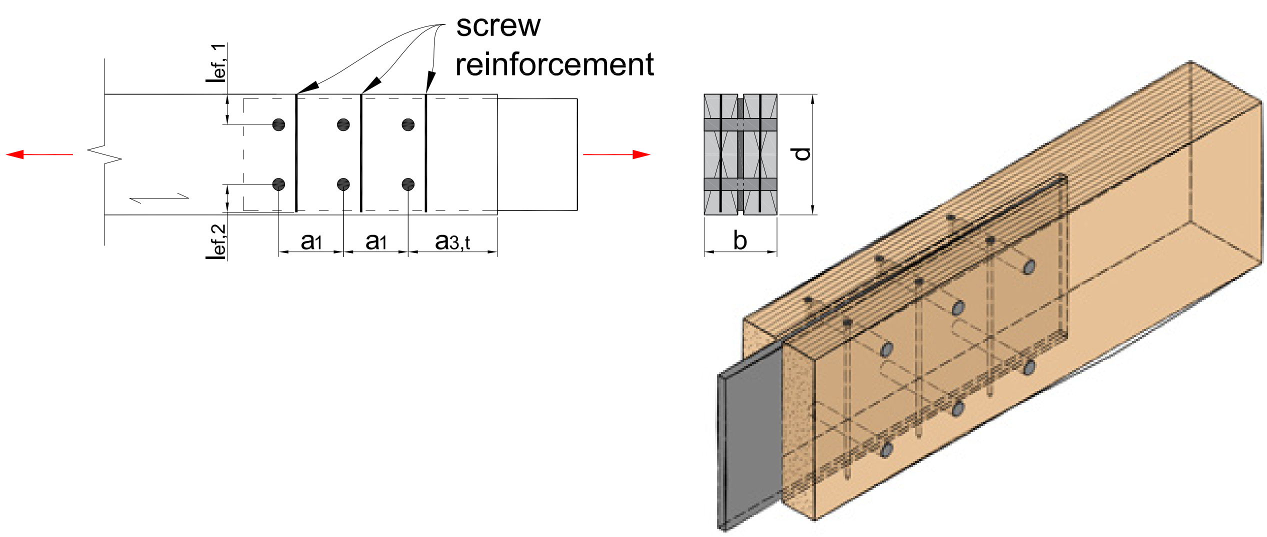


Figure 37: Reinforcement of a connection with multiple dowel type fasteners with multiple rows.

The screw reinforcement cannot prevent group tear-out failures, which should be checked in accordance with Appendix ZZ of NZS AS 1720.1.

C9.1

For load applications parallel to grain with large diameter dowel type fasteners, the onset of crack formation due to tension perpendicular to grain stresses can be limited by reinforcing the joint with fully threaded screws. The screw reinforcement acts as transverse restraint, avoiding timber splitting and row shear failure modes.

Reinforcement of large diameter dowel connections where they act as potential ductile elements is recommended , even if brittle failure modes of the timber members are prevented.

To account for potential splitting failure and shear lag in joints with multiple large dowel fasteners in a row, Eurocode 5 requires the use of an affective number of fasteners $n_{eff} \leq n$, with the ratio n_{eff}/n being the equivalent of factor k_{17} in NZS AS 1720.1 and k_{13} in NZS3603. By determining the row shear and group tear-out strength of the joints as required in NZS AS 1720.1, no reduction factors for multiple fasteners are now required.

The failure mechanisms of timber splitting and row shear cannot be strictly separated, as a loaded dowel would push the timber apart when deforming the fibres parallel to grain, causing splitting, it also tries to shear off the planes immediately adjacent to the dowel, causing row shear failures. Typically splitting is more dominant in smaller dowels and row shear in larger dowels, but the actual failure tends to be a mixed mode.

9.2 LOAD DEMAND PERPENDICULAR TO GRAIN IN CONNECTIONS WITH MULTIPLE DOWEL TYPE FASTENERS

The load demand in joints with multiple dowel type fasteners shall be calculated as:

$$N^*_{90,r} = 0.3\, k_{16}\; Q_k \qquad \textbf{(73)}$$

where

k_{16} = the factor for plywood or metal side plates as per NZS AS 1720.1

Q_k = the characteristic capacity per dowel type fastener and per shear plane

C9.2

The calculation model used for the derivation of the tension perpendicular to grain force in large dowel type fasteners was developed by Schmid (2002) and was derived for a single row of large dowel type fasteners. In case of multiple fastener rows it can be assumed that the fasteners in both rows develop the same tension force, which cancels out each other as shown in Figure C18.

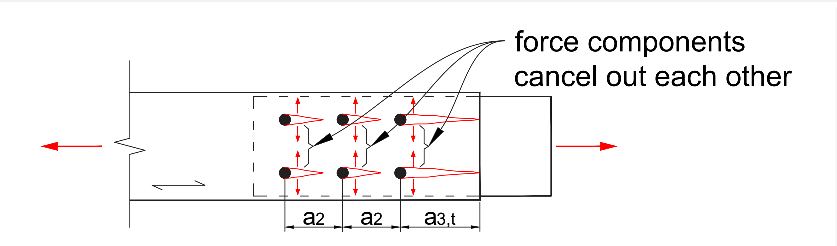


Figure C17: Tension forces perpendicular to grain in joints with multiple large dowel type fasteners in two rows.

9.3 REINFORCEMENT WITH FULLY THREADED SCREWS

The design capacity ($N_{d,j}$) of the screw reinforcement shall satisfy the following:

$$N_{d,j} \geq N^*_{90,r} \quad (74)$$

where $N_{d,j}$ is determined as per section 4.3.3 with the effective embedment lengths $l_{ef,1}$ and $l_{ef,2}$ as shown in Figure 36 and Figure 37.

The screw reinforcement shall be placed at a distance $2D$ from the dowel type fastener where D is the diameter of the dowel type fastener.

In case of load reversals in the joint, the screw reinforcement is to be positioned on both sides of the large diameter dowel type fastener.

C9.3

The crack propagation can be prevented by effectively connecting the two parts on either side of the potential crack location with fully threaded screws. In the case of multiple rows the screw reinforcement needs to connect the outer two portions of the member by either continuous screws with sufficient effective thread lengths $l_{ef,1}$ and $l_{ef,2}$ or by two screws inserted from either side of the member which can transfer the required force to each other over their lapping length.

The screw reinforcement installed at a distance from the dowel type fastener does not increase the capacity of the individual fasteners in the joint. For screw reinforcements installed in direct contact with the dowel type fasteners refer to Chapter E12 of Blaß and Sandhaas (2017).

10 MOMENT RESISTING CONNECTIONS

For moment resisting connections with more than one ring of dowel type fasteners, the connection strength shall be reduced by 15%, unless measures to prevent perpendicular to grain splitting are provided.

No reduction is required for moment resisting connections with a single ring of dowel type fasteners, unless the moment resisting connection is acting as a potential ductile element.

10.1 LOAD DEMAND PERPENDICULAR TO GRAIN IN CORNER AREAS OF MOMENT RESTING CONNECTIONS

The load demand perpendicular to grain in corner areas shall be calculated as

$$N^*_{90,r} = \frac{n_o}{12} \quad F^*_M \tag{75}$$

where

n_o = the number of dowel type fasteners in the outer ring

F^*_M = the design action effect in shear per fastener from the moment demand

NOTE: If the moment connection comprises fasteners inserted from both sides of the member, the number of dowel type fasteners in the outer rings on both sides needs to be considered.

C10.1

The load demand and direction of dowel type fasteners in moment resisting connections with fastener rings is dependent on the position of the fasteners. The loads create additional stresses in the timber members, which need to be considered in design. Any force components perpendicular to the member edges have the potential to create tension perpendicular to grain cracks.

Figure C18 shows the tension perpendicular to grain stresses for moment resisting connections with a circular and a rectangular fastener pattern. Rectangular connections lead to a more unfavourable situation, with perpendicular to grain tensile stresses over a larger area and unfavourable interaction between the tensile stresses perpendicular to grain and shear stresses across the member. For a rectangular fastener pattern, perpendicular to grain tension forces have been shown to be larger towards the end grain (Boult, 1988).

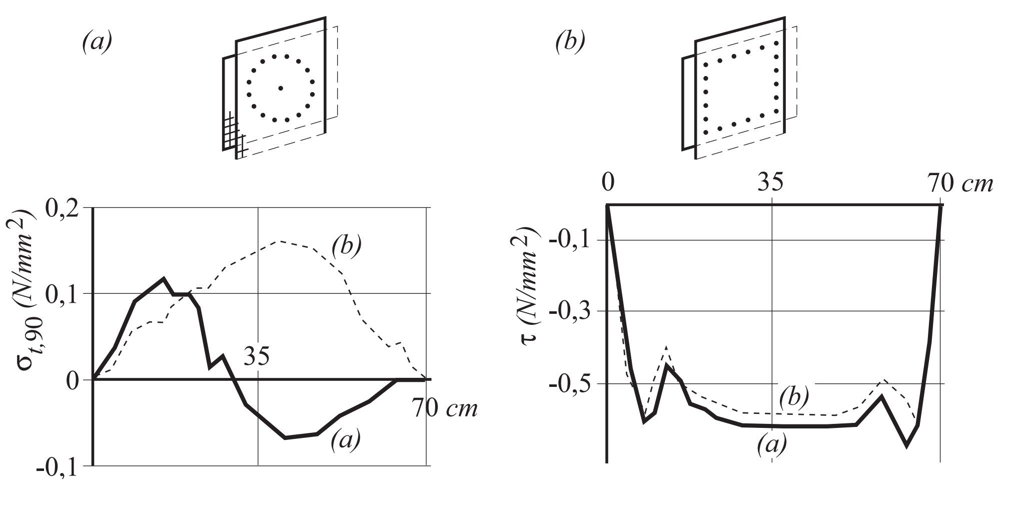


Figure C18: Stresses in moment resisting joints. Above: fastener arrangement in circular (a) and rectangular (b) pattern. Lower left: Perpendicular to grain stresses at 100mm from the member end. Lower right: shear stresses in the cross section at the centre of the joint (Racher and Gallimard, 1992).

C10.1 (continued)

The rules provided should be applied to all moment resisting connections in timber members, i.e. knee joints in portal knee connections, moment resisting frames in single and multi-storey construction, rigid beam column joints etc. Although no reduction is required for a single ring of fasteners, additional reinforcing should be installed if the connection is providing ductility under seismic loads, or when sensitive moisture variations in the members are expected.

In the case of gusset plates with nails or screws the load demand perpendicular to grain in corner areas $\underline{N^*_{90,r}}$ is normally small and can be resisted by the tension perpendicular to grain strength of the member. As a general check the following should be verified:

$$N^*_{90,r} \leq \phi\, k_1\;\; 0.5\, b\, l_{r,n} f'_{t,90} \qquad \textbf{C(18)}$$

where

ϕ = capacity factor for members

k_1 = the factor for load duration for members

$f'_{t,90}$ = the characteristic tensile strength perpendicular to grain (in MPa)

b = the breadth of the member

$l_{r,n}$ = $l_r - \frac{n_0}{8} D$

= half the length of the joint group l_r where onset of cracking is likely to occur minus the sum of holes created by the fasteners as shown in Figure C19

n_o = the number of dowel type fasteners in the outer ring

D = the diameter of the fastener

The factor 0.5 considers the non-uniform stress distribution and weakening of the member with the installation of the fasteners.

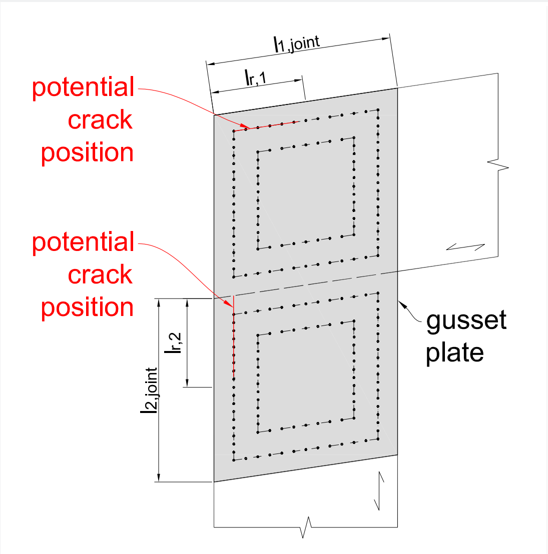


Figure C19: Possible crack locations joint (gravity load case) in a typical portal knee with gusset plates with small diameter dowel type fasteners.

10.2 REINFORCEMENT WITH FULLY THREADED SCREWS OR GLUED-ON PANELS

The design capacity ($N_{d,j}$) for the reinforcement shall satisfy the following:

$$N_{d,j} \geq N^{*}_{90,r} \qquad (76)$$

where $N_{d,j}$ is determined with screw reinforcement and glued-on panel reinforcement as per section 4.3.3 and section 4.3.5, respectively, with the effective embedment lengths $l_{ef,1}$ and $l_{ef,2}$ as shown in Figure 38. In moment resisting connections acting as potential ductile elements, the effective embedment length $l_{ef,2}$ should be measured from the inner dowel ring.

Reinforcement using fully threaded screws or glued-on panels shall be placed in the outer quarter of the connection area in both the rafter and the column in order to prevent any cracks parallel to grain along the outer ring of fasteners as shown in Figure 38.

For connections with plywood, cross-banded LVL or steel gusset plates, reinforcement only needs to be provided in the timber main member as per Figure 39.

Under seismic and wind loading, load reversals should be taken into consideration, requiring reinforcement also in the opposite corner area (note that these cases are not shown in Figure 38 and 39).

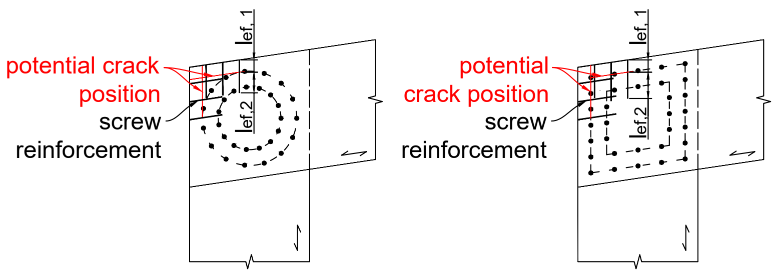


Figure 38: Corner reinforcement (gravity load case) for a lapped knee joint.

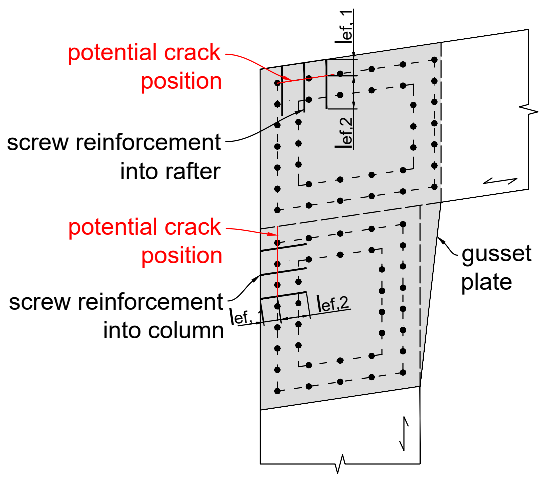


Figure 39: Corner reinforcement (gravity load case) with a gusset plate.

C10.2

Splitting of the timber members can be avoided by installing externally glued wood panels or reinforcements with fully threaded screws. The reinforcement should be applied at least in the outer quarter of the connection, i.e. in the 90° sector of the outer dowel ring.

Heimeshoff (1977) has shown that the tension perpendicular to grain forces are equal to the fastener shear demand in the 30° sector of the outer dowel ring as shown in Figure C20.

$$N^*_{90,r} = \frac{30}{360}\ n_o F^*_M = \frac{n_o}{12}\ F^*_M \qquad \textbf{C(19)}$$

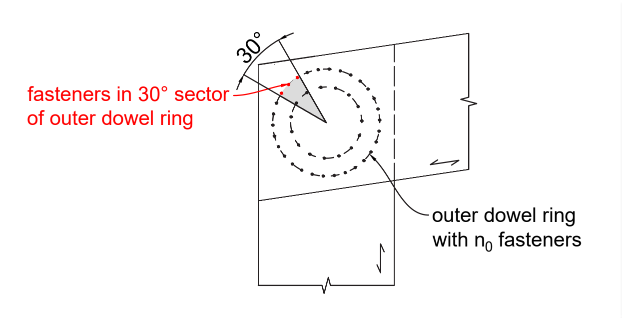


Figure C20: 30° sector of the outer dowel ring.

To prevent the formation of cracks, both the effective lengths $l_{ef,1}$ and $l_{ef,2}$ need to be long enough to transfer the load $N^*_{90,r}$ across the potential crack position. The withdrawal capacities of the screw given by the manufacturer's specification calculated for the smaller of $l_{ef,1}$ and $l_{ef,2}$ shall be used to verify the reinforcement. If $l_{ef,1}$ is not sufficient, a washer head screw could be used instead of fully threaded screws. In this case, instead of the withdrawal capacity for $l_{ef,1}$, the bearing capacity under the screw head shall be verified as per manufacturer's specification. The verification under withdrawal for $l_{ef,2}$ remains unchanged.

Under seismic and wind loading, load reversals should be taken into consideration, requiring reinforcement also in the opposite corner area.

For connections with external wood-based or steel gusset plates, the internal reinforcement should be installed as close as possible to the outer face of the inner member, as this is where cracking is induced by the moment resisting fasteners.

Figure C21 shows a moment resisting beam-column joint with the potential crack position and reinforcement of the beam. The figure also shows a potential secondary crack position in the column, but due to bigger area resisting perpendicular to grain stresses in the column when compared to the beam (smaller area due to the proximity to the end grain), cracks are less likely to form. Engineering judgement should be applied to determine if the column also needs reinforcement to prevent cracking.

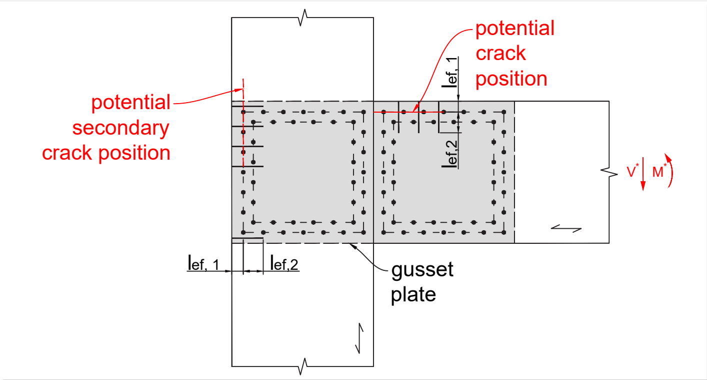


Figure C21: Reinforcement of a moment-resisting beam-column joint (reinforcement required for load reversals not shown for clarity).

C10.2 (continued)

Figure C22 shows the crack pattern for a simulated case of a moment-resisting beam-column joint with external steel plates. Due to displacement compatibility, cracks can only develop if the timber can displace on both side of the moment resisting connections. Onset of cracking normally occurs first towards the end of the member due to the smaller area resisting perpendicular to grain tension stresses. Typically, reinforcement is placed at the outer quarter of the connection towards the end of the member, thus preventing also cracking at the opposite corner.

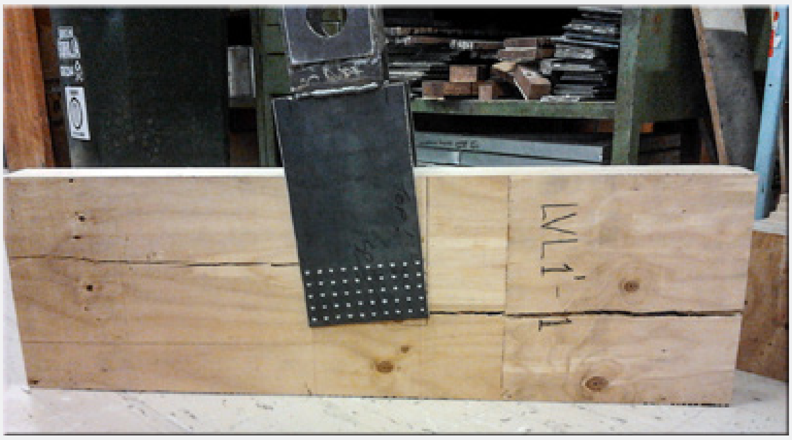
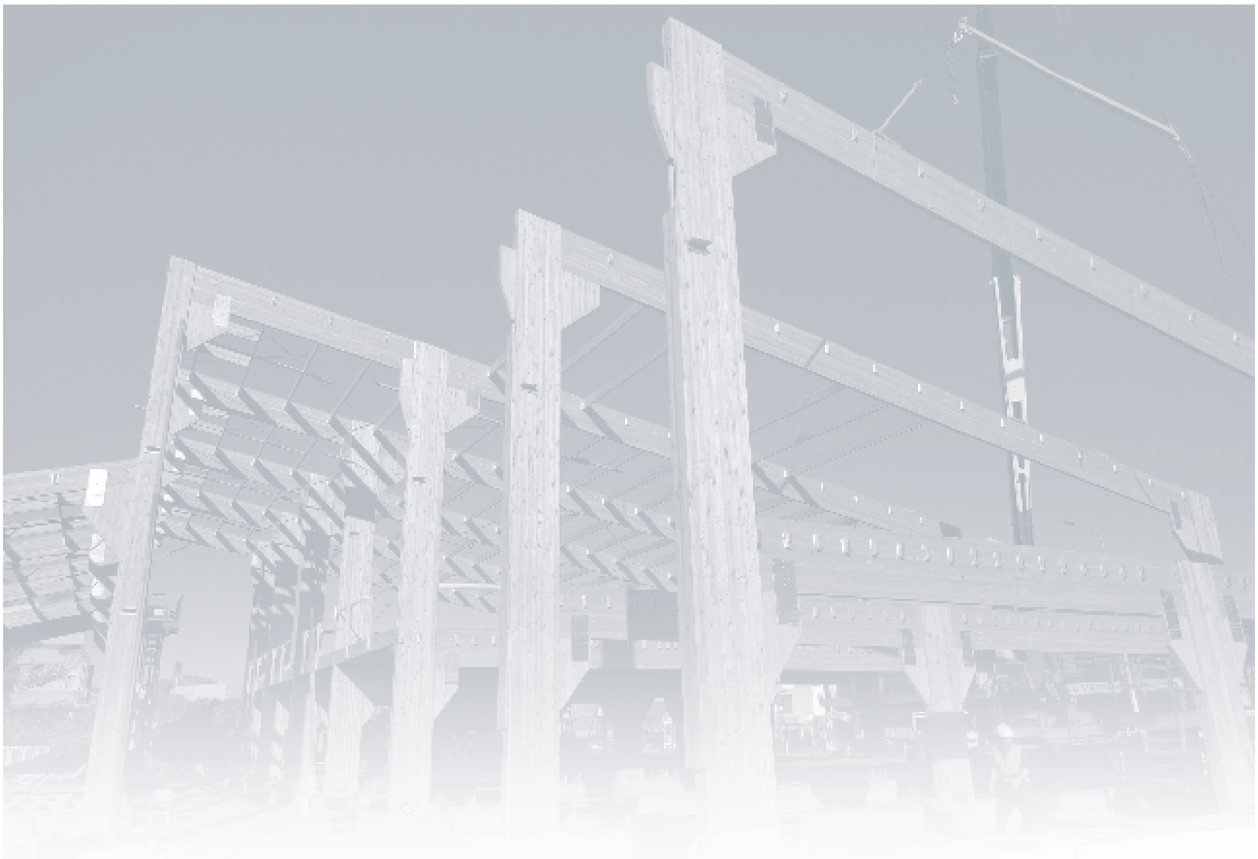
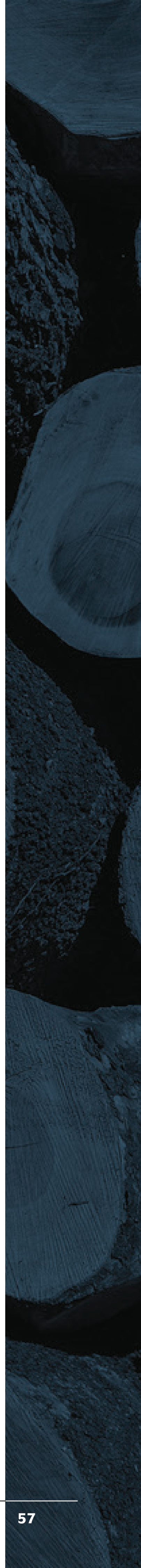


Figure C22: Experimental setup showing crack pattern of a moment resisting connection with external steel plates (c/o Pierre Quenneville, University of Auckland).

The requirement that the effective embedment length $l_{ef,2}$ is to be considered as starting from the inner dowel ring for ductile moment resisting connections is to account for the fact that under large displacement and force redistribution, large forces can also occur in the fasteners in the inner dowel ring and therefore cause splitting.

For more information refer to Blaß and Sandhaas (2017) and Blaß et al. (2004).





11 REINFORCEMENT AGAINST MOISTURE DEFORMATION IN THE PRESENCE OF STEEL PLATES

Moisture variation within a timber member will result in dimensional changes which may be restrained by steel plates in the connection. Perpendicular-to-grain shrinkage or swelling shall be considered in the joint design considering the moisture content at fabrication and in service. For more guidance on moisture effects and the likely shrinkage and swelling of wood refer to NZ Wood Design Guide Chapter 1.2 Trees, Timber, Species & Properties (NZ Wood, Design Guides, 2020).

NZS AS 1720.1 states that unless consideration is given to limit shrinkage stresses perpendicular-to-grain, a single steel splice plate shall not be used for rows of bolts, dowels or coach screws if the distance h_{max} between the two outside rows as shown in Figure 40 exceeds 125mm. For fasteners such as nails and screws, this limit is 200mm.

Considerations to limit shrinkage include the calculation and control of moisture around the timber member or the reinforcement against stresses created by moisture variation.

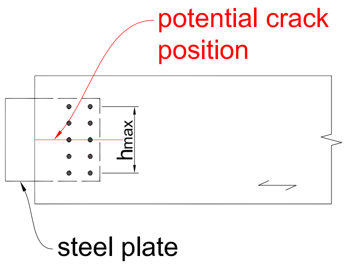


Figure 40: Maximum distance between fastener perpendicular to grain in presence of internal or external steel plates when in-service shrinkage or swelling is likely.

NOTE: The equation below will show high tension forces should large decreases in moisture content, Δ_{MC}, be used. These forces are possible and indicate that although reinforcement can be used to protect against small moisture movement, it does not replace careful moisture management, especially in the presence of large steel plates. The use of reinforcement will not prevent potentially visible shrinkage cracks from occurring but will ideally spread them to form several small cracks that will have less of an impact on connection performance. The screw must extend across the full width h_{max} and should ideally extend a nominal amount beyond the first and last bolt (or should be over the full member depth).

11.1 TENSION DEMAND IN REINFORCEMENT DUE TO SHRINKAGE

The tension demand from shrinkage between the two outermost fasteners in a connection shall be calculated as:

$$N^*_{90,r} = g_1 \frac{\Delta_{MC}}{(FSP - 12)} \gamma E A_s \tag{77}$$

where

g_1 = factor for the percentage shrinkage when drying from green to 12% moisture content, See C11.1.

Δ_{MC} = percentage change in moisture content

FSP = the fibre saturation point (as a percentage moisture content, 29% for NZ Radiata Pine, 27% for Douglas Fir. 25% for Macrocarpa. More values can be found in NZ Wood Design Guide Chapter 1.2 Trees, Timber, Species & Properties (NZ Wood, Design Guide, 2020)

E_s = the modulus of elasticity of the screw

$A_s = \frac{\pi D_2^2}{4}$

= the net area of the screw at the thread

D_2 = the inner diameter of the thread and

$$\gamma = \frac{1}{1 + \frac{12}{(2l_{ef})^2 \lambda^2}} \qquad \textbf{(78)}$$

where

$$l_{ef} = \frac{h_{max}}{2}$$

= the effective length of the screw taken as the distance between the two outside rows divided by 2

and

$$\lambda = al_{ef} + b \qquad \textbf{(79)}$$

$$a = (-0.039D^2 + 1.172D - 11.994) \cdot 10^{-6} \qquad \textbf{(80)}$$

$$b = (0.027D^2 - 0.858D + 12.285) \cdot 10^{-3} \qquad \textbf{(81)}$$

NOTE: The above equation provides an approximate λ that is valid for 150mm $\leq h_{max} \leq$ 1000mm

C11.1

A common situation where base plates are larger than the limits set in NZS AS 1720.1 and moisture movement may create cracking is at the column base, see Figure C23.

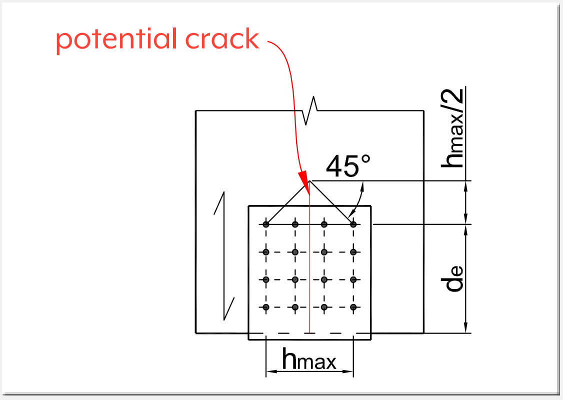


Figure C23: Column base connection with potential cracking under shrinkage.

The shrinkage characteristics (g_1 and FSP) change with the species of timber in which the plate is embedded. Some values for commonly used New Zealand species are provided in NZ Wood Guide Chapter 1.2 – Trees, timber species and properties (NZ Wood, Design Guides, 2020). It is usually unknown if the tangential or radial % shrinkage is appropriate so a conservative assumption should be made. The g_1 and FSP for Radiata Pine are 0.039 and 29%, respectively.

The stiffness perpendicular to the grain of timber E_{90}' can be taken as $E_0'/30$.

The factor γ is derived assuming a constant shrinkage induced strain along the reinforcing screw The axial load flow $n(x)$ can therefore also be assumed linear along the screw length:

$$n(x) = n_0 \left(1 - \frac{2x}{l_s}\right) \qquad \textbf{C(20)}$$

Where n_0 is the axial load flow at the head and point of the screw and l_s is the length of the screw (the distance between fasteners also in this case equal to h_{max}).

Differentiating the axial load flow in the screw gives the axial load in the screw, a quadratic:

$$N(x) = n_0 \left(x - \frac{x^2}{l_s}\right) \qquad \textbf{C(21)}$$

C11.1 (continued)

Where the maximum axial load occurs at mid-length:

$$N_{Max} = \frac{n_0 l_s}{4} \qquad \text{C(22)}$$

Re-writing Eq. 14 by Dietsch et al. (2013) the total deformation along the screw length can be written as

$$u(x) = \frac{n(x)}{k} + \frac{1}{EA_S}\int N(x)\,dx \qquad \text{C(23)}$$

Thus solving the integral of the screw axial force gives:

$$\frac{1}{EA_s}\int N(x)\;dx = \frac{n_0}{EA_s}\left(\frac{x^2}{2} - \frac{x^3}{3l_s}\right) + \text{const} \qquad \text{C(24)}$$

Where the integration constant is found by assuming that the displacement at mid-screw is zero:

$$\frac{n_0}{EA_s}\left(\frac{l_s^2}{8} - \frac{l_s}{24l_s}\right) + \text{const} = 0 \rightarrow \text{const} = \frac{n_0}{EA_s}\;\frac{l_s^2}{12} \qquad \text{C(25)}$$

Therefore, substituting in the above u_0 is as follows:

$$u_0 = \frac{n_0}{k} + \frac{n_0}{EA_s}\left(\frac{l_s^2}{12}\right) \qquad \text{C(26)}$$

and the factor γ for this loading condition is:

$$\gamma = \frac{1}{1+\dfrac{12EA_s}{(2l_{ef})^2\,k}} \qquad \text{C(27)}$$

The factor k is calculated in accordance with Eq. 13 by Dietsch et al. (2013) using the factor λ which (calculated is iteratively for a given K_{ax}). For screws with l_{ef} up to 500mm the factor λ can be found shown in Equations (79) to (81).

11.2 EFFECT OF MULTIPLE REINFORCING FASTENERS

The load sharing factor k_{65} shall be used to account for the use of multiple fasteners.

$$k_{65} = \frac{b\left(d_e + \dfrac{h_{max}}{2}\right)}{n\,h_{max}\;\max\,(h_{max}, b)} \qquad \textbf{(82)}$$

Where:

d_e = the distance from the loaded edge to the fastener furthest away from the loaded edge
n = the number of fasteners in the influence area
b = the breadth of the member

C 11.2

It should be noted that Equation (77) is independent of the area of timber being protected and in theory the addition of screw area will lead to a linear increase in screw force. This does not account, however, for the interaction between the stiffness of the screw group and the timber being reinforced. Should a single screw not be able to resist the tension force, k_{65} can be used to account for $N^*_{90,r}$ in multiple screws across the area being disturbed by the bolts.

As the two systems, the area being disturbed by the bolts and the area being reinforced, are bound together the factor k_{65} creates a ratio between the two areas to assess the influence of multiple fasteners. The two areas considered are shown in Figure C24. The area of influence of one screw is calculated by drawing a 45° line of influence from where the screw passes the outermost bolt. As such, the perimeter of the area of influence of one screw is h_{max} but bounded by the width of the member b. In order for the screw group to be as effective as possible, the bottom screw should be placed at a distance $h_{max}/2$ away from the edge of the member.

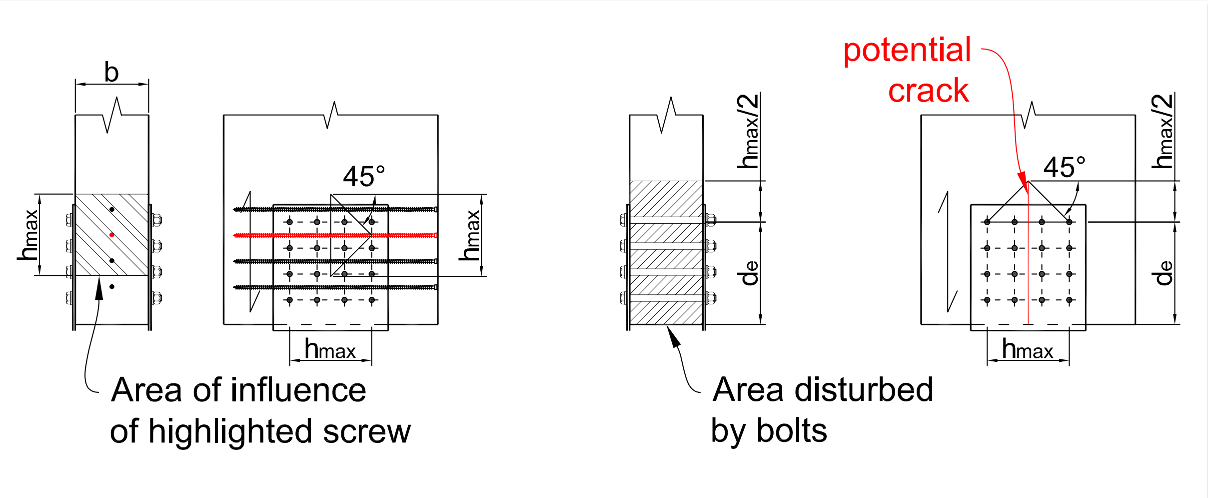


Figure C24: Area of influence of a screw and area disturbed by the bolt group.

NOTE: Engineering judgement will need to be used when assessing the area of influence of the screws group. If the screw area of influence is outside of the area distributed by the bolts or is too close to the end of the member this area should not be used in the calculation of the ratio k_{65}.

12 REINFORCEMENT OF GLUED-IN ROD CONNECTIONS

The effect of shrinkage will also create tension perpendicular to the grain stresses in glued-in rod connections where those rods are subsequently connected to rigid supports, such as a steel portal knee hub. The same considerations as described in Section 11.1 shall apply to these situations.

The addition of glued in rods as reinforcement will also provide protection against splitting failures in the location of the main glued-in rods.

C 12

A typical rule for the transverse reinforcement of glued rod connections is to provide at least 1/25th of the area of the main rods. (Buchanan and Moss, 1999). The steel area could be provided by glued in rods or fully threaded screws/ rods, should cross any potential split line and be approximately 50mm from the end of the member.

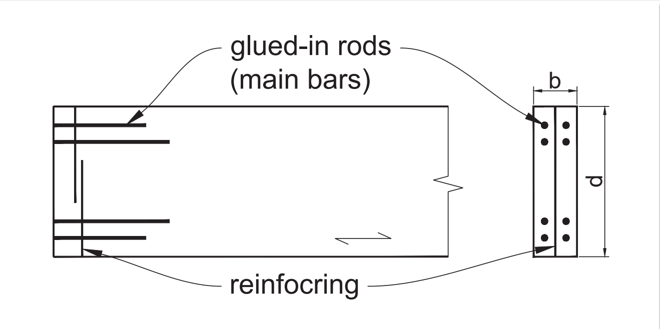


Figure C25: Transverse reinforcement due prevent splitting (modified from Buchanan 2007).

This reinforcement will also provide protection against sudden failure in shear should the main bars also be loaded in shear. This situation can be verified as a notched connection, as the rods loaded in shear are acting as a support at a distance d_{notch} away from the member edge, see section C4.1.

13 CURVED, DOUBLE-TAPERED AND PITCHED CAMBERED BEAMS

The design of curved, double-tapered and pitched-cambered beams is outside the scope of this guide, due to the relative complexity of stress distributions in curved members and the infrequent use of curved and cambered beams in New Zealand in general.

In cases where high moisture content variations are expected or the design check of the moment capacity due to the tensile strength perpendicular to grain is not verified, the *German National Annex to Eurocode 5* (DIN, 2013) or the guidebook *Timber engineering - Principles for design* (Blaß and Sandhaas, 2017) provide guidance to design the required reinforcement.

14 RETROFIT AND REPAIR OF TIMBER MEMBERS

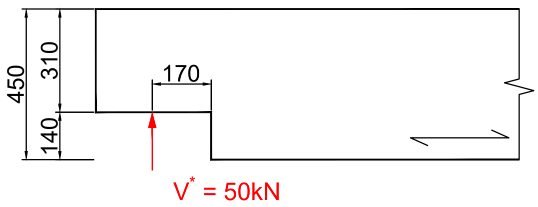
Retrofitting or repairing existing structures requires good knowledge of the structural behaviour of the members in question, as well as the state they are in and, in the case of repair work, the reason why they have failed. All reinforcement strategies shown in this guide can be applied to retrofit or repair existing members, with the best method depending on aesthetics, accessibility and state of the members.

Engineering judgment will be required to determine the best retrofit or repair strategy for the specific structure or member, and it is recommended that this type of work is carried out by engineers who have sufficient experience in the field. Valuable guidance can be found in Franke et al. 2014 and other international literature.

15 DESIGN EXAMPLES

15.1 REINFORCEMENT OF NOTCHES WITH FULLY THREADED SCREWS

A beam under a combination of 1.2G + 1.5Q is notched to fit over a support as shown below:



At the end of the notch there is a moment demand of:

$$M^* = 50 \times 0.17 = 8.5 \text{ kNm}$$

The full member is a 135mm wide GL10 beam. Using the unreinforced verification from Section E9 of NZS AS 1720.1 we see the section is not verified:

$$\frac{6M^*}{bd_n^2} + \frac{6v^*}{bd_n} \leq \phi g_{40}\, k_1\, k_4\, k_6\, k_{12}\, f'_{sj}$$

NOTE: The characteristic value in shear at joint details f'_{sj} is 4.2 MPa according to tables H2.2 and H2.4 of NZS AS 1720.1 for NZ Radiata Pine with a strength group SD6

l_{notch}/d_{notch} = 0 and $d_{notch} > 0.1d$ therefore:

$$g_{40} = 9.0/d^{0.45} = 9.0/450^{0.45} = 0.58$$

Therefore:

$$\frac{6 \times 8.5}{135 \times 310^2} + \frac{6 \times 50}{135 \times 310} = 7.17 > 0.8 \times 0.58 \times 0.8 \times 1 \times 1 \times 1 \times 4.2$$
$$= 1.56 \therefore \text{NO GOOD}$$

If we look then at the alternative verification from Equation (1):

$$\frac{1.5V^*}{bd_n} \leq \phi\ g'_{50}\ k_1\ k_4\ k_6\ k_{12}\ f'_{sj}$$

Where:

$$g'_{50} = \frac{k_{50}\left(1 + \frac{1.1\, i^{1.5}}{\sqrt{d}}\right)}{\sqrt{d}\left(\sqrt{(\alpha_r - \alpha_r^2} + 0.8\frac{l_{support}}{d}\sqrt{\frac{1}{\alpha_r} - \alpha_r^2}\right)}$$

$$\alpha_r = \frac{d_n}{d} = \frac{310}{450} = 0.69$$

$$i = \frac{l_{notch}}{d_{notch}} = \frac{0}{140} = 0$$

As the beam is made from GL10 k_{50} = 6.5:

$$g'_{50} = \frac{6.5 \times \left(1 + \frac{1.1 \times 0^{1.5}}{\sqrt{450}}\right)}{\sqrt{450}\left(\sqrt{0.69 - 0.69^2} + 0.8\frac{170}{450}\sqrt{\frac{1}{0.69} - 0.69^2}\right)} = 0.40$$

$$\frac{1.5 \times 50}{135 \times 310} = 1.79 > 0.8 \times 0.40 \times 0.8 \times 1 \times 1 \times 1 \times 4.2 = 1.08$$
$$\therefore \text{NO GOOD}$$

Therefore, the notch beam requires reinforcement. The force on the reinforcement is calculated using Equations (3) and (4):

$$k_{51} = 3(1-\alpha_r)^2 - 2(1-\alpha_r)^3 = 3(1-0.69)^2 - 2(1-0.69)^3 = 0.23$$

$$M^*_{90,r} = 1.3\,k_{51}\,V^* = 1.3 \text{ x } 0.23 \text{ x } 50 = 15.0\text{kN}$$

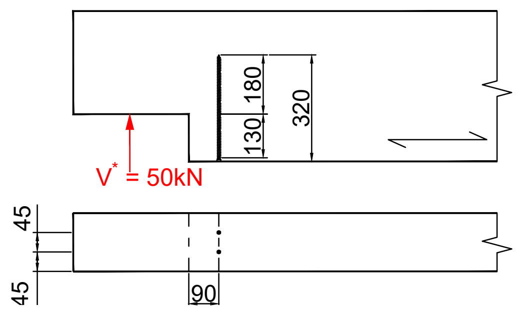
If we use screws to reinforce the panel, we need to develop the strength of the screw both above and b elow the location of the crack. This means that the withdrawal strength of the screw will be limited by $d_{notch} = 140\text{mm}$.

If we use 1 row of two 9mm screws with an embedment length of 125mm the manufacturer's literature provides us with a characteristic withdrawal capacity of $Q_k = 14.21$ kN and a tensile capacity of the screw of $N_{d.ts} = 0.9 \text{ X } 25.4 \text{ kN} = 22.9 \text{ kN}$. Using Equation (6) the design capacity becomes:

$$N_{d.j} = \ \phi\, k_1\, n\, Q_k = 0.8 \text{ x } 0.8 \text{ x } 2 \text{ x } 14.21 = 18.19 \text{ kN} \therefore \text{NO GOOD}$$

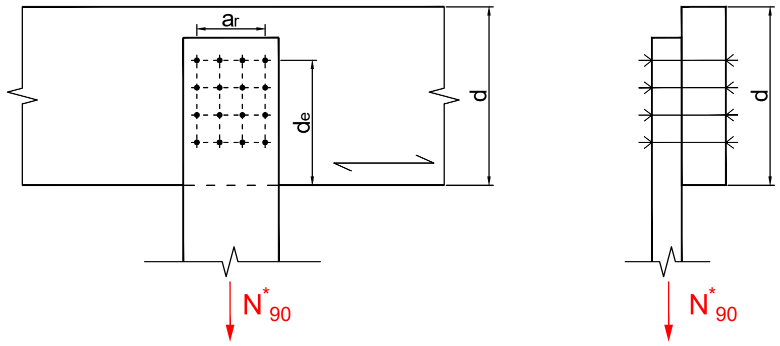
The other limiting factor on the reinforcement is that the screw embeds beyond 70% of the member so the effective length needs to be at least 0.7 X 410 = 287mm. Selecting a 320mm long screw satisfies this criterion.

The joint configuration therefore become



15.2 REINFORCEMENT OF JOINTS LOADED PERPENDICULAR TO GRAIN WITH FULLY THREADED SCREWS

A beam is to be hung off an LVL member and the bolted connection must be low in order for the joist connection to fit above the connection plate. The demand on the plates is 130 kN and a group of nine M16 bolts are used arranged as per the below:



Using Equation (22) we can check the beam to see is reinforcement is required. Firstly, we calculation the factors k53 and k54 using Equation C(3) 24 and C(4)25, respectively.

$$k_{53} = \max\left\{\begin{array}{l}1\\ 0.7+1.4\,\frac{a_r}{d}\end{array}\right. = \max\left\{\begin{array}{l}1\\ 0.7+1.4\times\frac{160}{550}\end{array}\right. = 1.11$$

$$k_{54} = \max\ \frac{n_r}{\sum_{i=1}^{n}\left(\frac{h_1}{h_i}\right)^2} = \frac{3}{\left(\frac{325}{325}\right)^2+\left(\frac{325}{405}\right)^2+\left(\frac{325}{485}\right)^2} = 1.43$$

$$N_{90,w} = \phi\ k_1\ k_{15}\ k_{53}\ k_{54}\left[6.5+18\left(\frac{d_e}{d}\right)^2\right]\ (t_{ef}\ d)^{0.8}\ f_{t,90}$$

$$= 0.9\times0.8\times1\times1.11\times1.43\ \times\left[6.5+18\left(\frac{225}{550}\right)^2\right](189\times550)\times0.5 = 56.1\text{ kN}$$

$$< 130\text{ kN} \therefore \text{NO GOOD}$$

Therefore, we need to reinforce the joint. Using Equations (20) and (21) we calculate the required strength of the reinforcement:

$$\alpha_r = \frac{d_e}{d} = \frac{225}{550} = 0.41$$

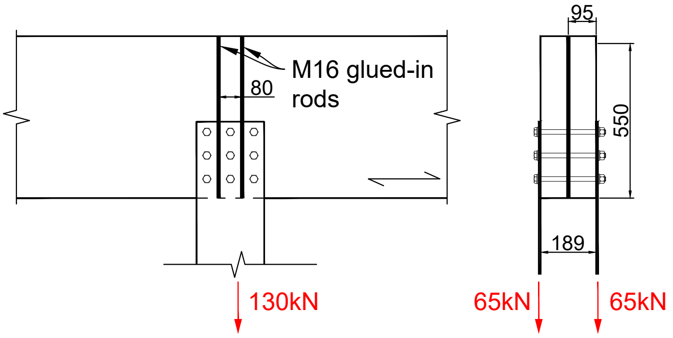
$$k_{56} = 1 - 3\alpha_r^2 + 2\alpha_r^3 = 1\text{-}3\times0.41^2 + 2\times0.41^3 = 0.63$$

$$N^*_{90,r} = k_{56}\ N^*_{90} = 0.63\times130 = 81.9\text{ kN}$$

Using Table 29.3 of Chapter 29 of the Timber Design Guide we know we need 163mm to fully develop the strength of a mild steel M16 threaded rod. As we have 225mm below location of the top row of bolts we need to check the tension strength of the rod to define the number required:

$$N^*_{90,r} \leq N_{d,r} = \phi_{st}\,nN_{tf} = \phi_{st}\,nA_s\,f_y = n\times0.9\times157\times300 = n\times42.4\text{ kN}$$

We therefore need 2 rods to achieve the required capacity ($2\times42.4 = 84.8\text{ kN} > 81.9\text{ kN} \therefore$ OK) and the joint configuration becomes:

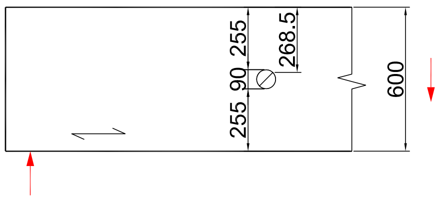


The spacing between bars and edge distance should be larger than $5D_r$ (80mm) and $2.5D_r$ (40mm), respectively. From the figure above we can see these limits are adhered to.

15.3 BEAMS WITH PENETRATIONS

15.3.1 Unreinforced penetration in an LVL beam

A 600mm deep and 126mm wide LVL 11 secondary beam requires a 90mm penetration in order to run a services duct. At the penetration location a shear and moment demand of 22 kN and 39 kNm, respectively, have been determined.



The hole diameter is less than 15% of the beam depth and hence the penetration does not need to be reinforced.

The design capacity in bending of the net section can be verified by using equation (26), where the terms $M^{*}_{b,a}$ and $M^{*}_{b,u}$ can be ignored for circular sections. The design action effect in shear can be verified by using equation (32). The net section is deemed verified by inspection.

The shear V_{cr} and moment M_{cr} at which onset of cracking occurs can be determined with equations (39) and (40), respectively. In order to use these equations, we first need to calculate the parameters λ and k_{cr} with equations (41) and (42):

$$k_{cr} = \frac{f'^{2}_{t,90}}{2\, G_f \left(\dfrac{1}{1 - \dfrac{d_{cr}}{d}}\right)^2} = \frac{(0.5\text{MPa})^2}{2 \times 0.7\ \text{N/mm} \left(\dfrac{1}{1 - \dfrac{268.5\text{mm}}{600\text{mm}}}\right)^2} = 0.0545$$

$$\lambda^4 = \frac{k_{cr}\ b}{4E\ I_{cr}} = \frac{0.0545 \times 126\text{mm}}{4 \times 11{,}000\ \text{MPa} \times 203{,}246{,}076\text{mm}^4} = 7.68 \times 10^{-13}$$

where the depth of the cracked beam section dcr for a circular hole is calculated as

$$d_{cr} = d_b + 0.15 d_d = 255\text{mm} + 0.15 \times 268.5\text{mm}$$

The moment of inertia of the cracked beam section I_{cr} can be calculated as

$$I_{cr} = \frac{b\, d_{cr}^3}{12} = \frac{126\text{mm} \times (268.5\text{mm})^3}{12} = 203{,}246{,}076\text{mm}^4$$

The section modulus of the cracked beam section Z_{cr} can be calculated as

$$Z_{cr} = \frac{b\, d_{cr}^2}{6} = \frac{126\text{mm} \times (268.5\text{mm})^2}{6} = 1{,}513{,}93\text{mm}^3$$

The shear V_{cr} and moment M_{cr} at which onset of cracking occurs hence are

$$V_{cr} = \frac{b f'_{t,90}}{\sqrt{4\lambda^2 + \dfrac{6 k_{cr}\, b}{5\, G A_{cr}}}} = \frac{126\text{mm} \times 0.5\text{MPa}}{\sqrt{4\sqrt{7.68 \times 10^{-13}} + \dfrac{6 \times 0.0545 \times 126\text{mm}}{5 \times 550\text{MPa}\ (126\text{mm} \times 268.5\text{mm})}}} = 31.7\text{kN}$$

$$M_{cr} = \frac{b\, f'_{t,90}}{2\lambda^2} = \frac{126\ \text{mm} \times 0.5\text{MPa}}{2\sqrt{7.68 \times 10^{-13}}} = 35.9\text{kNm}$$

The bending stress due to the cracking moment M_{cr} can be calculated as

$$\sigma_M = \frac{M_{cr}}{Z_{cr}} = \frac{35.9\text{kNm}}{1{,}513{,}937\text{mm}^3} = 23.7\text{MPa}$$

The design capacity in shear and moment of the beam at the penetration can be calculated with equations (37) and (38), respectively.

$$V_{d,t} = \phi\ k_1\ 2V_{cr} = 0.9 \times 0.8 \times 2 \times 31.7\text{kN} = 45.6\text{kN}$$

$$M_{d,t} = \phi\, k_1\, [2M_{cr} + b\, \sigma_M\ (d_{cr} + k_{64}\ d_d\,)(d - d_{cr})]$$
$$= 0.9 \times 0.8 \times [2 \times 35.9\text{kNm} + 126\text{ mm} \times 23.7\text{MPa}\,(268.5\text{mm} + 0.7 \times 90\text{mm})(600\text{mm} - 268.5\text{mm})] = 236.7\text{ kNm}$$

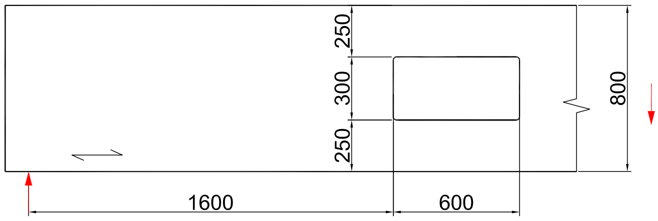
Finally, the beam section capacity can be verified with equation (36)

$$\frac{V^*}{V_{d,t}} + \left(\frac{M^*}{M_{d,t}}\right)^2 = \frac{22\text{kN}}{45.6\text{kN}} + \left(\frac{39\text{kNm}}{236.7\text{kNm}}\right)^2 = 0.48 + 0.03 = 0.51 \leq 1 \therefore \text{OK}$$

The LVL beam with the 90 mm unreinforced penetration is therefore verified.

15.3.2 Reinforced penetration in a glulam beam

An 800mm deep and 180 mm wide GL10 beam requires a 300mm high and 600 long penetration in order to run a services duct. At the penetration location a shear and moment demand of 45 kN and 60 kNm, respectively, have been determined.



The hole depth is less than 40% of the beam depth, which is the maximum limit for externally reinforced penetrations. The length a = 600mm of the penetration is less than 2.5 d_d = 750mm. The clear distance between the penetration and the support is larger than the beam depth and hence all geometric limitations for reinforced glulam beams are verified.

To determine the design action effects in tension perpendicular to grain $N^*_{90,r}$, the design action effect in tension perpendicular to grain from shear and moment action can be determined with equations (44) and (45), respectively. For this, factors k_{59} to k_{61} need to be determined:

$$k_{59} = 0.83 + 0.013\ \frac{a}{d_d} = 0.83 + 0.013\ \frac{600}{300} = 0.86$$

$$k_{60} = 0.57 + 0.533\ \frac{a}{d_d} = 0.57{+}0.533\ \frac{600}{300} = 1.64$$

$$k_{61} = 0.05 + 0.113\ \frac{a}{d_d} = 0.05 + 0.113\ \frac{600}{300} = 0.28$$

The design action effect in tension perpendicular to grain due to shear and bending can hence be determined.

$$N^*_{90,r,V} = \frac{V^*}{4}\left(\frac{k_{59}\,d_d}{d}\right)\left[3-\left(\frac{k_{59}\,d_d}{d}\right)^2\right]\left[1+k_{60}\left(\frac{k_{59}\,d_d}{d}\right)\right]$$

$$= \frac{45\text{kN}}{4}\left(\frac{0.86\times 300\text{mm}}{800\text{mm}}\right)\left[3-\left(\frac{0.86\times 300\text{mm}}{800\text{mm}}\right)^2\right]\left[1+1.64\left(\frac{0.86\times 300\text{mm}}{800\text{mm}}\right)\right] = 16\text{kN}$$

$$N^*_{90,r,M} = \frac{0.1\,M^*}{d}\left(\frac{k_{59}\,d_d}{d}\right)^2\left[1+k_{61}\left(\frac{k_{59}\,d_d}{d}\right)\right]$$

$$= \frac{0.1\times 60\text{kNm}}{800\text{mm}}\left(\frac{0.86\times 300\text{mm}}{800\text{mm}}\right)^2\left[1+0.29\left(\frac{0.86\times 300\text{mm}}{800\text{mm}}\right)\right] = 0.8\text{kN}$$

$$N^*_{90,r,} = N^*_{90,r,V} + N^*_{90,r,M} = 16\text{kN} + 0.8\text{kN} = 16.8\text{kN}$$

Although reinforcement will always be required for this penetration due to its size, the procedure to verify the capacity of the unreinforced section will be shown here for completeness.

The design capacities in tension perpendicular to grain resisting shear and moment action, respectively, can be determined with equations (34) and (35), respectively,

$$N_{90,d,n,V} = \phi\,k_1\,k_{57}\;0.5\,l_{t,90,V}\;b\,f'_{t,90}$$
$$= 0.8\times 0.8\times 1.22\times 0.5\times(1.3\times 300\text{mm})\times 180\text{mm}\times 0.5\text{MPa} = 13.7\text{kN}$$

$$N_{90,d,n,M} = \phi\,k_1\,k_{57}\;0.5\,l_{t,90,M}\;b\,f'_{t,90}$$
$$= 0.8\times 0.8\times 1.22\times 0.5\times(0.5\times 300\text{mm})\times 180\text{mm}\times 0.5\text{MPa} = 5.3\text{kN}$$

$$k_{57} = \left(\frac{10^7}{0.225\,b\,d_d^2}\right)^{0.2} = \left(\frac{10^7}{0.225\times 180\text{mm}\times(300\text{mm})^2}\right)^{0.2} = 1.22$$

The capacity of the beam with the unreinforced penetration can be verified with equation (33)

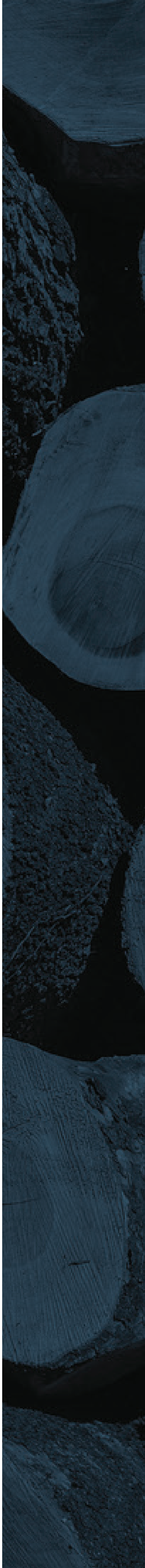
$$\frac{N^*_{90,r,V}}{N_{90,d,n,V}} + \frac{N^*_{90,r,M}}{N_{90,d,n,M}} = \frac{16\text{kN}}{13.7\text{kN}} + \frac{0.8\text{kN}}{5.3\text{kN}} = 1.17 + 0.15 = 1.32 > 1 \;\therefore \text{NO GOOD}$$

The penetration hence will need to be verified because of the high perpendicular to tension force.

Before designing the required reinforcement, the capacity net beam section at the penetration needs to be verified.

The moment capacity can be verified with equation (26), which is equal to equation (27) as the penetration is cantered at mid-height of the beam.

The shear demand in the portions of the beam above and under the penetration can be calculated as:



$$V_{b,a} = V_{b,u} = V^* \frac{d_{b,a}}{d_{b,a} + d_{b,u}} = 45\text{kN} \frac{250\text{mm}}{250\text{mm} + 250\text{mm}} = 22.5\text{kN}$$

The moment due to frame action hence becomes

$$M^*_{b,a} = M^*_{b,u} = V_{b,a} \frac{a}{2} = 22.5\text{kN} \times \frac{600\text{mm}}{2} = 6.75\text{kN}$$

The flexural beam strength can now be verified with equation (26)

$$\frac{M^*}{M_{d.n}} + \frac{M^*_{b,a}}{M_{b,d,a}} + \frac{N^*}{N_d} = \frac{60\text{kNm}}{194.6\text{kNm}} + \frac{6.8\text{kNm}}{24.3\text{kNm}} + 0 = 0.59 \leq 1 \therefore \text{OK}$$

This verification has been carried out with the following section properties

$$Z_{b,a} = Z_{b,u} = \frac{b\, d^2_{b,a}}{6} = \frac{180\text{mm} \times (250\text{mm})^2}{6} = 1{,}875{,}000\text{mm}^3$$

$$I_n = b\left[\frac{d^3}{12} + d\left(\frac{d}{2} - y_g\right)^2 - \frac{d^3_d}{12} - d_d\left(d_{b,a} + \frac{d_d}{2} - y_g\right)^2\right] = 7{,}275{,}000{,}000\text{mm}^4$$

$$Z_n = \min\left(\frac{I_{net}}{y_g}\,;\,\frac{I_{net}}{d - y_g}\right) = \frac{7{,}275{,}000{,}000\text{mm}^4}{400\text{mm}} = 18{,}187{,}500\text{mm}^3$$

and the following design capacities assuming a fully restrained beam

$$M_{d.n} = \phi\, k_1\, k_8\, k_{24}\, Z_n f'_b$$
$$= 0.8 \times 0.8 \times 1.0 \times \left(\frac{150}{800}\right)^{0.167} \times 18{,}187{,}500\text{mm}^3 \times 22\text{MPa} = 194.6\text{kNm}$$

$$M_{d,ba,n} = M_{d,bu,n} = \phi\, k_1\, k_8\, k_{24}\, Z_{b,a}\, f_b'$$
$$= 0.8 \times 0.8 \times 1.0 \times \left(\frac{150}{250}\right)^{0.167} \times 1{,}875{,}000\text{mm}^3 \times 22\text{MPa} = 24.3\text{kNm}$$

The shear strength of the beam can be verified with equation (32) with a net shear area of

$$A_{s,n} = \frac{2}{3} b\,(d - d_d) = \frac{2}{3} 180\text{mm}\,(800\text{mm} - 300\text{mm}) = 60{,}000\text{mm}^2$$

$$V_{d,n} = \phi\, k_1\, A_{s,n} f'_v = 0.8 \times 0.8 \times 60{,}000\text{mm}^2 \times 3.7\text{MPa} = 142\text{kN} > V^* = 45\text{kN} \therefore \text{OK}$$

Note that the additional requirements as per section 6.2.4 do not need to be applied for penetrations with external reinforcement.

To reinforce the penetration, an externally glued-on 19mm thick F8 grade plywood panel will be used. The design capacity $N_{d,r}$ as per equation (55) is based on design action effect in shear at the glued interface of the glued-on panel and the timber member $V_{d,sj}$ as per equation (58) and the design action effect in tension of the glued-on panel as per equation (59):

$$V_{d,sj} = \phi_a \ k_1 \ k_{19} f_a \ A_{sj} = \phi_a \ k_1 \ k_{19} f_a \ A_{sj}$$
$$= 0.7 \times 0.8 \times 1.0 \times 0.7\text{MPa} \times 40{,}000\text{mm}^2 = 15.7\text{kN}$$

$$N_{d,t} = \phi \ k_1 \ k_{52} \ t_r \ l_r \ f'_t$$
$$= 0.9 \times 0.8 \times 0.5 \times 11.2\text{mm} \times 200\text{mm} \times 15\text{MPa} = 12.1\text{kN}$$

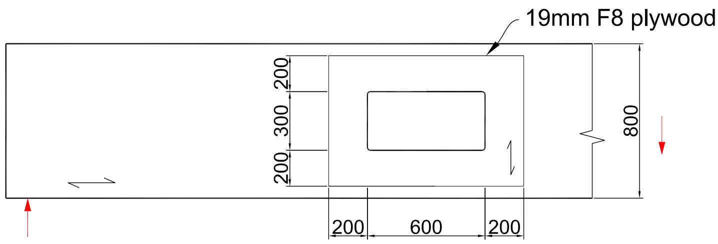
$$A_{sj} = (l_r \ d_{sj}) = 200\text{mm} \times 200\text{mm} = 40{,}000\text{m}^2$$

In the above equation it was assumed that the net total thickness t_r of the plywood plies in the parallel direction is 11.2mm. The reinforcement can hence be verified with equation (55) as

$$N_{d,r} = 2 \times \min\left(V_{d,sj}, N_{d,t}\right) = 2 \times \min\left(15.7\text{kN}, 12.1\text{kN}\right) = 24.2\text{kN} \geq N^*_{90,r} = 16.8\text{kN} \therefore \text{OK}$$

In addition, also the geometric limitations as per equations (60) and (61) need to be satisfied

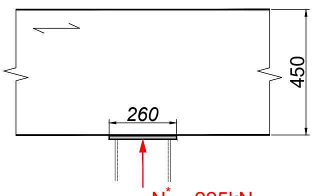
$$0.25\ a = 0.25 \times 600\text{mm} = 150 \leq l_r = 200\text{mm} \leq 0.3\ (d_d + d)$$
$$= 0.3\ (300 + 800) = 330\text{mm} \therefore \text{OK}$$
$$d_r = 200\text{mm} \geq 0.25\ a = 0.25 \times 600\text{mm} = 150\text{mm} \therefore \text{OK}$$



The 300mm x 600m with glued-on 19 mm plywood panels on either side of the beam is hence verified.

15.4 BEARING REINFORCEMENT WITH FULLY THREADED SCREWS

The connection below has a required capacity of $N^* = 225$ kN.



The width of the beam is 135mm and is made of glue laminated timber ($GL10$, $\rho_k = 500\ \text{kg/m}^3$). If we assume the plate is the full width of the member the strength in bearing becomes:

$$N_{d,p} = \phi \ k_1 \ k_4 \ k_6 \ k_7 \ f'_p \ A_p = 0.8 \times 0.8 \times 1 \times 1 \times 1 \times 4.5 \times 135 \times 260 = 101.09 \text{ kN}$$

We therefore require reinforcement to be able to transfer the bearing load.

First, we need to find the required strength on the screw group:

$$nN_{d,j,c} \geq N^*_p - \phi \ k_1 \ k_4 \ k_6 \ k_7 \ f_p \ A_p = 225 - 101.09 = 123.91 \text{ kN}$$

We also need to find out how far into the member the screw will need to penetrate to transmit the bearing force to allow for timber only bearing. The minimum area $A_{p,2}$ can be found by:

$$A_{p,2} \geq \frac{N^*_p}{\phi \ k_1 \ k_4 \ k_6 \ k_7 \ f'_p} = 78{,}125\text{mm}^2$$

If we assume that the two external screws are 200mm apart:

$$l_{ef} \geq \left(\frac{A_{p,2}}{b} - 200\right)/2 = \left(\frac{78125}{135} - 200\right)/2 = 189\text{mm}$$

We now need to know the strength of a screw in compression. If we use a screw with a thread diameter $D = 10\text{mm}$ and an inner thread or shank diameter $D_2 = 7\text{mm}$ with a yield strength fy $= 1000$ MPa, the characteristic pull out strength from supplier's literature is $Q_k = 26$ kN.

The member strength of the screw in accordance with equation C(11) is:

$$N_{d,cs} = \phi_{st}\,\kappa_c\,N_{k,pl}$$

where:

$$N_{k,pl} = \frac{\pi D_2^2}{4}\,f_y = \frac{\pi \times 7^2}{4} \times 1000 = 38.48\text{ kN}$$

in order to calculate κ_c we need the factors k_c from equation C(14), λ_k from C(15), $N_{k,e}$ from equation C(16) and c_h from equation C(17).

$$c_h = (0.19 + 0.012D_1)\,\rho_k\left(\frac{90°+\theta}{180°}\right) = (0.19 + 0.012 \times 10) \times 500 \times \left(\frac{90°+\theta}{180°}\right) = 155$$

$$N_{k,e} = \sqrt{c_h E_s I_s} = \sqrt{155 \times 210000 \times \frac{\pi \times 7^2}{64}} = 61.94\text{ kN}$$

$$\lambda_k = \sqrt{\frac{N_{k,pl}}{N_{k,e}}} = \sqrt{\frac{38.48}{61.94}} = 0.79$$

$$k_c = 0.5\,[\,1 + 0.49\,(\lambda_k - 0.2) + \lambda_k^2\,] = 0.5\,[1 + 0.49\,(0.79 - 0.2) + 0.79^2 = 0.96$$

as $\lambda_k > 0.79$:

$$k_c = \frac{1}{k_c + \sqrt{k_c^2 - \lambda_c^2}} = \frac{1}{0.96 + \sqrt{0.96^2 - 0.79^2}} = 0.66$$

therefore the design capacity of the screw in compression is:

$$N_{d,cs} = 0.9 \times 0.66 \times 38.48 = 22.86\text{ kN}$$

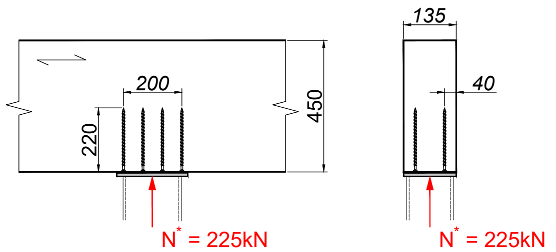
The design capacity is therefore:

$$N_{d,j,c} = \min\begin{cases}\phi k_1\,Q_k = 0.8 \times 0.8 \times 26 = 16.64\\ N_{d,cs} = 22.86\end{cases} = 16.64\text{ kN}$$

If we reinforce with 2 lines of 4 screws with an effective length $l_{ef} = 220\text{mm}$ ($> 189\text{mm}$) the joint strength becomes:

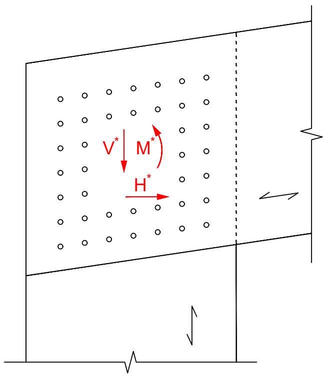
$$N_{d,p,r} = \phi\,k_1\,k_4\,k_6\,k_7\,f'_p\,A_p + nN_{d,j,c} = 101.09 + 8 \times 16.64 = 234.21\text{ kN} > 225\text{k N} \therefore \text{OK}$$

The joint configuration therefore becomes:



15.5 MOMENT RESISTING CONNECTIONS

The moment connection below comprises of an internal column between two rafters. It is to be reinforced against splitting under a design moment $M^* = 230\ \mathrm{kNm}$, a vertical shear of $V^* = 60\ \mathrm{kN}$ and horizontal shear of $H^* = 50\ \mathrm{kN}$.



From analysis of the dowel group this leads to a force on the outermost dowel of 27.6 kN at an angle of 45° in the column and 60° in the beam. The outer dowel ring is comprised of $n_0 = 24$ dowels.

Applying equation (75) from Section 10.1:

$$N^*_{90,r,} = \frac{24}{12} \times 27.6\mathrm{kN} = 55.2\ \mathrm{kN}$$

In order to find the reinforcing requirements on the rafter the force is divided by the number of rafters:

$$N^*_{90,r,} = \frac{55.2\mathrm{kN}}{2} = 27.6\ \mathrm{kN}$$

If we use three 280 mm long 9 mm screws we have an embedment length above and below the dowel at the top of the rafter of $l_{ef,1} = 155\mathrm{mm}$ and $l_{ef,2} = 125\mathrm{mm}$, respectively. The manufacturer's literature provides a characteristic strength for the screw of $Q_k = 14.21\ \mathrm{kN}$. If three screws are used the total capacity of the connection becomes:

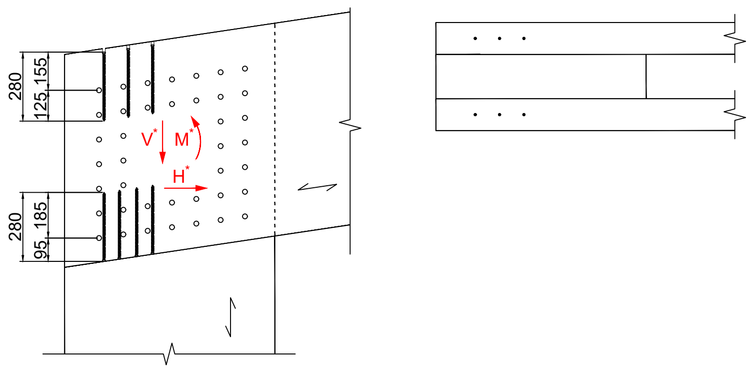
$$N_{d,j} = \phi\ k_1\ \ n\ Q_k = 0.6 \times 1.14 \times 3.0 \times 14.21 = 29.16\ \mathrm{kN} > 27.6\ \mathrm{kN}$$

A k_1 of 1.14 has been used in this case as the moment demand is from wind loading. If the moment demand was from an overstrength seismic force and the connection was being designed as a potential ductile element, the strength reduction factor could be set to 1.

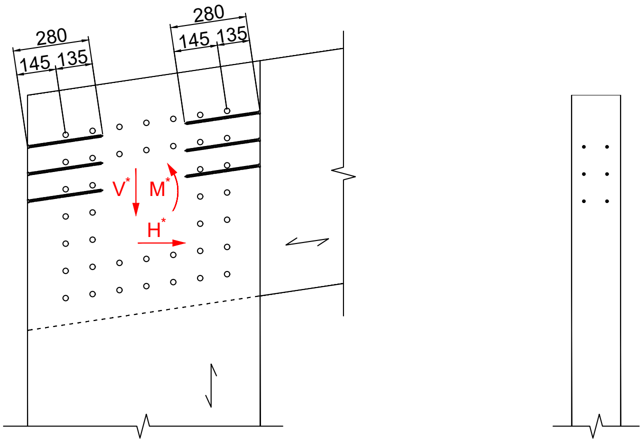
At the base of the rafter we have a reduced distance between the dowel at the bottom edge of the member and $l_{ef,1} = 95\ \mathrm{mm}$. For this reduced embedment length, the manufacturer's literature provides a characteristic strength for the screw of $Q_k = 10.80\ \mathrm{kN}$. If four screws are used the total capacity of the connection becomes:

$$N_{d,j} = \phi\ k_1\ \ n\ Q_k = 0.6 \times 1.14 \times 4.0 \times 10.80 = 29.54\ \mathrm{kN} > 27.6\ \mathrm{kN} \therefore \mathrm{OK}$$

The reinforcing pattern in the rafter therefore becomes:

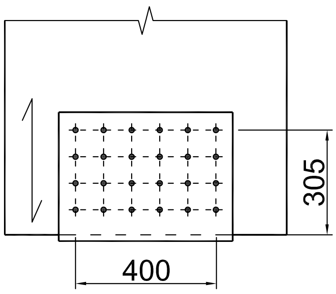


Due to the column being subjected to twice the demand, double the number of screws will be required to provide the required capacity. These can be place side by side as shown in the figure below.



15.6 REINFORCEMENT IN PRESENCE OF LARGE STEEL PLATES DUE TO TIMBER SHRINKAGE

A portal frame has been designed with a 800mm x 180mm column base connection and dimensions as shown below:



As the environment in the building when complete will be open there is concern that the moisture content in the wood may fluctuate between 18% and 10% over the life of the building. The decision is made to reinforce with fully threaded screws to protect against the opening of large cracks over the life of the building.

The column is made of Radiata Pine therefore the g_1 and FSP are 0.039 and 29%, respectively.

If we are to reinforce with a 9mm screw, $D = 9\text{mm}$ and $D_2 = 6.5\text{mm}$ ($A_s = 33.2\text{mm}^2$, $E = 210$ GPa).
Using equations (79) to (81) we can calculate λ:

$$a = (-0.039d^2 + 1.172d - 11.994) \cdot 10^{-6} = (-0.039 \times 9^2 + 1.172 \times 9 - 11.994) \cdot 10^{-6} = -4.61 \cdot 10^{-6}$$

$$b = (0.027d^2 - 0.858d + 12.285) \cdot 10^{-3} = 0.027 \times 9^2 - 0.858 \times 9 + 12.285) \cdot 10^{-3} = 6.75 \cdot 10^{-3}$$

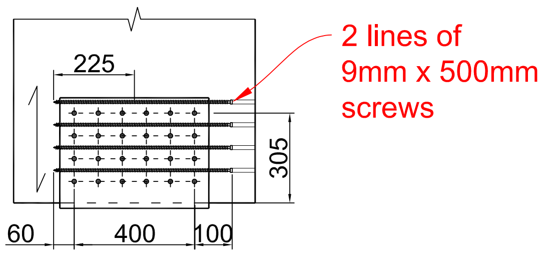
$$l_{ef} = \frac{h_{max}}{2} = \frac{400}{2} = 200\text{mm}$$

$$\lambda = al_{ef} + b = -4.61 \cdot 10^{-6} \times 200 + 6.75 \cdot 10^{-3} = 5.8 \cdot 10^{-3}$$

$$\gamma = \frac{1}{1 + \frac{12}{(2l_{ef})^2 \lambda^2}} = \frac{1}{1 + \frac{12}{(2 \times 200)2 \times (5.8 \cdot 10^{-3})^2}} = 0.31$$

$$N^*_{90,r,} = g_1 \frac{\Delta_{Mc}}{FSP - 12} \gamma E A_{st} = 0.039 \times \frac{8}{29 - 12} \times 0.31 \times 210000 \times 33.2 = 39.7 \text{ kN}$$

We want to reinforce the full area of influence of the bolts so we will place eight 9mm screws as shown below:



Calculating k_{65}

$$k_{65} = \frac{b\left(d_e + \dfrac{h_{max}}{2}\right)}{n\,\dfrac{h_{max}}{2}\,\max\left(\dfrac{h_{max}}{2}, b\right)} = \frac{180 \times \left(305 + \dfrac{400}{2}\right)}{8 \times 200 \times 180} = 0.32$$

Therefore, the load on one screw is $k_{65}\, N^{*}_{90,r} = 0.32 \times 39.7 = 12.7 \text{ kN}$

From here we need to check the embedment of the screw. The manufacturers information shows us that with 225mm embedment $Q_k = 26.5 \text{ kN}$. Therefore:

$$N_{d,j} = \phi k_1 Q_k = 0.6 \times 0.8 \times 26.5 = 12.7\text{kN} = 12.7\text{kN} \therefore \text{OK}$$

The screws extend 60mm past the last bolt, which is deemed sufficient.

NOTE: The potential moisture content change we have identified in this case is seasonal and likely to have a duration of 5 - 6 months, hence k_1 of 0.8 has been used. If the moisture content change was identified as being more permanent (i.e. drying of wet timber following manufacture or construction) k_1 should be adjusted accordingly (i.e. $k_1 = 0.57$)

16 ANNOTATION (NEW DESIGN FACTORS)

g_1	=	factor for the percentage shrinkage when drying from green to 12% moisture consent
g'_{50}	=	alternative modification factor for notched beams
k_{50}	=	factor for notched members
k_{51}	=	notch reinforcement factor
k_{52}	=	reduction factor due to non-uniform stress distribution
k_{53}	=	factor to account for the increased tension strength perpendicular to grain with multiple adjacent fasteners
k_{54}	=	factor to account for joints with multiple fastener rows
k_{55}	=	factor to account for multiple joint groups
k_{56}	=	reinforcement factor for joints loaded perpendicular to grain
k_{57}	=	factor to account for the stressed volume in beams with penetrations
k_{58}	=	factor to account for the increased shear demand in internally reinforced penetrations
k_{59}, k_{60}, k_{61}	=	factors for the load demand in timber and glulam beams with penetrations
$k_{62, 63}$	=	factors for the load demand in LVL beams with penetrations
k_{64}	=	shape factor for unreinforced LVL beams with penetrations
k_{65}	=	factor to account for load sharing in shrinkage reinforcement

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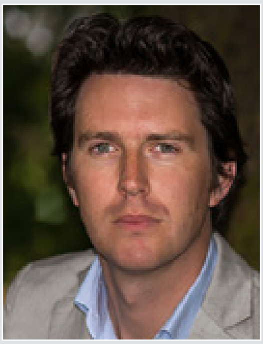
ABOUT THE AUTHORS

Daniel Moroder is a structural engineer at PTL | Structural Consultants. He obtained a master's degree in civil engineering from the University of Bologna in 2008 with an emphasis on structural and seismic design. During his studies, Daniel also spend time studying at the University of California, San Diego and the Universidad de Chile to further deepen his knowledge in seismic design. After the completion of his studies he returned to his hometown in Northern Italy, where he worked as a structural timber engineer designing residential, commercial and industrial timber buildings at one of Europe's leading glulam manufacturers. During this time Daniel learned many secrets and insights of timber engineering, which are in part covered in this guide. After moving to New Zealand, Daniel obtained a doctorate at the University of Canterbury focusing on the design of multi-storey timber buildings and diaphragm design.

Daniel has a wide range of experience in the design and review of single and multi-storey residential, commercial, and industrial buildings, with increased interest in pre-fabrication and multi-storey timber structures in recent years.

Daniel is currently the vice president of the NZ Timber Design Society and the co-editor of the Timber Design Society Journal. In addition to his ongoing effort in promoting the use of timber as a construction material and the design of healthy buildings, he is also involved in the development of New Zealand timber standards as well as the timber section of the European seismic design code.



Tobias Smith is a structural engineer and general manager of PTL | Structural Consultants. He attained a bachelor of civil Engineering from the University of Canterbury in 2007, and later, pursued a master's in civil engineering in 2008. Following a short period working as a structural engineer abroad, he commenced a doctorate in 2010 at the University of Canterbury in collaboration with the University of Basilicata in Potenza, Italy. During this time, he tested a three-story timber framed building on a shaking table. Since the completion of his doctoral studies in 2012 Tobias has worked for PTL in Christchurch.

Tobias has broad structural engineering consulting experience, having worked on a range of building projects such as residential, commercial, retail, and mixed-use within New Zealand and around the world. He has a special interest in timber buildings and their seismic design and is actively involved in the development of the New Zealand timber and seismic codes as well as the timber section of the European seismic design code, EC8. He has overseen various projects from the early concept design through to construction and the final sign-off.

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