

TIMBER JOINT DESIGN - 3

Bolts, Coach Screws and
Timber Connectors

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Introduction

Bolts, coach screws and timber connectors (split-rings and shear-plates) all have higher capacities than nails and screws and are therefore better suited to applications where a large load is imposed and the space available to accommodate fasteners is limited. Commonly, these are used in conjunction with proprietary and custom designed metal connectors but they are also used in direct load transfer from one piece of timber to another where they provide a basis for the design of elegant and economic connections. Typical applications include beam to column, beam to beam, truss, pole frame, marine structure and bridge connections as well as column, beam and truss supports.

It is often possible, by thoughtful design to provide a connection with a high level of architectural appeal. Several examples are provided within this and other datafiles. On the other hand, these fasteners can also be used in situations where the requirements are purely of an engineering nature in which strength, compactness, economy and durability are the only qualities sought. Examples include hidden joints and all joints in buildings where aesthetic qualities are usually unimportant such as farm buildings and some industrial and commercial buildings, bridges and marine structures.



Simple bolted joint between beams and poles provide supporting structure for pedestrian bridge



Exposed heavy timber bolted trusses provide clear span and airy atmosphere for food and beverage facility

Fastener Specification and Application

Bolts function by the action of timber bearing on the surface of the bolt and shearing action in the bolt itself. Refer Figure 1.

Split-rings and shear-plates, refer Figures 2 and 3, are collectively referred to as timber connectors. They are used in similar applications to bolts where even higher load capacity is required and/or space is even more restricted by edge and end distance requirements. These operate in a manner similar to short but large diameter bolts. Because of their large diameter, they achieve a large bearing area without the need to remove large volumes of timber, which might otherwise weaken the members being joined.

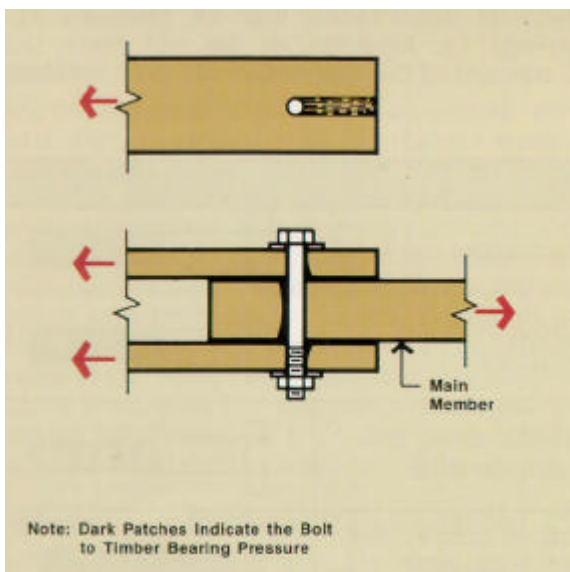


FIGURE 1 THREE MEMBER BOLTED JOINT

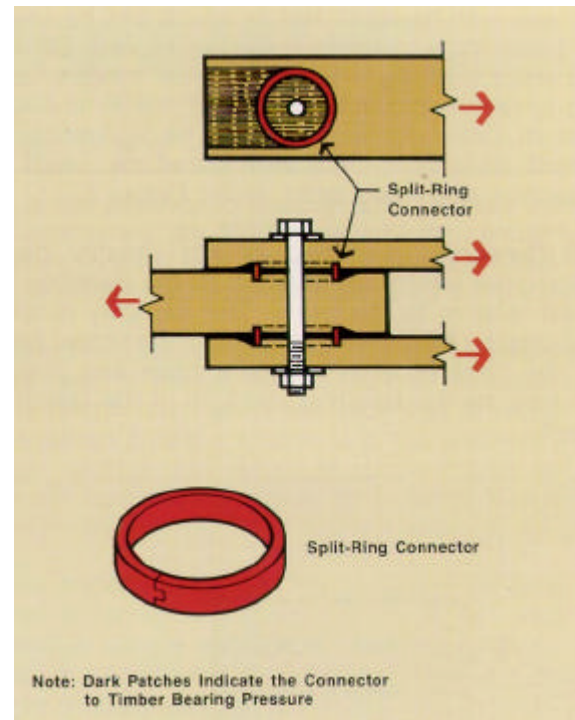


FIGURE 2 THREE MEMBER SPLIT-RING JOINT

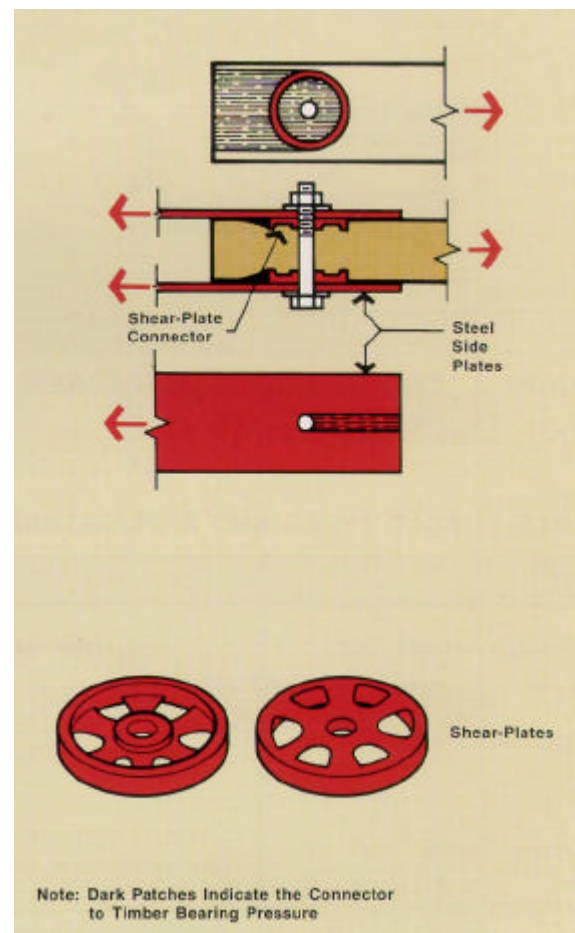


FIGURE 3 THREE MEMBER SHEAR-PLATE JOINT WITH SIDE PLATES

Coach screws, refer Figure 4, are simply large screws with hexagon heads which can be used in situations where bolts are undesirable or cannot be used such as in blind holes i.e. holes where no access is possible to place a nut on the bolt thread. Coach screws should not be confused with coach bolts, which is a term sometimes used to describe cup head bolts. Refer Figure 5.

All fasteners described in this datafile have capacities, which are affected by the direction of load relative to the grain. The capacity is also affected by the bearing strength of the timber and, in the case of small diameter bolts and coach screws, by the bending strength of the fastener itself.

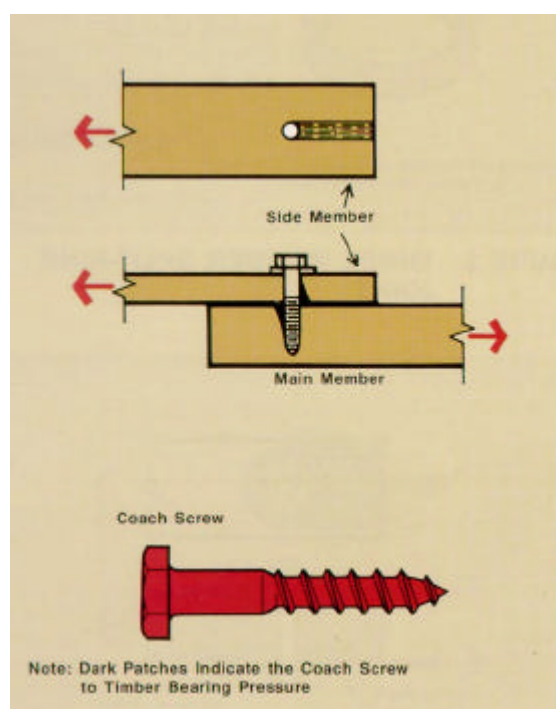


FIGURE 4 TWO MEMBER COACH SCREW JOINT

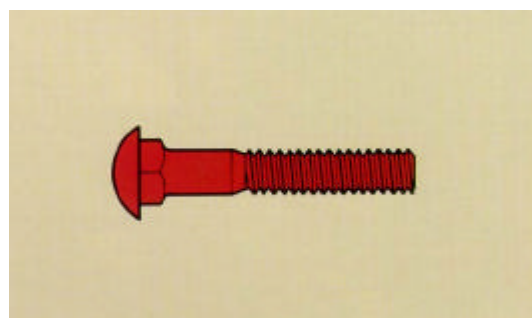


FIGURE 5 CUP HEAD BOLT

Bolts and Coach Screws

Bolt capacities given in AS 1720.1, apply to bolts manufactured in accordance with AS 1111, ISO METRIC HEXAGON COMMERCIAL BOLTS AND SCREWS. In using the bolt capacity tables given in AS 1720.1, designers should be aware that there is an implicit assumption that bolts of any diameter have a minimum bending strength corresponding to a material ultimate tensile strength of 400 MPa. In substituting bolts of other materials, care should be taken to ensure these have this minimum bending capacity.

Applications and Head Types

Only two types of bolt head are commonly available, hexagon and cup head types. Table 1 schedules the most commonly available bolt and coach screw types.

AS 1720.1, provides capacity data only for bolts fitted with a washer under both the bolt head and the nut. For practical purposes, hexagon head bolts are simpler to use than cup head bolts since, with the latter, the washers under the head and nut need to be of different diameters. It is possible to use cup head bolts without a head washer for non-structural applications and for relatively light loadings i.e. loadings of say not more than 50 percent of the respective AS 1720.1 values.

TABLE 1 BOLT TYPES AND APPLICATIONS

Type	Application	Illustrations
Hexagon Head Bolt	General Structural Purposes	
Cup Head Bolt	Occasional structural purposes where head must be flush with surface	
Coach Screw	Used to replace bolts in locations where nut is inaccessible or to improve appearance	
Threaded Rod	Applications where it is difficult to specify bolt length beforehand i.e. tie-down rods, pole construction and cross-bracing	

In the case of coach screws the load capacities provided in AS 1720.1, are for screws complying with AS 1393. Again, there is an implicit assumption that the steel has an ultimate tensile strength of 400 MPa and that there is a washer under the head.

For all bolt types, it is also important to ensure that washers have the minimum thicknesses and diameters specified in AS 1720.1, Table 4.12. These washers are thicker than commonly available types and need to be purchased from industrial hardware suppliers.

Materials

Bolts and threaded rod are available in brass and stainless steel as well as steel complying with AS 1111. Bolts are also available in high strength steel but the use of such special bolts usually cannot be justified on economic grounds nor is the necessary load capacity data available. Coach screws are only available in steel complying with AS 1393, which has the same specification as steel used in bolts to AS 1111.



Heavy hot dipped galvanised bolts connect timber wharf structure

Finishes

Zinc coated steel bolts and coach screws, either hot dipped galvanised or electro-plated are readily available. Hot dipped galvanised bolts and coach screws may be used to advantage in corrosive environments such as swimming pool structures, bridges, marine structures and some farm buildings such as piggeries. Washers and nuts should also be similarly coated. Cadmium and chrome finishes are available but like electro-plated zinc coatings these do more to improve the appearance than the corrosion resistance. Hot dipped galvanising is generally more effective because it deposits a thick sacrificial coating.

Preferred Length-Diameter Combinations

Bolt diameters and lengths are available only in certain combinations. Some indication is provided in Table 2 of commonly available lengths.

TABLE 2 MAXIMUM BOLT LENGTHS

Bolt Size	Maximum Length (mm)
M6 – M8	150
M10 – M36	200

It is the practice to specify bolts by their overall diameter e.g. an M20 hexagon commercial bolt refers to a 20 mm overall diameter bolt and the only additional information required is the length.

Length is always measured from the underside of the head to the tip. Refer Figure 6. For ordering, the length required is the thickness of timber to be joined plus the thickness of the washers plus the nut thickness, which is approximately equal to the bolt diameter, plus some small margin of 3 mm rounded up to the nearest preferred length.

The maximum commonly available length of metric hexagon commercial bolts is listed in Table 2. Longer lengths are usually available from bolt manufacturers for large orders on a delivery time of approximately six weeks. Nut thicknesses of commercial bolts are scheduled in Table 3.

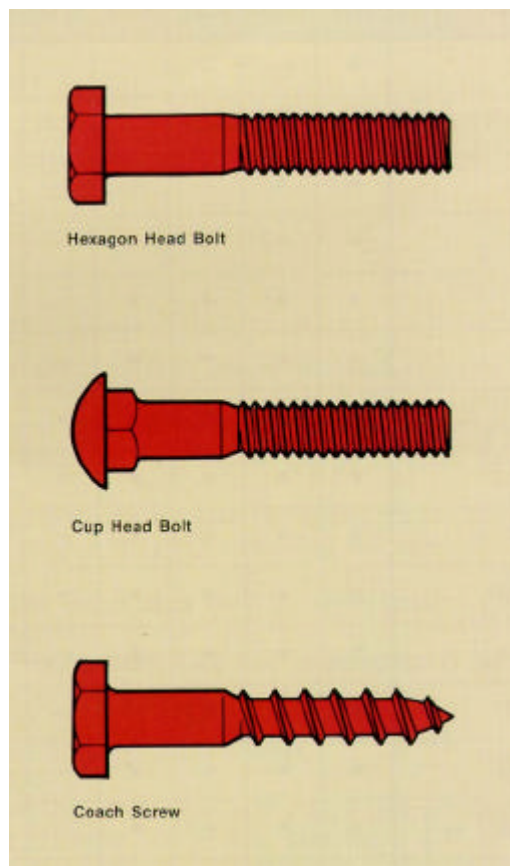


FIGURE 6 BOLT AND COACH SCREW TYPES

TABLE 3 BOLT/THICKNESS

Bolt Size	Nut Thickness (mm)
M6	5.2
M8	6.8
M10	8.4
M12	10.8
M16	14.8
M20	18.0
M24	21.5
M30	25.6
M36	29.4

Table 4 illustrates the range of lengths available for the various coach screw sizes.

TABLE 4 LENGTHS OF COACH SCREWS

Length	Coach Screw Size					
	M6	M8	M10	M12	M16	M20
25	-	-	-	-	-	-
30	-	-	-	-	-	-
35	-	-	-	-	-	-
40	-	-	-	-	-	-
50	-	-	-	-	-	-
60	-	-	-	-	-	-
70	-	-	-	-	-	-
80	-	-	-	-	-	-
90	-	-	-	-	-	-
100	-	-	-	-	-	-
110	-	-	-	-	-	-
120	-	-	-	-	-	-
130	-	-	-	-	-	-
140	-	-	-	-	-	-
150	-	-	-	-	-	-
180	-	-	-	-	-	-
200	-	-	-	-	-	-

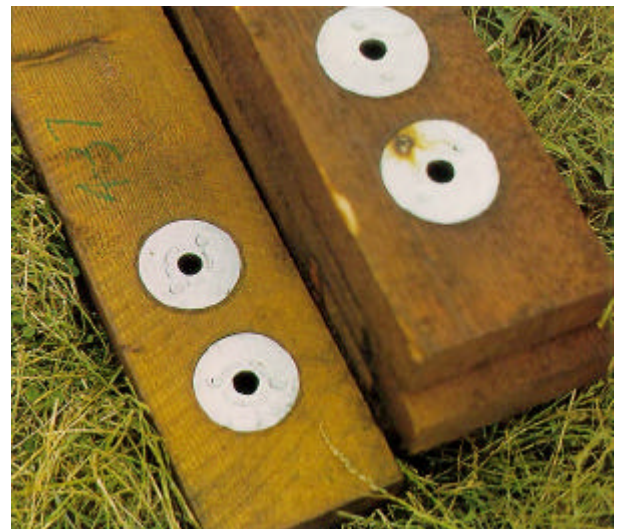
Split-Rings

Split-rings, refer Figure 2, transfer load from one piece of timber to another directly through a large diameter circular steel ring set into a groove. Two split-ring sizes are available, one having a diameter of 64 mm and the other 102 mm. The bolt through the centre serves only the purpose of holding the joint together i.e. it does not contribute in the transfer of shear. The split in the ring is for the purpose of permitting free movement of the wood with the moisture content changes, which occur in service.

The groove must be precision machined using a special grooving tool, which is usually available for hire or purchase from suppliers.

The installation procedure requires close supervision. First the joint must be assembled and the central bolt hole drilled. This hole is used to guide the post of the grooving tool. After the grooves are cut the ring can be inserted and the joint re-assembled. The assembled joint looks the same whether or not the ring is installed. Supervision of the installation is recommended.

Split-rings are manufactured from forged steel. Usually an M20 commercial grade bolt is used to assemble the joint. Rings are available with hot dipped galvanised coatings, which provide extra resistance against corrosion.



Shear-Plates in position ready for assembly

Shear-Plates

Shear-plates, refer Figure 3, are available in two diameters of 67 mm and 102 mm, have similar load carrying capacity to split-rings and require similar installation procedures. They function by transferring load to the shear-plate and then to the bolt which carries the shear across the interface. Thus the bolt in a shear-plate joint performs the dual role of holding the joint together and transferring shear.

Metal side plates may be used but it is also possible to use timber side plates if a second shear-plate is installed. This increases cost and the split-ring is then probably the best choice.

It is essential when using shear-plates to ensure that the timber is seasoned close to its service moisture content. Failure to observe this principle can result in severe shrinkage splitting, as the rigid shear-plate is intolerant to timber moisture movement. It is recommended that construction supervisors check the moisture content with electrical resistance moisture meters having long prongs to ensure that the timber moisture content is 15 percent or less unless the joint is designed for heavily downgraded loads.

Shear-plates are manufactured from malleable cast iron and are available with hot dipped galvanised coatings.

Joint Design

Bolts, split-rings and shear-plates are used more commonly to resist direct loads only but can also be used to form moment resisting connections. The latter is less common principally because of the cost involved.

Lateral Loads

The load capacity of bolt, split-ring and shear-plate connectors is determined by procedures outlined in AS 1649. The allowable capacity is determined by dividing the ultimate load by a suitable factor. Tests are carried with load both parallel and perpendicular to the grain. Separate data is provided for these two cases in AS 1720.1



Trusses featuring bolts and large washers in shopping complex

AS 1720.1, expresses laterally loaded shear capacity in the form:

$$\begin{aligned} \phi N_j &= \phi k_1 k_{16} k_{17} n Q_{sk} && \text{for bolts} && (1) \\ \phi N_j &= \phi k_1 k_{13} k_{16} k_{17} n Q_{sk} && \text{for coach screws} && (2) \end{aligned}$$

For bolts and coach screw systems where load is at an angle θ to the grain, the system capacity shall be calculated by use of Hankinson's formula:-

$$\begin{aligned} Q_{sk} &= \frac{Q_{skl} Q_{skp}}{Q_{skl} \sin^2 \theta + Q_{skp} \cos^2 \theta} \\ \phi N_j &= \phi k_1 k_{15} k_{17} K_{18} n Q_k && \text{for split-rings and shear plates} && (3) \end{aligned}$$

where

- Q_{sk} = System characteristic capacity of bolt or coach screw in shear for load parallel or perpendicular to grain.
- Q_{skl} = System characteristic capacity of bolt or coach screw in shear for load parallel to grain.
- Q_{skp} = System characteristic capacity of bolt or coach screw in shear for load perpendicular to grain.
- Q_k = characteristic capacity of split-rings or shear plates in shear given directly in AS 1720.1, for various grain angles and not requiring the use of Hankinson's formula.
- k_1 = duration factor – Table G1 AS 1720.1
- k_{13} = 1.0 for coach screws in side grain
= 0.6 for coach screws in end grain
- k_{15} = split rings and shear plates factor ranges from 1.25 – 1.5 depending on Strength Group and load to grain angle. Table 4.15 AS 1720.1.
- k_{16} = side plate factor (1.2 for metal side plates, 1.0 otherwise, Clauses 4.4.3.2 or 4.5.3.1 AS 1720.1)
- k_{17} = multiple fastener factor which ranges from 0.5 – 1.0 and has a major impact on joint capacity under adverse conditions; see Table 4.11 AS 1720.1.
- k_{18} = tension member factor which ranges from 0.3 – 1.0 depending on ring diameter and timber moisture content; see Table 4.16 AS 1720.1.

The method of computing the load acting on an individual fastener is not explicitly nominated in AS 1720.1, unlike the situation with nailed and screwed joints. Therefore it must be presumed that the intention is to revert to linear elastic formulae i.e. formulae where the force developed in a fastener is linearly related to the relative movement of the two pieces of wood being joined. Accordingly, the recommended procedure is to compute the maximum fastener loading using

$$q = \sqrt{[q_{dx} q_{mx}]^2 + (q_{dy} + q_{my})^2} \quad (4)$$

$$q_{dx} = F_x/n \quad (5)$$

$$q_{dy} = F_y/n \quad (6)$$

$$q_{mx} = My_m/I_p \quad (7)$$

$$q_{my} = Mx_m/I_p \quad (8)$$

where

- q = force on a bolt
- q_{dx}, q_{dy} = force components on a bolt in the x, y directions due to direct forces
- q_{mx}, q_{my} = force on a bolt in the x, y directions due to moment
- F_x, F_y = direct forces in the x, y directions
- n = number of fasteners in the joint
- M = moment acting on the joint
- X_m, Y_m = x, y coordinate to fastener furthest from the fastener group centroid
- I_p = polar moment of area of fastener group
- $I_p = r_i^2$
- r_i = polar distance of i^{th} fastener

For satisfactory design it is necessary that $q \leq \phi N_j$

As indicated above, linear theory is used for joint design where high capacity fasteners are involved and a non-linear theory used for joint design involving nails and screws. The use of non-linear theory produces a more economic joint design but the gain is small where relatively few fasteners are used as occurs with high load capacity fasteners. Linear theories are also much easier to use and accordingly, this minor Code inconsistency is justified.

In using AS 1720.1, to obtain fastener load capacity, the following steps are observed:

1. The timber species must be established.
2. The state of seasoning must also be known (and checked during construction especially when using shear-plates) and the service moisture content estimated.
3. Reference is then made to Table 2.1 of AS 1720.1, to determine the Joint Group. The Joint Group will be one of J1 – J6 for unseasoned timbers and seasoned timbers which have a service moisture content over 15 percent and JD1 – JD6 for seasoned timbers which remain at or below 15 percent moisture content in service.
4. The capacity is obtained from fastener capacity tables in the body of AS 1720.1. Where load is at other than 0° or 90° to the grain with bolts and coach screws, the use of Hankinson's formula is necessary.

It is also necessary to compute joint shear values. AS 1720.1, gives no guidance but Breyer⁽¹⁾ recommends the formula:

$$f_{sj} = 1.5V/bd_e$$

where V = shear, b = breadth and d_e = distance from the edge away from the load to the centre of the nearest fastener. Refer Figure 7. Finally, in the case of tension members, it is necessary to check the stresses in the member nett section (cross-section less holes etc).

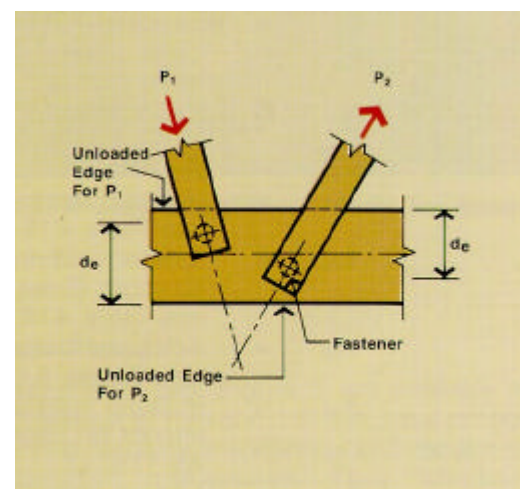


FIGURE 7 DIMENSION d_e FOR JOINT SHEAR COMPUTATIONS
Joint Layout

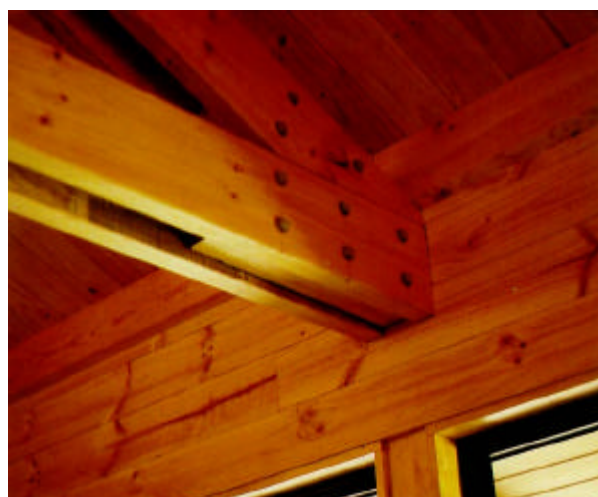
TIMBER JOINT DESIGN-3

The rules which apply to joints made from high capacity fasteners need to be strictly observed because the failure of only one fastener through premature splitting represents a proportionately much higher loss of joint load capacity than is the case with nailed and screwed joints.

The rules for spacing, edge distance and end distance are given in Sections 4.4.4 and Table 4.18 of AS 1720.1, and these should be observed.

Bolt and Coach Screw Spacings

The rules relating to bolts and coach screws are summarised in Table 5 and Figure 8. Where the end of a piece of timber is cut at an angle to the longitudinal axis, the end distance value is measured parallel to the axis to the intersection with the angled end. Refer Figure 9.



Multiple bolt joint in seasoned glued laminated truss

TABLE 5 BOLT/COACH SCREWS – SPACING, EDGE AND END DISTANCES

Detail	Value
D = bolt diameter b = timber width i.e. width measured along bolt axis	
Loads parallel to the grain	
Edge distance	2D
End distance (l_{par})	
Tension joints, unseasoned timber	8D
Tension joints, seasoned timber	7D
Compression joints	5D
Lesser end distances may be used in tension joints down to respectively, in unseasoned, seasoned timber but capacity must be reduced in proportion.	6D, 5D
Spacing	
Along grain	5D
Across grain	4D
Loads perpendicular to the grain	
Edge distance	4D
End distance	5D
Spacing	
Along grain (Dimension (a) Refer Figure 8)	$2.5D$ for $b/D = 2$ $1.25 + 0.625 \frac{b}{D}$ for $2 < \frac{b}{D} < 6$ $5D$ for $b/D \geq 6$
Across grain	5D
Loads at an angle to the grain	
0° - 30°	Use distance for load parallel to the grain
30°- 90°	Use distances for load perpendicular to the grain

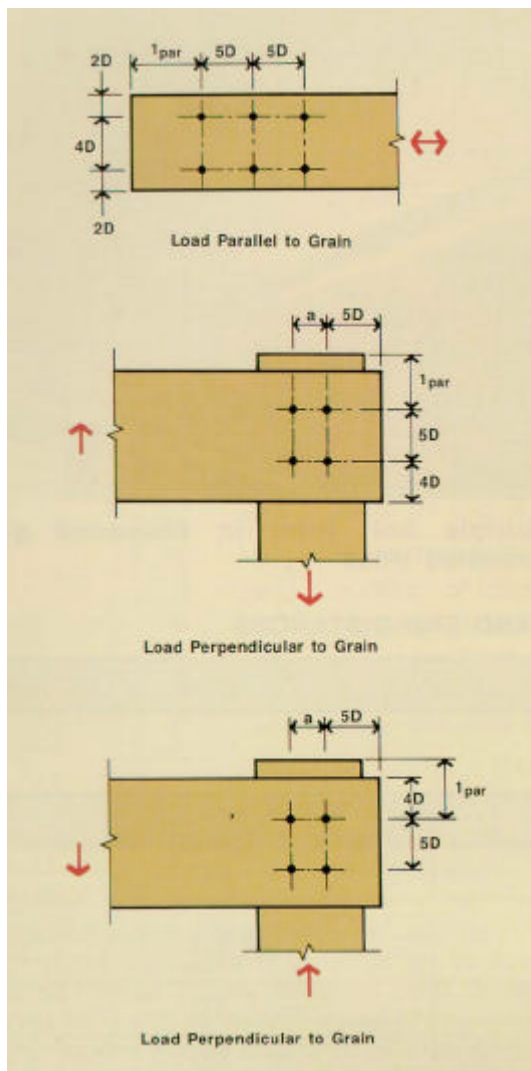


FIGURE 8 LAYOUT DETAILS FOR BOLTED JOINTS

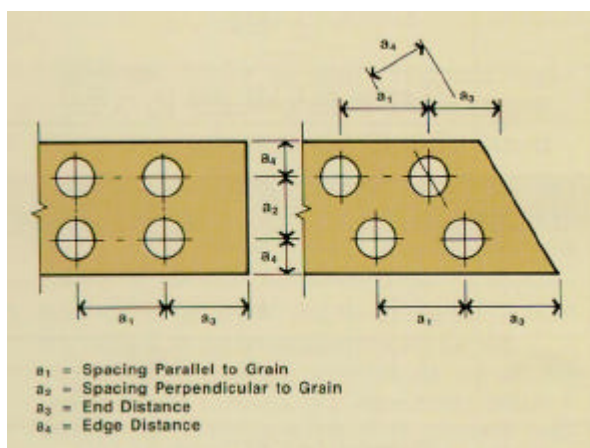


FIGURE 9 LAYOUT DETAILS FOR SPLIT-RING AND SHEAR-PLATE JOINTS

Split-Ring and Shear-Plate Spacings

Split-Ring and shear-plate geometry is determined by a common set of geometric rules given in Section 4.7.4 of AS 1720.1. The essentials are reproduced here in Table 6 and Figure 9.

Shrinkage

In this datafile, considerable emphasis has been placed on the need for careful design in circumstances where the combination of cross-grain shrinkage, rigid fasteners and rigid side plates can cause cross-grain splitting. Both bolts and timber connectors provide the rigid fasteners. Care is necessary with certain layout patterns. The situation has been put succinctly by Breyer (1) who states that cross-grain shrinkage problems can be minimised as follows:

- Design connections, which permit unrestricted movement of bolts across the grain. In the case of steel side plates, use separate plates for each bolt row or slotted holes.
- In timber to timber connections where members frame at right angles, use a single high capacity fastener in preference to multiple fasteners. Such steps are not necessary in initially dry timber, which remains dry.

Situations that pose no difficulty are illustrated in Figure 10.

The k_{17} factor represents the provision of AS 1720.1, for loss of capacity arising from possible cross-grain shrinkage effects. Reference should also be made to Datafile P4, TIMBER – DESIGN FOR DURABILITY.

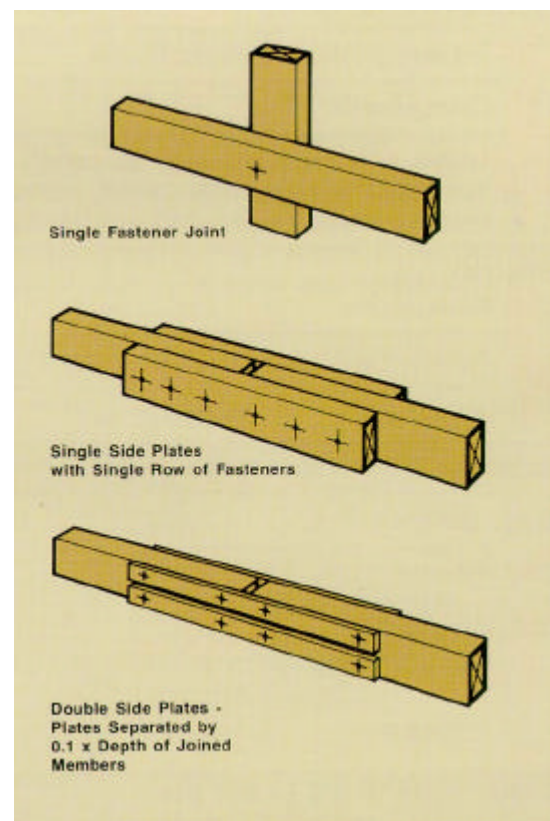


FIGURE 10 JOINT EXAMPLES WHERE CROSS-GRAIN SHRINKAGE IS NOT DETRIMENTAL

TABLE 6 SPLIT-RINGS/SHEAR-PLATES – SPACING, EDGE AND END DISTANCES

Detail	Minimum distance (D = ring or plate diameter)	
	D = 64 mm and 67 mm	D = 102 mm
End distance		
tension members	150	180
compression members	100	140
Edge distance		
0° - 30° angle of load to grain	45	70
0° - 90° angle of load to grain		
• compression side	70	95
• opposite compression side	45	70
Spacing		
0° - 30° angle of load to grain		
• parallel to the grain	180	230
• perpendicular to the grain	90	140
30° - 90° angle of load to grain		
• parallel to the grain	90	140
• perpendicular to the grain	115	165

Connection Design

Direct Force Connections

The design of connections subject to direct forces is straightforward. The allowable force and forces acting on individual fasteners are obtained using equations 1-8 (refer Joint Design) and the relevant tables in AS 1720.1, for the system characteristic capacity of Q_{sk}

In the case of direct force connections, equations 1-8 reduce to computing the number of fasteners by dividing the total load for the most critical load combination by the fastener capacity. Once the number of fasteners has been computed, the joint detailing can be finalised.

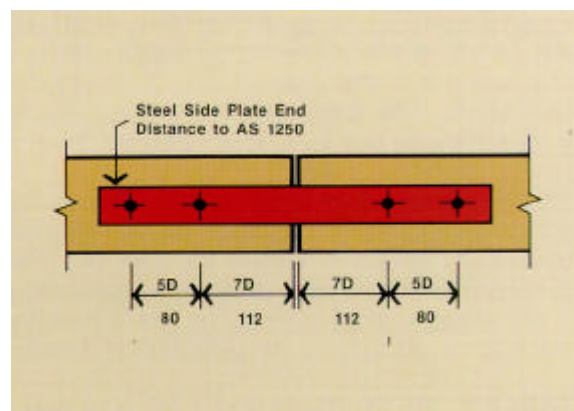


FIGURE 11 JOINT LAYOUT – EXAMPLE 1

Example 1

Assume that a splice joint (refer Figure 11) is to carry the tension forces $P_{DL} = 8 \text{ kN}$, $P_{LL} = 9 \text{ kN}$ in 120 x 35 mm Seasoned Softwood Joint Group JD4.

The load duration factor, K_1 , for 1.25D + 1.5 LL is 0.77.

The equivalent “instantaneous” load corresponding to this 1.25 DL + 1.5 LL combination is $(1.25 \times 8 + 1.5 \times 9)/0.77 = 30.5 \text{ kN}$.

The 1.25 DL + 1.5 LL combination will control the design. Assume that M16 bolts and metal sideplates are used.

The effective thickness = 35 mm and, from AS 1720.1, Table 4.9 (c), $Q_{kl} = 9900\text{N}$

For systems loaded parallel to the grain, $Q_{sk} = Q_{skl} = 2Q_{kl} \therefore$

$Q_{sk} = Q_{skl} = 2 \times 9900\text{N}$ where $K_1 = 0.77$, $K_{16} = 1.2$, $K_{17} = 1.0$, $\phi = 0.8$

Hence, design capacity $\phi N_j = 0.8 \times 0.77 \times 1.2 \times 1.0 \times n \times 9900 \times 2 = 14636n \text{ N}$

Factored DL & LL = $1.25 \times 8 + 1.5 \times 9 = 23.5 \text{ kN}$

The number of bolts = $\frac{23.5 \times 10^3}{14636} = 1.6$ say 2 bolts

Example 2

The design of the bolted joint at the heel of the bolted truss shown in Figure 12 illustrates a number of points, which arise in design. The truss is fabricated from seasoned mixed species timbers of Strength Group SD7 and Joint Group JD4. The top chord is designed as a single 90 x 35 mm member and the bottom chord as a double 90 x 35 mm member. Because a single fastener can support loads of this magnitude, only one is used. This removes any concerns about cross-grain shrinkage problems.

The critical load combination is taken to be 1.25 DL + 1.5 LL, the trusses are at 3 m centres and the total load is 0.4 kPa including self-weight. The nodal loads are taken as 1.8 kN. This leads to the member forces shown in Figure 12. At the heel the truss is supported eccentrically. It will be seen that this does not present an overstressing problem in terms of joint shear. Adopt M12 bolts as a first trial and consider the joint capacity calculated in two ways. First, assume that the double bottom chord limits the capacity, which is subject to a load of 10.1 kN parallel to the grain.

The force in the bottom chord is at 24° to grain of that member $\therefore$

$t_{\text{eff}} = 2t_1 = 2 \times 35 = 70\text{mm}$,

Table 4.9(C) and 4.10(C) AS 1720.1 the characteristic capacities parallel and normal to the grain are $Q_{kl} = 10.2 \text{ kN}$ and $Q_{kp} = 5.25 \text{ kN}$, respectively.

From Table 4.9(A) and 4.10(A) AS 1720.1 the system capacities are –

$Q_{skl} = 2Q_{kl} = 2 \times 10.2 = 20.4 \text{ kN}$ parallel to grain

$Q_{skp} = 2Q_{kp} = 2 \times 5.25 = 10.5 \text{ kN}$ perpendicular to grain.

For system loaded at an angle θ to grain, applying Hankinson's formula

$$Q_{sk} = \frac{20.4 \times 10.5}{20.4 \sin^2(24) + 10.5 \cos^2(24)} = 17.6 \text{ kN}$$

$$\begin{aligned} \text{Design capacity of connection } \phi N_j &= \phi k_1 k_{16} K_{17} n Q_{sk} \\ &= 0.75 \times 0.77 \times 1.0 \times 1.0 \times 1 \times 17.6 = 10.2 \text{ kN} > 10.1 \text{ kN} \end{aligned}$$

Secondly compute the capacity on the basis of the top chord which is subject to compression force of 11.1 kN parallel to the grain. From AS 1720.1, Table 4.9 (C), using an effective width = 35 mm the characteristic capacity $Q_{kl} = 7.5 \text{ kN}$.

From Table 4.9(A) AS 1720.1, the system capacity is given by :

$Q_{skl} = 2Q_{kl} = 2 \times 7.5 = 15.0 \text{ kN}$ parallel to grain

Clause 4.4.2.4(a) gives $Q_{sk} = Q_{skl} = 15.0 \text{ kN}$

$$\begin{aligned} \text{Design capacity of connection } \phi N_j &= 0.75 \times 0.77 \times 1.0 \times 1.0 \times 1.0 \times 15.0 \\ &= 8.7 \text{ kN} < 11.1 \text{ kN} \end{aligned}$$

This shortfall necessitates a shift to an M16 bolt or the use of 2 x M12 bolts. The former is the only possible alternative since there is insufficient space for two bolts which highlights the point that the choice of fastener is often constrained by the availability of space. Widening the timbers can create extra space but such widening is rarely justified in timber design. Because the support is eccentric (refer Figure 13), a check should be made of joint shear; $F'_{sj} = 4500 \text{ (V)} \times 1.5/2 \times 35 \text{ (b)} \times 45 \text{ (d}_e\text{)} = 2.14 \text{ MPa}$. The factor 2 in the denominator accounts for the bottom chord being a double member and d_e is its half depth. The characteristic strength in shear at joint for SD7 timber $f'_{sj} = 4.3 \text{ MPa}$.

Design capacity in transverse shear at an eccentric joint

$$\phi V_{sj} = \phi k_1 k_4 k_6 f_{sj} A_{sj}$$

= 0.75 x 0.77 x 1.0 x 1.0 x 4.3 = 2.5 MPa > 2.14 MPa and the joint is acceptable from the point of view of joint shear.

The geometric requirements provided in Table 5 now need to be applied to the tension chord (the compression chord distances are, consequently, automatically satisfied) and used to detail the joint; the details are shown in Figure 14(a) for the preliminary layout. The distances used are those appropriate to loading parallel to the grain since the load/grain angle is 24°. In interpreting AS 1720.1, the end distance is still measured parallel to the grain from the centre of the fastener to the intersection with the line of the sloping end cut. The required and actual layout distances for an M16 bolt are listed in Table 7.

TABLE 7 REQUIREMENTS FOR M16 BOLT

	Required Value (Ref. Table 5)	Actual Value (Initial Layout Fig. 18)
Bottom Chord		
end distance	7 x 16 = 112 mm	45/sin 24° = 111 mm assuming no support eccentricity
edge distance	2 x 16 = 32 mm	45 mm
Top Chord		
end distance	5 x 16 = 80 mm	111 mm

The edge distance and top chord end distance shortfalls are minor and can be ignored but the end distance shortfall in the bottom chord warrants more consideration. The end distance required is nominally 7D (= 112 mm) for the bottom chord and normally a designer would truncate the bottom chord at the top face of the top chord as illustrated in Figure 14(a). To increase this value to say 8D = 128, the lower chord can be extended as shown in Figure 14 (b) but this represents an overkill. Based on the lower chord, the joint load capacity spacing to 128 x 10.1/10.2 = 126 mm which is still greater than the minimum 5D permitted under AS 1720.1 design rules.

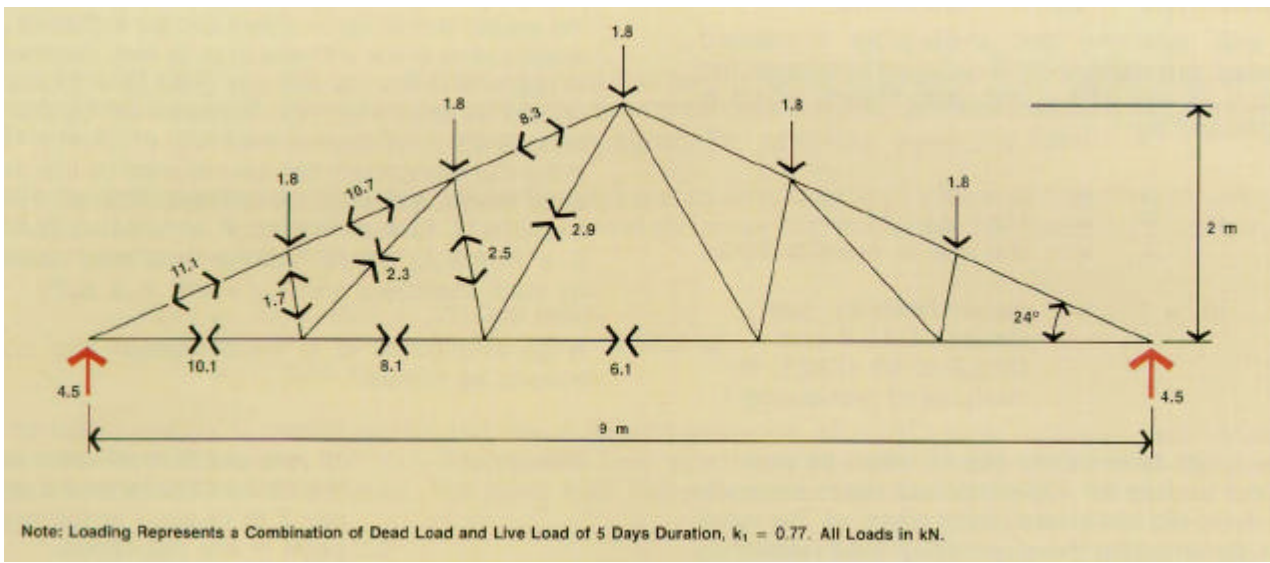


FIGURE 12 LOADING DETAILS OF TRUSS – EXAMPLE 2

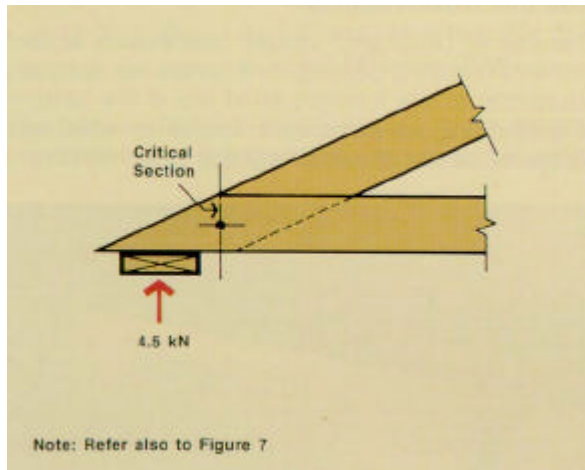
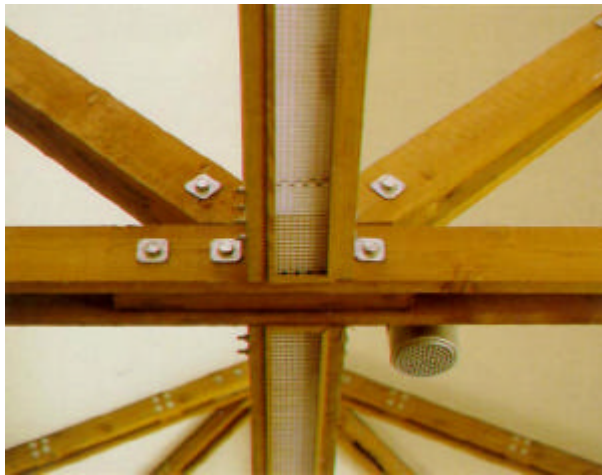


FIGURE 13 CRITICAL-SECTION FOR JOINT SHEAR



Single or in-line bolts in unseasoned timber truss

Special Note – Bolted Truss Design

In bolt, split-ring and shear-plate connected trusses, pre-cambering is required to counter the effects of slip and elastic deformation, which is expressed by:

$$\Delta = \Delta_i + fP$$

or $P = k(\Delta - \Delta_i)$

Δ_i = slip due to oversize holes

$l/k = f$ = elastic flexibility; see Clauses D2.3.2 and D2.3.3 of AS 1720.1 for methods of computing f .

The initial deformation of Δ_i is taken as zero for bolted trusses for superimposed loads because the initial slip has already been taken up. The value of k depends on the direction of load relative to the grain. Where load is at other than parallel or perpendicular to the grain, Hankinson's rule should be used to evaluate k .

From a purely theoretical point of view, the initial slip is equivalent to a lack of fit in which members

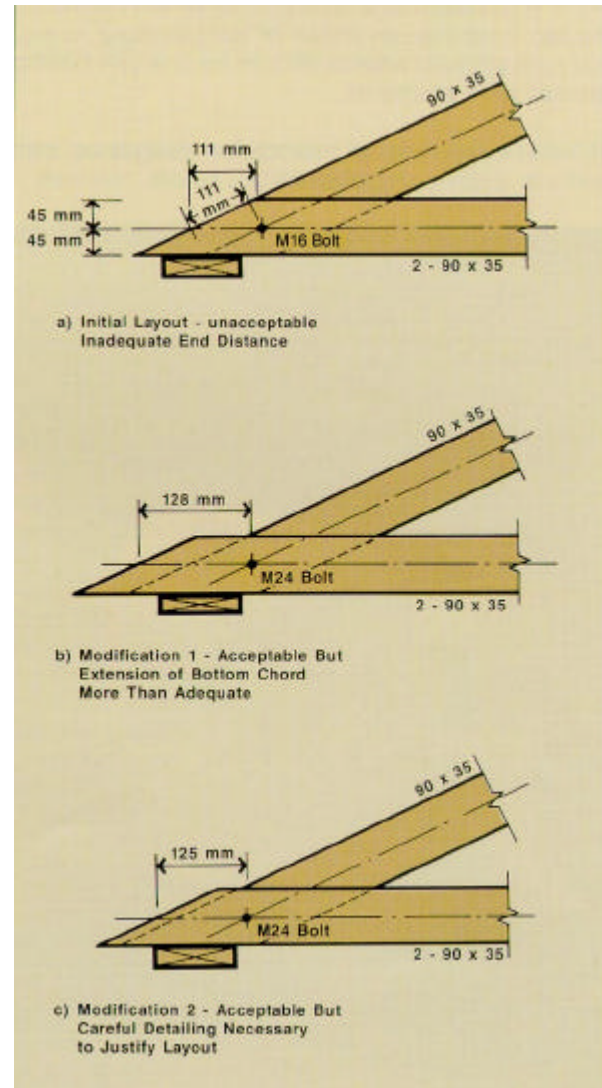


FIGURE 14 JOINT LAYOUTS FOR EXAMPLE 2

are too long (i.e. an initial or temperature strain) and joint stiffness effects should be analysed using special joint elements.

Unfortunately, not all computer programs can perform such computations.

If a designer does not have a computer program, the elastic deformation effect can be included by modification of the stiffness of truss web members. In place of EA/L , the stiffness $(EA/L)(1 + 2EA/kL)$ should be used which can be achieved by use of a modified cross-section area $A/(1 + 2EA/kL)$. The initial slip component can be adjusted by trial and error modifications of the stiffness. After an initial analysis, a further reduction in area to $(EA/L)(1 + 2EA/k_mL)$ where P is the axial load carried by such members and $k_m = k/(1 + k \Delta_i/P)$.

A general guide is to build camber into both trusses as follows:

- Triangular - 10 mm per 3 m of span on the bottom chord and 6 mm per 3 m of span at the mid-point of the top chord.
- Parallel chord - 10 mm per 3 m of span for both top and bottom chords.

Moment Connections

The use of bolts and timber connectors to form moment resisting connections arises occasionally. Situations where it might arise are in the splicing of a bending element with timber or steel side plates or at the base of a fixed base column.



Simple bolted joints in glued laminated beam roof structure

Example 3

Assume that the 150 x 150 mm softwood flag pole is to be connected to a pair of 100 x 100 mm hardwood posts. Refer Figure 15. Note firstly that split-rings cannot be used because access for fitting is not possible. This restricts the choice to either bolts or shear-plates. For simplicity, select the bolts.

Assume the base bending moment under the anticipated wind load is 11 kNm and that the connection is to be designed to resist this force. The bolts are at distances 800 mm apart. Following the procedures set out under Joint Design – Lateral Loads, the following are obtained.

$$\begin{aligned}
 I_p &= 2 \times 400^2 = 320000 \text{ mm}^2 \\
 q_{mx} &= 11 \times 10^6 \times 400 / 320000 = 13750 \text{ N} \\
 &= 13.75 \text{ kN}
 \end{aligned}$$

The load capacity is limited by the 150 mm wide JD4 softwood AS 1720.1, only tabulates load values for effective widths up to 120 mm (Tables 4.9(C), 4.10(C)). Beyond this limit, expressions given in Appendix C must be used. Assume that M16 bolts are used. The basic load capacity is the lesser of

$$\begin{aligned}
 Q_{skp} &= N t_e f_{pj} D / 2 &= 2 \times 150 \times 12.5 \times 16 / 2 &= 30.0 \text{ kN} \\
 \text{and } Q_{skp} &= N 17 f_{pj} \sqrt{D 3} &= 2 \times 17 \times 12.5 \times \sqrt{16^3} &= 27.2 \text{ kN}
 \end{aligned}$$

where

N	=	number of shear faces = 2
t_e	=	effective thickness = 150 mm
D	=	bolt diameter
f_{pj}	=	allowable stress for Joint Group, Table C6
	=	12.5 MPa for JD4

hence $Q_{skp} = 27.2 \text{ kN}$

The allowable load is given by equation 1 with $\phi = 0.65$ $K_1 = 1.3$ (wind load), $k_{16} = 1$ (timber sideplates), $K_{17} = 1$ (no transverse restraint i.e. only one bolt occurs at each position along the timber).

Thus $\phi N_j = 0.65 \times 1.3 \times 2 \times 27.2 = 46 \text{ kN}$

The final layout is shown in Figure 15.

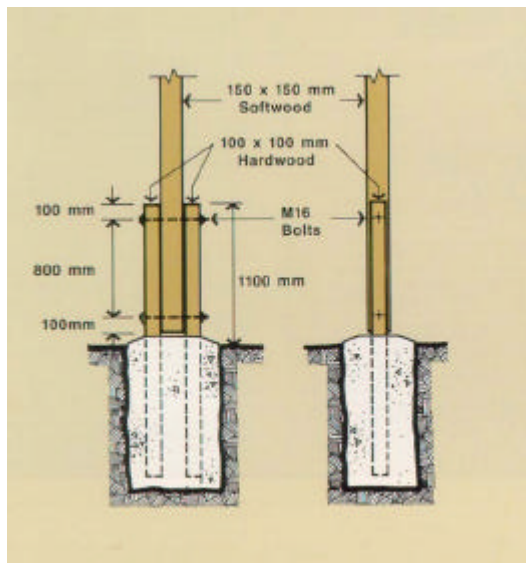


FIGURE 15 JOINT DETAILS FOR EXAMPLE 3



Bolted trusses fixed to central metal connector in school roof structure

Specifications

For detailed specification clauses, reference should be made to Datafile DP1, TIMBER SPECIFICATIONS.

The following is a check list of some items which should be included in the specification of fasteners/joints or alternatively, indicated on plans:

- Fastener type
- Fastener size/length etc.
- Washer/type/size
- Material/protection, e.g., galvanised steel, stainless steel
- End and edge distances/spacings
- Joint group of timber
- Moisture content of timber
- End-grain protection
- Workmanship
- Pre-drilling sizes

Other References

1. DESIGN OF WOOD STRUCTURES
Breyer, D.E., McGraw-Hill 1988
2. AS 1720.1, TIMBER STRUCTURES CODE
Standards Australia
3. AS 1111, ISO METRIC HEXAGON
COMMERCIAL BOLTS AND SCREWS
Standards Australia
4. AS 1393, COACH SCREWS (METRIC
SERIES) (WITH ISO HEXAGON HEADS)
Standards Australia
5. AS 1649, METHOD FOR THE
DETERMINATION OF BASIC WORKING
LOADS FOR METAL FASTENERS FOR
TIMBER
Standards Australia