TRUSS PLATE REINFORCED BOLTED CONNECTIONS IN PARALLEL STRAND LUMBER

by

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B.A.Sc., University of British Columbia, 1997

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

In

FACULTY OF GRADUATE STUDIES

Department of Wood Science

We accept this thesis as conforming

to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

April, 1999

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ABSTRACT

Research has shown that strength and ductility of bolted connections in sawn timber can be improved by reinforcing the connection area with such materials as fibreglass, plywood, densified veneer wood or truss plates. This thesis investigates the behaviour of truss plate reinforced bolted connections in parallel strand lumber (PSL). Static tension tests were performed on single bolt connections and multiple bolt connections in 38x140 mm specimens, 89x140 mm specimens, and 133x191 mm specimens. All connections were designed according to CAN/CSA 086.1-94. Each test configuration had a set of 10 unreinforced connection specimens and a set of 10 reinforced connection specimens.

Connections in the 38x140 mm test specimens consisted of one 12.7 mm diameter bolt and ten 12.7 mm bolts (two rows of 5 bolts). With the single bolt connections, average ultimate strength improvements of 36% and ductility improvements in excess of 278% were realized by reinforcing the connections. Reinforcement of the ten bolt connections did not effect the average ultimate strength, while ductility improved by 250%.

Connections in the 89x140 mm test specimens consisted of one 15.9 mm diameter bolt and ten 15.9 mm bolts (two rows of 5 bolts). With the single bolt connections, average ultimate strength improvements of 12% and ductility improvements of 104% were realized by reinforcing the connections. For the ten bolt connections, reinforcement did not significantly change the average ultimate strength, while ductility improved about 5%.

Connections in the 133x191 mm test specimens consisted of one 22.2 mm diameter bolt and four 22.2 mm bolts (two rows of 2 bolts). With the single bolt connections, average ultimate strength improvements of 3% and ductility improvements greater than 143% were realized by reinforcing the connections. For the four bolt connections, the average ultimate strength was increased by 6%, while ductility improved by about 35%.

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ACKNOWLEDGEMENTS

I would like to thank my supervisors Dr. Frank Lam¹ and Dr. Helmut Prion² for their guidance and advice during the course of this study. I would also like to express my gratitude to Bruce Craig of Trus-Joist MacMillan (Vancouver Plant) for providing the Parallam[®] material. As well, I would like to thank Forest Renewal BC, which provided funding for this project through grant HQ 96327-RE. Finally, a special thanks to my family for their support during my educational studies at the University of British Columbia.

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1 INTRODUCTION

Connections have always been the critical elements in heavy timber construction in old and modern cultures. In Europe and the Far East carpentry skills on the fabrication of connections have been perfected over many centuries and were passed on from one generation to the next. Much of this skill has been lost due to high labour costs and the emergence of more cost-effective and simpler construction methods, such as platform construction. When architects and engineers took over the function of prescribing connection methods, a severe lack of design rules resulted in a regression of the art of timber construction, accompanied by a shift towards a more technocratic approach. In North America a large amount of research was done on common mechanical timber fasteners, which led to the design information found in applicable model codes, such as the Canadian Code of Timber Design (CAN/CSA 086.1-94). Typically, only the more commonly used fasteners, such as bolts and nails, are included in design codes and design values are applicable to sawn or glued-laminated products only.

Another shift in the last few decades has been the development of advanced engineered wood products such as parallel strand lumber (PSL-Parallam[®]), laminated veneer lumber, and laminated strand lumber. With the development of these advanced engineered wood products, heavy timber construction is making a comeback as a building alternative to steel and concrete for commercial and non residential buildings. At this stage, however, very little information is available on how to design efficient connections utilizing the enhanced properties of these products. In an attempt to increase the connection choices available to engineers, there has been much research to develop alternative connections not found in design codes. A brief literature review of some of these innovative connection alternatives can be found in Section 2.1.

With regards to seismic design, heavy timber structures have to dissipate the input energy from earthquakes mainly through its connections. With heavy timber connections usually failing in a brittle manner due to excessive perpendicular to grain stresses causing

splitting, there is a need to develop not only more efficient connections, but also more ductile connections. Efficient and ductile connections almost go hand in hand, as a ductile connection allows an even distribution of forces throughout a multiple fastener connection, thereby creating a more efficient connection.

A promising solution to address the ductility of sawn timber connections has been the development of reinforced connections. The majority of research on reinforced connections has been on bolted connections with the surface application of fibreglass, truss plates, or glued-on plywood in the connection region. This type of reinforcement has been successful in improving the ultimate strength and ductility of these connections. A detailed review of bolted connections is discussed in Section 2.2, including past research on innovative ways to reinforce bolted connections.

With this information on reinforced connections, and a desire to further understand the behaviour of mechanical fastener connections in structural composite lumber, a study was initiated at UBC to determine the influence that reinforcement would have on bolted connections in Parallam®. Preliminary work with single bolt connections in 38x89 mm PSL, tested in static tension, showed that the ultimate strength and ductility of single bolted connections in PSL can be improved significantly by reinforcing the connection area with fibreglass, plywood or truss plates (Schubert, 1997). Surprisingly, similar improvements were obtained for all three types of reinforcement. From this preliminary study, it was decided to further investigate truss plate reinforced connections, and expand the scope to include varying sizes of PSL and multiple bolt connections. Truss plate reinforcement was chosen over glued-on plywood and fibreglass as it seemed to be the most practical reinforcement method, using components that can be purchased from local building suppliers and installed easily.

The aim of this study was to investigate the efficiency of truss plate reinforcing methods for bolted connections in large PSL cross sections. It was decided to test single and multiple bolt connections with the following combinations of bolt and PSL sizes: 12.7 mm bolts in 38x140 mm PSL, 15.9 mm bolts in 89x140 mm PSL, and 22.2 mm bolts in 133x191 mm PSL. Multiple bolt connections for the 38x140 mm and 89x140 mm specimens consist of 2 rows of 5 bolts. Multiple bolt connections for the 133x191 mm specimens consist of 2 rows of 2 bolts (Note: the number of bolts was dictated by the load capacity of the testing equipment).

2 LITERATURE REVIEW

2.1 INNOVATIVE CONNECTIONS

The Wood Design Manual (Canadian Wood Council, 1995) is the main reference manual for timber design in Canada. Chapter 7 of the Wood Design Manual contains the necessary information to design the following connections utilizing the more common mechanical fasteners (in accordance with CAN/CSA-086.1-94):

- nails and spikes
- bolts
- glulam rivets
- drift pins
- lag screws
- shear plates and split rings
- truss plates
- joist hangers
- framing anchors

Non-traditional fastener methods have been available as proprietary products and design values are typically provided by the manufacturer. Since connections play such a crucial role in successfully constructing a timber building, many innovative methods have been devised in the past. The following sections present some innovative ways of connecting timber members together that are not included in the design code.

2.1.1 Nail/Glue Joints

The nail/glue joint has been used in light structural framing systems for many years to reduce floor squeaking and nail popping. As well, a greater stiffness and strength of the flooring system is realized, enhancing the efficient use of construction material. The increases in stiffness and strength are dependent on nail diameter and spacing, as well as the species of wood. Unfortunately most design codes give the designer no benefit for using the nail/glue joint (Pellicane, 1991).

For example, design information provided in CAN/CSA-086.1-94 states that for nail joints without glue, the factored lateral resistance of the nailed or spiked connections is calculated with the following formula:

$$N_r = \phi * n_u * n_F * n_{se} * K' * J'$$

where:

N_r	=factored lateral resistance
$\phi * n_u$	=basic factored lateral resistance(tabulated value)
n_F	=number of nails or spikes
n_{se}	=the effective number of shear planes
K', J'	=modification factors depending on actual conditions of use

The factored lateral resistance is based on the European yield model (Johansen, 1949 and Larsen, 1977), which assumes the ultimate joint strength depends on the yielding moment of the nail and the embedding strength of the wood only. For a two-member connection, four different failure modes are identified:

- wood crushing in both members before the nail yields (stiff nail, thin members)
- wood failure in only the thinner member
- a combination of wood failure in both members with nail yielding in the thicker member
- a failure that is governed by nail yielding and wood failure in both members

Brittle wood failures are avoided by adhering to spacing requirements, which are based on work done by Ramos (1960) and Lau and Tardiff (1987). Tabulated spacing requirements for nails parallel to grain were checked to ensure that they were at least twice the maximum crack lengths observed by Lau and Tardiff.

In addition to the nail resistance, much can be gained by including glue in the timber joints. Because of different load-slip characteristics of nailed and glued joints, resistance values are not additive, however, and care must be taken to properly determine the behaviour of this

dual connection method. Early work by Johannes (1961) showed that the following factors influence glue line strength:

- type of glue: must have good gap-filling properties, must be suited to curing environment and wood moisture content,
- type of nails: need high withdrawal resistance,
- wood species,
- gluing pressures and surface conditions: planed surfaces are desired,
- grain direction: parallel grain bonding yields highest strengths, look out for cupping due to moisture content variations in the wood causing shrinking and swelling,
- moisture content,
- climatic conditions,
- size of specimen: when the gluing area is large, which is found in full size specimens, there tend to be serious stress concentrations in the joint that cause collapse before the average shear stress has reached a value which compares to the shear strength as observed on shear block specimens of the standard type,
- quality of workmanship.

Hoyle (1988), presents a design method for nailed and elastomeric glued joints; the joint is superior in strength to nailed joints and has similar slip characteristics (as nailed joints) at allowable joint loading. His work shows that one and one half square inches (968 mm²) of glue line is roughly equivalent to one 8d common nail, and slip characteristics of the nails and the adhesive are compatible. Elastomeric compounds behave much like rubber. This joint is different from a synthetic resin glue/nail joint, for in synthetic resin glue/nail joints the nails do not carry any load until the joint slip has become so great that the glue line ruptures.

2.1.2 Resin Injection

By increasing the friction between a fastener and the timber on which it bears, the load bearing properties of the fastener are improved. Resin injection produces an infinite coefficient of friction between the bolt and timber. As well, resin injection eliminates slack in the connection, thus load is taken up without any initial slip (Rodd et al, 1991).

2.1.3 Glued-in Ties

Turkowskij (1991) has done some work on glued-in steel ties. Glued-in ties parallel to grain have not found much use in practice, although they have higher strength and rigidity indices when compared to bolts, screws, and dowels. The disadvantage is that the layers where the bars are glued in, experience high stresses. There also remains a high probability that cross-cut end cracks will coincide with the glued-in bar; hence, increasing the possibility of failure. Gluing the bars in at angles removes some of the aforementioned deficiencies, but manufacturing problems arise when fabricating this connection. Figure 1 shows an example of glued-in steel rods parallel to grain.

Figure 1: Glued-in Steel Rods (from Larsen, 1994)

Madsen (1998) developed glued-in steel rod connections using the following design criteria:

- The connection should be able to transmit 100% of the capacity of the glulam cross section.
- The connection should be stiff enough to meet the requirements for inter-storey drift during wind and earthquakes.
- The connection should be able to withstand reversal of the loads.
- The connection must fail in a ductile manner.
- The connection must be able to transmit the loads without relying on contact between the structural members.
- The glulam members should have a moisture content of less than 15%.
- An Epoxy adhesive designed specifically for this application shall be used. All gluing must be arranged in such a manner that it can be done indoors under controlled temperature conditions.
- All welding should be done prior to the epoxy gluing due to possible heat damage of the glue line.
- The use of bolted connections for field joints is preferred.

Results of his work showed that his glued-in steel tie connection could transmit 100% of the capacity of the glulam, but found problems with keeping deformations within tolerable limits. Figure 2 shows the general connection type that Madsen tested and Figure 3 illustrates some applications of the connection.

Figure 2: Glued-in Steel Rod Connection (Madsen, 1998)

Figure 3: Applications for Glued-in Steel Rod Connections (Madsen, 1998)

2.1.4 Tight Fitting Dowel Connections

Dowels are made of high-quality steel, with high ultimate strength and very narrow tolerances on the diameter. They are typically driven into undersized holes, which ensures both a stiff joint and a uniform load distribution in multiple dowel connections because initial slip is avoided. Doweled joints also tend to be more aesthetically pleasing than their bolted counterparts, which contain nuts and washers. Fabrication of dowel joints consists of temporarily assembling the timber members, then drilling the holes in all members in one operation; the dowels are installed immediately after or later (Larsen, 1994). Figure 4 shows an example of a doweled joint.

Figure 4: Moment Connection Made with Dowels (from Larsen, 1994)

Manufacturing of this joint is labour intensive needing highly skilled workers. As well, tolerances between the timber members and the dowels are influenced by moisture fluctuations within the joint. Steel plates are often used in combination with dowel fasteners in which case fabrication is more complicated because the holes have to be drilled separately in the wood and steel. As well, special drilling equipment is necessary to ensure that the holes are perpendicular to the surfaces of the timber and steel plate.

2.1.5 Glued Joints

Glued joints are increasingly being used in steel and concrete designs, but are rarely applied for load bearing timber structures. This may be due to the lack of design rules for use in timber structures. Glos and Horstmann (1991) offer the following design formula:

$$P_c(\tau) = v_\tau * \tau_c * L_{Kc} * k_d * k_\alpha * k_B * k_K$$

where

$P_c(\tau)$ =characteristic ultimate shear load
 v_τ =factor covering joint design
 $\tau_c L_{Kc}$ =factor covering glue area
 k_d =factor covering tension chord thickness
 k_α =factor covering gluing angle
 k_B =factor covering load transmission mode
 k_K =factor covering shear modulus of the glue used

A similar formula is given for the characteristic ultimate tension load perpendicular to grain. The essential parameters that determine the load bearing behaviour of glued wooden joints are the following: design of joint, geometry of joint, geometry of glued area, load transmission mode. If designers had formulae or procedures outlined in the design manual on how to efficiently design a glued connection, it would give engineers another connection design option.

2.1.6 Prestressed Joints

An increased load capacity of bolted connections can be achieved by applying cross-grain prestressing and therefore preventing splitting. Shorter end distances can be used as a result; however, the benefits of prestressing a timber joint will be realized only if two requirements are met (Quenneville et al, 1991):

- the prestress must increase the capacity of the connector enough to compensate for the effects of the less than adequate end distance
- the prestress level must remain above a required value throughout the life of the structure.

The second point is the main reason why one doesn't see many prestressed joints. The maintenance involved with consistently retightening the prestressing in the connection (due to the wood relaxing) is usually not economically feasible.

2.1.7 Seismic Considerations

While working on a project for predicting the hysteresis response of nailed joints to dynamic loads, Ni and Chui (1994) confirmed the common knowledge that connections with nails or small diameter bolts tended to fail in a ductile manner. As well, lumber members in their test groups were matched based on density, and they found that nail breakage is a failure mode in higher density species under cyclic loading. Work done by Cruz et al (1991) reinforces the fact that designers have to realize that the only ductility that may be considered from a timber structure is that obtained from suitable design and production of structural joints which are able to dissipate energy by inelastic deformations.

2.2 BOLTED TIMBER CONNECTIONS

General information on bolted connection design will be presented in section 2.2.1, while section 2.2.2 will examine past research on reinforced bolted connections .

2.2.1 General Bolted Connection Design

The following design information is taken from section 7.3 of the Wood Design Manual-1995. To prevent premature splitting and to ensure adequate ductility of timber connections utilizing mechanical fasteners, the design code specifies minimum end distance, edge distance, and spacing requirements for fastener arrangements. Figure 5 details the general spacing requirements.

Figure 5: Bolted Connection Geometry (from Wood Design Manual, 1997)

The lateral resistance of most fasteners depends on the angle of load to the grain.

When the load is parallel to the grain the angle is 0°, and when the load is perpendicular to grain the angle is 90°. Resistance with loading at an intermediate angle to grain is calculated using Hankinson's formula:

$$N_r = \frac{P_r * Q_r}{P_r * \sin^2 \theta + Q_r * \cos^2 \theta}$$

N_r = factored resistance at an angle θ to grain
 P_r = lateral strength for parallel to grain loading
 Q_r = lateral strength for perpendicular to grain loading
 θ = angle between grain direction and direction of load

Bolted connections must be designed so that the factored resistance of the connection is greater than or equal to the factored load. The factored lateral resistance is calculated as follows:

$$P_r = \phi * P_u * n_s * n_F * J_F$$

$$Q_r = \phi * Q_u * n_s * n_F * J_R$$

where:

ϕ = 0.7
 P_r = factored lateral strength resistance for parallel to grain loading
 P_u = $p_u(K_D K_{SF} K_T)$
 p_u = lateral strength resistance for parallel to grain loading (Clause 10.4.4.2), N
 Q_r = factored lateral strength resistance for perpendicular to grain loading
 Q_u = $q_u(K_D K_{SF} K_T)$
 q_u = lateral strength resistance for perpendicular to grain loading (Clause 10.4.4.2), N
 n_F = number of bolts
 n_s = number of shear planes
 K 's = modification factors depending on actual conditions of use
 J_F = $J_G J_L J_R$ – connection configuration adjustment factor
 J_R = group reduction factor for row effect
 J_L = reduction factor when reduced end distance is used
 J_G = group reduction factor for uneven load sharing of bolts in a row (2-12 bolts)

The CSA commentary found in the Wood Design Manual gives the background of the code philosophy for bolted connections. Work done by Yasumura (1987) allowed for new group factors found in the code, and work done by Masse (1988) provided guidance for end distance factors. Originally lateral resistance values for a single bolt (P_u and Q_u) were based on work done by Trayer (1932). CAN/CSA-086.1-94 used work done by Johansen (1949) and Larsen (1973) to refine P_u and Q_u values. Larsen's prediction equations assumed the following failure modes:

- failure by full bearing in one or more wood members (for short bolts)
- bending yield of the bolt (for longer bolts) accompanied by local wood crushing at the member interfaces, or by a combination of both

P_u and Q_u values in CAN/CSA-086.1-94 are based on modified forms of Larsen's equations by Whale (1987). These assume ductile behaviour and are based on the yield strength of the fasteners and the embedding strength of the wood. It is evident that no benefit is gained in the code if designers modify bolted connections in some manner, with the exception of using densified wood with higher embedding strength. The following section describes methods and benefits of reinforced bolted connections.

2.2.2 Reinforced Bolted Connections

When most bolted timber connections fail, they fracture in a brittle manner, giving little warning of the impending failure. Brittle failure of the connection is usually due to shear failure or the ultimate tension strength of the timber in the perpendicular to wood grain direction being exceeded by the imposed loading conditions (see Figure 6).

Figure 6: Brittle Failure of Bolted Connection), a) splitting; b) shear block (modified picture from Soltis, 1996)

Brittle failure of connections in any structure is undesirable, as sudden collapse of the structure may occur, giving occupants no time to escape from the building. One type of reinforced connection consists of a reinforcing layer bonded to the connection area of a timber member. The reinforcing layer improves the strength of the timber member in the perpendicular to grain direction and in shear. It also distributes concentrated loads over a larger area. Figure 7 illustrates this concept.

Figure 7: Reinforced Bolted Connection (modified picture from Soltis, 1996)

If appropriate levels of reinforcement is applied, the failure mode in the connection can change from wood failure (non-reinforced case) to yielding of the fasteners. Therefore, the strength of the reinforced connection is then determined by the strength and quality of the mechanical fasteners (Chen et al, 1994). Reinforced bolted connections fail in a more ductile fashion under cyclic loading due to plastic deformations of the bolt, and not splitting of the timber. Figure 8 illustrates the concept of a reinforced connection using densified veneer wood (DVW) to enhance the embedment strength.

Figure 8: Reinforced Bolted Connection (from Rodd and Leijten, 1996)

Leijten (1991) mentions that using glued-on steel as a reinforcing material leads to problems in the connection. Stress concentrations develop when the elastic moduli of the reinforcement and wood differ greatly, resulting in glue-line cracks between the timber member and the reinforcement. The glue-line transfers the shear stresses from the reinforcement to the timber members. A breakdown in the glue-line results in a breakdown of the connection. His solution to this problem was to use densified veneer wood (DVW) as reinforcement. In his research, the ductility of the connection depended on the ratio of reinforcement thickness to fastener dimensions. When the reinforcement withstood the high concentrated loads transferred by the fastener, very ductile behaviour was observed. Because DVW and steel

dowel type fasteners have low variability of their mechanical properties, the variability in joint strength is reduced when compared to non-reinforced type joints. The tighter distribution of connection strengths could lead to an increase in allowable connection strengths, as the lower fifth percentile, which is the basis for design formulae, would be closer to the mean value.

Rogers et al (1994) performed preliminary experiments to investigate nailplate reinforced bolted connections. They found that nailplate reinforcement increased the strength of bolted joints by up to 60%. They observed that the size of the nail plate has a major effect on joint strength.

Chen et al (1994) conducted experiments using fibreglass to reinforce timber joints. A joint in a Warren truss girder was tested because of complex stress states developed in the connection. The reinforcement in the connections that they tested allowed the mechanical fasteners to yield plastically. Plastic deformation of the mechanical fasteners meant the failure of the joint was no longer governed by the strength of the timber, and ductile behaviour of the connection was observed under ultimate loads.

Larsen and Enquist (1996), experimented with glass fibre reinforced dowel-type connections. Test specimens had their connection area reinforced with glass fibres glued on with polyester glue. Specimens were loaded parallel and perpendicular to the wood grain. For dowel type connections they found marginal increases of failure load (up to 10%). They did find that connection behaviour becomes more ductile and spacing and end distances could be reduced considerably. When they tested glass fibre reinforced nail connections, they found that the failure loads increased by more than 50%, even though spacing requirements outlined in Eurocode 5 were not met.

Leijten (1996) took his concept of densified reinforced joints one step further, moving from dowel-type fasteners to expanded tube type fasteners. The expanded tube fastener works in the following manner. Oversized holes are predrilled in the timber members to be connected. A hollow tube (approx. 2 mm smaller in diameter) is inserted into the predrilled hole. The tube is then expanded by inserting a central rod through the tube and attaching two movable flaring

dies on each end of the rod. A hydraulic jack squeezes the flaring dies together, thereby expanding the hollow tube and effectively riveting the tube ends over a washer. Figure 9 illustrates the concept.

Figure 9: DVW Reinforced Expanded Tube Joint (from Leijten, 1996)

The expanded tube prestresses the timber joint and removes any slack that is often encountered when assembling joints using conventional dowel type fasteners. Stiffness of the joint is improved as minimal clearance between the expanded tube and timber members eliminates any initial slip upon loading. Leijten suggests the following practical benefits:

- the oversized holes makes the assemblage of large parts on site and transport much easier,
- the drilling precision is not so restrictive as traditionally required and can be done on site if necessary,
- the number of holes required is much less than with traditional dowels and therefore does not take so much effort,
- the adhesives used to glue DVW to timber are the familiar structural types well known in the glued laminated industry.

Soltis et al (1998) conducted a study examining the use of fibreglass and epoxy resin as a means of reinforcing wood at the bolted connection. He tested single bolt connections in tension parallel and in tension perpendicular to grain. One, two, and three layers of fibreglass reinforcement were tested. Results showed that connection strength and ductility increased as the number of reinforcing layers was increased, with the biggest improvement occurring from zero layers of reinforcement to one layer of reinforcement. This improvement was observed in both parallel and perpendicular to grain load cases. The average ultimate strength for three layered reinforced connections was 33% higher than non-reinforced connections in parallel to grain loading, and more than double for perpendicular to grain loading.

Reinforcement does not always have to be externally applied. A small unpublished study at UBC showed that threaded rods inserted perpendicular to grain within the connection region can significantly improve strength and ductility of bolted connections. The connection design and load displacement behaviour of unreinforced connections from this study is shown in Figure 10. The behaviour of the same connection with threaded rod reinforcement is shown in Figure 11.

Figure 10: Unreinforced Bolted Connection Tests in Glulam Members

Figure 11: Threaded Rod Reinforced Connection Tests in Glulam Members

After reviewing the literature, it was decided to further investigate truss plate reinforced bolted connections because truss plates are readily available at local suppliers, and can be installed easily. The following section outlines the materials and methods used in this experiment.

3 MATERIALS AND METHODS

The methodology of determining the performance of truss plate reinforced bolted timber connections under static tension loading is described in this chapter. As mentioned in the introductory chapter, it is the aim to prevent brittle failure modes, commonly observed in traditional unreinforced bolted connections. Not only is the deformation capacity affected, but the strength can potentially be increased; thus the amount of load distribution can improve to achieve equal fastener contribution. Current timber codes control premature brittle failures by specifying minimum bolt spacing requirements, as well as minimum end and edge distances for bolt placement. It is the goal of this study to show that by reinforcing the timber connection area in the perpendicular to grain direction with truss plates, the capacity and ductility of bolted connections can be increased. The material chosen for the study was Parallam[®], as it is a high end composite structural material that is being used in high stress applications. The economics thus warrants the additional expenses to enhance connection behaviour, while keeping the member sizes at a minimum. Section 3.1.1 discusses the history of Parallam[®], including how the composite wood material is made and its material characteristics. Section 3.1.2 describes the truss plates used in this experiment, while section 3.1.3 describes the steel side plates and bolts.

3.1 MATERIALS

3.1.1 Parallam[®]

A paper entitled "Parallam-The History and Future of a New Structural Wood Composite" by Churchland (1984) gives a good overview of the Parallam[®] PSL product. It is made from strands of veneer, usually Douglas-Fir, with dimensions as follows: width = 12.0 mm, thickness = 2.5-3.0 mm, and length = 600-2500 mm. Phenol formaldehyde adhesive is used to bond the

strands together, which is similar to the resin used in exterior grade plywood. The assembly of Parallam[®] is summarized in the following steps:

- Logs are turned on a lathe to produce veneer sheets
- When available, reject sheets of veneer from plywood production plants are used
- Veneer sheets are clipped into strands with dimensions previously mentioned
- Strands are dried in a continuous feed kiln
- Defective strands and debris are removed
- Resin is applied to all surfaces of the strands
- The strands are randomly distributed into a trough on a continuous feed conveyor belt forming a continuous billet, where each strand is oriented approximately parallel to the product axis
- The billet travels into a belt press where the volume of the billet is reduced about 30% and microwave generated heat cures the adhesive

Finished billets are up to 280x406 mm in cross section and can be ripped down to smaller sizes.

The manufacturing process is illustrated in Figure 12.

Figure 12: Manufacturing of Parallam[®] (from Introduction to Wood Design, CWC, 1996)

Figure 13 shows a cross section of a Parallam[®] billet, and the orientation of the veneer strips.

Figure 13: Parallam[®] Cross Section (NRC, 1997)

Churchland (1984) lists the following four attributes of Parallam[®] as reasons why it is superior to other wood based structural products:

- “First, the product is very strong. The stranding process inherently weeds out knots, severe grain deviations and weak wood. Such weaknesses cause the strands to break up into short lengths. These are automatically discarded. The mat when formed from the remaining strands has evenly distributed small strand ends. Each of these ends, although theoretically a weakness, has little impact and the inherent strength of each individual strand is fully utilized.”
- “Second, the product is very consistent. Because a single 3.5 inches by 16 inches beam contains over a thousand strands in any given cross-section, the weakness of a single strand or the placement of a strand end does not significantly affect strength. In the testing of full size beams, the beam-to-beam variability in strength and stiffness is less than 10% (coefficient of variation). Solid wood varies from 25% to 50% (coefficient of variation).”
- “Third, the product is very durable. Because of the waterproof resin and the even microwave heating, the product behaves like solid wood in the presence of water. It is strong even when wet and has minimal swelling, effectively recovering both strength and original dimensions on re-drying. In a recent 5-year exterior exposure test conducted by the American Plywood Association, Parallam PSL retained 100 per cent of its original strength after conditioning back to initial moisture levels.”

- “Fourth, the product properties are controllable. Because the process steps are controllable, the final product can be engineered to ensure the advertised strength properties are met. The combination of strength, consistency, durability and controllability combine to give a range of usable strength properties unparalleled in the industry.”

3.1.2 Truss Plates

The truss plates used in this study were 0.80 mm thick and of the “Gang Nail” type with dimensions 92x148 mm and 92x200 mm. They are manufactured by LumberLok, a division of Mitek Canada, and are a general purpose type with no structural rating. Figure 14 shows a drawing of the nail plates.

Figure 14: Gang Nail Truss Plates

3.1.3 Side Plates and Fasteners

Steel side plates were made from 19 mm ASTM A36 mild steel flat bar. All bolts used were Grade 5 bolts with a specified manufacturers yield strength of 635 MPa. No yield coupons of steel side plates or bolts were tested since these were not expected to yield. This was confirmed in the tests. Nominal specified values were used in code calculations (CAN/CSA 086.1-94).

3.2 METHODS

3.2.1 Scope/Connection Design

Three different sizes of Parallam[®] specimens were used in this investigation; 38x140 mm, 89x140 mm, and 140x191mm. The 38x140 mm specimens were ripped from excess billets of 89x140 mm material. It was decided to use 12.7 mm (1/2") bolts for the 38 mm thick specimens, 15.9 mm (5/8") bolts for the 89 mm thick specimens, and 22.2 mm (7/8" mm) bolts for the 133 mm thick specimens. Table 1 gives the resulting L/D ratios (length of bolt to depth of specimen).

Table 1 : L/D Ratios

Member Thickness	Bolt Size	L/D Ratio
38 mm	12.7 mm (1/2")	3.0
89 mm	15.9 mm (5/8")	5.6
133 mm	22.2 mm (7/8")	6.0

Clause 10.4.3.1 of CAN/CSA 086.1-94 requires a minimum bolt spacing of 4 bolt diameters in the parallel to grain direction. Clause 10.4.3.2.1 states that for parallel to grain loading, the spacing between rows shall be not less than twice the bolt diameter. Edge distance requirements of the code state that for members loaded parallel to grain, the edge distance shall be at least 1-1/2 times the bolt diameter, or half the distance between rows of bolts, whichever is greater. In each multiple bolt connection tested, this criterion was met. In all cases, holes were drilled 0.8 mm (1/32") in excess of the bolt size to accommodate bolt insertion and replicate industry standards. Figure 15 shows the single bolt connection specimen layouts, while Figure 16 shows the multiple bolt configurations.

Figure 15: Single Bolt Connection Specimen Layout

Figure 16: Multiple Bolt Connection Specimen Layout

Clause 10.4.4.2 has the wood member embedding strength dependent on the mean relative density of the wood material used. A density value for Parallam was taken to be 628.1 kg/m³ with standard deviation of 17.32 kg/m³ and coefficient of variation of 2.8%, resulting in a mean relative density of 0.628 (Moss et al, 1998). As well, a short term load factor of 1.15 was used in the code calculations to reflect short term loading.

Table 2 summarizes the tensile resistance of each connection according to the design code (see Appendix I), including the net tensile resistance of the test specimens (detailed calculations can be found in Appendix II), as well as the ratio of factored connection capacity to factored net tensile strength.

Table 2: Code Connection Strengths and Member Tensile Resistance's(CAN/CSA 086.1-94)

Specimen	Connection	Connection Resistance	Tension Resistance (Net Section)	Connection Capacity/Net Tensile Strength
38x140 mm	One 12.7 mm bolt	10.7 kN	101.1 kN	10%
	Ten 12.7 mm bolts	39.9 kN	91.1 kN	44%
89x140 mm	One 15.9 mm bolt	30.3 kN	230.9 kN	13%
	Ten 15.9 mm bolts	154.4 kN	201.3 kN	77%
133x191 mm	One 22.2 mm bolt	61.6 kN	469.4 kN	13%
	Four 22.2 mm bolts	175.2 kN	407.6 kN	43%

It was initially intended to test multiple bolt connections that would give a connection capacity to member strength ratio around 60%. In the case of the 38 mm thick members it was felt that using ten 12.7 mm bolts with L/D ratios of 3.0 was more practical than using ten 16 mm bolts with L/D ratios of 2.3. This resulted in a lower connection capacity to tensile strength ratio. For the 133 mm thick specimens, it turned out that the testing apparatus could only fail a four bolt connection, thus a lower connection capacity to tensile strength ratio resulted. Table 3 summarizes the scope of the testing program.

Table 3: Number of Tests for each Parallam Member Size

Member Size	Unreinforced Members			Truss Plate Reinforced Members		
	38 x89mm	89 x140mm	133 x191mm	38 x89mm	89 x140mm	133 x191mm
Connection						
Single bolt	10	10	10	10	10	10
Multiple bolt	10 (10 bolts)	10 (10 bolts)	10 (4 bolts)	10 (10 bolts)	10 (10 bolts)	10 (4 bolts)

For the truss plate reinforced members, 92 x 148 mm and 92 x 200 mm truss plates were pressed on the connection region. The location of the truss plates can be seen in Figures 17 and 18.

Figure 17: Truss Plate Locations on Single Bolt Connections

Figure 18: Truss Plate Locations on Multiple Bolt Connections

The following section describes the procedure used to fabricate the truss plate reinforced connections.

3.2.2 Fabrication Procedure for Reinforced Connections

For each connection, the steel side plates were drilled first. The steel side plates were then used as templates to drill the wood specimens. The truss plate reinforcement was pressed on using the Baldwin universal testing machine in the structures laboratory at UBC. The truss plates were installed in accordance with CSA 086.1-94 section 10.8. For the reinforced connections, it was necessary to apply a layer of reinforcement on one side of the test member and then drill through the existing holes in the test member from the opposite side to guide the drill bit through the truss plate. This was necessary because truss plates have an uneven surface which throws the drill bit off when attempting to drill through them directly. Thus the following procedure was established as the most efficient.

- Drill wood specimen using steel side plate as the template
- Apply truss plate reinforcement in connection region on one side of wood member
- Use predrilled holes on the other face of the wood member as template to drill through truss plate reinforcement from the backside
- Apply truss plate reinforcement on remaining face and drill through reinforcement from other side

3.2.3 Test Apparatus

A modular frame was set up in the structures laboratory to allow for a hydraulic actuator to be suspended from the middle of the cross beam. The cross beam was adjustable in the vertical direction to accommodate different lengths of test specimens. Figure 19 shows a typical test setup.

Figure 19: Typical Test Setup

Two types of actuator/load-cell combinations were used in this experiment; a 222 kN (MTS-50,000 lb-Model 202.01) and a 445 kN (MTS-100,000 lb-Model 661.22). The 445 kN actuator/load-cell combination was necessary to test the larger connections. The load cells screwed onto the end of the hydraulic actuator. A coupling device was screwed into the bottom of the load cell. The steel side plate mounts were then pin connected to the coupling device.

Likewise a coupling device was fastened to the bottom of the floor of the structures lab. Steel side plate mounts were then pin connected to the floor coupling device. By having the steel side plate mounts pin connected to the actuator and the floor, the test specimen experienced axial loads only, with moments minimized within the connection region (due to eccentric loading). Figure 20 shows a close-up of the specimen attachment configuration.

Figure 20: Load Cell-to-Side Plate Mount Configuration

Three different side plate mounts were fabricated by the machine shop to accommodate the different member thicknesses. All tests in this study were of the <steel side plate-wood member-steel side plate> nature. All the steel side plates used were 19 mm thick. This was to ensure that no localized yielding occurred in the steel side plates, as each set of side plates had to be reused 20 times (consistency across all tests was needed). The steel side plates were shimmed to allow for an approximate gap of about 0.5-1.0 mm between the side plates and the wood members. This was done to ensure that friction did not play a significant role in the behaviour of the connection. A washer was placed on the bolt between the side plate and nut, with the nut being finger tightened. Bolts were tightened on the bottom end of the specimen to

ensure that no slip occurred in the bottom connection, thereby forcing failure to occur at the top connection. The loading rate was displacement controlled at 2.0 mm/min using a MTS 458.10 MircoConsole and an Exact (Model 340) Materials Testing Generator. This produced failure of the connection in about 5 minutes. Relative displacements between the steel side plates and the timber member were monitored by LVDT's located on each side of the test specimen (see Figure 21 for a typical setup).

Figure 21: LVDT Placement

Load and displacement data were sampled and recorded at 5 Hertz by a 386 personal computer running LabTech Notebook version 4.0.

4 RESULTS

The following sections summarize the experimental results. Each section presents the statistics of each group of tests including average ultimate strength, standard deviation, and the coefficient of variation (calculations can be found in Appendix III). As well, each section will show the load deformation plots of the experimental results, including average curves. Because loading was displacement controlled and sampling rate constant between tests, average curves could be obtained by zeroing all ten individual load deformation curves, and averaging forces and displacements at each sampling point. The code characteristic strength value plotted on the average curve graphs was calculated by dividing the code predicted connection strength value by the performance factor 0.7. Pictures of failed specimens for each connection configuration can be found in Appendix IV.

Section 4.1 shows the results of the experiments that involved 38x140 mm sized Parallam® members with unreinforced and reinforced 12.7 mm single bolted connections and unreinforced and reinforced bolted connections using ten 12.7 mm bolts. Section 4.2 shows the results of the experiments on 89x140 mm sized Parallam® members with unreinforced and reinforced single bolted connections and ten bolt connections using 15.9 mm bolts. Finally, section 4.3 covers the experiment results on 133x191 mm sized Parallam® members with unreinforced and reinforced single bolted connections and four bolt connections using 22.2 mm bolts.

Failure modes for the connections can usually be described by one of the following four failure mechanisms; row shear-out, group tear-out, bolt bearing, or row splitting. Figure 22 illustrates the failure modes.

Figure 22: Connection Failure Modes (Quenneville, 1998)

4.1 38x140 mm TEST SPECIMENS

4.1.1 Single Bolt Connection

Ten specimens measuring 38x140 mm containing one 12.7 mm bolt connection were tested in static tension. Figure 23 shows the plot of all ten load deformation curves. The failure mode in each case was by wood splitting. Ten more tests were conducted using truss plate reinforcement in the connection region of the 12.7 mm bolt. Figure 24 shows the plots of the 10 load deformation curves. Failure of the reinforced connections was by wood bearing. Figure 25 is a plot of the average curves. Table 4 shows the statistics for both test series.

Table 4: Statistics-38x140 mm-One12.7 mm Bolt

Specimens	Average Ultimate strength	Standard Deviation	Coefficient of Variation (COV)
Unreinforced 38x140mm-One 12.7 mm Bolt	24.89 kN	3.79 kN	15.22%
Reinforced 38x140mm-One 12.7 mm Bolt	33.97 kN	1.86 kN	5.48%

Comparing the unreinforced and reinforced average curves in Figure 25 and the statistics in Table 4, the improvements achieved by reinforcing the connection are quite evident. As a distinct change in failure mode from wood splitting to wood bearing is observed (with the reinforced connections), average ultimate strength is significantly improved with less variability resulting in a reduction in the COV. The shape of the reinforced average curve is quite interesting, showing an increase in connection strength at displacements greater than 20 mm. A possible explanation for this is that as the bolt was bearing on the wood, the reinforcing plate was piling up on one side of the bolt between the wood member and the steel side plate creating binding and friction forces (even though originally the side plates were shimmed so they were not touching the specimen) . As more reinforcing plate piles up, greater friction forces result between the side plates and wood member, resulting in an increase in connection strength. Detailed comments on ductility and comparing code strength values with ultimate strengths can be found in the discussion.

38x140 mm UNREINFORCED SPECIMENS-ONE 12.7 mm BOLT

Figure 23: Load-Displacement Curves-38x140 mm Unreinforced Tests-One 12.7 mm Bolt

38x140 mm REINFORCED SPECIMENS-ONE 12.7 mm BOLT

Figure 24: Load-Displacement Curves-38x140 mm Reinforced Tests-One 12.7 mm Bolt

Figure 25: Average Load-Displacement Curves-38x140 mm Tests-One 12.7 mm Bolt

4.1.2 Ten Bolt Connection

Ten specimens measuring 38x140 mm containing ten 12.7 mm bolts each (2 rows of 5 bolts) were tested in static tension. Figure 26 shows the plot of all ten load displacement curves. Row splitting, row shear, and group shear dominated the failure modes. Three specimens failed in group shear with post ultimate strength values dropping close to zero immediately after failure. The brittle failure mode of all the unreinforced specimens is reflected in the jagged nature of the average curve shown in Figure 28.

Ten more tests were conducted using truss plate reinforcement in the connection region of the 12.7 mm bolts. Figure 27 shows the plots of the 10 load deformation curves. Failure modes observed consisted of either group shear or row shear. Row splitting seemed to be eliminated as a failure mode, possibly due to the truss plate teeth lining up along the bolt rows making row shear more favourable as a failure mode than splitting. One can clearly see the three specimens that failed in group shear in Figure 27 as they are the curves showing the abrupt drop in load. These three failures are also picked up as discontinuities in the average curve shown in Figure 28. The specimens failing in row shear had gradual declines in strength post ultimate load, making for a more ductile connection than their unreinforced counterparts. Table 5 shows the statistics for both tests.

Table 5: Statistics-38x140 mm-Ten 12.7 mm Bolts

Specimens	Average Ultimate strength	Standard Deviation	Coefficient of Variation (COV)
Unreinforced 38x140mm-Ten 12.7 mm Bolts	123.83 kN	12.13 kN	9.80%
Reinforced 38x140mm-Ten 12.7 mm Bolts	121.71 kN	11.13 kN	9.15%

Table 5 shows the average ultimate strength for the unreinforced connections higher than the average ultimate strength for the reinforced connections, whereas Figure 28 shows the average curve for the reinforced connections with a higher peak load than the unreinforced average curve. This is due to the fact that the values in Table 5 are calculated by averaging the ultimate loads from each individual curve, which can occur at different displacements. Average curves are calculated by zeroing all individual curves and averaging all force and displacement values at each corresponding sample point; loading rate was constant for all tests, thus time and displacement is assumed to be interchangeable. The fact that the reinforcement did not change the failure modes between unreinforced and reinforced connection to any great extent, is probably why similar average ultimate strength and COV's were observed for both tests. Average ultimate strengths are a bit lower for the reinforced connections, which may be due to the fact that the truss plate teeth lined up along the bolt rows, making row shear more favourable than splitting. The truss plates did significantly improve the post ultimate behaviour of the reinforced connections, with the tail end of the reinforced average curve much higher than the unreinforced average curve.

38x140 mm UNREINFORCED SPECIMENS-TEN 12.7 mm BOLTS

Figure 26: Load-Displacement Curves-38x140 mm Unreinforced Tests-Ten 12.7 mm Bolts

38x140 mm REINFORCED SPECIMENS-TEN 12.7 mm BOLTS

Figure 27: Load-Displacement Curves-38x140 mm Reinforced Tests-Ten 12.7 mm Bolts

Figure 28: Average Load-Displacement Curves-38x140 mm Tests-Ten 12.7 mm Bolts

4.2 89x140 mm TEST SPECIMENS

4.2.1 Single Bolt Connection

Ten specimens measuring 89x140 mm containing one 15.9 mm bolt were tested in static tension. Figure 29 shows the plot of all the load displacement curves. The failure mode in each case was by splitting. Ten more tests were conducted using truss plate reinforcement in the connection region of the 15.9 mm bolt. Figure 30 shows the plots of the 10 load deformation curves. The reinforcement proved to be successful in changing the failure mode from wood splitting to wood crushing. The reinforced connections did eventually split due to the imposed large displacements, but the splitting action was restrained by the reinforcement and monitored load levels remained high even after large displacements. Figure 31 shows a plot of average curves. The smoothness of the reinforced average curve is indicative of the ductility of the reinforced connections, as brittle failures would show up as abrupt load drops. The average curve for the unreinforced connections has a few of the abrupt load drops. Table 6 shows the statistics for both tests. As Table 6 demonstrates, the reinforcement improved average ultimate strengths and reduced the COV.

Table 6: Statistics-89x140 mm-One 15.9 mm Bolt

Specimens	Average Ultimate strength	Standard Deviation	Coefficient of Variation (COV)
Unreinforced 89x140mm-One 15.9 mm Bolt	76.39 kN	6.96 kN	9.11%
Reinforced 38x140mm-One 15.9 mm Bolt	85.56 kN	6.27 kN	7.33%

89x140 mm UNREINFORCED SPECIMENS-ONE 15.9 mm BOLT

Figure 29: Load-Displacement Curves-89x140 mm Unreinforced Tests-One 15.9 mm Bolt

89x140 mm REINFORCED SPECIMENS-ONE 15.9 mm BOLT

Figure 30: Load-Displacement Curves-89x140 mm Reinforced Tests-One 15.9 mm Bolt

Figure 31: Average Load-Displacement Curves-89x140 mm Tests-One 15.9 mm Bolt

4.2.2 Ten Bolt Connection

Ten specimens measuring 89x140 mm containing ten 15.9 mm bolts each (2 rows of 5 bolts) were tested in static tension. The brittle nature of this unreinforced connection is dramatically shown in Figure 32, which shows the plot of all ten load displacement curves. The ultimate load for each curve occurred at a spiked peak, after which the load drops off significantly and abruptly. Failure occurred by group shear, row shear and splitting.

Ten more tests were conducted using truss plate reinforcement in the connection region of the 15.9 mm bolts. Figure 33 shows the plots of the 10 load deformation curves. One data set was lost due to a file corruption, so statistics are based on the results of nine tests. The reinforced connection behaviour was brittle at failure with row shear, group shear, and row splitting occurring. No improvements in ultimate strength were realized, as failure modes at ultimate did not differ from the unreinforced connections. The post ultimate behaviour of the reinforced connection was better than the unreinforced connections, as the reinforcement restrained splitting after ultimate failure and prevented the shear plugs from pulling out easily. Figure 34 shows a plot of average curves. Table 7 shows the statistics for both tests.

Table 7: Statistics-89x140 mm-Ten 15.9 mm Bolts

Specimens	Average Ultimate strength	Standard Deviation	Coefficient of Variation
Unreinforced 89x140mm-Ten 15.9 mm Bolts	372.50 kN	24.89 kN	6.68%
Reinforced 89x140mm-Ten 15.9 mm Bolts	364.93 kN	34.96 kN	9.58%

Table 7 indicates the average ultimate strength of the reinforced connections is lower than the average ultimate strength of the unreinforced connections, whereas Figure 34 shows the reinforced average-load displacement curve with a higher peak than the unreinforced average-load displacement curve (just like what occurred in the 38x140 mm ten bolt connections). This is again explained by the way values in Table 7 are calculated and the curves in Figure 34 are plotted. The ultimate strength values in Table 7 are averages of the ultimate loads of the individual load displacement curves, which can occur at different displacements. Average curves are calculated by zeroing all individual curves and averaging all force and displacement values at each corresponding sample point; loading rate was constant for all tests, thus time and displacement is assumed to be interchangeable. As was the case with the ten bolt connection in the 38x140 mm specimens, the lower value for average ultimate strength of the reinforced connections may be attributed to the fact that the truss plate teeth lined up along the edges of the bolt rows, increasing the likelihood of row shear failures.

89x140 mm UNREINFORCED SPECIMENS-TEN 15.9 mm BOLTS

Figure 32: Load-Displacement Curves-89x140 mm Unreinforced Tests-Ten 15.9 mm Bolts

89x140 mm REINFORCED SPECIMENS-TEN 15.9 mm BOLTS

Figure 33: Load-Displacement Curves-89x140 mm Reinforced Tests-Ten 15.9 mm Bolts

Figure 34: Average Load-Displacement Curves-89x140 mm Tests-Ten 15.9 mm Bolts

4.3 133x191 mm TEST SPECIMENS

4.3.1 Single Bolt Connection

Ten specimens measuring 133x191 mm in cross section containing one 22.2 mm bolt were tested in static tension. Figure 35 shows the plot of all ten load displacement curves. The curves are more rounded at the peak than the single bolt curves in the other PSL sizes, as the failure mode was wood crushing with eventual splitting at large displacements (the splitting is shown by the abrupt drop in the load displacement curves).

Ten more tests were conducted using truss plate reinforcement in the connection region of the 22.2 mm bolt. Figure 36 shows the plots of the 10 load displacement curves. The failure mode again was wood crushing followed by splitting, but the load displacement curves do not have abrupt load drops, as the reinforcement controls the splitting action, thereby forcing more wood crushing. Figure 37 contains the average load displacement curves. Table 8 shows the statistics for both tests. Again, the brittle failures of the unreinforced connections are picked up in the average load displacement curve as abrupt drops. Ductility improvements can be seen in the reinforced average load displacement curve, which is very smooth and has the tail end of its curve well above that of the unreinforced average load displacement curve.

Table 8: Statistics-133x191 mm-One 22.2 mm Bolt

Specimens	Average Ultimate strength	Standard Deviation	Coefficient of Variation (COV)
Unreinforced 133x191 mm-One 22.2 mm Bolt	152.87 kN	18.62 kN	12.18%
Reinforced 133x191 mm-One 22.2 mm Bolt	157.82 kN	17.80 kN	11.28%

There were gains in average ultimate strength with the reinforced connections, but not very much. This is due to the fact that both the reinforced and unreinforced connections had similar initial failure modes. The gains in strength could be due to the truss plates prolonging the wood crushing. Once the wood starts to split, there is transverse displacement of the fibres, causing the truss plates to pick up load in the perpendicular to grain direction and limit the size of the split, resulting in more wood crushing by the bolt and more ductile behaviour. The coefficient of variation did decrease by about 1%, indicating the reinforced connection behaviour at ultimate was more consistent than the unreinforced connections.

133x191 mm UNREINFORCED SPECIMENS-ONE 22.2 mm BOLT

Figure 35: Load-Displacement Curve-133x191 mm Unreinforced Tests-One 22.2 mm Bolt

133x191 mm REINFORCED SPECIMENS-ONE 22.2 mm BOLT

Figure 36: Load-Displacement Curves-133x191 mm Reinforced Tests-One 22.2 mm Bolt

Figure 37: Average Load-Displacement Curves-133x191 mm Tests-One 22.2 mm Bolt

4.3.2 Four Bolt Connection

Ten specimens measuring 133x191 mm containing four 22.2 mm bolts each were tested in static tension. Figure 38 shows the plot of all ten load displacement curves. Failure of this connection was by brittle splitting with each load displacement curve showing a characteristic spiked peak at ultimate load.

Ten more tests were conducted using truss plate reinforcement in the connection region of the 22.2 mm bolts. Figure 39 shows the plots of the 10 load deformation curves. The failure mode was still by splitting, but once again, the restraining forces imposed by the reinforcement limited the size of the split, forcing more wood bearing. This is shown by the gradual decline in load after ultimate. The test apparatus was unable to fail one of the reinforced specimens, therefore, the statistics are based on nine tests. Figure 40 contains the plots of the average curves. The reinforcement improved average ultimate strength and decreased the variability (Table 9). Out of all the multiple bolt reinforced connections tested (ten bolts in 38x140 mm specimens, ten bolts in 89x140 mm specimens, and 4 bolts in 133x191 mm specimens), this is the only set of tests that showed an increase in average ultimate strength with the application of reinforcement. This may indicate that there is a limit to the effectiveness of external reinforcement depending on member cross sections and number of fasteners in a connection.

Table 9: Statistics-133x191 mm-Four 22.2 mm Bolt

Specimens	Average Ultimate strength	Standard Deviation	Coefficient of Variation (COV)
Unreinforced 133x191 mm-Four 22.2 mm Bolts	401.84 kN	26.34 kN	6.55%
Reinforced 133x191 mm-Four 22.2 mm Bolts	427.97 kN	24.57 kN	5.74%

133x191 mm UNREINFORCED SPECIMENS-FOUR 22.2 mm BOLTS

Figure 38: Load-Displacement Curves-133x191 mm Unreinforced Tests-Four 22.2 mm Bolts

133x191 mm REINFORCED SPECIMENS-FOUR 22.2 mm BOLTS

Figure 39: Load-Displacement Curves-133x191 mm Reinforced Tests-Four 22.2 mm Bolts

Figure 40: Average Load-Displacement Curves-133x191 mm Tests-Four 22.2 mm Bolts

5 DISCUSSION

5.1 EFFECT OF REINFORCEMENT

5.1.1 Ultimate Strength

From the test data it is evident that the benefits of truss plate reinforcement in the connection region vary widely. Table 10 gives the effect of the reinforcement on the average ultimate strengths of the connections tested.

Table 10: Effect of Reinforcement on Average Ultimate Strength

Specimen	Connection	Effect of Reinforcement on Average Ultimate Strength
38x140 mm	One 12.7 mm bolt	+36.5%
	Ten 12.7 mm bolts	-1.7%
89x140 mm	One 15.9 mm bolt	+12.0%
	Ten 15.9 mm bolts	-2.0%
133x191 mm	One 22.2 mm bolt	+3.2%
	Four 22.2 mm bolts	+6.5%

In the cases where the average ultimate strength was increased by the addition of reinforcement, failure modes had changed from splitting of the Parallam[®] above the bolt, to wood crushing combined with eventual splitting. Shear plug failures were not eliminated by use of reinforcement in the 38x140 mm ten bolt specimens or the 89x140 mm ten bolt specimens. Because the failure modes did not change with these test specimens, the reinforcement did not improve the ultimate strength of the connections. In fact it caused a slight decrease in average ultimate strengths. One possible explanation of this is that the truss plates only resist splitting forces in the perpendicular to grain direction, and have little effect in preventing shear planes forming in the longitudinal direction. In other words, the truss plate only resists load when there is deformation of the wood in the perpendicular to grain direction, which occurs with splitting of

the rows. With row or group shear failures, there is little deformation of the wood in the perpendicular to grain direction, as the wood fibre is pushed out in the longitudinal direction by the bolts. Where shear failures dominated in the unreinforced and reinforced 38x140 mm and 89x140 mm 10 bolt connections, the placement of the reinforcement may have actually created a weak path for shear planes to form, as the truss plate teeth lined up on the edge of the bolt holes, and a lower average ultimate strength was realized. In order to maximize the effectiveness of the reinforcement, it is proposed that in future the reinforcement be placed in the regions of the connection where it minimizes its influence on shear plug failures. That means placing the truss plate reinforcement so that the teeth penetrate the wood at least 2 bolt diameters away from the center of the bolt holes.

Table 10 also gives a good indication of the range of effectiveness of this type of reinforcement on ultimate strength. Looking at the improvement values for the single bolt connections one sees a trend that the degree of effectiveness decreases with the larger cross sections (36.5% increase with the 38x140 mm specimens down to 3.2% with the 133x191 mm specimens). As well, reinforcement had a positive effect on the four bolt connection in the 133x191 mm specimens, whereas it had a negative effect on the 10 bolt connections in the 38x140 mm and 89x140 mm specimens. Thus, the degree of effectiveness of the reinforcement on ultimate strength will depend on the cross section size and number of fasteners in the connection.

5.1.2 Stiffness

To calculate the stiffness for each connection, the load at 50% of ultimate is divided by its associated displacement (calculations can be found in Appendix V). In each case, the average stiffness of the connections decreased when reinforcement was added to the specimens. Table 11 shows the results.

Table 11: Effect of Reinforcement on Stiffness

Connection		Average Stiffness (kN/mm)		Effect of Reinforcement
		Unreinforced	Reinforced	
38x140 mm	One 12.7 mm bolt	18.9	13.0	-31.3%
	Ten 12.7 mm bolts	72.6	32.9	-54.8%
89x140 mm	One 15.9 mm bolt	22.3	19.1	-14.3%
	Ten 15.9 mm bolts	200.6	137.2	-31.6%
133x191 mm	One 22.2 mm bolt	36.8	30.1	-18.2%
	Four 22.2 mm bolts	167.8	141.0	-16.0%

5.1.3 Ductility

In all cases, the truss plate reinforcement did improve the residual strength of the connection. On average, the tail ends of the load deformation curves for reinforced connections were well above those of unreinforced connections, and in some instances were well above the predicted code resistance even after large deformations. This improvement in residual strength of the reinforced connections translates into more ductile connections, which are capable of absorbing more energy than unreinforced connections. For critical connections in timber buildings, this may be a valuable benefit during earthquakes. To quantify this ductility improvement, a ductility factor will be defined as the post ultimate displacement at 80% of ultimate load (after ultimate has been reached), divided by the displacement at 50% of ultimate load (before ultimate is reached). Table 12 shows the ductility improvement results (calculations can be found in Appendix V). Values with a (>) sign indicate the load never dropped to 80% of ultimate after failure, so a conservative displacement value of 30 mm was picked to calculate a lower bound value.

Table 12: Ductility Improvements

Specimen	Connection	Effect of Reinforcement on Ductility
38x140 mm	One 12.7 mm bolt Ten 12.7 mm bolts	>+278.5% +250.4%
89x140 mm	One 15.9 mm bolt Ten 15.9 mm bolts	+104.5% +5.3%
133x191 mm	One 22.2 mm bolt Four 22.2 mm bolts	>+143.1% +35.3%

5.2 CONNECTION COMPARISON WITH CAN/CSA 086.1-94

5.2.1 Normal Distribution

Assuming a normal distribution of the calculated ultimate connection strengths, the lower 5th percentile strengths can be calculated from the following formula :

$$x_p = m_x \pm k\sigma$$

where x_p is the strength property at the P -th percentile, m_x is the mean value, σ is the standard deviation, and k varies for different P values. To calculate the lower 5th percentile strengths the value for k is -1.645. Table 13 gives the lower 5th percentile strengths for the tested connections, as well as a ratio of the lower 5th percentile connection strengths to the code predicted characteristic strength values.

Table 13: Lower 5th Percentiles of Connection Strengths (Normal Distribution)

Connection		Reinforced	Code Predicted Connection Strength ^a (kN)	Code Predicted Characteristic Strength ^b (kN)	Lower 5 th Percentile Connection Strengths (kN)	Ratio of Lower 5 th Percentile Strengths vs. Code Predicted Characteristic Strength
38x140 mm	single bolt	No	10.7	15.3	18.7	1.2
		Yes	10.7	15.3	30.9	2.0
	ten bolt	No	39.9	57.0	103.9	1.8
		Yes	39.9	57.0	103.4	1.8
89x140 mm	single bolt	No	30.3	43.3	64.9	1.5
		Yes	30.3	43.3	75.3	1.7
	ten bolt	No	154.4	220.6	331.6	1.5
		Yes	154.4	220.6	307.4	1.4
133x191 mm	single bolt	No	61.6	88.0	122.2	1.4
		Yes	61.6	88.0	128.5	1.5
	four bolt	No	175.2	250.3	358.5	1.4
		Yes	175.2	250.3	387.6	1.5

^a calculated using mean relative density of 0.628 for parallam, and short load duration factor $K_D=1.15$

^b calculated by dividing code predicted connection strength by performance factor 0.7

As Table 13 indicates, CAN/CSA 086.1-94 is conservative in predicting the connection strength values of the unreinforced and reinforced connections. One contribution to the above results may come from the fact that Parallam[®] is a stronger and more consistent construction material than sawn timber from which the design equations were derived.

5.2.2 2-P Weibull Distribution

Perhaps a better distribution to model the connection strengths is the 2-P Weibull distribution, as the probability density function of the normal distribution is symmetrical, and negative values are possible. The estimated percentile of the 2-P Weibull distribution is given by the following formula:

$$x_p = x_0 + m*[-\log(1 - P)]^{\frac{1}{k}}$$

where:

- x_p = the estimated P -th percentile value
- x_0 = estimated location parameter ($x_0 = 0$ for 2-P Weibull)
- m = estimated scale parameter
- k = estimated shape parameter

Using a computer program called EXTREME, the parameters (m) and (k) can be estimated by entering in the mean and standard deviation for each data set. Once (m) and (k) have been determined the 5th percentile can be calculated. Table 14 gives the lower 5th percentile strengths for the tested connections (assuming a 2-P Weibull distribution). Table 15 compares the lower 5th percentile strengths estimated from the 2-P Weibull distribution to the code predicted characteristic strength values.

Table 14: Lower 5th Percentiles of Connection Strengths (2-P Weibull Distribution)

Connection		Reinforced	Estimated Scale Parameter (m)	Estimated Shape Parameter (k)	Lower 5 th Percentile Connection Strengths (kN)
38x140 mm	single bolt	No	26.47	7.77	16.22
		Yes	34.79	22.68	29.42
	ten bolt	No	129.06	12.40	94.96
		Yes	126.53	13.33	95.12
89x140 mm	single bolt	No	79.40	13.38	59.75
		Yes	88.30	16.78	70.39
	ten bolt	No	383.42	18.47	312.05
		Yes	380.03	12.69	281.59
133x191 mm	single bolt	No	160.78	9.86	109.31
		Yes	165.43	10.69	115.89
	four bolt	No	413.40	18.84	337.81
		Yes	438.80	21.60	367.94

Table 15: Connection Strength Comparisons to CAN/CSA 086.1-94

Connection		Reinforced	Code Predicted Connection Strength ^a (kN)	Code Predicted Characteristic Strength ^b (kN)	Lower 5 th Percentile Connection Strength (kN)	Ratio of Lower 5 th Percentile Strengths vs. Code Predicted Characteristic Strength
38x140 mm	single bolt	No	10.7	15.3	16.2	1.1
		Yes	10.7	15.3	29.4	1.9
	ten bolt	No	39.9	57.0	95.0	1.7
		Yes	39.9	57.0	95.1	1.7
89x140 mm	single bolt	No	30.3	43.3	59.8	1.4
		Yes	30.3	43.3	70.4	1.6
	ten bolt	No	154.4	220.6	312.1	1.4
		Yes	154.4	220.6	281.6	1.3
133x191 mm	single bolt	No	61.6	88.0	109.3	1.2
		Yes	61.6	88.0	115.9	1.3
	four bolt	No	175.2	250.3	337.8	1.3
		Yes	175.2	250.3	367.9	1.5

^a calculated using mean relative density of 0.628 for parallam, and short load duration factor $K_D=1.15$

^b calculated by dividing code predicted connection strength by performance factor 0.7

Comparing Tables 13 and 15, one can see that using a 2-P Weibull distribution to estimate the lower 5th percentile strengths of the connections yields more conservative estimates than the normal distribution.

5.3 CONNECTION EFFICIENCY

As previously mentioned, the factored parallel to grain resistance of a bolted connection is calculated with the following formula (CAN/CSA 086.1-94):

$$P_r = \phi * P_u * n_s * n_F * J_F$$

where:

ϕ	=0.7
P_r	=factored lateral strength resistance for parallel to grain loading
P_u	= $p_u(K_D K_{SF} K_T)$
p_u	=lateral strength resistance strength resistance for parallel to grain loading(Clause 10.4.4.2), N
n_F	=number of bolts
n_s	=number of shear planes
K 's	=modification factors depending on actual conditions of use
J_F	= $J_G J_L J_R$ – connection configuration adjustment factor
J_R	=group reduction factor for row effect
J_L	=reduction factor when reduced end distance is used
J_G	=group reduction factor for uneven load sharing of bolts in a row (2-12 bolts)

As indicated the J_F factor depends on three other factors which include J_R , J_L , and J_G . The factor J_R has values of 1.0 for 1 row, or for 1 bolt per row, the factor J_L has values of 0.75 for end distances of 7d, or 1.0 for 10d (for intermediate values, interpolate linearly), and the factor J_G has values which depend on the following empirical formula:

$$J_G = 0.33 * \left(\frac{l}{d} \right)^{0.5} * \left(\frac{s}{d} \right)^{0.2} * N^{-0.3}$$

where:

J_G	=not greater than 1.0 =1.0 in cases with only one bolt per row
l	=member thickness, mm
s	=bolt spacing in the row, mm
d	=bolt diameter, mm
N	=number of bolts in a row

For a given bolt size and specimen size actual J_F values can be calculated from experiment data by the following formula (values calculated from normal distribution were used):

$$J_F^* x = \left(\frac{LWR_{MULT}}{LWR_{SING} * N_{MULT}} \right) x$$

where:

J_F^*	=calculated J_F from experiment data
x	=connection results (R for reinforced, U for unreinforced)
LWR_{MULT}	=lower 5 th percentile strength of multiple bolt connection
LWR_{SING}	=lower 5 th percentile strength of single bolt connection of same bolt size and specimen size as the multiple bolt connection
N_{MULT}	=number of bolts in multiple bolt connection

Because the reinforced single bolt connections had significant improvements of ultimate strength, whereas the ultimate strengths did not improve much for the reinforced multiple bolt connections, using the above formula results in a low value for J_F^* for the reinforced connections. For interest sake, a third factor, J_F^{**} , is calculated for the reinforced connections by dividing the lower 5th percentile strength of the reinforced multiple bolt connections by the lower 5th percentile strength of the unreinforced single bolt connection times the number of bolts in the multiple bolt connection. This can be shown with the following formula.

$$J_F^{**} = \frac{(LWR_{MULT})_R}{(LWR_{SING})_U * N_{MULT}}$$

where:

J_F^{**}	=modified J_F from experiment data for reinforced connections
LWR_{MULT}	=lower 5 th percentile strength of multiple bolt connection
LWR_{SING}	=lower 5 th percentile strength of single bolt connection of same bolt size and specimen size as the multiple bolt connection.
R	=reinforced connection results
U	=unreinforced connection results
N_{MULT}	=number of bolts in multiple bolt connection

Using the previously mentioned formula, J_F^* for the ten bolt connection in the 38x140 mm specimens was calculated to be 0.557 for the unreinforced connections, and 0.335 for the reinforced connections, while J_F^{**} was calculated to be 0.554. Figure 41 shows the plot of J_F values for two rows of 12.7 mm bolts in 38x140 mm specimens (according to CAN/CSA 086.1), with J_F^* and J_F^{**} values plotted as individual points. J_F^* for the unreinforced connections lies above the code predicted value, reflecting a conservative approach towards connection efficiency. J_F^* for the reinforced connections lies below the code value due to the reinforcement increasing strength values significantly with the single bolt connection, whereas it had little effect on the ten bolt connection. Thus, taking a ratio of the two 5th percentile strengths, which is necessary to calculate J_F^* , a low value results.

Figure 41: Comparison of CAN/CSA 086.1-94 J_F and Actual J_F for 38 mm Thick Specimens

For the ten bolt connection in the 89x140 mm specimens J_F^* was calculated to be 0.511 for the unreinforced connections, and 0.409 for the reinforced connections, while J_F^{**} was calculated to be 0.473. Figure 42 shows the plot of J_F values for two rows of 15.9 mm bolts in 89x140 mm specimens (according to CAN/CSA 086.1), with J_F^* and J_F^{**} values plotted as individual points. In this instance, the actual J_F^* for the unreinforced connections was right on the code predicted value. For the reinforced connections, J_F^* was below the code value due to the significant improvements in strength the reinforcement made on the single bolt connection and not on the ten bolt connection.

Figure 42: Comparison of CAN/CSA 086.1-94 J_F and Actual J_F for 89 mm Thick Specimens

For the four bolt connection in the 133x191 mm specimens J_F^* was calculated to be 0.733 for the unreinforced connections, and 0.754 for the reinforced connections, while J_F^{**} was calculated to be 0.792. Figure 43 shows the plot of J_F values for two rows of 22.2 mm bolts in 133x191 mm specimens (according to CAN/CSA 086.1), with J_F^* and J_F^{**} values plotted as individual points.

Figure 43: Comparison of CAN/CSA 086.1-94 J_F and Actual J_F for 133 mm Thick Specimens

The actual J_F^* for the unreinforced connections was slightly above the code predicted value. For the reinforced connections, J_F^* was above the code value due improvements in strength the reinforcement made on both the single bolt connection and the four bolt connection.

6 CONCLUSION

The study reported in this thesis was meant to address the issue of improving the strength and ductility of bolted connections in PSL. Because of the variable nature of timber products, design codes world-wide are very conservative in predicting resistance values, which are largely based on a ductile behaviour pattern, combined with empirical rules to avoid premature failures through splitting or shear failures. The use of truss plates to reinforce the connection region to assure ductile behaviour has been shown to be effective in a certain range of connection configurations and PSL cross sections.

For single bolt connections, it was found that reinforcement was less effective on the larger cross sections on improving ultimate strength, but did significantly improve ductility in all cases. Reinforcement had little effect on ultimate strengths of the ten bolt connections in the 38x140 mm specimens and 89x140 mm specimens, whereas small gains in ultimate strength were observed with the four bolt connection in the 133x191 mm specimens. Ductility gains were achieved in all the multiple bolt cases (Table 16).

Table 16: Summary of Reinforcement Effects

Connection		Effect of Reinforcement	
		Average Ultimate Strength	Ductility
38x140 mm	One 12.7 mm bolt	+36.5%	>+278.5%
	Ten 12.7 mm bolts	-1.7%	+250.4%
89x140 mm	One 15.9 mm bolt	+12.0%	+104.5%
	Ten 15.9 mm bolts	-2.0%	+5.3%
133x191 mm	One 22.2 mm bolt	+3.2%	>+143.1%
	Four 22.2 mm bolts	+6.5%	+35.3%

In all instances, the connections (reinforced or unreinforced) failed well above the code predicted values. Comparing the lower 5th percentiles of the actual connection strengths to their corresponding code predicted characteristic strength value, results in the lower 5th percentiles being anywhere from 1.1 to 2.0 times greater than their code predicted values.

As mentioned earlier, the effectiveness of the truss plate reinforcement depends on cross section sizes and connection configurations. More work needs to be done to determine the upper bound where one should change from truss plate reinforcement to a reinforcement technique that would be more beneficial to the connection in question. With that said, future research with reinforced bolted connections in parallel strand lumber should investigate the following:

- Trying different truss plates
- Using other external reinforcement methods such as fibreglass and plywood to determine their effectiveness on improving the behaviour of multiple bolt connections
- Using an internal reinforcement technique such as reinforcing rods glued or threaded in perpendicular to grain for multiple bolt connections, as opposed to an external type reinforcement
- Establishing a framework to assist in deciding where and when to use external reinforcement
- Changing the static tension loading to cyclic loading to determine the energy dissipating capacity and degradation of reinforced connections under repeated cycles of displacement
- Dynamic testing of reinforced multiple bolt connections on the shake table

7 LITERATURE CITED

- Canadian Wood Council: Introduction to Wood Building Technology, Ottawa, ON, 1997.
- Canadian Wood Council: Introduction to Wood Design, Ottawa, ON, 1996.
- Canadian Wood Council: Wood Design Manual 1995, Ottawa, ON, 1997.
- Canadian Wood Council: Wood Reference Handbook. Ottawa, ON, 1997.
- Chen, C.; Haller, P.; Natterer, J. 1994. Experimental Study on Glass Fibre Reinforced Timber Joints. Proceedings, Pacific Timber Engineering Conference; July 1994; Gold Coast, Australia, pp. 66-72.
- Cruz, H. M. P.; Hilson, B. O.; Whale, L. R. J. 1991. Dynamic Behaviour of Nailed Timber Joints: Embedment tests under cyclic load. Proceedings, International Timber Engineering Conference; September, 1991; London, United Kingdom, pp. 3.11-3.19.
- Glos, P; Horstmann, H. 1991. Design of Glued Joints. Proceedings, International Timber Engineering Conference; September, 1991; London, United Kingdom, pp. 3.77-3.84.
- Haller, P.; Chen, C. J.; Natterer, J. 1996. Experimental Study on Glassfiber Reinforced and Densified Timber Joints. Proceedings, International Wood Engineering Conference; October, 1996; New Orleans, Louisiana, USA, 1.308-1.314.
- Haygreen, John G., and Jim L. Bowyer. 1996. Forest Products and Wood Science. Iowa State University Press, Ames Iowa, USA.
- Hoyle, Robert J., Jr. 1988. Design of Wood Shear Joints Fastened with Nails and Structural Elastomeric Adhesive. Proceedings, International Conference on Timber Engineering; September 1988; Washington State University, United States, pp. 38-45.
- Johansen, K.W. 1949. Theory of Timber Connectors. Publications of the International Association of Bridge and Structural Engineering. No. 9: 249-262. Bern, General Secretariat.
- Larsen, H.J. 1973. The Yield Load of Bolted and Nailed Joints. Proceedings, International Union of Forestry Research Organization meeting – 5th Congress. South Africa.
- Larsen, H.J. 1977. K.W. Johansen's Nail Tests. Bygningsstatisk Meddelels No. 48: 9-30.
- Larsen, H. 1994. Connections and Joints. Proceedings, Pacific Timber Engineering Conference; July 1994; Gold Coast, Australia, pp. 60-66.
- Larsen, Hans Jorgen; Enquist, Bertil. 1996. Glass Fibre Reinforcement of Dowel-Type Joints. Proceedings, International Wood Engineering Conference; October, 1996; New Orleans, Louisiana, USA, pp. 1.293-1.302.
- Lau, P.W.C. and Y Tardiff. 1987. Cracks Produced by Driving Nails into Wood Effects of Wood and Nail Variables. Unpublished progress report, Building Systems Dept., Forintek Canada Corp., Ottawa, ON.

- Leijten, A. J. M. 1991. Development of Reinforced Joints with Densified Veneer Wood. Proceedings, International Timber Engineering Conference; September, 1991; London, United Kingdom, pp. 3.36-3.41.
- Leijten, A. J. M. 1996. The Concept of the Prestressed DVW Reinforced Joint with Expanded Tubes. Proceedings, International Timber Engineering Conference; October, 1996; New Orleans, Louisiana, USA, pp. 2.295-2.299.
- Madsen, Borg. 1998. Timber Connections with Strength and Reliability of Steel. Timber Connection Design Seminar-Session 7 Notes, October, 1998; Vancouver, British Columbia, Canada.
- Massé, D.I., J.J. Salinas and J.E. Turnbull. 1988. Lateral Strength and Stiffness of Single and Multiple Bolts in Glued-laminated Timber Loaded Parallel to Grain. Unpublished contract. No. C-029, Eng. And Stat. Res. Centre, Research Branch, Agriculture Canada, Ottawa, ON.
- Moe, Johannes. 1961. A Study of Nail-Glued Timber Truss Joints. Proceedings, First International Conference on Timber Engineering; September 1961, Southampton, England, pp. 51-59.
- Moss, Darren; McKay, Paul; Kuo, Andy; Hsu, Jack; Mui, Jeromy. 1998. The Behaviour of Multiple Bolt Connections in Parallel Strand Lumber with Claw Truss Plate Reinforcement. Unpublished Civil 321 Project Report, University of British Columbia, Dept. of Civil Engineering, Vancouver, BC.
- National Research Council (NRC). Registry of Product Evaluations, Summer 1997. Institute for Research in Construction, Ottawa, ON, 1997.
- Ni, C; Chui, Y. 1994. Response of Nailed Wood Joints to Dynamic Loads. Proceedings, Pacific Timber Engineering Conference; July 1994; Gold Coast, Australia, pp. 2.9-2.12.
- Odeen, K. 1991. Wood as an Engineering Material. Proceedings, International Timber Engineering Conference; September, 1991; London, United Kingdom, pp. 4.413-4.420.
- Quenneville, Pierre. 1998. Connection Design-Effect of Configuration on Failure Modes. Timber Connection Design Seminar-Session 2 Notes, October, 1998; Vancouver, British Columbia, Canada.
- Quenneville, J H P; Dalen, K Van. 1991. Reduction of Timber Fastener End Distance Requirements through Joint Prestressing. Proceedings, International Timber Engineering Conference; September, 1991; London, United Kingdom, pp. 4.27-4.34.
- Ramos, A.N. 1960. Spacing of Sixpenny and Eightpenny Wire Nails in Douglas Fir Multi-nail Joints. U.S. Department of Agriculture, Forest Products Laboratory. Report No. 2155. Madison, WI.
- Rodd, P. D.; Leijten, A. J. M. 1994. The Bending Moment Capacity of DVW Reinforced Glulam Joints. Proceedings, International Wood Engineering Conference; October, 1996; New Orleans, Louisiana, USA.

- Rodd, P D; Spriggs, R A; Hilson B O. 1991. Prediction of Embedment Characteristics For Laterally Loaded Resin Injected Bolts in Timber. Proceedings, International Timber Engineering Conference; September, 1991; London, United Kingdom, pp. 3.50-3.55.
- Rogers, C.; R. Thomsom; G. Smith. 1994. Nailplate Reinforcement of Bolted Joints. Proceedings, Pacific Timber Engineering Conference; July 1994; Gold Coast, Australia, pp. 349-352.
- Schubert, Cristoph. 1997. Testing of Reinforced Bolted Connections of Parallel Strand Lumber (PSL) Under Cyclic Loading. Unpublished report, University of British Columbia, Dept. of Wood Science, Vancouver, BC.
- Soltis, Lawrence A.; Ross, Robert J. 1996. US Patent #5501054.
- Soltis, Lawrence A.; Robert J. Ross; Daniel F. Windorski. 1998. Fiberglass-Reinforced Bolted Wood Connections. Forest Products Journal, Vol. 48, No. 9.
- Trayer, G.W. 1932. Bearing Strength of Wood Under Bolts. U.S. Department of Agriculture, Technical Bulletin No. 332.
- Turkowskij, S. 1991. Prefabricated Joints of Timber Structures on Inclined Glued-In Bars. Proceedings, International Timber Engineering Conference; September, 1991; London, United Kingdom, pp. 3.143-3.148.
- Werner, Hartmut. 1994. Reinforced Joints with Dowels and Expanded Tubes Loaded in Tension. Proceedings, International Wood Engineering Conference; October, 1996; New Orleans, Louisiana, USA, 2.307-2.311.
- Whale, L.R.J., I. Smith and H.J. Larsen. 1987. Design of Nailed and Bolted Joints Proposals for the Revision of Existing Formulae in Draft Eurocode V and the CIB Code. International Council for Building Research Studies and Documentation Working Commission W18A. Paper No. 20-7-1.
- Williams, J.; Smith, G.; 1994. Load Transfer Mechanisms in Toothed Nail Plates. Proceedings, Pacific Timber Engineering Conference; July 1994; Gold Coast, Australia.
- Yasumura, M., T. Murota and H. Sakai. 1987. Ultimate Properties of Bolted Joints in Glued-laminated Timber. Report to the Working Commission W18 – Timber Structures. Dublin, Ireland.

APPENDICES

APPENDIX I

DESIGN CALCULATIONS FOR BOLTED TIMBER CONNECTIONS

Figures 44 through 49 show the design code calculations (CAN/CSA 086.1-94) for the parallel to grain resistance values of the connections tested in this experiment.

BOLTED CONNECTION DESIGN SPREADSHEET: 38x140 mm specimens, one 12.7 mm bolt			
Member Geometry			
l1l1	= member thickness	= 38.00 mm	<input>
ddd	= bolt diameter	= 12.70 mm	<input>
end	= number of bolt diameters from member end	= 10.00	<input>
sss	= ddd*bolt	= 127.00 mm	<calc>
NNN	= bolt spacing in the row	= 0.00 mm	<input>
row	= number of bolts in a row	= 1.00	<input>
nss	= number of rows	= 1.00	<input>
nff	= number of shear planes	= 2.00	<input>
	= NNN*row	= 1.00	<calc>
Modification Factors			
phi	= performance factor	= 0.70	<input>
Kdd	= load duration factor	= 1.15	<input>
Ksf	= service condition factor for fastening	= 1.00	<input>
Ktt	= treatment factor	= 1.00	<input>
Bolt Group Modification Factors			
Jrr	= @if(row=1,1.0,@if(row=2,0.8,@if(row=3,0.6,1.0)))	= 1.00	<calc>
Jll	= (0.25/(3*ddd))*end+1/6	= 1.00	<calc>
Jgg	= @if(sss=0,1.0,@if(0.33*(l1l1/ddd)*0.5*(sss/ddd)*0.2*(NNN*(-0.3))>1.0,1.0,0.33*(l1l1/ddd)*0.5*(sss/ddd)*0.2*(NNN*(-0.3))))	= 1.00	<calc>
Jff	= Jgg*Jll*Jrr	= 1.00	<calc>
Wood member embedding strength			
ggw	= mean relative density (Douglas-fir)	= 0.63	<input>
fwpar	= 63*ggw*(1-0.01*ddd)	= 34.54 MPa	<calc>
fwper	= 27.4*ggw*(1-0.01*ddd)	= 15.02 MPa	<calc>
Steel plate embedding strength			
Fuu	= ultimate tensile strength of steel	= 0.00 MPa	<input>
phiSteel	= resistance factor for steel member in bolted connection	= 0.90	<input>
phiWood	= resistance factor for wood member in bolted connection	= 0.70	<input>
Fsteel	= 1.5*(phiSteel/phiWood)*Fuu	= 0.00 MPa	<calc>
FA36	= ASTM A36 grade steel	= 574.00 MPa	<input>
FCSA	= CSA-G40.21M steel, grade 300 W	= 646.00 MPa	<input>
Material Strengths			
l1l1	= side member thickness	= 19.00 mm	<input>
l1l2	= main member thickness	= 38.00 mm	<input>
fyf	= bolt yield strength (Hex Gr5)	= 634.30 MPa	<input>
ff1	= embedding strength of side member (ASTM A36)	= 574.00 MPa	<input>
ff2	= embedding strength of main member	= 34.54 MPa	<input>
FFFF1	= 0.8*ff1	= 459.20 MPa	<calc>
Failure Modes			
faila	= FFFF1*ddd*2*(l1l1/ddd)	= 110805 N	<calc>
failb	= FFFF1*ddd*2*(ff2/ff1)*(l1l2/ddd)	= 13335 N	<calc>
failc	= FFFF1*ddd*2*0.5*(ff2/ff1)*(l1l2/ddd)	= 6668 N	<calc>
faild	= FFFF1*ddd*2*(@sqrt(0.6*(ff2/(ff1+ff2)))*(fyf/ff1))+0.2*(l1l1/ddd)	= 36529 N	<calc>
faile	= FFFF1*ddd*2*(@sqrt(0.6*(ff2/(ff1+ff2)))*(fyf/ff1))+0.2*(l1l2/ddd)	= 58690 N	<calc>
failf	= FFFF1*ddd*2*0.2*(l1l1/ddd)+(ff2/ff1)*(l1l2/ddd)	= 24828 N	<calc>
failg	= FFFF1*ddd*2*(@sqrt((2/3)*(ff2/(ff1+ff2)))*(fyf/ff1)))	= 15145 N	<calc>
Results			
twomem	= @min(faila,failb,faild,failf,failg)	= 13335 N	<calc>
threemem	= @min(faila,failc,faild,failg)	= 6668 N	<calc>
nummem	= number of members: 0 for two, 1 for three	= 1	<input>
puu	= @if(nummem=0, twomem,@if(nummem=1, threemem, 0))*kdd*kt*ksf	= 7668 N	<calc>
Prr	= phi*Puu*nss*nff*Jff/1000	= 10.7 kN	<calc>

Figure 44: Design calculations for one 12.7 mm bolt in 38x140 mm PSL members

BOLTED CONNECTION DESIGN SPREADSHEET: 38x140 mm specimens, ten 12.7 mm bolts

Member Geometry			
lli	= member thickness	=	38.00 mm
ddd	= bolt diameter	=	12.70 mm
end	= number of bolt diameters from member end	=	10.00
ssd	= ddd*bolt	=	127.00 mm
ss	= bolt spacing in the row	=	51.00 mm
NNN	= number of bolts in a row	=	5.00
row	= number of rows	=	2.00
nss	= number of shear planes	=	2.00
nff	= NNN*row	=	10.00
Modification Factors			
phi	= performance factor	=	0.70
Kdd	= load duration factor	=	1.15
Ksf	= service condition factor for fastening	=	1.00
Ktt	= treatment factor	=	1.00
Bolt Group Modification Factors			
Jrr	= @if(row=1,1.0,@if(row=2,0.8,@if(row=3,0.6,1.0)))	=	0.80
Jll	= (0.25/(3*ddd))*end+1/6	=	1.00
Jgg	= @if(ss=0,1.0,@if(0.33*(lli/ddd)*0.5*(ss/ddd)*0.5*(lli/ddd)>1.0,1.0,0.33*(lli/ddd)*0.2*(NNN*(-0.3))))	=	0.47
Jff	= Jgg*Jll*Jrr	=	0.37
Wood member embedding strength			
ggw	= mean relative density	=	0.63
fwpar	= 63*ggw*(1-0.01*ddd)	=	34.54 MPa
fwper	= 27.4*ggw*(1-0.01*ddd)	=	15.02 MPa
Steel plate embedding strength			
Fuu	= ultimate tensile strength of steel	=	0.00 MPa
phisteel	= resistance factor for steel member in bolted connection	=	0.90
phiwood	= resistance factor for wood member in bolted connection	=	0.70
Fsteel	= 1.5*(phisteel/phiwood)*Fuu	=	0.00 MPa
FA36	= ASTM A36 grade steel	=	574.00 MPa
FCSA	= CSA G40.21M steel, grade 300 W	=	646.00 MPa
Material Strengths			
lli1	= side member thickness	=	19.00 mm
lli2	= main member thickness	=	38.00 mm
fy	= bolt yield strength (Hex Gr5)	=	634.30 MPa
ffi1	= embedding strength of side member (ASTM A36)	=	574.00 MPa
ffi2	= embedding strength of main member	=	34.54 MPa
FFFF1	= 0.8*ffi1	=	459.20 MPa
Failure Modes			
faila	= FFFF1*ddd*2*(lli1/ddd)	=	110805 N
failb	= FFFF1*ddd*2*(ffi2/ffi1)*(lli2/ddd)	=	13335 N
failc	= FFFF1*ddd*2*0.5*(ffi2/ffi1)*(lli2/ddd)	=	6668 N
faild	= FFFF1*ddd*2*(@sqrt(0.6*(ffi2/ffi1+ffi2))*(fy/ffi1))+0.2*(lli1/ddd)	=	36529 N
faile	= FFFF1*ddd*2*(@sqrt(0.6*(ffi2/ffi1+ffi2))*(fy/ffi1))+0.2*(lli2/ddd)	=	58690 N
failf	= FFFF1*ddd*2*0.2*(lli1/ddd)-(ffi2/ffi1)*(lli2/ddd)	=	24828 N
failg	= FFFF1*ddd*2*(@sqrt((2/3)*(ffi2/ffi1+ffi2))*(fy/ffi1))	=	15145 N
Results			
twomem	= @min(faila,failb,faild,failf,failg)	=	13335 N
threemem	= @min(faila,failc,faild,failg)	=	6668 N
nummem	= number of members: 0 for two, 1 for three	=	1
puu	= @if(nummem=0, twomem,@if(nummem=1, threemem, 0))*kdd*kt*ksf	=	7668 N
Prr	= phi*Puu*nss*nff/Jff/1000	=	39.9 kN

Figure 45: Design calculations for ten 12.7 mm bolts in 38x140 mm PSL members

BOLTED CONNECTION DESIGN SPREADSHEET: 89x140 mm specimens, one 15.9 mm bolt

Member Geometry			
l1	= member thickness	=	89.00 mm
ddd	= bolt diameter	=	15.90 mm
bolt	= number of bolt diameters from member end	=	10.00
end	= ddd*bolt	=	159.00 mm
sss	= bolt spacing in the row	=	0.00 mm
NNN	= number of bolts in a row	=	1.00
row	= number of rows	=	1.00
nss	= number of shear planes	=	2.00
nff	= NNN*row	=	1.00

Modification Factors			
phi	= performance factor	=	0.70
Kdd	= load duration factor	=	1.15
Ksf	= service condition factor for fastening	=	1.00
Kt	= treatment factor	=	1.00

Bolt Group Modification Factors			
Jrr	= @if(row=1,1.0,@if(row=2,0.8,@if(row=3,0.6,1.0)))	=	1.00
Jll	= (0.25/(3*ddd))*end+1/6	=	1.00
Jgg	= @if(sss=0,1.0,@if(0.33*(l1/ddd)*0.5*(sss/ddd)*0.5*(l1/ddd)*0.5*(NNN*(-0.3)))>1.0,1.0,0.33*(l1/ddd)*0.5*(sss/ddd)*0.5*(NNN*(-0.3)))	=	1.00
Jff	= Jgg*Jll*Jrr	=	1.00

Wood member embedding strength			
ggw	= mean relative density	=	0.63
fwpar	= 63*ggw*(1-0.01*ddd)	=	33.27 MPa
fwper	= 27.4*ggw*(1-0.01*ddd)	=	14.47 MPa

Steel plate embedding strength			
Fuu	= ultimate tensile strength of steel	=	0.00 MPa
phisteel	= resistance factor for steel member in bolted connection	=	0.90
phiwood	= resistance factor for wood member in bolted connection	=	0.70
Fsteel	= 1.5*(phisteel/phiwood)*Fuu	=	0.00 MPa
FA36	= ASTM A36 grade steel	=	574.00 MPa
FCSA	= CSA G40.21M steel, grade 300 W	=	646.00 MPa

Material Strengths			
l1f1	= side member thickness	=	19.00 mm
l1f2	= main member thickness	=	89.00 mm
fy	= bolt yield strength (Hex G15)	=	634.30 MPa
ff1	= embedding strength of side member (ASTM A36)	=	574.00 MPa
ff2	= embedding strength of main member	=	33.27 MPa
FFFF1	= 0.8*ff1	=	459.20 MPa

Failure Modes			
faila	= FFFF1*ddd*2*(l1f1/ddd)	=	138724 N
failb	= FFFF1*ddd*2*(ff2/ff1)*(l1f2/ddd)	=	37664 N
failc	= FFFF1*ddd*2*0.5*(ff2/ff1)*(l1f2/ddd)	=	18832 N
faild	= FFFF1*ddd*2*(@sqrt(0.6*(ff2/ff1+ff2)))*(fy/ff1)+0.2*(l1f1/ddd)	=	49871 N
faile	= FFFF1*ddd*2*(@sqrt(0.6*(ff2/ff1+ff2)))*(fy/ff1)+0.2*(l1f2/ddd)	=	152089 N
failf	= FFFF1*ddd*2*0.2*(l1f1/ddd)*(ff2/ff1)*(l1f2/ddd)	=	35278 N
failg	= FFFF1*ddd*2*(@sqrt((2/3)*(ff2/ff1+ff2)))*(fy/ff1))	=	23323 N

Results			
twonem	= @min(faila,failb,faild,failf,failg)	=	23323 N
threemem	= @min(faila,failc,faild,failg)	=	18832 N
nummem	= number of members: 0 for two, 1 for three	=	1
puu	= @if(nummem=0, twonem, @if(nummem=1, threemem, 0))*kdd*kt*ksf	=	21657 N
Prr	= phi*Fuu*nss*nff/Jff/1000	=	30.3 kN

Figure 46: Design calculations for one 15.9 mm bolt in 89x140 mm PSL members

BOLTED CONNECTION DESIGN SPREADSHEET: 89x140 mm specimens, ten 15.9 mm bolts		
Member Geometry		
l1l1	= member thickness	<input>
ddd	= bolt diameter	<input>
end	= number of bolt diameters from member end	<input>
ss	= ddd*bolt	<calc>
NNN	= bolt spacing in the row	<input>
row	= number of bolts in a row	<input>
nss	= number of rows	<input>
nff	= number of shear planes	<input>
	= NNN*row	<calc>
Modification Factors		
phi	= performance factor	<input>
Kdd	= load duration factor	<input>
Ksf	= service condition factor for fastening	<input>
Ktt	= treatment factor	<input>
Bolt Group Modification Factors		
Jrr	= @if(row=1,1.0,@if(row=2,0.8,@if(row=3,0.6,1.0)))	<calc>
Jll	= (0.25/(3*ddd))*end+1/6	<calc>
Jgg	= @if(ss=0,1.0,@if(0.33*(l1l1/ddd)*0.5*(ss/ddd)*0.2*(NNN*(-0.3))>1.0,1.0,0.33*(l1l1/ddd)*0.5*(ss/ddd)*0.2*(NNN*(-0.3)))	<calc>
Jff	= Jgg*Jll*Jrr	<calc>
Wood member embedding strength		
ggw	= mean relative density	<input>
fwpar	= 63*ggw*(1-0.01*ddd)	<calc>
fwper	= 27.4*ggw*(1-0.01*ddd)	<calc>
Steel plate embedding strength		
Fuu	= ultimate tensile strength of steel	<input>
phisteel	= resistance factor for steel member in bolted connection	<input>
phwood	= resistance factor for wood member in bolted connection	<input>
Esteel	= 1.5*(phisteel/phwood)*Fuu	<calc>
FA36	= ASTM A36 grade steel	<input>
FCSA	= CSA G40.21M steel, grade 300 W	<input>
Material Strengths		
l1l1	= side member thickness	<input>
l1l2	= main member thickness	<input>
fy	= bolt yield strength (Hex Gr5)	<input>
ff1	= embedding strength of side member (ASTM A36)	<input>
ff2	= embedding strength of main member	<input>
FFFF1	= 0.8*ff1	<calc>
Failure Modes		
faila	= FFFF1*ddd*2*(l1l1/ddd)	<calc>
failb	= FFFF1*ddd*2*(ff2/ff1)*(l1l2/ddd)	<calc>
failc	= FFFF1*ddd*2*0.5*(ff2/ff1)*(l1l2/ddd)	<calc>
faild	= FFFF1*ddd*2*(@sqrt(0.6*(ff2/ff1+ff2))*(fy/ff1))+0.2*(l1l1/ddd)	<calc>
faile	= FFFF1*ddd*2*(@sqrt(0.6*(ff2/ff1+ff2))*(fy/ff1))+0.2*(l1l2/ddd)	<calc>
failf	= FFFF1*ddd*2*0.2*(l1l1/ddd)-(ff2/ff1)*(l1l2/ddd)	<calc>
failg	= FFFF1*ddd*2*(@sqrt((2/3)*(ff2/ff1+ff2)*(fy/ff1)))	<calc>
Results		
twomem	= @min(faila, failb, faild, faile, failf, failg)	<calc>
threemem	= @min(faila, failc, faild, failg)	<calc>
nummem	= number of members: 0 for two, 1 for three	<input>
puu	= @if(nummem=0, twomem, @if(nummem=1, threemem, 0))*kdd*kt*ksf	<calc>
Prr	= phi*Puu*nss*nff/Jff/1000	<calc>

Figure 47: Design calculations for ten 15.9 mm bolt in 89x140 mm PSL members

BOLTED CONNECTION DESIGN SPREADSHEET: 133x191 mm specimens, one 22.2 mm bolt									
Member Geometry									
l1	=	member thickness	=	140.00 mm	<input>				<input>
ddd	=	bolt diameter	=	22.20 mm	<input>				<input>
bolt	=	number of bolt diameters from member end	=	10.00	<input>				<input>
end	=	ddd*bolt	=	222.00 mm	<calc>				<calc>
sss	=	bolt spacing in the row	=	0.00 mm	<input>				<input>
NNN	=	number of bolts in a row	=	1.00	<input>				<input>
row	=	number of rows	=	1.00	<input>				<input>
nss	=	number of shear planes	=	2.00	<input>				<input>
nff	=	NNN*row	=	1.00	<calc>				<calc>
Modification Factors									
phi	=	performance factor	=	0.70	<input>				<input>
Kdd	=	load duration factor	=	1.15	<input>				<input>
Ksf	=	service condition factor for fastening	=	1.00	<input>				<input>
Kt	=	treatment factor	=	1.00	<input>				<input>
Bolt Group Modification Factors									
Jrr	=	@if(row=1,1.0,@if(row=2,0.8,@if(row=3,0.6,1.0)))	=	1.00	<calc>				<calc>
Jll	=	(0.25/(3*ddd))*end+1/6	=	1.00	<calc>				<calc>
Jgg	=	@if(sss=0,1.0,@if(0.33*(l1/ddd)*0.5*(sss/ddd)*0.2*(NNN*(-0.3))>1.0,1.0,0.33*(l1/ddd)*0.5*(sss/ddd)*0.2*(NNN*(-0.3)))	=	1.00	<calc>				<calc>
Jff	=	Jgg*Jll*Jrr	=	1.00	<calc>				<calc>
Wood member embedding strength									
ggw	=	mean relative density	=	0.63	<input>				<input>
fwpar	=	63*ggw*(1-0.01*ddd)	=	30.78 MPa	<calc>				<calc>
fwper	=	27.4*ggw*(1-0.01*ddd)	=	13.39 MPa	<calc>				<calc>
Steel plate embedding strength									
Fuu	=	ultimate tensile strength of steel	=	0.00 MPa	<input>				<input>
phisteel	=	resistance factor for steel member in bolted connection	=	0.90	<input>				<input>
phiwood	=	resistance factor for wood member in bolted connection	=	0.70	<input>				<input>
Fsteel	=	1.5*(phisteel/phiwood)*Fuu	=	0.00 MPa	<calc>				<calc>
FA36	=	ASTM A36 grade steel	=	574.00 MPa	<input>				<input>
FCSA	=	CSA G40.21M steel, grade 300 W	=	646.00 MPa	<input>				<input>
Material Strengths									
l11	=	side member thickness	=	19.00 mm	<input>				<input>
l12	=	main member thickness	=	140.00 mm	<input>				<input>
fy	=	bolt yield strength (Hex Gr5)	=	634.30 MPa	<input>				<input>
ff1	=	embedding strength of side member (ASTM A36)	=	574.00 MPa	<input>				<input>
ff2	=	embedding strength of main member	=	30.78 MPa	<input>				<input>
FFFF1	=	0.8*ff1	=	459.20 MPa	<calc>				<calc>
Failure Modes									
faila	=	FFFF1*ddd*2*(l11/ddd)	=	193691 N	<calc>				<calc>
failb	=	FFFF1*ddd*2*(ff2/ff1)*(l12/ddd)	=	76531 N	<calc>				<calc>
failc	=	FFFF1*ddd*2*0.5*(ff2/ff1)*(l12/ddd)	=	38266 N	<calc>				<calc>
faild	=	FFFF1*ddd*2*(@sqrt(0.6*(ff2/(ff1+ff2)))+(fy/ff1))+0.2*(l11/ddd)	=	80311 N	<calc>				<calc>
faile	=	FFFF1*ddd*2*(@sqrt(0.6*(ff2/(ff1+ff2)))+(fy/ff1))+0.2*(l12/ddd)	=	327012 N	<calc>				<calc>
failf	=	FFFF1*ddd*2*0.2*(l11/ddd)+(ff2/ff1)*(l12/ddd)	=	54044 N	<calc>				<calc>
failg	=	FFFF1*ddd*2*(@sqrt((2/3)*(ff2/(ff1+ff2)))+(fy/ff1)))	=	43822 N	<calc>				<calc>
Results									
twomem	=	@min(faila,failb,faild,failf,failg)	=	43822 N	<calc>				<calc>
threemem	=	@min(faila,failc,faild,failg)	=	38266 N	<calc>				<calc>
nummem	=	number of members: 0 for two, 1 for three	=	1	<input>				<input>
puu	=	@if(nummem=0, twomem,@if(nummem=1, threemem, 0))*kd*kt*ksf	=	44008 N	<calc>				<calc>
Prr	=	phi*Puu*nss*nff*Jff/1000	=	61.6 kN	<calc>				<calc>

Figure 48: Design calculations for one 22.2 mm bolt in 133x191 mm PSL members

BOLTED CONNECTION DESIGN SPREADSHEET: 133x191 mm specimens, four 22.2 mm bolts									
Member Geometry									
l1	=	member thickness							<input>
ddd	=	bolt diameter							<input>
bolt	=	number of bolt diameters from member end							<input>
end	=	ddd*bolt							<calc>
sss	=	bolt spacing in the row							<input>
NNN	=	number of bolts in a row							<input>
row	=	number of rows							<input>
nss	=	number of shear planes							<input>
nff	=	NNN*row							<calc>
Modification Factors									
phi	=	performance factor							<input>
Kdd	=	load duration factor							<input>
Ksf	=	service condition factor for fastening							<input>
Ktt	=	treatment factor							<input>
Bolt Group Modification Factors									
Jrr	=	@if(row=1,1.0,@if(row=2,0.8,@if(row=3,0.6,1.0)))							<calc>
Jll	=	(0.25/(3*ddd))*end+1/6							<calc>
Jgg	=	@if(sss=0,1.0,@if(0.33*(l1/ddd)*0.5*(sss/ddd)*0.5*(l1/ddd)*0.2*(NNN*(-0.3))))							<calc>
Jff	=	Jgg*Jll*Jrr							<calc>
Wood member embedding strength									
ggw	=	mean relative density							<input>
fwpar	=	63*ggw*(1-0.01*ddd)							<calc>
fwper	=	27.4*ggw*(1-0.01*ddd)							<calc>
Steel plate embedding strength									
Fuu	=	ultimate tensile strength of steel							<input>
phisteel	=	resistance factor for steel member in bolted connection							<input>
phiwood	=	resistance factor for wood member in bolted connection							<input>
Fsteel	=	1.5*(phisteel/phiwood)*Fuu							<calc>
FA36	=	ASTM A36 grade steel							<input>
FCSA	=	CSA G40.21M steel, grade 300 W							<input>
Material Strengths									
l1l1	=	side member thickness							<input>
l1l2	=	main member thickness							<input>
fy	=	bolt yield strength (Hex Gr5)							<input>
ff1	=	embedding strength of side member (ASTM A36)							<input>
ff2	=	embedding strength of main member							<input>
FFFF1	=	0.8*ff1							<calc>
Failure Modes									
faila	=	FFFF1*ddd*2*(l1l1/ddd)							<calc>
failb	=	FFFF1*ddd*2*(ff2/ff1)*(l1l2/ddd)							<calc>
failc	=	FFFF1*ddd*2*0.5*(ff2/ff1)*(l1l2/ddd)							<calc>
faild	=	FFFF1*ddd*2*(@sqrt(0.6*(ff2/ff1+ff2)))*(fy/ff1)+0.2*(l1l1/ddd)							<calc>
faile	=	FFFF1*ddd*2*(@sqrt(0.6*(ff2/ff1+ff2)))*(fy/ff1)+0.2*(l1l2/ddd)							<calc>
failf	=	FFFF1*ddd*2*0.2*(l1l1/ddd)+(ff2/ff1)*(l1l2/ddd)							<calc>
failg	=	FFFF1*ddd*2*(@sqrt((2/3)*(ff2/ff1+ff2)))*(fy/ff1))							<calc>
Results									
twonem	=	@min(faila,failb,faild,failf,failg)							<calc>
threemem	=	@min(faila,failc,faild,failg)							<calc>
nummem	=	number of members: 0 for two, 1 for three							<input>
puu	=	@if(nummem=0, twonem, @if(nummem=1, threemem, 0))*Kdd*Ktt*Ksf							<calc>
Prr	=	phi*Puu*nss*nff/1000							<calc>

Figure 49: Design calculations for four 22.2 mm bolts in 133x191 mm PSL members

APPENDIX II

NET TENSILE RESISTANCE CALCULATIONS FOR PARALLEL TO GRAIN LOADING

Figures 50 through 52 show the design code calculations (CAN/CSA 086.1-94) for the net tensile strength of the PSL members for parallel to grain loading. A tension parallel to grain strength value of 20.2 MPa taken from M-26 machine evaluated lumber was chosen to estimate parallel to grain strength resistance for the PSL, as there are no limit state strength values tabulated for PSL. M-26 lumber was chosen because it has the same elastic modulus value (E) as PSL (13 800 MPa).

Tension Members: 38x140 mm Specimens-One 12.7 mm Bolt

System Geometry				
G	Grade: Assuming M-26 Machine Evaluated Lumber			
fti	= Tensile Strength	=	20.20 MPa	<input>
ddd	= Depth of section	=	38.00 mm	<input>
bbb	= Width of section	=	140.00 mm	<input>

Modification Factors				
KD	= Load duration factor	=	1.15	<input>
KH	= System factor	=	1.00	<input>
KSt	= Service condition factor for tension	=	1.00	<input>
KT	= Treatment factor	=	1.00	<input>
KZt	= Size factor for parallel to grain loading	=	1.00	<input>

Member Strengths				
Ft	= fti*KD*KH*KSt*KT*KZt	=	23.23 Mpa	<calc>

Tension Resistance				
Trr	= $0.9 \cdot F_t \cdot B \cdot D / 1000$	=	111.23 kN	<calc>
bolt	= Bolt Diameter	=	12.70 mm	<input>
nbc	= Number of Bolts in Column	=	1.00	<input>
netbbb	= $(b - n \cdot b)$	=	127.30 mm	<calc>
netarea	= $d \cdot \text{net}b$	=	4837.40 mm ²	<calc>
gross	= $d \cdot b$	=	5320.00 mm ²	<calc>
AreaChk	= @if($\text{netarea}/\text{gross} \geq 0.75$, "OK", "FAIL")	=	OK	<calc>
Trn	= $T_r \cdot \text{netarea}/\text{gross}$	=	101.14 kN	<calc>

Tension Members: 38x140 mm Specimens-Two Rows of 12.7 mm Bolts

System Geometry				
G	Grade: Assuming M-26 Machine Evaluated Lumber			
fti	= Tensile Strength	=	20.20 MPa	<input>
ddd	= Depth of section	=	38.00 mm	<input>
bbb	= Width of section	=	140.00 mm	<input>

Modification Factors				
KD	= Load duration factor	=	1.15	<input>
KH	= System factor	=	1.00	<input>
KSt	= Service condition factor for tension	=	1.00	<input>
KT	= Treatment factor	=	1.00	<input>
KZt	= Size factor for parallel to grain loading	=	1.00	<input>

Member Strengths				
Ft	= fti*KD*KH*KSt*KT*KZt	=	23.23 Mpa	<calc>

Tension Resistance				
Trr	= $0.9 \cdot F_t \cdot B \cdot D / 1000$	=	111.23 kN	<calc>
bolt	= Bolt Diameter	=	12.70 mm	<input>
nbc	= Number of Bolts in Column	=	2.00	<input>
netbbb	= $(b - n \cdot b)$	=	114.60 mm	<calc>
netarea	= $d \cdot \text{net}b$	=	4354.80 mm ²	<calc>
gross	= $d \cdot b$	=	5320.00 mm ²	<calc>
AreaChk	= @if($\text{netarea}/\text{gross} \geq 0.75$, "OK", "FAIL")	=	OK	<calc>
Trn	= $T_r \cdot \text{netarea}/\text{gross}$	=	91.05 kN	<calc>

Figure 50: Net Tensile Resistance for Parallel to Grain Loading of 38x140 mm Specimens (CAN/CSA 086.1-94)

Tension Members: 89x140 mm Specimens-One 15.9 mm Bolt

System Geometry				
G	Grade: Assuming M-26 Machine Evaluated Lumber			
fti	= Tensile Strength	=	20.20 MPa	<input>
ddd	= Depth of section	=	89.00 mm	<input>
bbb	= Width of section	=	140.00 mm	<input>

Modification Factors				
KD	= Load duration factor	=	1.15	<input>
KH	= System factor	=	1.00	<input>
KSt	= Service condition factor for tension	=	1.00	<input>
KT	= Treatment factor	=	1.00	<input>
KZt	= Size factor for parallel to grain loading	=	1.00	<input>

Member Strengths				
Ft	= $f_{ti} \cdot K_D \cdot K_H \cdot K_{St} \cdot K_T \cdot K_{Zt}$	=	23.23 Mpa	<calc>

Tension Resistance				
Trr	= $0.9 \cdot F_t \cdot B \cdot D / 1000$	=	260.50 kN	<calc>
bolt	= Bolt Diameter	=	15.90 mm	<input>
nbc	= Number of Bolts in Column	=	1.00	<input>
netbbb	= $(b - nbc \cdot bolt)$	=	124.10 mm	<calc>
netarea	= $d \cdot netbbb$	=	11044.90 mm ²	<calc>
gross	= $d \cdot b$	=	12460.00 mm ²	<calc>
AreaChk	= @if($netarea/gross \geq 0.75$, "OK", "FAIL")	=	OK	<calc>
Trn	= $Trr \cdot netarea/gross$	=	230.92 kN	<calc>

Tension Members: 89x140 mm Specimens-Two Rows of 15.9 mm Bolts

System Geometry				
G	Grade: Assuming M-26 Machine Evaluated Lumber			
fti	= Tensile Strength	=	20.20 MPa	<input>
ddd	= Depth of section	=	89.00 mm	<input>
bbb	= Width of section	=	140.00 mm	<input>

Modification Factors				
KD	= Load duration factor	=	1.15	<input>
KH	= System factor	=	1.00	<input>
KSt	= Service condition factor for tension	=	1.00	<input>
KT	= Treatment factor	=	1.00	<input>
KZt	= Size factor for parallel to grain loading	=	1.00	<input>

Member Strengths				
Ft	= $f_{ti} \cdot K_D \cdot K_H \cdot K_{St} \cdot K_T \cdot K_{Zt}$	=	23.23 Mpa	<calc>

Tension Resistance				
Trr	= $0.9 \cdot F_t \cdot B \cdot D / 1000$	=	260.50 kN	<calc>
bolt	= Bolt Diameter	=	15.90 mm	<input>
nbc	= Number of Bolts in Column	=	2.00	<input>
netbbb	= $(b - nbc \cdot bolt)$	=	108.20 mm	<calc>
netarea	= $d \cdot netbbb$	=	9629.80 mm ²	<calc>
gross	= $d \cdot b$	=	12460.00 mm ²	<calc>
AreaChk	= @if($netarea/gross \geq 0.75$, "OK", "FAIL")	=	OK	<calc>
Trn	= $Trr \cdot netarea/gross$	=	201.33 kN	<calc>

Figure 51: Net Tensile Resistance for Parallel to Grain Loading of 89x140 mm Specimens (CAN/CSA 086.1-94)

Tension Members: 133x191 mm Specimens-One 22.2 mm Bolt

System Geometry				
G	Grade	2.0E Doug - Fir PSL		
fti	= Tensile Strength	=	20.20 MPa	<input>
ddd	= Depth of section	=	133.00 mm	<input>
bbb	= Width of section	=	191.00 mm	<input>

Modification Factors				
KD	= Load duration factor	=	1.15	<input>
KH	= System factor	=	1.00	<input>
KSt	= Service condition factor for tension	=	1.00	<input>
KT	= Treatment factor	=	1.00	<input>
KZt	= Size factor for parallel to grain loading	=	1.00	<input>

Member Strengths				
Ft	= fti*KD*KH*KSt*KT*KZt	=	23.23 Mpa	<calc>

Tension Resistance				
Trr	= $0.9 \cdot Ft \cdot BBB \cdot DDD / 1000$	=	531.10 kN	<calc>
bolt	= Bolt Diameter	=	22.20 mm	<input>
nbc	= Number of Bolts in Column	=	1.00	<input>
netbbb	= $(bbb - nbc \cdot bolt)$	=	168.80 mm	<calc>
netarea	= $ddd \cdot netbbb$	=	22450.40 mm ²	<calc>
gross	= $ddd \cdot bbb$	=	25403.00 mm ²	<calc>
AreaChk	= @if(netarea/gross>=0.75,"OK","FAIL")	=	OK	<calc>
Trn	= $Trr \cdot netarea / gross$	=	469.37 kN	<calc>

Tension Members: 133x191 mm Specimens-Two Rows of 22.2 mm Bolts

System Geometry				
G	Grade: Assuming M-26 Machine Evaluated Lumber			
fti	= Tensile Strength	=	20.20 MPa	<input>
ddd	= Depth of section	=	133.00 mm	<input>
bbb	= Width of section	=	191.00 mm	<input>

Modification Factors				
KD	= Load duration factor	=	1.15	<input>
KH	= System factor	=	1.00	<input>
KSt	= Service condition factor for tension	=	1.00	<input>
KT	= Treatment factor	=	1.00	<input>
KZt	= Size factor for parallel to grain loading	=	1.00	<input>

Member Strengths				
Ft	= fti*KD*KH*KSt*KT*KZt	=	23.23 Mpa	<calc>

Tension Resistance				
Trr	= $0.9 \cdot Ft \cdot BBB \cdot DDD / 1000$	=	531.10 kN	<calc>
bolt	= Bolt Diameter	=	22.20 mm	<input>
nbc	= Number of Bolts in Column	=	2.00	<input>
netbbb	= $(bbb - nbc \cdot bolt)$	=	146.60 mm	<calc>
netarea	= $ddd \cdot netbbb$	=	19497.80 mm ²	<calc>
gross	= $ddd \cdot bbb$	=	25403.00 mm ²	<calc>
AreaChk	= @if(netarea/gross>=0.75,"OK","FAIL")	=	OK	<calc>
Trn	= $Trr \cdot netarea / gross$	=	407.64 kN	<calc>

Figure 52: Net Tensile Resistance for Parallel to Grain Loading of 133x191 mm Specimens (CAN/CSA 086.1-94)

APPENDIX III

STATISTIC CALCULATIONS FOR BOLTED TIMBER CONNECTIONS

Figure 53 shows the statistic calculations for the strengths of the connections tested in this experiment.

Test Specimen-Ultimate Strength (kN)											
Member	Size	Single/Multi	Unrein/Rein	1	2	3	4	5	6	7	Average
AUS	38x140	S	U	24.76	21.93	25.08	26.82	28.56	15.42	29.22	24.89
ARS	38x140	S	R	33.66	35.07	33.01	30.51	30.73	35.62	34.75	33.97
AUM	38x140	M	R	130.53	132.27	124.78	111.10	95.35	132.39	139.55	123.83
ARM	38x140	M	R	108.92	110.23	118.80	110.01	142.37	122.93	140.63	121.71
BUS	89x140	S	U	77.30	71.57	72.65	75.91	86.44	79.60	70.15	76.39
BRS	89x140	S	R	96.11	79.49	90.35	80.25	81.34	81.99	82.65	85.56
BUM	89x140	M	U	339.21	352.36	383.88	381.39	327.64	388.87	390.68	372.50
BRM	89x140	M	R	337.40	361.88	346.69	325.60	347.82	447.59	394.31	364.83
CUS	133x191	S	U	115.87	180.28	154.43	164.09	155.29	163.44	168.65	152.87
CRS	133x191	S	R	133.14	167.35	192.87	153.12	165.94	141.83	174.74	157.82
CUM	133x191	M	U	413.12	380.03	372.77	419.70	415.17	410.40	364.38	401.84
CRM	133x191	M	R	436.03	452.36	428.95	427.77	427.87	417.89	438.29	427.97

Statistic Calculations for Connection Strengths											
Spec	Ult	Mean	(k-m) ²	1	2	3	4	5	6	7	Average
1	24.76	24.89	0.02	33.66	35.07	33.01	30.51	30.73	35.62	34.75	33.97
2	21.93	24.89	8.76	35.07	33.01	30.51	30.73	30.73	35.62	34.75	33.97
3	25.08	24.89	0.04	33.01	30.51	30.73	30.73	30.73	35.62	34.75	33.97
4	26.82	24.89	3.73	30.51	30.73	30.73	30.73	30.73	35.62	34.75	33.97
5	28.56	24.89	13.48	30.73	30.73	30.73	30.73	30.73	35.62	34.75	33.97
6	15.42	24.89	89.66	35.62	34.75	33.97	33.97	33.97	35.62	34.75	33.97
7	29.22	24.89	18.76	34.75	33.97	33.97	33.97	33.97	35.62	34.75	33.97
8	24.21	24.89	0.46	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
9	25.09	24.89	0.04	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
10	27.80	24.89	8.47	35.62	33.97	33.97	33.97	33.97	35.62	34.75	33.97
Sum(k-m) ²				143.41	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Mean				24.89	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Std Dev				3.79	1.88	1.88	1.88	1.88	1.88	1.88	1.88
COV				15.22%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%

Statistic Calculations for Connection Strengths											
Spec	Ult	Mean	(k-m) ²	1	2	3	4	5	6	7	Average
1	24.76	24.89	0.02	33.66	35.07	33.01	30.51	30.73	35.62	34.75	33.97
2	21.93	24.89	8.76	35.07	33.01	30.51	30.73	30.73	35.62	34.75	33.97
3	25.08	24.89	0.04	33.01	30.51	30.73	30.73	30.73	35.62	34.75	33.97
4	26.82	24.89	3.73	30.51	30.73	30.73	30.73	30.73	35.62	34.75	33.97
5	28.56	24.89	13.48	30.73	30.73	30.73	30.73	30.73	35.62	34.75	33.97
6	15.42	24.89	89.66	35.62	34.75	33.97	33.97	33.97	35.62	34.75	33.97
7	29.22	24.89	18.76	34.75	33.97	33.97	33.97	33.97	35.62	34.75	33.97
8	24.21	24.89	0.46	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
9	25.09	24.89	0.04	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
10	27.80	24.89	8.47	35.62	33.97	33.97	33.97	33.97	35.62	34.75	33.97
Sum(k-m) ²				143.41	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Mean				24.89	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Std Dev				3.79	1.88	1.88	1.88	1.88	1.88	1.88	1.88
COV				15.22%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%

Statistic Calculations for Connection Strengths											
Spec	Ult	Mean	(k-m) ²	1	2	3	4	5	6	7	Average
1	24.76	24.89	0.02	33.66	35.07	33.01	30.51	30.73	35.62	34.75	33.97
2	21.93	24.89	8.76	35.07	33.01	30.51	30.73	30.73	35.62	34.75	33.97
3	25.08	24.89	0.04	33.01	30.51	30.73	30.73	30.73	35.62	34.75	33.97
4	26.82	24.89	3.73	30.51	30.73	30.73	30.73	30.73	35.62	34.75	33.97
5	28.56	24.89	13.48	30.73	30.73	30.73	30.73	30.73	35.62	34.75	33.97
6	15.42	24.89	89.66	35.62	34.75	33.97	33.97	33.97	35.62	34.75	33.97
7	29.22	24.89	18.76	34.75	33.97	33.97	33.97	33.97	35.62	34.75	33.97
8	24.21	24.89	0.46	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
9	25.09	24.89	0.04	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
10	27.80	24.89	8.47	35.62	33.97	33.97	33.97	33.97	35.62	34.75	33.97
Sum(k-m) ²				143.41	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Mean				24.89	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Std Dev				3.79	1.88	1.88	1.88	1.88	1.88	1.88	1.88
COV				15.22%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%

Statistic Calculations for Connection Strengths											
Spec	Ult	Mean	(k-m) ²	1	2	3	4	5	6	7	Average
1	24.76	24.89	0.02	33.66	35.07	33.01	30.51	30.73	35.62	34.75	33.97
2	21.93	24.89	8.76	35.07	33.01	30.51	30.73	30.73	35.62	34.75	33.97
3	25.08	24.89	0.04	33.01	30.51	30.73	30.73	30.73	35.62	34.75	33.97
4	26.82	24.89	3.73	30.51	30.73	30.73	30.73	30.73	35.62	34.75	33.97
5	28.56	24.89	13.48	30.73	30.73	30.73	30.73	30.73	35.62	34.75	33.97
6	15.42	24.89	89.66	35.62	34.75	33.97	33.97	33.97	35.62	34.75	33.97
7	29.22	24.89	18.76	34.75	33.97	33.97	33.97	33.97	35.62	34.75	33.97
8	24.21	24.89	0.46	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
9	25.09	24.89	0.04	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
10	27.80	24.89	8.47	35.62	33.97	33.97	33.97	33.97	35.62	34.75	33.97
Sum(k-m) ²				143.41	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Mean				24.89	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Std Dev				3.79	1.88	1.88	1.88	1.88	1.88	1.88	1.88
COV				15.22%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%

Statistic Calculations for Connection Strengths											
Spec	Ult	Mean	(k-m) ²	1	2	3	4	5	6	7	Average
1	24.76	24.89	0.02	33.66	35.07	33.01	30.51	30.73	35.62	34.75	33.97
2	21.93	24.89	8.76	35.07	33.01	30.51	30.73	30.73	35.62	34.75	33.97
3	25.08	24.89	0.04	33.01	30.51	30.73	30.73	30.73	35.62	34.75	33.97
4	26.82	24.89	3.73	30.51	30.73	30.73	30.73	30.73	35.62	34.75	33.97
5	28.56	24.89	13.48	30.73	30.73	30.73	30.73	30.73	35.62	34.75	33.97
6	15.42	24.89	89.66	35.62	34.75	33.97	33.97	33.97	35.62	34.75	33.97
7	29.22	24.89	18.76	34.75	33.97	33.97	33.97	33.97	35.62	34.75	33.97
8	24.21	24.89	0.46	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
9	25.09	24.89	0.04	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
10	27.80	24.89	8.47	35.62	33.97	33.97	33.97	33.97	35.62	34.75	33.97
Sum(k-m) ²				143.41	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Mean				24.89	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Std Dev				3.79	1.88	1.88	1.88	1.88	1.88	1.88	1.88
COV				15.22%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%

Statistic Calculations for Connection Strengths											
Spec	Ult	Mean	(k-m) ²	1	2	3	4	5	6	7	Average
1	24.76	24.89	0.02	33.66	35.07	33.01	30.51	30.73	35.62	34.75	33.97
2	21.93	24.89	8.76	35.07	33.01	30.51	30.73	30.73	35.62	34.75	33.97
3	25.08	24.89	0.04	33.01	30.51	30.73	30.73	30.73	35.62	34.75	33.97
4	26.82	24.89	3.73	30.51	30.73	30.73	30.73	30.73	35.62	34.75	33.97
5	28.56	24.89	13.48	30.73	30.73	30.73	30.73	30.73	35.62	34.75	33.97
6	15.42	24.89	89.66	35.62	34.75	33.97	33.97	33.97	35.62	34.75	33.97
7	29.22	24.89	18.76	34.75	33.97	33.97	33.97	33.97	35.62	34.75	33.97
8	24.21	24.89	0.46	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
9	25.09	24.89	0.04	35.19	33.97	33.97	33.97	33.97	35.62	34.75	33.97
10	27.80	24.89	8.47	35.62	33.97	33.97	33.97	33.97	35.62	34.75	33.97
Sum(k-m) ²				143.41	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Mean				24.89	33.97	33.97	33.97	33.97	33.97	33.97	33.97
Std Dev				3.79	1.88	1.88	1.88	1.88	1.88	1.88	1.88
COV				15.22%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%	5.49%

Statistic Calculations for Connection Strengths											
Spec	Ult	Mean	(k-m) ²	1	2	3	4	5	6	7	Average
1	24.76	24.89	0.02	33.66	35.07	33.01	30.51	30.73	35.62	34.75	33.97
2	21.93	24.89	8.76	35.07	33.01	30.51	30.73	30.73	35.62	34.75	33.97
3	25.08	24.89	0.04	33.01	30.51	30.73	30.73	30.73	35.62	34.75	33.97
4	26.82	24.89	3.73	30.51	30.73	30.73	30.73	30.73	35.62	34.75	33.97
5	28.56	24.89	13.48	30.73	30.73	30.73	30.73	30.73</			

APPENDIX IV
FAILURE MODE PICTURES

Figures 54 through 59 show failure mode pictures of the connections tested.

Figure 54: Failure Modes For Single Bolt Connections in the 38x140 mm Specimens

Figure 55: Failure Modes For Ten Bolt Connections in the 38x140 mm Specimens

Figure 56: Failure Modes For Single Bolt Connection in the 89x140 mm Specimens

Figure 57: Failure Modes For Ten Bolt Connection in the 89x140 mm Specimens

Figure 58: Failure Modes For Single Bolt Connection in the 133x191 mm Specimens

Figure 59: Failure Modes For Four Bolt Connection in the 133x191 mm Specimens

APPENDIX V

DUCTILITY AND STIFFNESS CALCULATIONS FOR BOLTED TIMBER CONNECTIONS

Figures 60 through 62 show the calculations used to determine the stiffness and the ductility factor for the connections tested in this study.

DUCTILITY AND STIFFNESS CALCULATIONS FOR 38X140 MM SPECIMENS

Single Bolt Connections-Unreinforced

Single Bolt Connections-Unreinforced										Stiffness (kN/mm)		
Specimen	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltafult	80%Fult	Delta80	Ductility	K50	K70	Average K(K50+K70)
aus1	12.38	0.59	17.33	0.89	24.76	2.55	19.81	4.05	6.9	21.0	19.5	20.2
aus2	10.97	0.81	15.35	1.29	21.93	3.25	17.54	5.65	7.0	13.5	11.9	12.7
aus3	12.54	1.17	17.56	1.50	25.08	3.50	20.06	5.38	4.6	10.7	11.7	11.2
aus4	13.41	0.62	18.77	0.89	26.82	2.46	21.46	3.27	5.3	21.6	21.1	21.4
aus5	14.28	0.29	19.99	0.35	28.56	1.52	22.85	3.67	12.7	49.2	57.1	53.2
aus6	7.71	1.05	10.79	1.46	15.42	3.95	12.34	7.35	7.0	7.3	7.4	7.4
aus7	14.61	0.79	20.45	0.95	29.22	2.05	23.38	3.21	4.1	18.5	21.5	20.0
aus8	12.11	0.65	16.95	0.92	24.21	2.48	19.37	4.47	6.9	18.6	18.4	18.5
aus9	12.55	0.92	17.56	1.08	25.09	2.22	20.07	3.15	3.4	13.6	16.3	15.0
aus10	13.90	0.92	19.46	1.14	27.80	2.52	22.24	2.98	3.2	15.1	17.1	16.1
AVG>									6.1	18.9	20.2	19.6

Single Bolt Connections-Reinforced

Single Bolt Connections-Reinforced										Stiffness (kN/mm)		
Specimen	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltafult	80%Fult	Delta80	Ductility	K50	K70	Average K(K50+K70)
ars1	16.83	1.27	23.56	1.66	33.66	4.66	26.93	30.00	23.6	13.3	14.2	13.7
ars2	17.54	1.04	24.55	1.60	35.07	6.23	28.06	30.00	28.8	16.9	15.3	16.1
ars3	16.51	2.38	23.11	2.84	33.01	6.25	26.40	30.00	12.6	6.9	8.1	7.5
ars4	15.23	1.00	21.36	1.43	30.51	6.10	24.40	30.00	30.0	15.2	14.9	15.1
ars5	15.37	1.08	21.51	1.48	30.73	3.66	24.58	30.00	27.8	14.2	14.5	14.4
ars6	17.81	1.79	24.93	2.31	35.62	6.10	28.50	30.00	16.8	9.9	10.8	10.4
ars7	17.38	1.05	24.33	1.60	34.75	7.10	27.80	30.00	28.6	16.6	15.2	15.9
ars8	17.76	0.99	24.86	1.27	35.51	6.05	28.41	30.00	30.3	17.9	19.6	18.8
ars9	17.60	2.98	24.63	3.69	35.19	5.50	28.15	30.00	10.1	5.9	6.7	6.3
ars10	17.81	1.35	24.93	1.98	35.62	5.85	28.50	30.00	22.2	13.2	12.6	12.9
AVG>									23.1	13.0	13.2	13.1
Avg Improv>									278.5%	-31.3%	-34.6%	-33.0%

Ten Bolt Connections-Unreinforced

Ten Bolt Connections-Unreinforced										Stiffness (kN/mm)		
Specimen	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltafult	80%Fult	Delta80	Ductility	K50	K70	Average K(K50+K70)
aum1	65.27	0.81	91.37	0.98	130.53	1.45	104.42	2.03	2.5	80.6	93.2	86.9
aum2	66.14	1.36	92.59	1.69	132.27	2.16	105.82	2.58	1.9	48.6	54.8	51.7
aum3	62.39	1.18	87.35	1.40	124.78	2.03	99.82	2.59	2.2	52.9	62.4	57.6
aum4	55.55	0.48	77.77	0.60	111.10	0.94	88.88	1.30	2.7	115.7	129.6	122.7
aum5	47.68	0.72	66.75	0.86	95.35	1.27	76.28	1.56	2.2	66.2	77.6	71.9
aum6	66.20	0.66	92.67	0.84	132.39	1.14	105.91	1.66	2.5	100.3	110.3	105.3
aum7	69.78	0.97	97.69	1.16	139.55	1.80	111.64	2.33	2.4	71.9	84.2	78.1
aum8	64.67	1.38	90.54	1.62	129.34	2.16	103.47	2.60	1.9	46.9	55.9	51.4
aum9	62.28	0.76	87.19	0.95	124.56	1.26	99.65	1.70	2.2	81.9	91.8	86.9
aum10	59.19	0.97	82.87	1.16	118.38	1.79	94.70	2.25	2.3	61.0	71.4	66.2
AVG>									2.3	72.6	83.1	77.9

Ten Bolt Connections-Reinforced

Ten Bolt Connections-Reinforced										Stiffness (kN/mm)			
Specimen	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltafult	80%Fult	Delta80	Ductility	K50	K70	Average K(K50+K70)	
arm1	54.46	2.29	76.24	2.73	108.92	4.69	87.14	14.63	6.4	23.8	27.9	25.9	
arm2	55.12	1.31	77.16	1.88	110.23	3.50	88.18	8.53	6.5	42.1	41.0	41.6	
arm3	59.40	2.19	83.16	2.60	118.80	4.87	95.04	16.93	7.7	27.1	32.0	29.6	
arm4	55.01	1.95	77.01	2.35	110.01	5.10	88.01	31.27	16.0	28.2	32.8	30.5	
arm5	71.19	1.57	99.66	2.02	142.37	3.29	113.90	23.98	15.3	45.3	49.3	47.3	
arm6	61.47	1.95	86.05	2.28	122.93	4.82	98.34	6.84	3.5	31.5	37.7	34.6	
arm7	70.32	1.53	98.44	2.02	140.63	4.25	112.50	13.01	8.5	46.0	48.7	47.3	
arm8	60.38	2.08	84.53	2.51	120.76	3.75	96.61	5.21	2.5	29.0	33.7	31.4	
arm9	61.74	2.18	86.44	2.83	123.48	6.05	98.78	18.68	8.6	28.3	30.5	29.4	
arm10	59.46	2.19	83.24	2.74	118.92	6.20	95.14	13.68	6.2	27.2	30.4	28.8	
AVG->									8.1	32.9	36.4	34.6	
Avg Improv->									256.0%	-54.8%	-56.2%	-55.5%	

**Fult Values in (kN), Delta Values in (mm), Ductility Values in (%)

Figure 60: Ductility and Stiffness Calculations for 38x140 mm Specimens

DUCTILITY AND STIFFNESS CALCULATIONS FOR 89X140 MM SPECIMENS

Single Bolt Connections-Unreinforced										Stiffness (kN/mm)		
Specimen	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltault	80%Fult	Delta80	Ductility	K50	K70	Average K(K50+K70)
bus1	38.65	1.75	54.11	2.45	77.30	4.45	61.84	9.95	5.7	22.1	22.1	22.1
bus2	35.79	2.33	50.10	2.93	71.57	7.50	57.26	10.23	4.4	15.4	17.1	16.2
bus3	36.33	2.00	50.86	2.73	72.65	5.36	58.12	7.36	3.7	18.2	18.6	18.4
bus4	37.96	1.51	53.14	2.44	75.91	6.60	60.73	9.14	6.1	25.1	21.8	23.5
bus5	43.22	1.48	60.51	2.54	86.44	5.35	69.15	7.58	5.1	29.2	23.8	26.5
bus6	39.80	1.79	55.72	2.74	79.60	6.36	63.68	12.79	7.1	22.2	20.3	21.3
bus7	35.08	1.67	49.11	2.46	70.15	5.22	56.12	14.80	8.9	21.0	20.0	20.5
bus8	41.33	1.76	57.86	2.77	82.65	6.36	66.12	8.71	4.9	23.5	20.9	22.2
bus9	42.41	1.51	59.37	2.40	84.82	5.30	67.86	8.27	5.5	28.1	24.7	26.4
bus10	31.39	1.70	43.94	2.46	62.77	5.29	50.22	6.32	3.7	18.5	17.9	18.2
AVG>									5.5	22.3	20.7	21.5

Single Bolt Connections-Reinforced										Stiffness (kN/mm)		
Specimen	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltault	80%Fult	Delta80	Ductility	K50	K70	Average K(K50+K70)
brs1	48.06	2.37	67.28	3.79	96.11	8.99	76.89	27.90	11.8	20.3	17.8	19.0
brs2	39.75	1.89	55.64	2.97	79.49	7.26	63.59	27.10	14.3	21.0	18.7	19.9
brs3	45.18	1.90	63.25	3.04	90.35	8.73	72.28	15.42	8.1	23.8	20.8	22.3
brs4	40.13	1.82	56.18	3.07	80.25	7.69	64.20	30.90	17.0	22.0	18.3	20.2
brs5	40.67	3.21	56.94	4.90	81.34	9.56	65.07	26.24	8.2	12.7	11.6	12.1
brs6	41.00	2.40	57.39	3.58	81.99	7.29	65.59	29.92	12.5	17.1	16.0	16.6
brs7	40.62	2.12	56.86	3.46	81.23	9.83	64.98	36.40	17.2	19.2	16.4	17.8
brs8	44.91	2.49	62.87	4.03	89.81	9.65	71.85	18.86	7.6	18.0	15.6	16.8
brs9	39.91	2.43	55.87	4.03	79.82	9.64	63.86	19.74	8.1	16.4	13.9	15.1
brs10	47.82	2.29	66.67	3.50	95.24	7.96	76.19	18.22	8.0	20.8	19.0	19.9
AVG>									11.3	19.1	16.8	18.0
Avg Improv>									104.5%	-14.3%	-18.8%	-16.5%

Ten Bolt Connections-Unreinforced										Stiffness (kN/mm)		
Specimen	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltault	80%Fult	Delta80	Ductility	K50	K70	Average K(K50+K70)
bum1	169.61	0.85	237.45	1.08	339.21	1.71	271.37	3.13	3.7	199.5	219.9	209.7
bum2	176.18	0.98	246.65	1.24	352.36	1.92	281.89	3.33	3.4	179.8	198.9	189.3
bum3	191.94	0.90	268.72	1.22	383.88	1.95	307.10	3.56	4.0	213.3	220.3	216.8
bum4	190.70	0.95	266.97	1.26	381.39	1.80	305.11	3.39	3.6	200.7	211.9	206.3
bum5	163.82	0.85	229.35	1.11	327.64	1.89	262.11	3.53	4.2	192.7	206.6	199.7
bum6	194.44	1.06	272.21	1.33	388.87	1.90	311.10	3.57	3.4	183.4	204.7	194.0
bum7	195.34	0.96	273.48	1.25	390.68	2.11	312.54	4.20	4.4	203.5	218.8	211.1
bum8	195.91	1.08	274.27	1.41	391.81	2.39	313.45	3.96	3.7	181.4	194.5	188.0
bum9	180.04	0.95	252.05	1.26	360.07	1.84	288.06	3.63	3.8	189.5	200.0	194.8
bum10	204.52	0.78	286.33	1.05	409.04	1.65	327.23	3.39	4.3	262.2	272.7	267.4
AVG>									3.8	200.6	214.8	207.7

Ten Bolt Connections-Reinforced										Stiffness (kN/mm)		
Specimen	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltault	80%Fult	Delta80	Ductility	K50	K70	Average K(K50+K70)
brm1	168.70	1.31	236.18	1.72	337.40	3.60	269.92	11.34	8.7	128.8	137.3	133.0
brm2	180.94	1.20	253.32	1.54	361.88	2.18	289.50	3.88	3.2	150.8	164.5	157.6
brm3	173.35	1.32	242.68	1.65	346.69	2.37	277.35	3.7	2.8	131.3	147.1	139.2
brm4	162.80	1.39	227.92	1.88	325.60	3.24	260.48	5.85	4.2	117.1	121.2	119.2
brm5	173.91	1.25	243.47	1.64	347.82	2.38	278.26	3.84	3.1	139.1	148.5	143.8
brm6	223.80	1.60	313.31	2.04	447.59	3.01	358.07	4.76	3.0	139.9	153.6	146.7
brm7												
brm8	197.16	1.41	276.02	1.81	394.31	2.76	315.45	4.99	3.5	139.8	152.5	146.2
brm9	187.29	1.20	262.21	1.58	374.58	2.33	299.66	3.57	3.0	156.1	166.0	161.0
brm10	174.25	1.32	243.95	1.85	348.50	3.98	278.80	6.43	4.9	132.0	131.9	131.9
AVG>									4.0	137.2	146.9	142.1
Avg Improv>									5.3%	-31.6%	-31.6%	-31.6%

**Fult Values in (kN), Delta Values in (mm), Ductility Values in (%)

Figure 61: Ductility and Stiffness Calculations for 89x140 mm Specimens

DUCTILITY AND STIFFNESS CALCULATIONS FOR 133x191 MM SPECIMENS

Single Bolt Connections-Unreinforced

Specimen	Stiffness (kN/mm)								Ductility			Average K(K50+K70)
	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltault	80%Fult	Delta80	K50	K70		
cus1	57.34	2.74	81.11	3.41	116.87	6.80	92.70	8.21	3.0	21.1	23.8	22.5
cus2	90.14	2.17	126.20	3.10	180.28	7.76	144.22	11.87	5.5	41.5	40.7	41.1
cus3	77.22	2.27	108.10	3.38	154.43	7.20	123.54	9.27	4.1	34.0	32.0	33.0
cus4	82.05	2.64	114.86	3.81	164.09	9.05	131.27	10.51	4.0	31.1	30.1	30.6
cus5	77.65	1.62	108.70	2.46	155.29	6.50	124.23	10.02	6.2	47.9	44.2	46.1
cus6	81.72	2.03	114.41	2.88	163.44	6.60	130.75	12.02	5.9	40.3	39.7	40.0
cus7	84.33	1.96	118.06	2.87	168.65	6.38	134.92	12.13	6.2	43.0	41.1	42.1
cus8	76.73	2.05	107.42	3.43	153.45	8.50	122.76	11.32	5.5	37.4	31.3	34.4
cus9	61.96	1.99	86.74	2.80	123.92	7.20	99.14	13.07	6.6	31.1	31.0	31.1
cus10	74.66	1.86	104.52	2.64	149.32	6.50	119.46	9.36	5.0	40.1	39.6	39.9
AVG>									5.2	36.8	35.4	36.1

Single Bolt Connections-Reinforced

Specimen	Stiffness (kN/mm)								Ductility			Average K(K50+K70)
	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltault	80%Fult	Delta80	K50	K70		
crs1	66.57	1.93	93.20	3.04	133.14	6.48	106.51		0.0	34.5	30.7	32.6
crs2	83.68	2.47	117.15	3.86	167.35	8.43	133.88	37.56	15.2	33.9	30.3	32.1
crs3	96.44	3.38	135.01	5.11	192.87	10.07	154.30	27.92	8.3	28.5	26.4	27.5
crs4	76.56	2.85	107.18	4.44	153.12	9.75	122.50	31.34	11.0	26.9	24.1	25.5
crs5	82.97	2.89	116.16	4.29	165.94	11.81	132.75		0.0	28.7	27.1	27.9
crs6	70.92	3.17	99.28	4.58	141.83	10.62	113.46		0.0	22.4	21.7	22.0
crs7	87.37	2.96	122.32	4.88	174.74	11.22	139.79	28.98	9.8	29.5	25.1	27.3
crs8	81.67	1.93	114.34	3.25	163.34	9.69	130.67	41.76	21.6	42.3	35.2	38.7
crs9	75.59	3.15	105.82	4.65	151.17	11.15	120.94	31.1	9.9	24.0	22.8	23.4
crs10	67.33	2.24	94.26	3.45	134.66	8.84	107.73		0.0	30.1	27.3	28.7
AVG>									12.6	30.1	27.1	28.6
Avg Improv>									143.1%	-18.2%	-23.5%	-20.8%

Four Bolt Connections-Unreinforced

Specimen	Stiffness (kN/mm)								Ductility			Average K(K50+K70)
	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltault	80%Fult	Delta80	K50	K70		
cum1	206.56	1.73	289.18	2.04	413.12	3.06	330.50	4.65	2.7	119.4	141.8	130.6
cum2	190.02	1.16	266.02	1.63	380.03	3.04	304.02	5.72	4.9	163.8	163.2	163.5
cum3	186.39	1.38	260.94	1.87	372.77	3.20	298.22	6.78	4.9	135.1	139.5	137.3
cum4	209.85	1.19	293.79	1.66	419.70	3.60	335.76	7.28	6.1	176.3	177.0	176.7
cum5	207.59	1.08	290.62	1.51	415.17	4.52	332.14	6.18	5.7	192.2	192.5	192.3
cum6	205.20	1.22	287.28	1.63	410.40	3.76	328.32	5.38	4.4	168.2	176.2	172.2
cum7	182.19	1.09	255.07	1.49	364.38	4.19	291.50	5.64	5.2	167.1	171.2	169.2
cum8	225.95	1.14	316.33	1.66	451.90	3.41	361.52	6.71	5.9	198.2	190.6	194.4
cum9	208.49	1.07	291.89	1.46	416.98	3.01	333.58	6.14	5.7	194.9	199.9	197.4
cum10	186.95	1.15	261.73	1.59	373.90	3.33	299.12	6.41	5.6	162.6	164.6	163.6
AVG>									5.1	167.8	171.6	169.7

Four Bolt Connections-Reinforced

Specimen	Stiffness (kN/mm)								Ductility			Average K(K50+K70)
	50%Fult	Delta50	70%Fult	Delta70	Fult	Deltault	80%Fult	Delta80	K50	K70		
crm1	218.02	1.48	305.22	2.05	436.03	6.00	348.82	9.11	6.2	147.3	148.9	148.1
crm2	226.18	1.64	316.65	2.25	452.36	3.93	361.89	10.06	6.1	137.9	140.7	139.3
crm3												
crm4	213.48	1.49	298.87	2.09	426.95	6.18	341.56	9.27	6.2	143.3	143.0	143.1
crm5	213.94	1.50	299.51	2.00	427.87	6.31	342.30	8.27	5.5	142.6	149.8	146.2
crm6	208.95	1.44	292.52	2.05	417.89	6.88	334.31	13.89	9.6	145.1	142.7	143.9
crm7	219.15	1.62	306.80	2.17	438.29	7.17	350.63	10.14	6.3	135.3	141.4	138.3
crm8	184.12	1.44	257.76	1.97	368.23	6.02	294.58	13.52	9.4	127.9	130.8	129.4
crm9	212.35	1.37	297.28	2.04	424.69	6.64	339.75	10.32	7.5	155.0	145.7	150.4
crm10	229.69	1.71	321.57	2.33	459.38	5.15	367.50	9.33	5.5	134.3	138.0	136.2
AVG>									6.9	141.0	142.3	141.7
Avg Improv>									35.3%	-16.0%	-17.1%	-16.5

**Fult Values in (kN), Delta Values in (mm), Ductility Values in (%)

Figure 62: Ductility Calculations for 133x191 mm Specimens