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AN INVESTIGATION OF TRUSS HOLD DOWN DETAILS

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ABSTRACT

Wide span trusses and especially girder trusses can concentrate wind uplift pressures to high reaction forces. Innovative solutions are often needed to cope with these forces. This research program investigates the strength of some innovative solutions and compares them with more conventional ones. The program includes two strengths of timber, different size and strength bolts, one or two side cleats together with an overstrap solution.

One hundred and forty 2 m long trusses were tested in the program. The effect of transverse bolts through the nailplate and above it were also investigated. Results showed that the strength of conventional hold down systems was greater than predicted because of the reinforcing effect of the nailplate. This applied both to systems with bolts through the nailplates and to those with bolts above the nailplates.

The overstrap increased the strength of the hold down systems, although estimates of its effect were sometimes clouded by changes in other parameters. The overstrap worked much better in hold down systems including two side cleats rather than one. Indicative design values have been calculated for each system.

1 INTRODUCTION

During a severe wind storm uplift wind pressures can overcome the mass of the roofing and roof structure. Roof trusses concentrate that uplift pressure at their reactions and thus may require design of specific hold down details. For example, conventional trusses with a 10 m span and 900 mm overhangs, spaced 900 mm apart supporting a sheet roof and ceiling on a house in a suburban location of the cyclone region would need to be designed for a permissible stress total uplift force of about 15 kN. The hold down force needed at each reaction point is 7.5 kN. In exposed locations these forces would be considerably higher.

Significantly higher hold down forces are needed for the girder truss of a hip roof. Figure 1 shows a roof plan and compares the tributary area for a conventional truss with that of a girder truss. Depending on the configuration of the roof structure at the hip, the tributary area and therefore the truss reaction can be nearly three times that of conventional trusses.

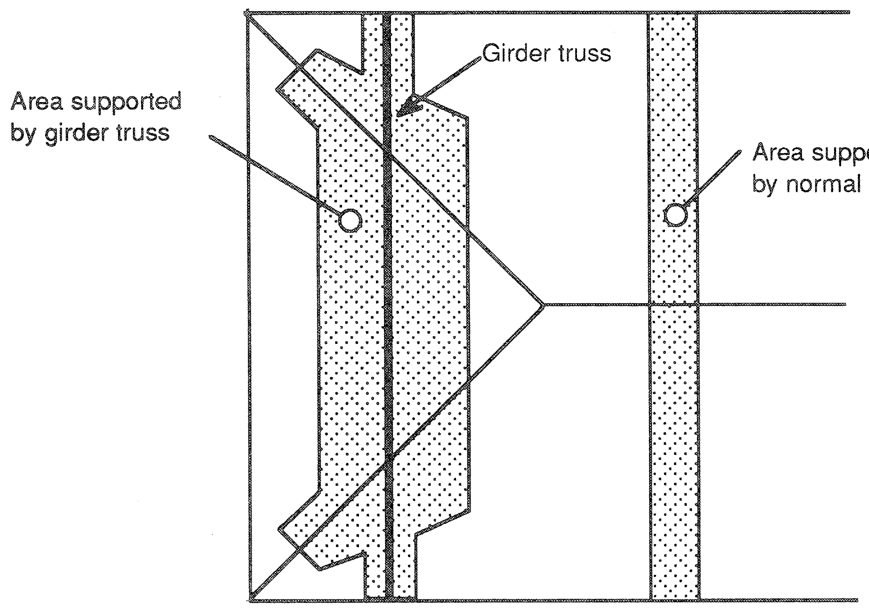


Figure 1 Comparison of Tributary Areas

It is common practice for truss manufacturers to provide details of reaction forces when trusses are ordered by a builder. For high uplift reactions, such as occur in high wind areas, the builder will normally consult a structural engineer for a suitable hold down detail, and would certainly need to do so for the girder truss reaction. But design of the hold down detail is not necessarily straight forward as a number of constraints are inherent in the roof system.

The method of clamping the truss down by bolts through an overbatten is limited by the fact that the depth of the overbatten is restricted to the same depth as the normal battens, and the bolt heads need to be countersunk into the battens. If the roofing is also fixed to those overbattens, it is likely that, somewhere on the roof, the bolt heads through the overbattens will coincide with the location of the roofing fasteners and impede their installation.

The most common method of hold down for higher loads is to bolt the truss to one or two side plates. The restrictions here are that there is only room for one transverse bolt and it may have to be installed through the nailplate at the heel of the truss, which may affect accurate location of the bolt hole.

The effect of the truss nailplates on the strength of the bolted joint is not obvious, and the timber code offers no guidance on it. Further it is certainly not obvious whether it is better to put the transverse hold down bolt through the nailplate or above it.

These uncertainties have led to innovative solutions such as that shown in Figure 2 where the addition of a light gauge steel strap over the top chord provides somewhat of a "belt and braces" solution as an additional force path is introduced. The uplift force on the truss can be transferred to the angle hold down plate by the top surface of the top chord bearing on the overstrap as well as by the transverse bolt bearing on the timber. This system is complex to analyse as the distribution of force between the two devices is dependent upon their relative effective stiffness, which includes effects of the tolerance of the bolt holes in both the timber and the overstrap and the tightness of fit of the overstrap (which is normally bent in-situ). The two systems may not necessarily achieve their maximum strength at the same time.

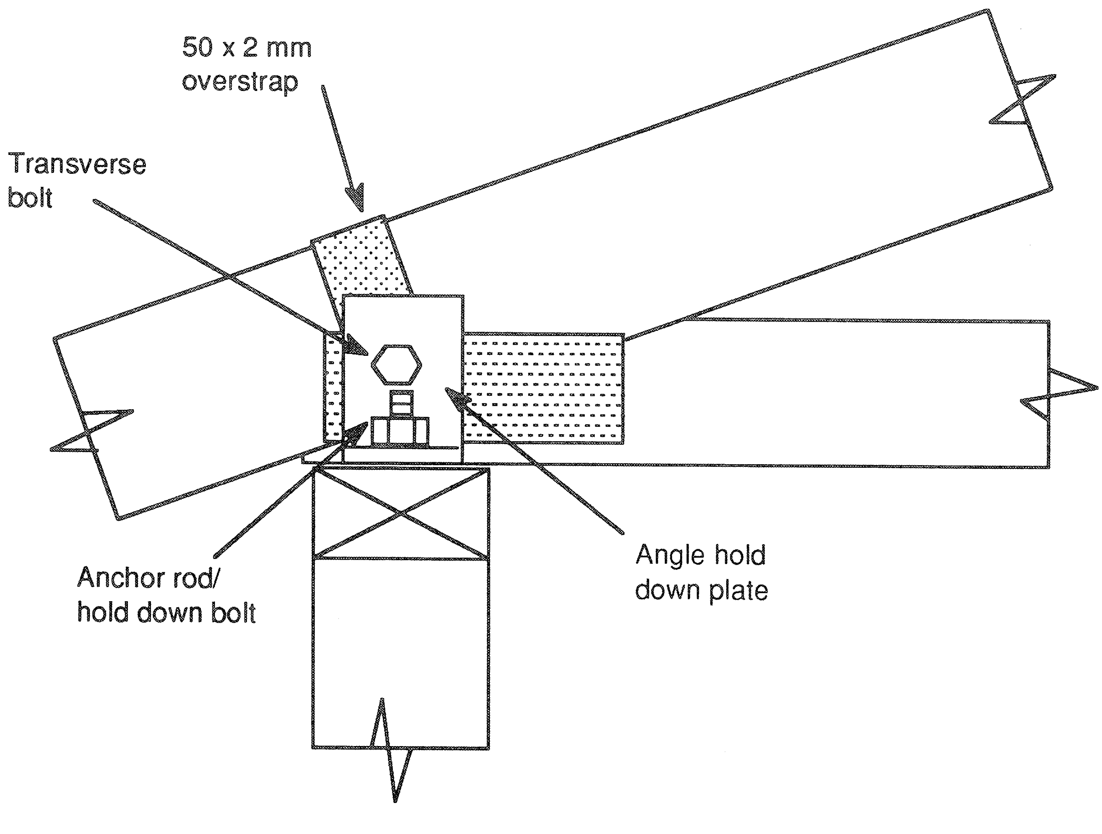


Figure 2 Illustration of Overstrap Innovation

To achieve its full strength potential, the overstrap has to be installed so that it is tight, or some packing piece should be inserted after installation. Obviously, inserting a packing piece is not the preferred option. One method explained to the authors is to predrill one end of the strap, align the hole with the one in the side cleat and partly install a bolt to maintain the alignment. The strap can then be pulled tight and bent over the truss, using a hammer to achieve the sharp corners. It can then be clamped to the side of the truss so that the bolt hole can be drilled accurately. There are probably other equally suitable methods.

The research program was initiated to investigate the strength of some of these innovative truss hold-down details for high uplift forces and compare them with more conventional methods.

2. CURRENT PRACTICE

At present structural engineering in Australia is in a transitional phase from permissible stress design to limit states design. The current loading codes include details for both. The timber engineering code (Standards Australia, 1988) is one of the last to change over to limit states methods. In June 1996 the limit states version is still in draft form, in its final committee stages.

To make this report more useful in the short term, details are included for bolt strengths using both forms of design. However, it must be emphasised that the limit states design is based on the latest draft of the code (Standards Australia, 1995) and as such there is the possibility, although unlikely at this stage, that important details may be changed before the final edition is published.

The current timber engineering code is not very useful for the design of transverse bolt hold down details for trusses. There is no guidance on the effect of nailplates on bolt strength. Rogers et al (1994) showed that nailplates increased the strength of conventional bolted joints loaded parallel to the grain by up to 60%, but no research has been done on such joints loaded perpendicular to the grain. In most instances truss hold down forces are nearly perpendicular to the grain.

Further the strength of a bolt through the top chord directly above the nailplate is likely to be affected by the presence of that plate. The code offers no advice on this situation. Thus the innovative solutions mentioned above are often based on reasonable estimates of performance rather than accurate knowledge of such.

Ignoring the effect of the nailplate, design strengths for wind for bolts at an angle of 70° to the grain can be calculated from the permissible stress code for two member and three member joints in the timbers used in this research program. Lhuede (1987) concluded that code recommendations were conservative for joints away from the ends of members. Based on his test results, an average failure load can be estimated by multiplying the basic design loads by eight. Estimates of mean load at failure for bolts loaded at an angle of 70° to the grain are given in Table 1 for three combinations of bolt diameter and timber thickness for hardwood timber (joint group J1) and seasoned pine (joint group JD4), as were used in this research program.

TABLE 1
ESTIMATED MEAN LOADS AT FAILURE

Timber	Thickness (mm)	Bolt diameter (mm)	Failure load 2-member joint (kN)	Failure load 3-member joint (kN)
J1	33	12	21	22
J1	33	16	29	30
J1	66	16	35	60
JD4	35	12	13	14
JD4	35	16	17	18
JD4	70	16	33	36

The estimates should only be used as guide against which the test joints in this research program can be measured.

3. TEST PROGRAM

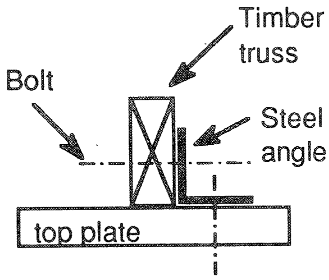
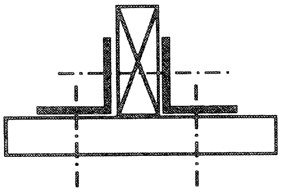
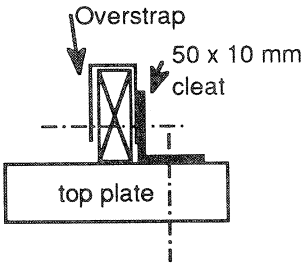
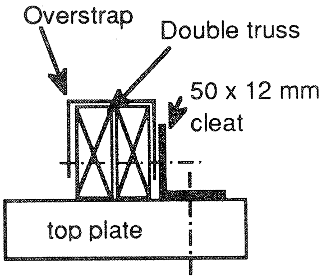
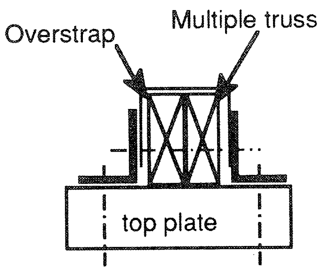
3.1 Test Parameters

The aim of the program was to determine the design strength of different devices that are used to secure trusses to walls. Table 2 includes details of the different configurations of hold down that were tested for both timber types. All involve a single transverse bolt through one or two side plates which are attached to the wall. Unless specified otherwise the bolts were mild steel. Thirty two millimetre diameter washers were used with the 12 mm bolts and 35 mm diameter washers were used for the 16 mm bolts. Cuphead bolts were used for joint Type 1 for both timbers, but the other types used machine bolts. Cuphead bolts were chosen because field investigations showed that they are used in practice for this type of joint.

In addition to the hold down angles or cleats, Types 3, 4 and 5 had an overstrap as illustrated in Figure 2. The thickness of the overstrap was 2 mm for hardwood, but was increased to 3 mm for the pine trusses, to compensate for the lower bearing strength of that timber. The stronger details, Types 4 and 5, required a multiple truss to resist the specified hold down forces.

One overriding parameter in the whole test program was to produce failures in the timber members rather than break a bolt. The original test program had to be modified in some instances to ensure that this occurred. For example, the first tests

TABLE 2
HOLD DOWN DETAILS

Type	Illustration	Connection details	Through truss plate?
1		Single 75 x 75 x 8 mm angle bracket with 12 mm cuphead bolt for JD4 and 16 mm cuphead bolt for J1.	yes and no
2		Two 75 x 75 x 8 mm angle brackets with 12 mm bolt	yes and no
3		50 x 2 mm* overstrap, M12 HS bolt for JD4 and M16 bolt for J1, single 50 x 10 mm cleat.	yes
4		50 x 2 mm* overstrap, M16 HS bolt, single 50 x 12 mm cleat	yes
5		50 x 2 mm* overstrap, M16 HS bolt, two 50 x 10 mm cleats	yes

*3 mm strap used for JD4 trusses

showed that the ultimate strength of a Type 1 joint with a 12 mm bolt in J1 timber was the shear strength of the M12 bolt. In order to achieve timber failure an M16 bolt was used in the test program. The series most affected by this reassessment was the joint Type 3 one, where the joint had to be strengthened for both timber types. The bolt size was increased for J1 timber and the bolt strength was increased for JD4 timber. As will be discussed later, this upgrading of the hold down details without upgrading the trusses may have adversely affected the overall performance of this Type 3 system.

Another variable included in the test program was the location of the bolt, either it passed through the heel plate of the truss, or was located above the heel plate. As indicated in Table 2 Types 1 and 2 were tested for both of these conditions whereas the other types had the bolt through the nailplate. Other variables included timber type and thickness and bolt diameter and strength.

All of the hold down details would apply equally to masonry wall construction as to timber framed walls.

For clarity the illustrations in Table 2, and in the appendix, show the overstrap clear of the truss. In practice the strap must be tight against the truss, as it was for this test program. Otherwise the recommendations given in this paper may not apply.

3.2 Trusses

For convenience, and to reduce cost, the test trusses were made only 2 m long. But they were still designed to resist high reaction forces. In effect, they were virtually two full size heel joints joined together. Figure 3 shows the truss configuration.

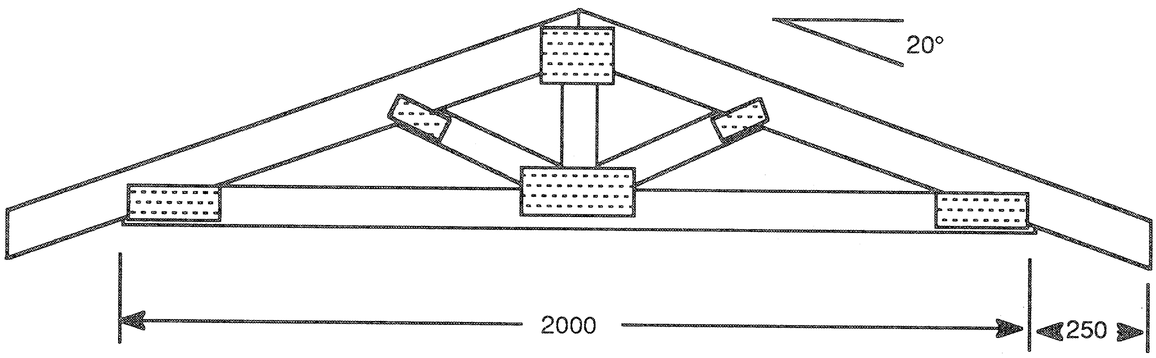


Figure 3 Test Truss

The trusses were made by commercial fabricators. The pine trusses were made by one fabricator and the hardwood trusses by another (see Acknowledgments). They were designed for a pair of concentrated loads on each top chord causing specified permissible stress design reactions of 10 kN, 30 kN and 40 kN. These values were specified as being in excess of estimates of strength of each of the hold down details. As previously mentioned, multiple trusses were required for the high reaction forces.

Although the timber was specified to be joint strengths J1 and JD4, the design of the truss members was based on bending stress, thus the hardwood was defined as green F17 (basic bending stress 17 MPa) and the pine was seasoned F8 (basic bending strength 8 MPa). It is accepted that these stress grades and joint strengths are compatible. All of the hardwood trusses were fabricated from spotted gum which is low in the range for J1 timbers and the JD4 was seasoned radiata pine free of any pith.

Ten replications of each truss/hold down combination were tested. Thus the test program included 140 truss tests.

Table 3 summarises the truss details.

TABLE 3
TRUSS DETAILS

Hold down types (Permissible stress design Reaction)	Hardwood trusses (J1)		Pine trusses (JD4)	
	Top chord	Bottom chord	Top chord	Bottom chord
T1, T2, T3 (10 kN)	100 x 35	100 x 35	120 x 35	90 x 35
T4 (30 kN)	2/100 x 35	2/100 x 35	2/190 x 35	2/90 x 35
T5 (40 kN)	2/100 x 35	2/120 x 35	3/140 x 35	3/90 x 35

3.3 Loading

Design of a loading rig proved somewhat of a challenge, as it was anticipated that loads in excess of 200 kN (20 tonnes force) would need to be applied to the trusses to fail the strongest hold down details. The physical constraints of the short span truss with full size members meant that only two loads could be applied per top chord, midway along each bay of each top chord. Figure 4 shows the loading configuration. Hydraulic rams were used to apply loads "P" normal to the top chord and a force transducer measured the vertical reaction "R" at one end. Care was taken to ensure that the two reactions were equal. This was checked on a number of tests of the lighter trusses, where a pair of force transducers of suitable capacity was available.

To prevent failure by lateral buckling, both the top chords and the bottom chord of each truss were braced laterally at two locations along their length. The braces were designed to allow unrestricted vertical movement without any vertical force transfer into the bracing structure.

Despite the fact that the trusses had been designed for the above loading system, a comparison was made with the forces produced by uniform uplift normal to the top

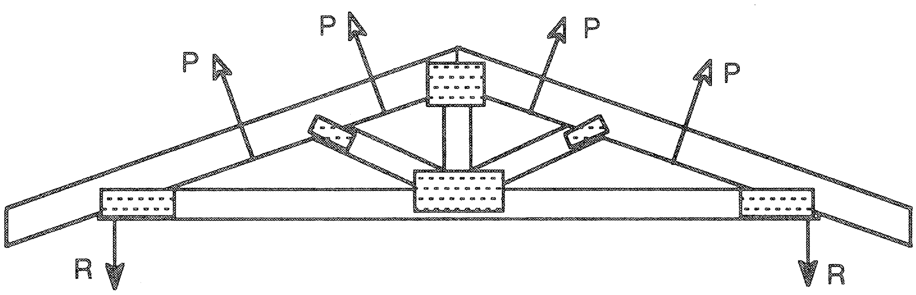


Figure 4 Loading Configuration

chord to ensure that the loading system did not create any abnormal stresses. Table 4 lists the differences in maximum load between the two systems for vertical reactions of 10 kN at each end. As would be expected the main discrepancy was in bending moment at the points of application of the test load. As all test joints used a single bolt connection through the top chord, it was assumed that this difference in bending moment would not affect the performance of the hold down device. The small differences in axial load and shear were considered acceptable.

TABLE 4
COMPARISON BETWEEN UNIFORM LOADING AND TEST LOADING FOR 10 KN REACTION FORCES

Maximum force in top chord	Uniform loading	Test loading	Percent difference
Axial	23.4 kN	23.6 kN	+1%
Shear	3.1 kN	3.0 kN	-3%
Moment	2.6 kN.m	4.6 kN.m	+77%

To maintain uniformity during the test program, the bolt holes were located in the same position for each type of test. The holes were 75 mm in from the end of the span and 25 mm from the bottom edge of the top chord when the bolt went through the nailplate, or 25 mm from the top of the top chord when the bolt went above the nailplate. These edge distances were obtained from observations of building practice. Figure 5 illustrates the alternative locations of the transverse bolt.

Figure 6 shows a typical test configuration at one end for a Type 1 joint. The trusses were supported on a pair of RHS steel beams so that the force transducer measuring reaction could be located directly beneath the transverse hold down bolt. For Type 1, Type 3 and Type 4 joints flat side plates were used rather than angle pieces to improve local stability of the test trusses. In reality that stability would be provided by battens, roofing and lining. A steel rod was welded to the side plate and connected to the force transducer which measured the reaction force. For Type 2

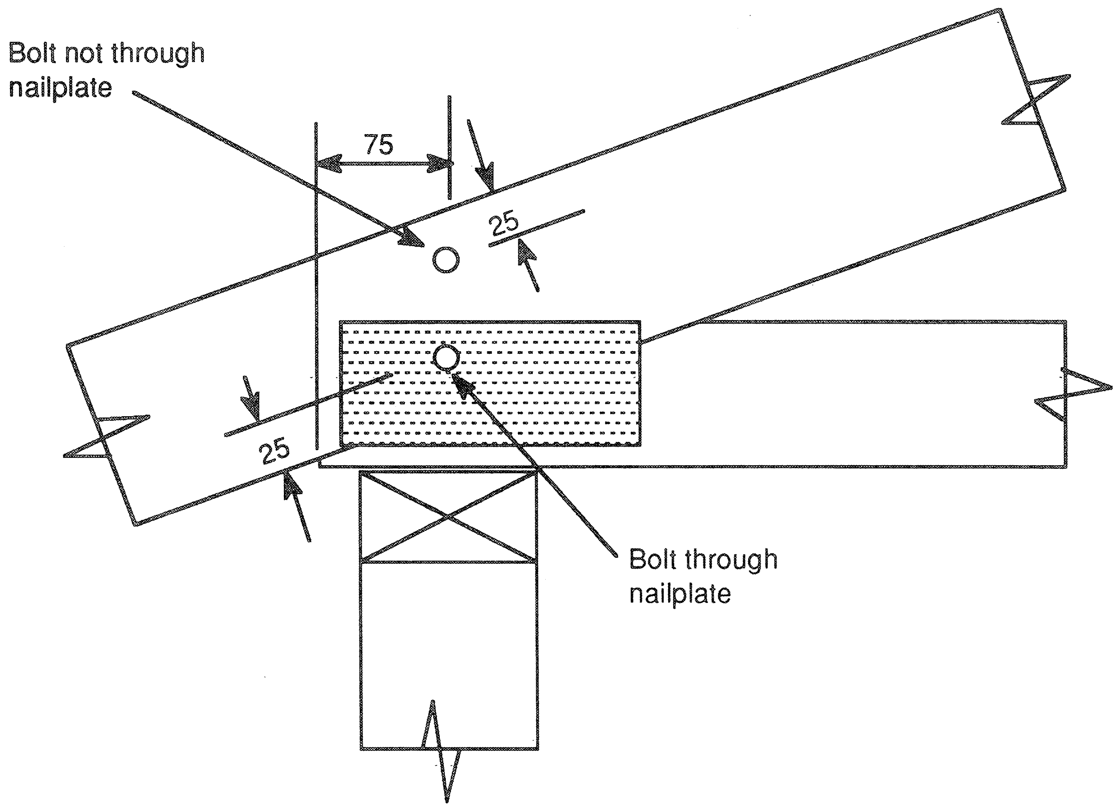


Figure 5 Locations for Transverse Bolts

and Type 5 joints U-shaped hold down devices were used to simulate the pair of side plates. This allowed the force transducer to be located on the longitudinal centre line of the truss and eliminated eccentricities at the reaction points.

Both the first hardwood Type 1 truss and the first pine Type 1 truss failed because the tensile forces in the top chords tore the apex nailplate apart. There was no sign of failure at the hold down position. As this type of failure was not part of the investigation, all subsequent trusses in the test program were reinforced by a 40 x 5 mm steel strap screwed to the top face of each top chord and bridging across the apex joint. For multiple trusses, multiple straps were used. This decision is considered acceptable as the truss reactions often exceeded three times the permissible stress design value nominated for the trusses. The apex nailplates would not be expected to have this same degree of reserve strength, because of the relatively low variability in the strength of steel.

Those first Type 1 trusses were tested again with reinforced apex joints.

4. TEST RESULTS

This section summarises the test results and presents a statistical analysis of them. Individual results of each test are given in Appendix A. The appendix also includes descriptions of failure for each of the different joints tested.

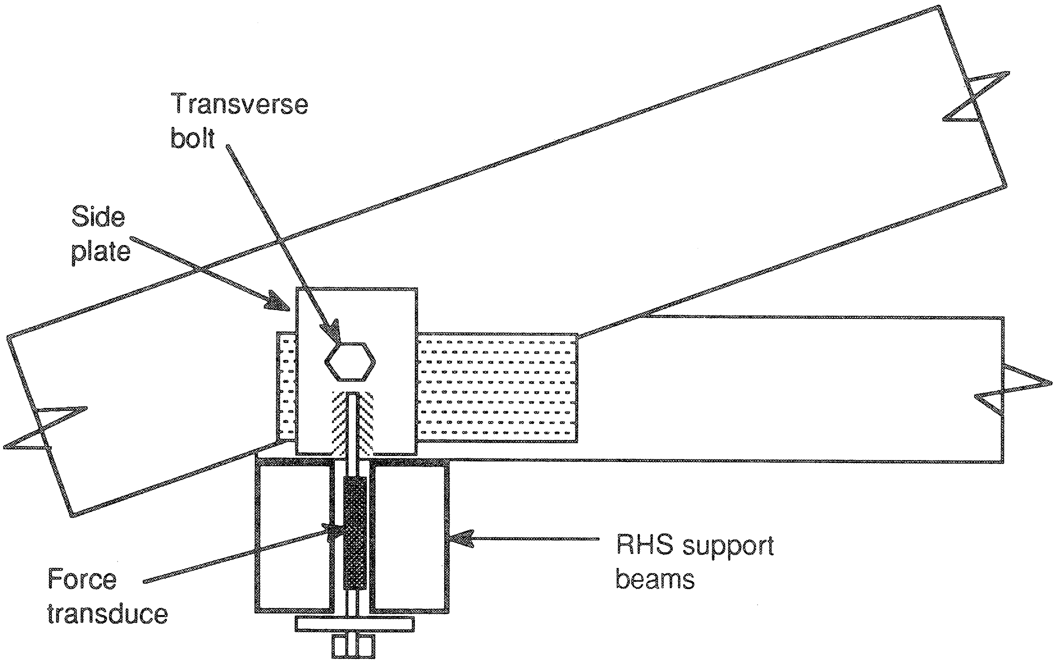


Figure 6 Reaction Configuration for a Type 1 Test Truss

As explained in Section 3.1, the original test program was sometimes modified to achieve timber failure at the joints rather than bolt failure. However in some instances, notably with Type 3 joints, this upgrading was only partially successful as it resulted in the bending failure of some top chords. This was not surprising as reaction forces as high as 50 kN were applied to trusses that were designed for only 10 kN reactions. Thus the Type 3 joints have a greater potential than is indicated by this test program.

4.1 Hardwood Trusses

Nominally ten replications were conducted on each hold down type, but in fact the number varied from 9 to 12. For those joint types where the first test resulted in shear of the transverse bolt, it was changed to a larger diameter or higher strength bolt. In some instances an additional test could be made on that truss if the other end was not badly damaged, but for Type 4 in JD4 timber only nine trusses were tested with M12 high strength bolts.

In summarising test results for hardwood trusses, Table 5 lists the mean, standard deviation, the percentage coefficient of variation (CoV) and minimum failing load for each joint type in hardwood trusses. For Type 1 and Type 2 joints the results include tests where the transverse bolt went through the heel nailplate and tests when it was above the nailplate. The nomenclature used in the column headings is as follows: T is the hold down type as defined in Table 2, Y indicates the bolt passed through the nailplate, alternatively N indicates it was above the nailplate, 12 and 16 are bolt diameters, HS indicates high strength bolts and MS indicates mild steel bolts. Thus the column headed T2-Y-12 MS gives test results for hardwood trusses having two angle brackets and a 12 mm mild steel bolt passing through the nailplate.

TABLE 5
SUMMARY OF REACTION FORCES AT FAILURE
HARDWOOD TRUSSES

	T1-Y-16 MS	T1-N-16 MS	T2-Y-12 MS	T2-N-12 MS	T3-Y-16 MS	T4-Y-16 HS	T5-Y-16 HS
No in Sample	10	10	12	12	10	10	10
Mean React. (kN)	37.8	39.1	34.2	33.5	46.9	73.5	114
Std Dev.	4.0	4.1	4.4	8.3	4.1	3.7	12.3
CoV %	10.7	10.5	12.9	24.9	8.8	5.1	10.7
Min. React. (kN)	29.8	32.3	29	23.1	41	65.8	92

The original test program had both Type 1 joints and Type 2 joints using 12 mm transverse bolts so a direct comparison could be made with softwood, but the bolts broke in combined shear and tension at 30 kN. This could be taken as an upper limit for 12 mm cuphead bolts in a T1 joint. While T1-12 MS joints could be used in practice, their design is governed by the strength of the transverse bolt in single shear rather than the strength of the timber.

Both T1 and T2 indicate that there is virtually no difference in average strength whether the transverse bolt goes through the heel nailplate or is located above it. However the unusually high variability of results of the T2-N-12 MS trusses produced a much lower minimum value for that type. The other coefficients of variation are about 10% to 12% which is not unreasonable for timber that is reinforced by nailplates. Initial analysis of the T2-N-12 results shows no obvious reason for the wide variability. The mean loads at failure for both T1 and T2 are well in excess of the predicted mean of 29 kN and 22 kN respectively listed in Table 1 for timber without nailplates.

A comparison of the results from T1 and T3 shows the effect of the 50 x 2 mm overstrap. It resulted in an increase in mean strength of approximately 25%. As the coefficients of variation of the two sets of results are similar, the design strength would also be about 25% higher. Even this increase in strength was somewhat surprising as the lopsided nature of the joints with only one hold down cleat meant that one end of the overstrap was free to move relative to the other, as the bolt bent. As mentioned previously, the average performance of the T3 series was limited by the strength of the trusses which had been underestimated for this joint type.

The main difference in joint detail between T3 and T4 in the hardwood trusses was that the M16 bolt in T4 was high tensile, but the higher specified design loads for T4 required double trusses. This combination improved the performance of the T4 trusses by more than 50%, as both the bolt and the overstrap had twice the bearing area and the high strength bolts did not bend as readily. However the larger eccentricity caused by the double truss with the single cleat hold down meant that these T4 trusses tended to roll more as they were loaded.

The T5 trusses with two hold down cleats, overstrap and 16 mm high strength bolts were very strong with many resisting reaction forces in excess of 100 kN. There were several different types of failure. In some instances nailplates split or released, in others the top chord split at a reaction position and broke in bending. The only direct comparison that can be made is with the single cleated system of T4, where more than 50% increase in mean load was achieved by the T5 system.

4.2 Pine Trusses

Table 6 summarises the performance of the pine trusses.

TABLE 6
REACTION FORCES AT FAILURE
PINE TRUSSES

	T1-Y-12 MS	T1-N-12 MS	T2-Y-12 MS	T2-N-12 MS	T3-Y-12 HS	T4-Y-16 HS	T5-Y-16 HS
No in Sample	10	10	10	10	9	10	10
Mean React. (kN)	21.1	25.9	23.8	28.5	35.8	51.6	84.4
Std Dev	2.84	2.16	2.69	3.07	5.26	5.47	9.00
CoV %	13.5	8.3	11.3	10.7	14.7	10.6	10.7
Min. (kN)	17.8	23.0	20.9	23.4	31.3	41.5	66.4

Again types T1 and T2 showed that the joints with the transverse bolt above the heel nailplate were not weaker than when the bolt passed through the nailplate. In fact in both instances the joint with the bolt above the nailplate was stronger. The reason for this is not clear. In five of the ten tests of T1 configuration with the bolt above the nailplate, the bolt broke in combined shear and tension. The values have been included in the analysis because they represent the four of the top six reaction strengths, and thus the truss strengths were even higher. The mean loads at failure for T1 and T2 were well in excess of the estimated values of 13 kN and 14 kN listed in Table 1.

Comparison of types T1 and T2 indicate that the double shear case of type T2 was stronger than T1 for both locations of the transverse bolt. The severe bending of the M12 MS bolt in single shear would have concentrated a high stress at the edge of the top chord adjacent to the cleat, leading to failure.

In the original test program 12 mm mild steel cuphead bolts were meant to be used for the T3 series. One group of trusses was tested with those bolts, all of which failed in combined shear and tension. The test program was then rearranged to replace them with high strength bolts. This could be achieved because one end of this group of trusses was supported by a very strong clamp arrangement which caused no damage to the truss at the loads at which the other end failed.

This need to use high strength bolts for T3 joints eliminated the planned direct comparison between them and T1 joints, to ascertain the effect of the overstrap. The combined effect of the high strength bolt and the overstrap resulted in T3 being on average about 70% stronger than T1. But the strength of T3 was limited by a number of bending failures of the top chords, because the trusses had been designed for only 10 kN reaction force. Thus the strength of the T3 joints is potentially greater than will be concluded from this study.

The average strength of T4 was almost 45% higher than T3. This was due both to the increase in bolt size and to the use of double trusses. The higher design specifications for T4 trusses ensured hold down failures rather than bending failures of the top chord, as occurred with the T3 series.

Series T5 with the transverse M16 HS bolt in double shear averaged 64% stronger than T4. This ratio is higher than was achieved for the hardwood trusses, but it compares a triple truss with a double one, so there was a 50% greater bearing area for both the bolt and the overstrap. Again the additional strength was achieved in part by securing each end of the overstrap to a cleat, rather than the lopsided arrangement of T4.

4.3 Bolt Performance

As mentioned in Section 3.1 the aim of the program was to produce failures relating to the timber at the joints rather than break the bolts. But in some preliminary tests the mild steel M12 bolts failed in a combination of single shear and tension. For the 8 Type 3 tests in JD4 trusses where this was allowed to happen, the average load at failure was 31 kN, with a minimum load of 29.5 and a coefficient of variation of nearly 4%. These figures could be used to determine design loads for joints of Type 3 configuration using mild steel M12 bolts.

Another hold down detail that was tried but discarded was a Z-shaped bracket made from 75 x 8 mm steel. The top flange was angled to 20° to suit the slope of the trusses and a lip added to prevent it slipping off the top chord. The natural eccentricity of this shape caused considerable distortion under load, despite the lateral support provided. The test broke the M12 anchor rod in tension and bending. It was decided not to pursue this system further, as fabrication of the bracket would make it an expensive system in the field.

The overall performance of joints with high strength transverse bolts was significantly better than those with mild steel bolts. Most of the mild steel bolts were severely bent as the joint failed, whereas the high strength ones only bent a small amount.

4.4 Displacements

The investigation concentrated on the strength of the hold down joints rather than their serviceability. Displacement measurements were not formally recorded. The only joint where displacement was measured was replication 10 of JD4-T4-Y. It was taken for that truss because the overstrap appeared slightly loose and the double truss with single cleat represented a large eccentricity at the joint. This combination indicated that large displacements were likely. The displacement was measured at the edge of the bottom chord away from the cleat, so it represents a combination of vertical movement and rotation.

Figure 7 shows the load displacement curve for the joint which was measured as about 35 mm at failure. Also shown on the graph is the permissible stress design load for wind for the joint (27 kN, see Table 8) and the matching displacement of about 11 mm.

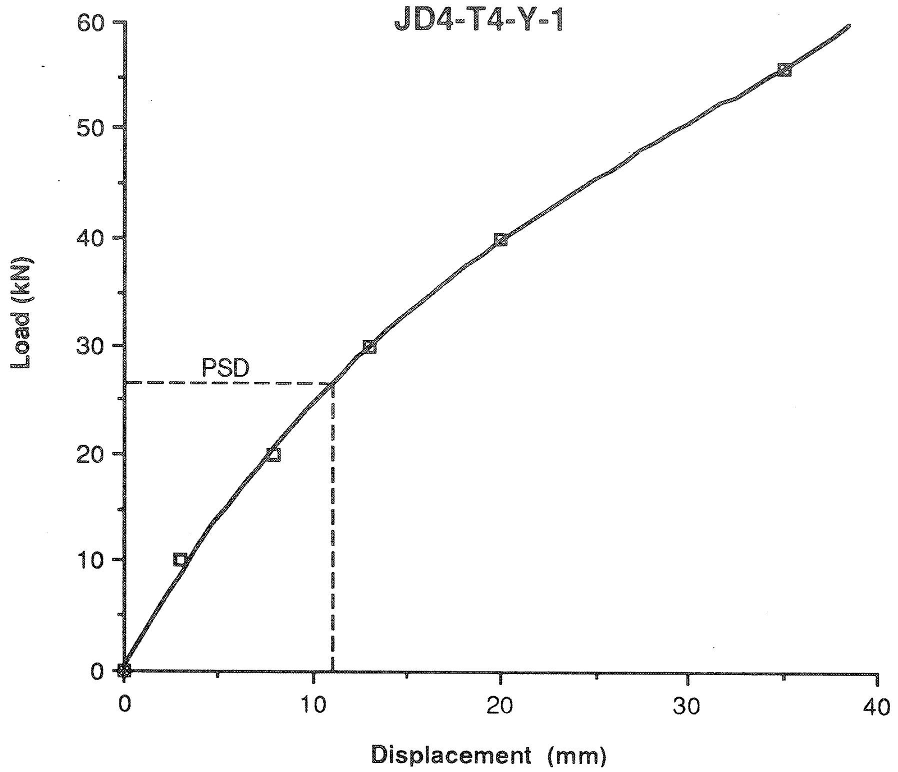


Figure 7 Typical Load Displacement Curve for Type 4 Joint

In general, those joints with balanced hold down provisions, Type 2 and Type 5, had relatively small displacement whereas the eccentric joints, Type 1, Type 3 and Type 4 displaced and twisted significantly under load.

5. DESIGN LOADS

While the Timber Engineering Code (AS 1720) is at present being rewritten in limit states format, the limit states version of the code for derivation of design loads for fasteners is only just being addressed. The following derivations are based on a draft proposed by Foliente (1996) and private correspondence (Leicester and Foliente, 1996), but this draft is likely to be revised in some form before final publication. Therefore the design capacities calculated in this section should be taken as interim.

The characteristic value R_k is calculated by

$$R_k = (N/27)^V R_{min} \tag{1}$$

where N is the number of samples, V is the coefficient of variation and R_{min} is the weakest test result.

The characteristic capacity (defined as Q_{sk} in draft AS 1720) is then given by

$$Q_{sk} = (0.85\text{-}0.95V)\ R_k/\phi \tag{2}$$

The value of ϕ for bolted joints has been taken as 0.7, as recommended in the draft AS 1720.

When designing joints for wind forces the draft code allows the characteristic capacity to be increased by a duration of loading factor of 1.14 to obtain the design capacity Q, to satisfy the fundamental strength limit state requirement

$$\phi Q \geq S^* \tag{3}$$

where S^* is the design action effect.

The ultimate limit states (ULS) design capacity Q for wind design for each test type is listed in Table 7 for hardwood and in Table 8 for pine.

The permissible stress design loads for bolts have been derived using Foliente's recommended statistical approach rather then that given in the old AS 1649-1972. The specified basic load capacity, R_{bws}, is defined as

$$R_{bws} = (0.85\text{-}0.95V)\ R_k/(1.35\ k_d) \tag{4}$$

where R_k is the characteristic value derived from equation 1 and k_d is the duration of load factor for 5 seconds. The permissible stress design load for wind is

$$PSD = k_d \times R_{bws}. \tag{5}$$

These permissible stress design (PSD) values for wind loading are also listed in Tables 7 and 8.

It must be emphasised that the design values listed in Tables 7 and 8 relate only to the joints tested and defined in Table 2. Any change in geometry, timber joint strength, bolt diameter, bolt strength, truss thickness or even cleat thickness is likely to affect the performance of the joints. Some engineering judgement may be

TABLE 7
DESIGN CAPACITY FOR WIND LOAD
J1 HARDWOOD JOINTS

	Design Capacity (kN) for Joint Types						
	T1-Y-16 MS	T1-N-16 MS	T2-Y-12 MS	T2-N-12 MS	T3-Y-16 MS	T4-Y-16 HS	T5-Y-16 HS
ULS Wind (Q)	33	36	31	19	47	82	101
PSD Wind	15	16	14	9	21	37	46

TABLE 8
DESIGN CAPACITY FOR WIND LOAD
JD4 PINE JOINTS

	Design Capacity (kN) for Joint Types						
	T1-Y-12 MS	T1-N-12 MS	T2-Y-12 MS	T2-N-12 MS	T3-Y-12 HS	T4-Y-16 HS	T5-Y-16 HS
ULS Wind (Q)	18	27	23	26	31	60	73
PSD Wind	8	12	10	12	14	27	33

required in using the T2-N-12 MS design values for J1 joints.

Further, the laboratory test program did not include normal details for the transfer of the high reaction forces to the foundations. In some instances this force transfer may prove a significant challenge to the designer.

6 CONCLUSIONS

This research program demonstrates that the presence of heel nailplates significantly increases the strength of truss hold down systems based on a transverse bolt. Mean loads at failure were more than 50% greater than predicted for both the J1 hardwood and JD4 pine trusses. With one exception, this increased strength was achieved whether the bolt penetrated the nailplate or was located directly above it.

Where direct comparison was available, for the JD4 joints, the system with two cleats was slightly stronger than that with one cleat only. The former system is recommended as it resulted in significantly less displacement and distortion at failure.

For joints in J1 timber, the presence of the overstrap increased the design strength of the joint by about 50%, however a direct comparison cannot be made for JD4 joints as the bolt strength had to be increased to cope with the higher forces.

The overstrap performed significantly better for the system with two side cleats than that with only one. This is because both ends of the overstrap were retained in position.

The Type 3 joints with overstrap have the potential for even higher strength than indicated here, as the design of the trusses underestimated the joint strength and partly limited their potential.

The design data listed in Tables 7 and 8 should be taken as indicative only, as the sample size for each joint type was relatively small and the derivation of design loads was based on draft procedures. Further investigations need to be made into the results of the rogue T2-N-12 MS system.

7 REFERENCES

Lhuede E. P. (1987) Tests for bolted timber joints loaded perpendicular to grain. In: Proceedings, First National Structural Engineering Conference, Institution of Engineers, Australia. August 26-28 Melbourne. pp 325 - 329.

Rogers C., Thomson R. and Smith G. (1994) Nailplate reinforcement of bolted joints. In: Proceedings Pacific Timber Engineering Conference, Gold Coast, Australia. July 11 - 15. pp 349 - 352.

Foliente G. C. (1996) Test methods and evaluation for mechanical joint systems - Draft proposal for Committee purposes.

Leicester R. H. and Foliente G. C. (1996) Private correspondence.

Standards Australia (1988) Timber structures code. Part 1 - Design methods. AS1720.1-1988. Standards Association of Australia.

Standards Australia (1995) Timber structures code. Part 1 - Design methods (Revision of AS 1720.1-1988).

8 ACKNOWLEDGMENTS

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APPENDIX

Individual maximum loads and modes of failure for the test trusses are listed in this appendix.

It should be noted that the descriptions of failure are summaries of the actual modes of failure. For example, all trusses with a single angle hold down cleat tried to roll and distort laterally at the heel due to the eccentricity of loading and reaction. In most cases this is not recorded in the description of failure although it would have been a contributing factor to the withdrawal of heel nailplates, which tends to have been recorded.

In all cases the timber crushed at the bolt hole and the mild steel bolts usually bent, especially when the joint had a single cleat. These are usually not recorded, as they are expected to happen. There was also considerable crushing of the timber at the overstrap as well as the bolt before the mechanism to cause ultimate failure occurred.

For the Type 2 joints, with two cleats, virtually the only method of failure that could occur at the support position was for the top chord to split through the bolt hole. This was the predominant mode of failure for the pine trusses and for the hardwood ones with the joint above the nailplate. It is unlikely to have been influenced by either the edge distance or end distance dimensions.

For the very high test loads achieved by the Type 5 joints in multiple trusses, an apt description of failure may well be "truss disintegrated", as a number of elements broke and contributed to the overall failure. There was usually significant crushing of the top chord under the overstrap, but this was not seen as the principal mode of failure and therefore is rarely mentioned.

Although mention is made of the apex nailplate splitting in the highly loaded trusses, this was rarely the principal mode of failure as the reinforcing straps still transferred load between the top chords. The authors felt that this mode should be noted to remind the user that failure can occur at this location.

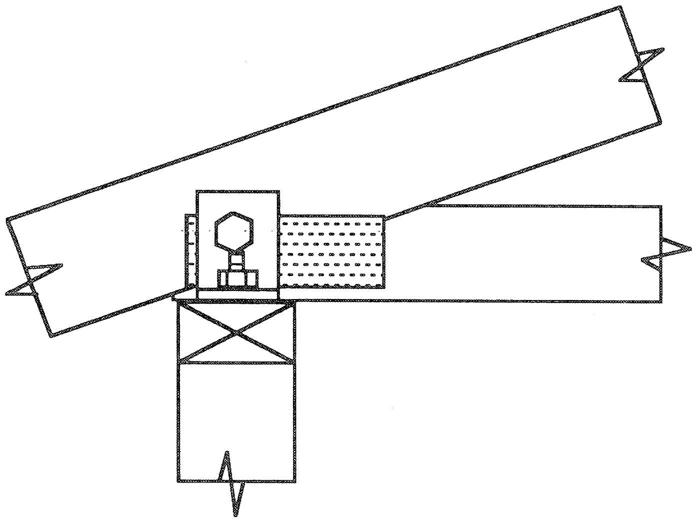
TEST RESULTS

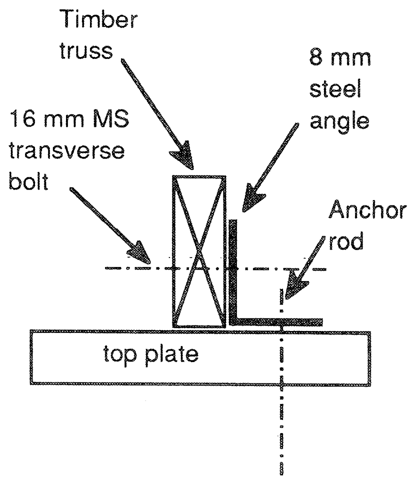
Hardwood trusses (J1) Single truss designed for 10 kN uplift reactions.

Joint Type T1

Description: M16 mild steel cuphead bolt passing through a single 8 mm thick steel angle and heel nailplates.

Legend: T1 - Y - 16 - MS





Maximum Load (kN)	Failure Mode
39.6	Heel joint twisting, heel nailplate releasing
37.8	Timber split along top chord through bolt hole
42.6	Heel joint twisting, heel nailplate releasing
39.6	Heel joint twisting, heel nailplate releasing
36.3	Timber split along top chord
40.0	Heel joint twisting, heel nailplate releasing
35.5	Timber split along top chord, heel nailplate releasing
29.8	Timber split along gum vein
43.0	Heel joint twisting, heel nailplate releasing
34.1	Timber split along top chord, heel nailplate releasing

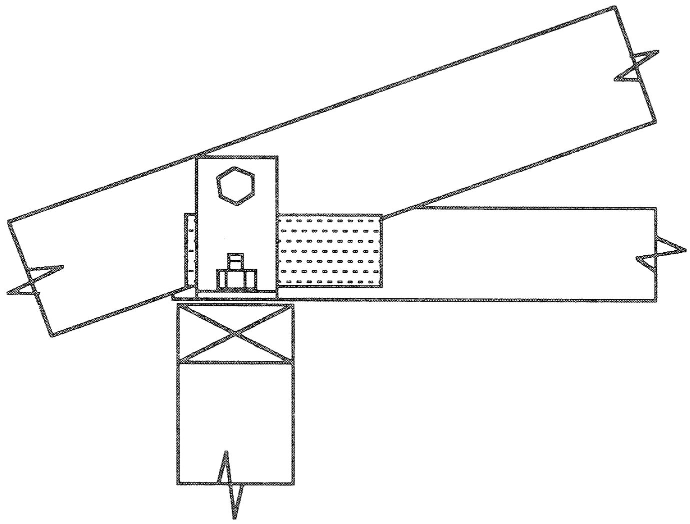
TEST RESULTS

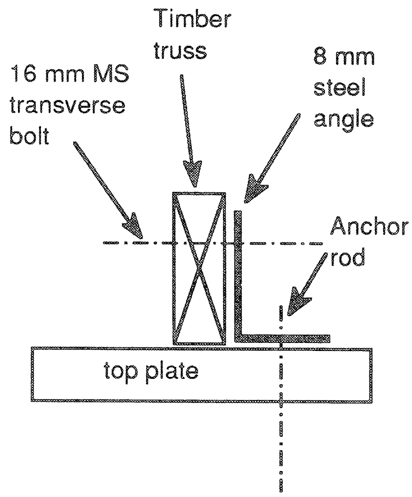
Hardwood trusses (J1) Single truss designed for 10 kN uplift reactions.

Joint Type T1

Description: M16 mild steel cuphead bolt passing through a single 8 mm thick steel angle above heel nailplates.

Legend: T1 - N - 16 - MS





Maximum Load (kN)	Failure Mode
32.3	Bolt bearing in top chord, splitting, shear failure of nailplate
39.2	Timber twisting, splitting, nailplate shearing
36.7	Timber twisting, splitting, nailplate shearing
38.8	Timber twisting, splitting, nailplate shearing, eventual breaking of top chord
43.0	Timber twisting, splitting, apex nailplate splitting
38.6	Timber twisting, splitting, apex nailplate splitting
47.8	Timber twisting, splitting, apex nailplate splitting
37.4	Timber twisting, splitting, apex nailplate splitting, strut nailplate splitting
37.3	Timber twisting, heel nailplate shearing, top chord split through bolt hole.
40.3	Timber twisting, heel nailplate shearing, top chord split through bolt hole.

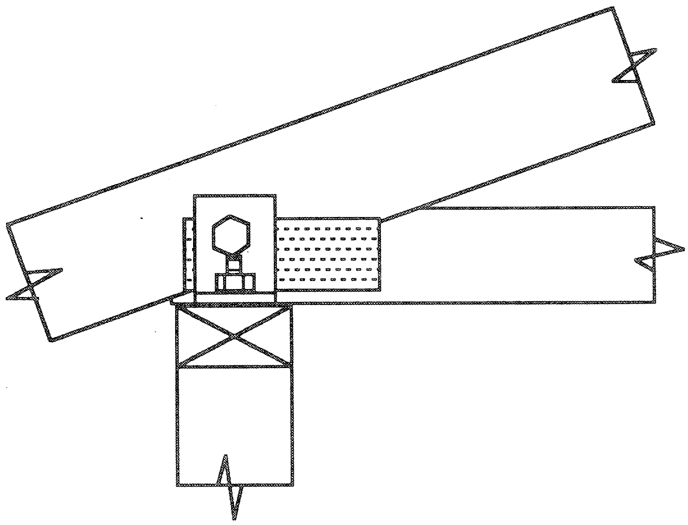
TEST RESULTS

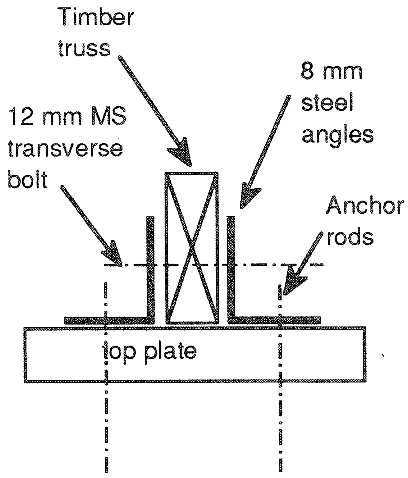
Hardwood trusses (J1) Single truss designed for 10 kN uplift reactions.

Joint Type T2

Description: M12 mild steel bolt passing through two 8 mm thick steel angles and heel nailplates.

Legend: T2 - Y- 12 - MS





Maximum Load (kN)	Failure Mode
37.2	Top chord split from bolt hole to end, through nailplate
30.2	Top chord split
32.3	Bolt bearing in nailplate, timber splitting, nailplates starting to withdraw
37.8	Ridge nailplate split
33.3	Bolt bearing in nailplate, timber splitting, nailplates starting to withdraw
30.1	Bolt bearing in nailplate, timber splitting, nailplates starting to withdraw
29.0	Bolt bearing in nailplate, timber splitting, nailplates starting to withdraw
32.6	Top chord split from bolt hole to end, through nailplate
34.0	Top chord split from bolt hole to end, through nailplate
32.6	Bolt bearing in nailplate, timber splitting, nailplates starting to withdraw
36.5	Top chord split from bolt hole to end, through nailplate
45.0	Bolt bearing in nailplate, timber splitting, nailplates starting to withdraw

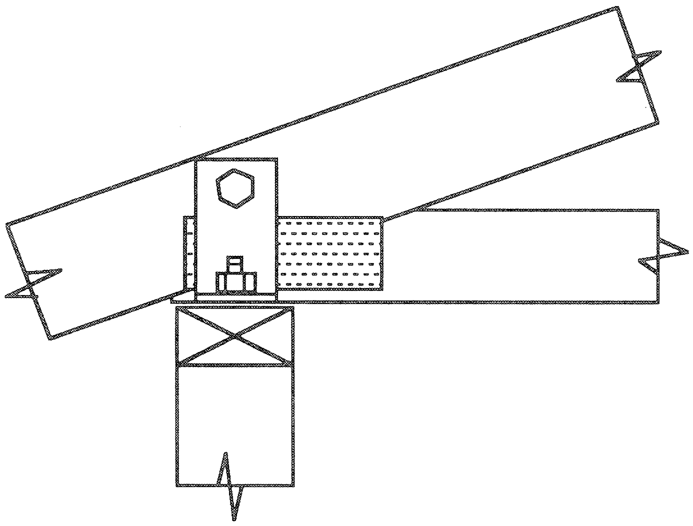
TEST RESULTS

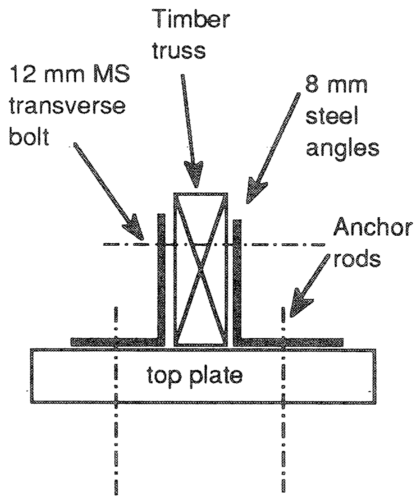
Hardwood trusses (J1)   Single truss designed for 10 kN uplift reactions.

Joint Type T2

Description: M12 mild steel bolt passing through two 8 mm thick steel angles above heel nailplates.

Legend: T2 - N- 12 - MS





Maximum Load (kN)	Failure Mode
42.2	Ridge nailplate split
41.8	Top chord split through bolt hole
39.0	Heel nailplate shearing
43.4	Top chord splitting, heel nailplate shearing
35.2	Top chord split through bolt hole
39.5	Top chord split through bolt hole
23.7	Top chord split through bolt hole
23.1	Top chord split through bolt hole
37.3	Top chord splitting, heel nailplate shearing
24.0	Top chord split through bolt hole
30.3	Ridge nailplate split
22.0	Top chord split through bolt hole

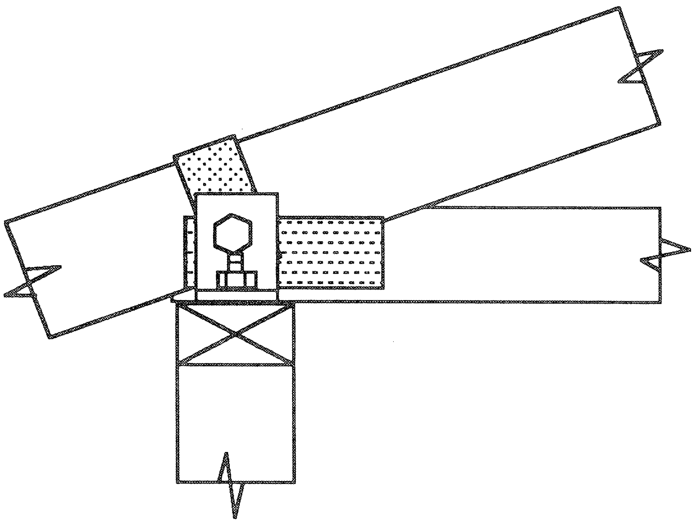
TEST RESULTS

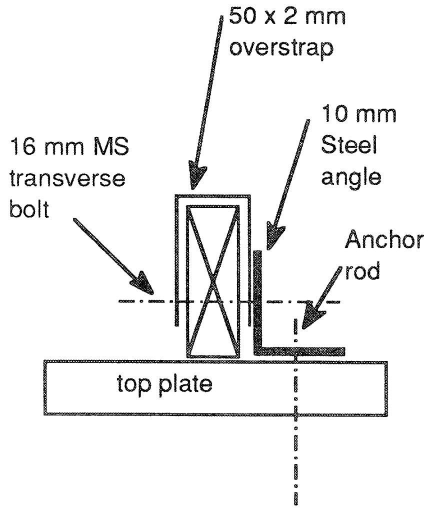
Hardwood trusses (J1)   Single truss designed for 10 kN uplift reactions.

Joint Type T3

Description: M16 mild steel bolt passing through a single 10 mm thick steel angle, heel nailplates and 50 x 2 mm overstrap.

Legend: T3 - Y- 16 - MS





Maximum Load (kN)	Failure Mode
51.0	Apex nailplate split, top chord broke
43.1	Apex nailplate split, top chord broke
41.0	Top chord broke, apex nailplate and heel nailplate failed
47.0	Apex nailplate split, top chord broke
50.8	Apex nailplate and heel nailplate failed, top chord broke
45.0	Apex nailplate split, top chord split along screws in reinforcing strap
41.7	Apex nailplate split, top chord broke
47.5	Apex nailplate split, top chord split along screws in reinforcing strap
48.7	Apex nailplate and heel nailplate failed, top chord broke
52.9	Apex nailplate split, top chord broke

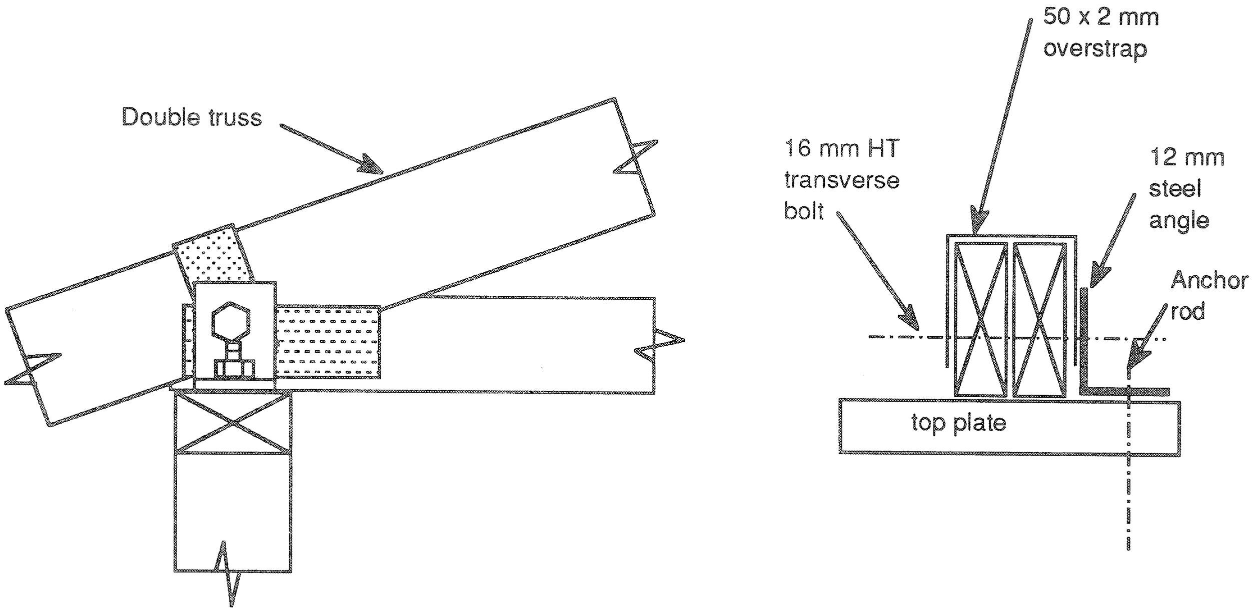
TEST RESULTS

Hardwood trusses (J1) Double truss designed for 30 kN uplift reactions.

Joint Type T4

Description: M16 high strength steel bolt passing through a single 12 mm thick steel angle, heel nailplates and 50 x 2 mm overstrap.

Legend: T4 - Y- 16 - HT



Maximum Load (kN)	Failure Mode
65.8	Transverse bolt broke
70.7	Top chord twisted, heel nailplate withdrawing
74.0	Heel nailplate withdrawing, truss rolling at support, cleat badly bent.
73.6	Heel nailplate withdrawing, truss twisting, top chord split through bolt hole
73.6	Apex plate split, heel nailplate withdrawing, truss twisting, top chord split through bolt hole
72.0	Heel nailplate withdrawing, truss twisting, top chord split through bolt hole
74.0	Heel nailplate withdrawing, truss twisting, top chord split through bolt hole
77.0	Apex plate split, heel nailplate withdrawing, truss twisting, top chord split through bolt hole
80.0	Heel nailplate withdrawing, truss twisting, heel plate badly crushed
74.0	Heel nailplate withdrawing, truss twisting, top chord split through bolt hole

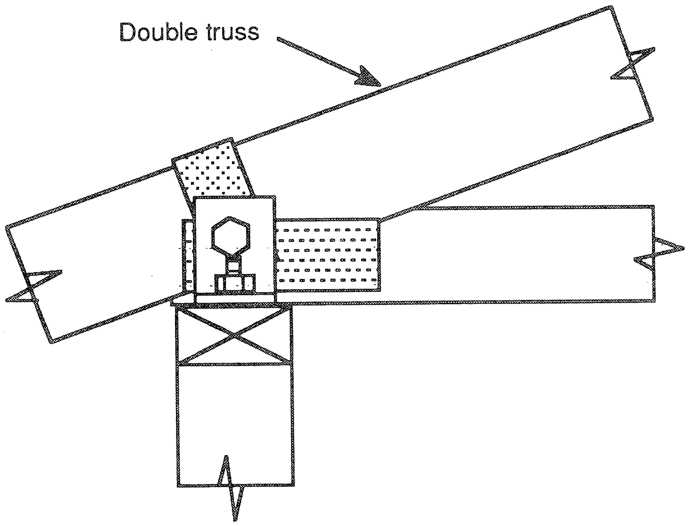
TEST RESULTS

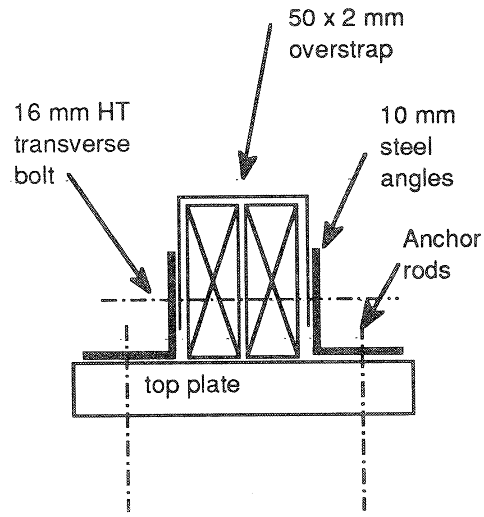
Hardwood trusses (J1) Double truss designed for 40 kN uplift reactions.

Joint Type T5

Description: M16 high strength steel bolt passing through two 10 mm thick steel angles, heel nailplates and 50 x 2 mm overstrap.

Legend: T5 - Y- 16 - HT





Maximum Load (kN)	Failure Mode
110.2	Heel joint distorting, nailplate withdrew from top chord
114.5	Apex plates and heel plates withdrawing as members move relative to each other
116.6	Apex plates and heel plates withdrawing as members move relative to each other. Apex nailplates split
128.6	Apex plates and heel plates withdrawing as members move relative to each other. Apex nailplates split
92.0	Truss top chords split
122.8	Heel nailplates withdrawing, top chords broke
100.0	Heel nailplates withdrawing, top chords broke
131.6	Apex nailplate split, heel nailplates withdrawing, top chords failed in longitudinal shear
119.2	Heel nailplates withdrawing, top chords broke
109.0	Heel nailplates withdrawing, top chords broke

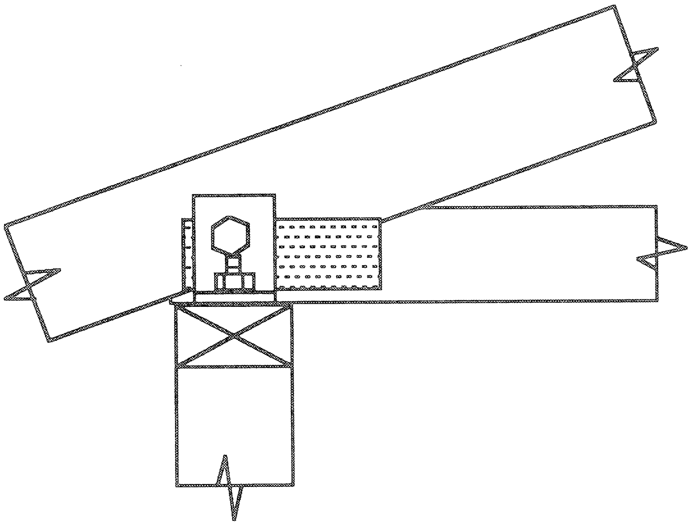
TEST RESULTS

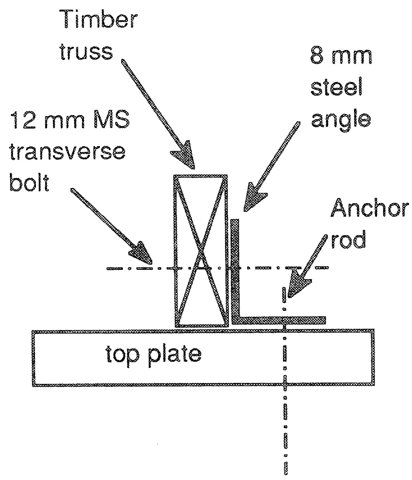
Pine trusses (JD4) Single truss designed for 10 kN uplift reactions.

Joint Type T1

Description: M12 mild steel cuphead bolt passing through a single 8 mm thick steel angle and heel nailplates.

Legend: T1 - Y - 12 - MS





Maximum Load (kN)	Failure Mode
26.5	Top chord and heel nailplates split through bolt hole
25.4	Top chord and heel nailplates split through bolt hole
20.0	Top chord and heel nailplates split through bolt hole
20.0	Top chord and heel nailplates split through bolt hole
21.2	Top chord and heel nailplates split through bolt hole
18.8	Top chord and heel nailplates split through bolt hole
21.8	Top chord split above nailplates and bolt hole
18.7	Top chord split above nailplates and bolt hole
17.8	Top chord and heel nailplates split through bolt hole
20.7	Top chord and heel nailplates split through bolt hole

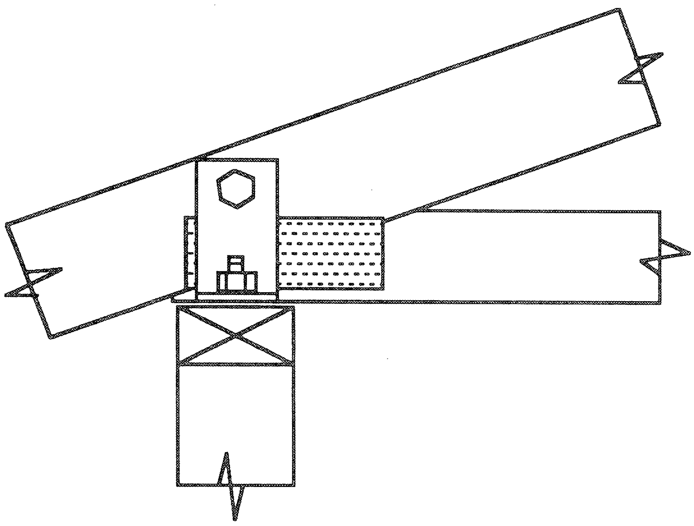
TEST RESULTS

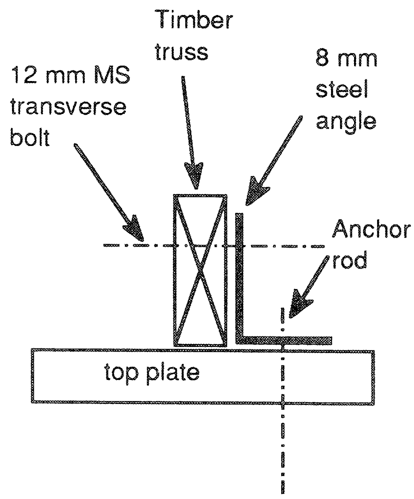
Pine trusses (JD4) Single truss designed for 10 kN uplift reactions.

Joint Type T1

Description: M12 mild steel cuphead bolt passing through a single 8 mm thick steel angle and above heel nailplates.

Legend: T1 - N - 12 - MS





Maximum Load (kN)	Failure Mode
30.0	Top chord split through bolt hole, bolt broke
27.2	Top chord cracking at bolt hole
24.3	Top chord cracking at bolt hole, bolt broke
23.6	Top chord cracking at bolt hole, then split
25.3	Top chord cracking at bolt hole, bolt broke
24.8	Top chord cracking at bolt hole, bolt deeply embedded
27.8	Top chord cracking at bolt hole, bolt broke
25.8	Top chord cracking at bolt hole, bolt deeply embedded
27.3	Top chord cracking at bolt hole, bolt broke
23.0	Top chord cracking at bolt hole, bolt deeply embedded

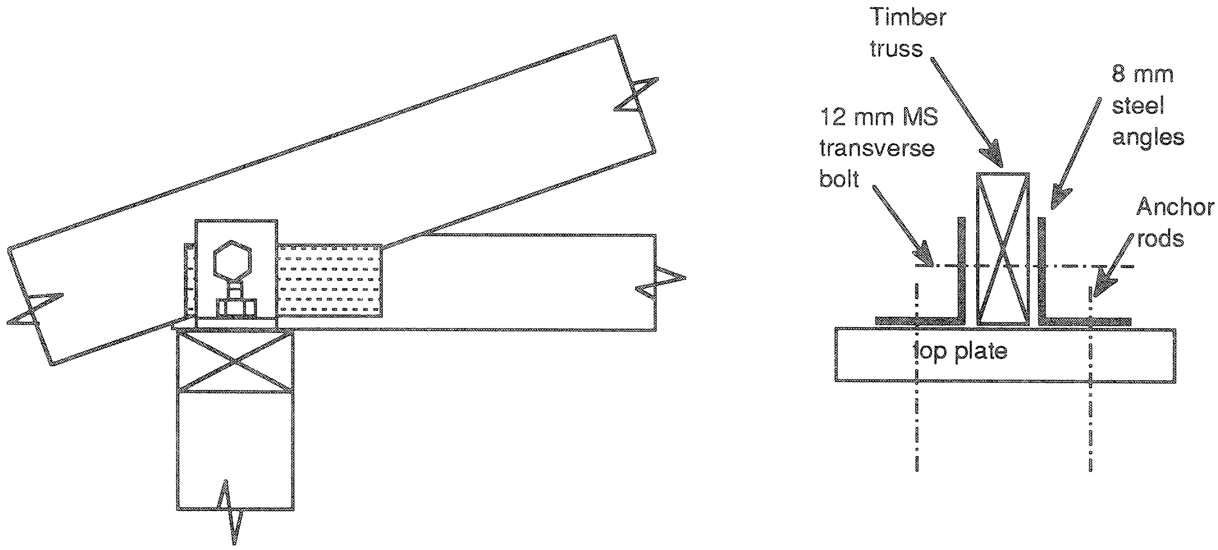
TEST RESULTS

Pine trusses (JD4)	Single truss designed for 10 kN uplift reactions.

Joint Type T2

Description:	M12 mild steel cuphead bolt passing through two 8 mm thick steel angles and heel nailplates.

Legend:	T2 - Y- 12 - MS



Maximum Load (kN)	Failure Mode
21.9	Top chord and heel nailplates split through bolt hole
22.0	Top chord and heel nailplates split through bolt hole
23.6	Top chord and heel nailplates split through bolt hole
28.3	Top chord and heel nailplates split through bolt hole
21.8	Top chord and heel nailplates split through bolt hole
23.2	Top chord and heel nailplates split through bolt hole
22.5	Split diagonally through nailplates
26.4	Split diagonally through nailplates
27.9	Top chord and heel nailplates split through bolt hole
20.9	Top chord and heel nailplates split through bolt hole

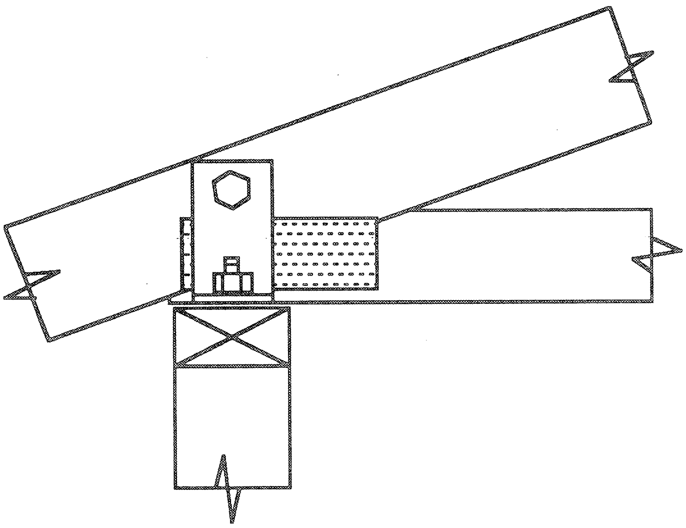
TEST RESULTS

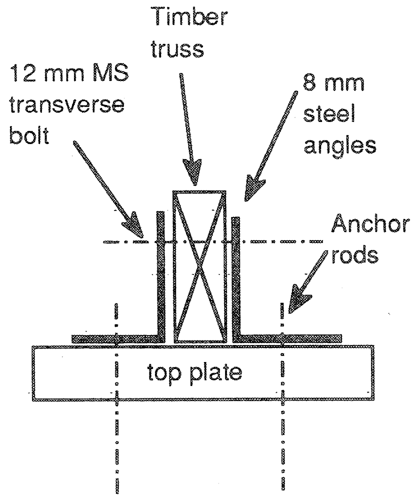
Pine trusses (JD4) Single truss designed for 10 kN uplift reactions.

Joint Type T2

Description: M12 mild steel cuphead bolt passing through two 8 mm thick steel angles and above heel nailplates.

Legend: T2 - N- 12 - MS





Maximum Load (kN)	Failure Mode
25.0	Top chord split through bolt hole
25.2	Timber split along top and bottom chords
31.0	Timber split along top and bottom chords
28.2	Top chord split through bolt hole
23.4	Top chord split through bolt hole
30.1	Timber split along top and bottom chords
32.7	Top chord split through bolt hole
29.0	Top chord sheared
29.4	Top chord sheared
31.3	Top chord split through bolt hole

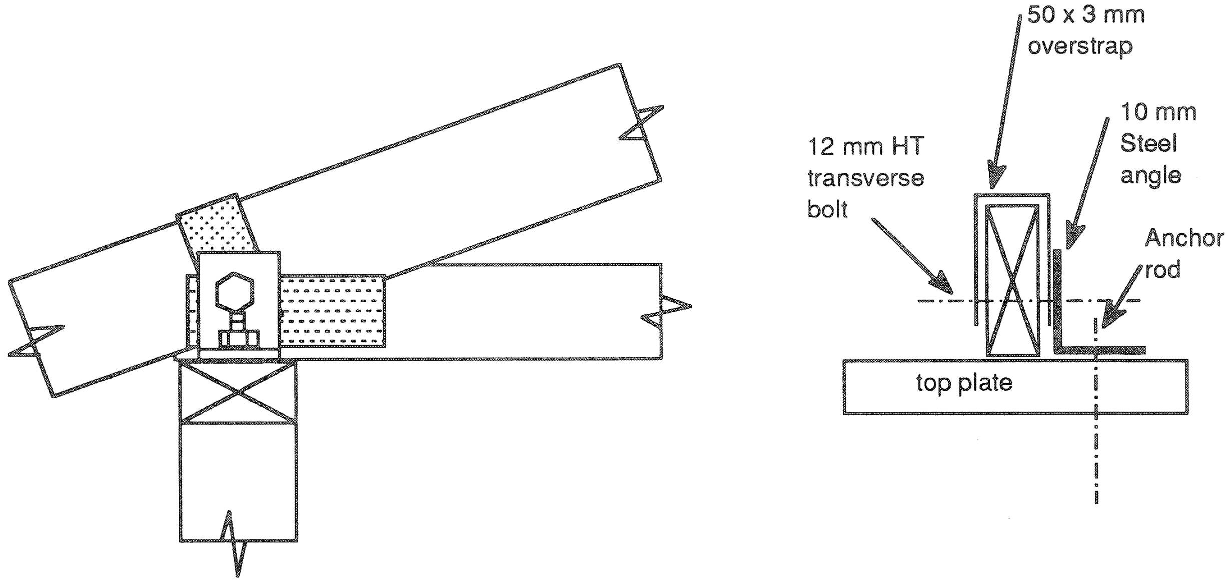
TEST RESULTS

Pine trusses (JD4) Single truss designed for 10 kN uplift reactions.

Joint Type T3

Description: M12 high strength steel bolt passing through a single 10 mm thick steel angle, heel nailplates and 50 x 3 mm overstrap.

Legend: T3 - Y- 12 - HS



Maximum Load (kN)	Failure Mode
34.2	Apex nailplate split
40.0	Apex nailplate split top chord broke
31.7	Top chord broke
34.4	Top chord broke
31.8	Apex nailplate split top chord broke through knot
47.7	Top and bottom chords shattered
34.2	Apex nailplate split top chord broke
31.3	Apex nailplate split
37.2	Apex nailplate split

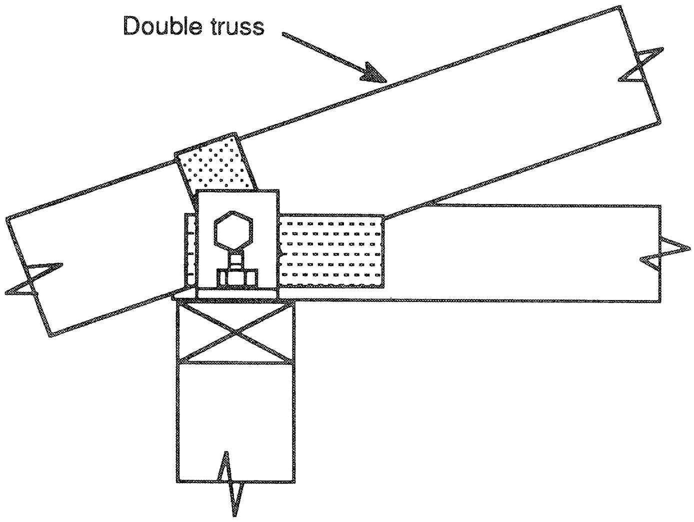
TEST RESULTS

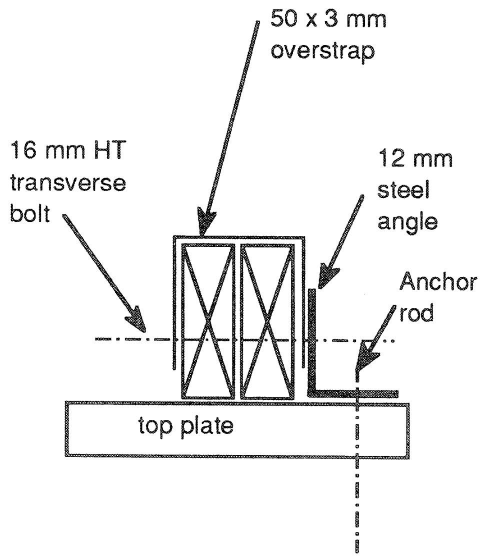
Pine trusses (JD4) Double truss designed for 30 kN uplift reactions.

Joint Type T4

Description: M16 high strength steel bolt passing through a single 12 mm thick steel angle, heel nailplates and 50 x 3 mm overstrap.

Legend: T4 - Y- 16 - HT





Maximum Load (kN)	Failure Mode
52.5	Top chords split through bolt hole, to end of one truss
52.3	Top chords split through bolt hole, to end of one truss
49.2	Both top chords split
55.8	Both trusses split through bolt holes to end
51.6	Both trusses split through bolt holes to end
54.0	Both trusses split through bolt holes to end
56.2	Both trusses split through bolt holes to end
59.0	Both trusses split through bolt holes to end
60.0	Both trusses split through bolt holes to end
56.0	Both trusses split through bolt holes to end

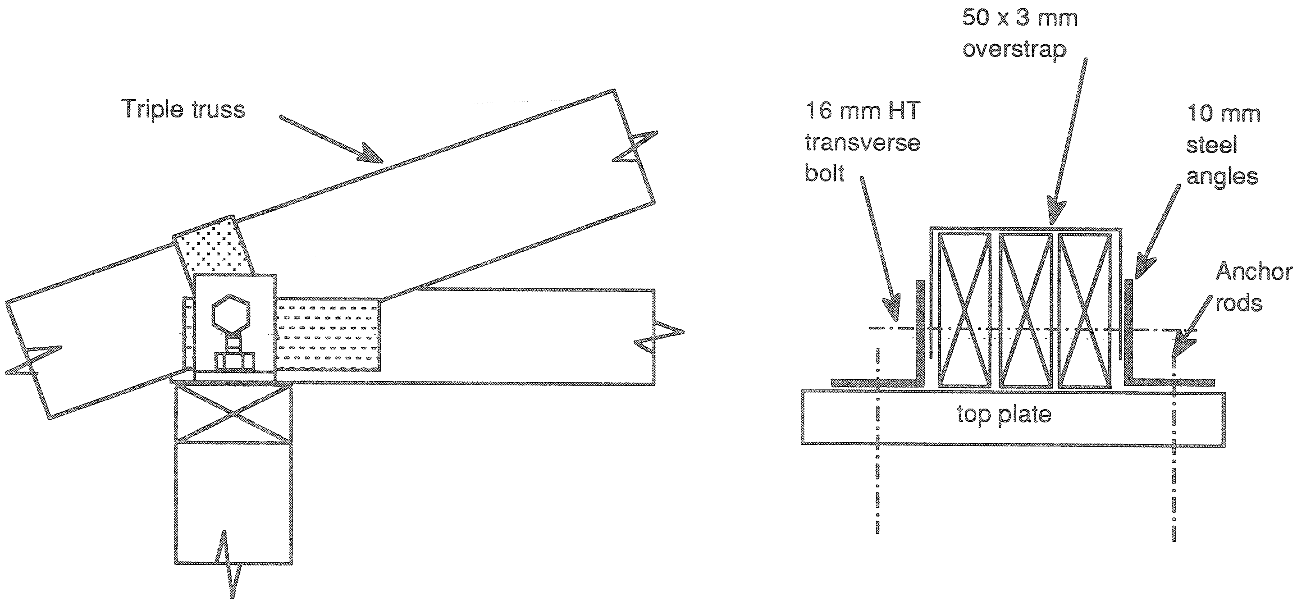
TEST RESULTS

Pine trusses (JD4) Triple truss designed for 40 kN uplift reactions.

Joint Type T5

Description: M16 high strength steel bolt passing through two 10 mm thick steel angles, heel nailplates and 50 x 3 mm overstrap.

Legend: T5 - Y- 16 - HT



Maximum Load (kN)	Failure Mode
85.0	Top chord broke
82.2	Two trusses split through bolt hole, other broke at knot
83.6	Two trusses split through bolt hole, other broke at knot
102.0	Top chords split through bolt holes
66.4	One top chord broke at a knot others split from apex
81.4	One top chord broke at a knot others split from apex
93.0	All top chords broke near apex
85.4	One top chord split through bolt hole, others broke
82.4	One top chord split through bolt hole, others broke
83.0	All top chords split through bolt holes