Fire Resistance of Connections in Laminated Veneer Lumber (LVL)

by

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Abstract

The fire resistance of timber connections is relatively unknown in the construction and design industries even though they are widely used. This research focuses on the fire resistance of nailed, screwed, bolted and self-drilling doweled connections in laminated veneer lumber (LVL) timber. These connections have been found to have high strength under cold or normal temperature but hardly achieved 30 minutes fire rating in the furnace tests.

To establish the performance of connections, an investigation was carried out on the compressive strength of connections by having compressive tests using an Instron Testing Machine. Similar connections were tested at simulated fire conditions under constant load in a custom-built furnace. The different fasteners used and the arrangement of the connections gave different connection strengths at ambient and elevated temperature.
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Chapter 1  Introduction

1.1 Overview

At one time, timber was the predominant structural material used in building construction. The human species has used wood to make a variety of objects from wagon wheels to multi-storey dwellings. Today there is healthy competition between steel, aluminium, concrete, masonry and timber in buildings. In New Zealand, timber is widely used because of its structural adequacy, ready availability and aesthetics, especially when compared to alternative building materials, such as steel and concrete.

Timber structures are commonplace around the world, as timber is used in large industrial and commercial buildings as well as residential houses. The advent of the relatively new, engineered wood product, laminated veneer lumber (LVL) along with glue laminated timber (Glulam) has the potential to initiate the resurgence of structural wood being used as a major building material, especially in large clear span buildings using portal frames.

Connections in timber structures have to be strong in order to match the strength of the timber. In the building industry, many types of connections are being used such as nails, screws, bolts, epoxy steel rod and dowels. These types of connections provide the strength and rigidity required, but there is uncertainty as to the fire resistance of these connections. There is no simple way of expressing the behaviour of a material with respect to fire. There are two distinct phases to fire, the developing phase and the fully developed phase. Performance of the timber has to be categorised in both these conditions. The behaviour of timber connections is very important in the fully developed phase as the connections play an important role in maintaining the building structure to enable occupants to escape from fire and enable fire fighters to fight fires.

Our knowledge about the behaviour of fasteners joints in timber structures exposed to fire is limited. So far little is known about the reduction in the strength properties of the wood members due to elevated temperature and changes in moisture content.
This project will provide information on different types of fasteners such as screws, nails, bolts and dowels used in timber connections. The different connections can be normally seen in timber trusses, or in roof or floor beams of buildings. The effect of heat and fire upon all different types of connections will be investigated in this research using LVL timber. The purpose is to determine the fire resistance of each connection using the different fasteners in timber connections.

1.2 Objectives

The main objectives of this research were to determine and improve the fire resistance of nailed, screwed, bolted and doweled connections in LVL timber.

Particular aims of this research were:

- To discover unanticipated failure modes of the connections in fire situations
- To investigate the performance of different connections in fires
- To determine the limitations of each connection
- To investigate possible fire protection methods
- To investigate the influence of protection on the connections

1.3 Scope of Research

This research is essentially contained in two parts:

- Experimental laboratory testing of different connections in cold conditions:
  — In this phase of testing, different fasteners such as bolts, nails, screws and self-drilling dowels were tested in the various connection arrangements. The testing was done in an Instron Testing Machine by loading the connections in compression to determine their
ultimate load. Comparisons were made between the ultimate loads to compare the behaviour and strength of the different fasteners making up the connections.

- Experimental laboratory testing of different connections in a fire situation:
  — In this phase of testing, the connections were held under constant tensile load while they were exposed to an external heat flux in a furnace. The objective of these tests was to determine and evaluate the fire performance of the connection. The quantification of the fire resistance was made by comparing the results in the test furnace to those achieved by Lane (2004) in an ISO 834 fire and using “radiant exposure area correlation” concept developed by Nyman (2001). Intumescent materials were also used for application to the connections to evaluate the impact of using these materials in the fire resistance of connections.

1.4 Outline of Thesis

Chapter 1 talks about the overview of the timber connections in the construction world and introduction to the experiments. A literature review was undertaken and is outlined in Chapter 2 before the initiation of any laboratory testing to establish the types of joints available and used at present. Chapter 3 discusses the design processes for the connections in ambient temperature and fire exposure. Then the testing materials and specimens’ preparation will be discussed in Chapter 4. The analysis of the cold and furnace testing results will be shown in Chapters 5 and 6 respectively. Chapter 7 is about the discussion of the results and comparison from previous research done. Chapter 8 discusses the conclusions and further recommendations for future research.
Chapter 2 Literature Review

2.1 Overview

A literature review was undertaken before any analyses or laboratory experiments were carried out. This chapter gives a summary of the literature review.

2.2 Joint Types and Their Fire Performance

2.2.1 Nails

Nails are probably the most common and familiar form of mechanical fastenings because they penetrate the wood much better than surface adhesive. Besides that, they do not weaken the wood with drilled holes and they can distribute forces over a larger part of the surface than bolts. They are manufactured in many sizes, shapes and materials.

![Different size, shapes and type of nails (Buchanan, 2002)](image)

The load capacity of nailed joints in timber structures exposed to fire depends on the charring of the wood and the reduction of strength in the residual cross-section of the members (Noren, 1996). The capacity of nails to transmit forces in shear between wood members in joints depends on the embedding strength of the wood and on the yield moment of the nails.
Literature Review

Research by Frangi A and Mischler A (2004) showed that unprotected connections with steel side plates and annular shank nails failed after about 12 minutes due to large deformations of the nails and the steel side plates directly exposed to the fire. By protecting the steel side plates by a fire proof-coat the fire resistance of the connections was increased up to around 15 minutes.

Noren (1996) states that larger side members should be used to increase the time to failure. However, at the same time the rounding of the edges and the charring parallel to the side members will be come more decisive for the nails close to the edge. Consequently, the edge distance must be increased in proportion to the extra thickness of the side members.

In addition, Noren (1996) showed that a theoretical method can be used for calculation of the load bearing capacity of nailed joints exposed to fire. He has shown that there is a considerable reduction of the embedding strength during the initial period of the fire, which is probably due to the moisture content and to the fact that there is a major influence from transient effects as the temperature reaches 100°C. Problems are encountered because our knowledge regarding the mechanical properties of wood at elevated temperature is limited. Also, the available data on the mechanical properties at elevated temperatures is related to the compression and tension strength but not to the embedding strength.

However, there is a complication as calculation may give values which are different from the test results. The calculated values represent a single nail having at a certain failure mode, while the test results are based on average values of all load bearing nails in the joint. Since the charring depth increases more rapidly close to the edges of the side members than in the central parts, the capacity of the nails may be reduced close to the edges of the side members.

In the fire test by Noren (1996), the nail temperature in the central member of the joint was less than 200°C when failure occurs. The influence of temperature on the yield moment of the nail may therefore be neglected in the calculation of the load bearing capacity.

An in-depth study into the performance of nailed connections was undertaken by Noren (1996). This study investigated the failure modes and load carrying capacity of nails when
Literature Review

exposed to fire conditions. It was found that for load ratios of around 0.3 in unprotected nailed gusset connections that a 15 minute fire resistance could be achieved.

Nailed connections using steel gusset plates and plywood gusset plates were analysed by Buchanan and King (1991). The steel gusset plates which are usually 5mm thick are manufactured with pre-drilled holes for nails. The plywood gusset plates are fixed with nails driven by nail gun. Nailed connections using plywood gussets are simple to construct as they do not require any lining up of holes or predrilling. The strength of the gusset plate is limited by the thickness of plywood and also by the plywood sheet size (1.2 x 2.4m). The design guidance for steel and plywood gusset plates is given by Buchanan (2002).

2.2.2 Bolts

Bolts can provide very strong connections, using fewer fasteners than nailed connections. Bolts are commonly made from ordinary mild steel with hexagonal or square heads and nuts. The diameters range between 12 and 30 mm. Generally, bolts are used in double or multiple shear joints. To ensure the performance of the joints, a minimum thickness is required for timber elements: 30 mm for side members and 40 mm for internal members. All the tightened fasteners should be installed with a washer under any heads or nuts in contact with the timber.

Bolted connections range from simple types with bolts passing through holes in each member to those using external steel plates or steel rings to transmit forces between members. These are known as “Trailer Connection”. No fire tests have been carried out for this type of bolted connections. The research reported herein will not involve this type of connections as it is not widely used in construction nowadays.

The major disadvantages of bolted joints are their intrusive nature as the bolt heads and nuts will be seen on the exterior of the connections. Spacing of bolts in the joint area is crucial as splitting may occur if bolts are placed too close together. Besides, bolted joints typically use slightly oversize holes for ease of construction although this reduces their stiffness.

2.2.3 Screws

Screws have many of the advantages of nails. They also have much better gripping capacity than nails because of the threaded shaft. A disadvantage is the poorer ductility of screws
compared with nails. The fire performance of screwed connections in wood has not been studied extensively. (Buchanan, 2001a)

2.2.4 Dowelled or Pinned Joints

Dowelled joints are steel pins similar to large diameter nails or like bolts with no heads or nuts. Dowels are usually inserted into tight fitting pre-drilled holes. Dowelled joints hence operate on a very similar basis to nailed or bolted joints, with the mode of load transfer being shear in the pin joining the lapped timber pieces and or steel (Scheibmair, 2003). Instead of a nail group holding two external steel plates or timber on either side of the member to be joined, typically one or more steel plates are inserted into a slot cut in the timber member, and held in place by steel dowels. There are holes that are pre-drilled through both steel plate and timber members for the steel dowels.

2.2.5 Self Drilling Dowels

Another recent development is the use of self-drilling dowel in timber connections. This type of connection is frequently used in Europe and other parts of the world. However, this is still a rather new technology in timber construction in New Zealand. This type of connection provides hidden fasteners in heavy timber construction which are very strong.

The self-drilling steel dowel connection has the benefit of not requiring pre-drilling of holes for the dowels prior to construction. This makes for easier construction as there are no holes to line up which would prove difficult if the timbers are left in the open as the timber would expand or shrink due to the influence of moisture. These connections were shown to develop high strength and good ductility during testing. (Scheibmair, 2003)

Multiple shear steel-to-timber connections with dowels and slotted-in steel plates were studied in Switzerland in research carried out for the Swiss Agency for the Environment, Forests and Landscape (Frangi A and Mischler A, 2004). The diameters of the dowels tested were 6.3 and 12 mm. In the experiment, the load levels during the fire tests were 0.3 $F_u$, 0.15 $F_u$ and 0.075 $F_u$ with $F_u$ = load-carrying capacity at room temperature. Some connections tested were protected by timber boards or gypsum plasterboard in addition to unprotected connections. The influences of load level during fire tests were also studied. The thickness of the steel plates slotted in the timber was 5 mm.
The connections were subjected to tension in the parallel to grain direction of the timber and the fire behaviour was analysed. During the fire test, the slotted-in steel plates were not exposed directly to the fire as the char layers on the outer of the timber remain intact.

The connections which are exposed to the fire and loaded with 0.3 $F_u$ failed around 30 minutes which means a fire resistance of 30 minutes. However, a reduction of the load level to 0.15 $F_u$ and 0.075 $F_u$ led to only an increased failure time of between 3 – 8 minutes, thus the load level seems not to be too relevant a design parameter in order to increase the fire resistance of the connections. Besides, an increase or reduction of the number of the dowels as well as an increase of the diameter of the steel dowels did not significantly improve the fire resistance of the connections.

When the timber dimensions were increased from 200x200 mm to 280x280 mm, thus the timber cover of the slotted-in steel plates as well as the length of the steel dowels was also increased by 40mm. The end distance to the dowels was also increased by 40mm. From the experiment, the connection failed after 70 minutes which was an increase in the fire resistance of about 40 minutes. The observed increased fire resistance corresponds to an improvement of about 1 minute for 1 mm of increased timber cover of the connections.

The connections which were protected by timber board and gypsum plasterboard which has combined thickness of 27mm have failure times between 57 to 72 minutes. Thus by protecting the connection the fire resistance was increased by around 25 minutes which corresponded to an improvement of about 1 minute for 1 mm timber protection of the connection.

Thus, the modification of increasing the timber cover of the steel dowels and the slotted-in steel plates were gave a significant increase in the fire resistance of the connections from a fire design point of view.

2.3 Laminated Veneer Lumber (LVL)

Of all, the characteristics of the timber, variability in strength is the weakest link among all the materials used in this project. This section will address the development of LVL, the manufacturing process and its properties.
2.3.1 Development of LVL

Laminated timber was first discovered in mid 1500’s (Muller, 2000) where multiple timber pieces were used to form members, but these pieces were not glued together. Glue-laminated timbers really become building materials in late 18th century (Muller, 2000). The main advantage of glue-laminated timber was the ability to produce curved members without damaging the timber members. This was particularly important in the building construction where timber arches were introduced in roof and bridges.

LVL introduces a new dimension of the widely used glue-laminated timber. First introduced in 1998 in New Zealand market (CHH, 2001) as a new building material, it gives better strength characteristics and much greater behavioural property uniformity.

2.3.2 Manufacturing of LVL

Manufacturing of LVL is similar in principle to glue-laminated timbers which involve the process of gluing laminations of timber to give the finished product. Nevertheless, they do not have similar material properties which will be discussed in the next section.

According to Scheibmair (2003), LVL is produced by shaving thin layers, typically 3.2mm thick off a rotating soaked log. Veneers are then cut into sheets, dried and graded to stiffness and visual quality. Since the veneers are machine graded, weaker veneers that may have been discarded in the glue-laminated processes can now be matched with high stiffness veneers, to give the desired average properties.

Each veneer then passes through a glue curtain and is layered up. The application of glue is very important as too much or too little glue can cause undesirable effects such as flushing, premature drying and creating air voids. The layered up veneers are then pulled through the continuous press after being heated by passing through microwaves. The 1.2m nominal width continuous billet emerging from the other end of the press is then cut into 12 m lengths. All billets and cut to size members are checked several times and by machine and as well as visual inspection to ensure any glue faults are identified and rectified. Finally the LVL members are painted to identify their type of LVL and then packed.
Figure 2-2 – LVL manufacture process at Marsden Point Mill, Northland, New Zealand (Scheibmair, 2003)
2.3.3 Properties of LVL

The material properties of LVL are much more closely spread about their average value due to the number of layers of veneers used in LVL compared to glue-laminated timber. In natural timber, the largest variability would be expected, with glue-laminated timber varying less than its value and LVL showing the smallest spread. As a consequence, the material properties of LVL can be determined confidently and closer to the mean value than natural or glue-laminated timber.

![Figure 2-3 – Material properties variance (Scheibmair, 2003)](image)

2.4 LVL Products

There are several different ranges of LVL products such as Hyspan, Hy90, Hybeam, Hychord, Truform, Edgeform and Hyplank. These products are produced for different applications and the functions of the different LVL products are explained below. This information was obtained from the Futurebuild website.

Hyspan structural laminated veneer lumber is ideal for application in residential, commercial, rural and industrial construction. Their popular uses include rafters, floor joints, beams and lintels.
Features include:
- Long lengths
- Easy to handle
- Lightweight
- Dimensionally stable
- Straight and true
- Cost effective

Hyspan was used for the experiments described in this report.

Hy90 is the new 90 mm thick LVL product. It is designed for use in structural beams for high strength and consistent performance. Applications include lintels, larger span garage door lintels and beams where the 90 mm width is desired.

Hybeam is an engineered wood 'I' Joist, manufactured utilising both Hyspan LVL and structural plywood. Hybeam is ideally suited for floor joist applications in commercial, housing and multi-residential constructions. Hybeam can be treated to H3 level, for use in weather – protected applications.

Hychord is a new LVL truss fabrication material for high load applications where strength and predictable deflection performance are paramount. With Hychord, designers can confidently exploit the excellent strength properties for minimum sections and then utilize the predicted deflection performance to apply just the right amount of camber to ensure flat ceilings and a no 'dips or bumps' roof line.

Truform is a solid LVL beam specifically manufactured for use in structural concrete formwork applications. It is suitable for joists, bearers, walers and soldiers. Truform comes painted bright orange for moisture protection and ready identification.

Hyplank is a strong yet lightweight LVL scaffold plank. The defined structural properties of LVL and strict quality control processes ensure safe and consistent performance and mean that Hyplank has significantly higher structural reliability than sawn timber. This makes Hyplank ideal to use in place of conventional timber, and where modular systems cannot
accommodate the size and shape of the scaffolding requirement. Hyplank is also used to considerable advantage where corrosion is a hazard for metal planks.

Edgeform LVL is arised and painted red for use as edge boards in concrete formwork projects. Edgeform is light, straight and more uniform than traditional alternatives.

2.5 Design Code

This section refers to the New Zealand Timber Design Code, NZS3603. For the purpose of joint design, timber species shall be assigned to the appropriate group. Collins and Britton (2001) concluded that for nail connections the classification of LVL is group J4 when nails are loaded perpendicular to the glue line. Therefore, the classification of LVL for joint design is group J4 due to the high tensile strength.

Design data for nails, screws, bolts and dowels are given in sections 2.5.1, 2.5.2, 2.5.3 and 2.5.4 respectively.

NZS 3603:1993 is the New Zealand Timber Structures Standard governing joint detail such as minimum spacing of fasteners and design strength of timber. This standard is used in this research to determine appropriate joint layout and detail. The formulae and requirements from the NZS 3603:1993 are outlined in the subsections that follow:

Figure 2-4 – Position of nails and screws (NZS3603:1993)
Figure 2-5 – Position of bolts (NZS3603:1993)

Table 2-1 – Minimum spacing of nails and screws in joints (NZS3603:1993)

<table>
<thead>
<tr>
<th>Distance</th>
<th>Hole not prebored (nails only)</th>
<th>Hole prebored to 0.8(d_a) or as given by 4.3.1.2 for screws</th>
</tr>
</thead>
<tbody>
<tr>
<td>From end of member</td>
<td>20(d_a) may be reduced to 12(d_a) for radiata pine</td>
<td>10(d_a)</td>
</tr>
<tr>
<td>From edge of member</td>
<td>5(d_a)</td>
<td>5(d_a)</td>
</tr>
<tr>
<td>Between nails along grain</td>
<td>20(d_a) may be reduced to 10(d_a) for radiata pine</td>
<td>10(d_a)</td>
</tr>
<tr>
<td>Between nails across grain</td>
<td>10(d_a) may be reduced to 5(d_a) for radiata pine</td>
<td>3(d_a)</td>
</tr>
</tbody>
</table>
2.5.1 Nails

According to NZS3603, Clause 4.2.2.2, laterally loaded nailed joints shall be proportioned to satisfy

\[ S^* \leq \Phi Q_n \]

where

- \( S^* \) = design load effects on joint
- \( \Phi \) = strength reduction factor
- \( Q_n \) = nominal strength of a joint appropriate to the mode of loading

The nominal strength shall be taken as

\[ Q_n = nKQ_k \]

where

- \( n \) = number of fasteners
- \( Q_k \) = characteristic strength of fasteners as given in Table 2-2
- \( K \) = product of modification factors listed below:

a) Green timber 0.85  
b) Nails in end grain 0.67  
c) Nails in double shear 2.0  
d) Steel side plate < 3.0 mm thickness 1.25  
   Steel side plate \( \geq 3.0 \) mm thickness 1.5  
   Plywood or particle board with flat head nails 1.4  
e) Number of nails. For connections containing 50 or more nails the design strength shall be increased by 1.3. For fewer nails, the factor shall be obtained by linear interpolation to value of 1.0 for four nails. The position for the nail arrangement in the connection is shown in Figure 2-4.

<table>
<thead>
<tr>
<th>Timber Group</th>
<th>Nail Shank Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>J4</td>
<td>2.0 2.24 2.50 2.80 2.87 3.15 3.33 3.55 3.75 4.00 4.50 5.00 5.30 6.00</td>
</tr>
<tr>
<td></td>
<td>391 476 577 703 733 863 951 1060 1165 1310 1610 1930 2140 2660</td>
</tr>
</tbody>
</table>
2.5.2 Screws

Clause 4.3.1.2 of NZS 3603:1993 states that the correct sizes of lead holes shall be bored for all screws except self drilling screws. The diameter of the hole for the shank shall be equal to the diameter of the shank and the lead hole for the threaded portion of the shank shall not exceed the root diameter of the screw.

Edge and end distances and spacing of screws in a screwed joint shall be not less than is given in Figure 2-4 and Figure 2-5.

Laterally loaded screwed joints shall be so proportioned to satisfy

\[ S^* \leq \Phi Q_n \]

where

- \( S^* \) = design load effects on joint
- \( \Phi \) = strength reduction factor
- \( Q_n \) = nominal strength of a joint appropriate to the mode of loading

The nominal strength shall be taken as

\[ Q_n = nKQ_k \]

where

- \( n \) = number of fasteners
- \( Q_k \) = characteristics strength of fasteners as given in Table 2-3
- \( K \) = product of modification factors listed below:
  
  a) Green timber 0.85
  b) Screws in end grain 0.67
  c) Screws in double shear 2.0
  d) Steel side plate 1.25

<table>
<thead>
<tr>
<th>Timber Group</th>
<th>Minimum Screw Shank Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>J4</td>
<td>775 960 1155 1371 1614 1855 2118 2692 3303</td>
</tr>
</tbody>
</table>
2.5.3 Bolts

Clause 4.4.1 of NZS 3603:1993 states that the diameter of the hole for a bolt shall be not less than the bolt diameter and shall not exceed it by more than 10%. In addition, every bolt shall be provided with a washer at each end of sizes not less than:

- 20 mm x 20 mm x 1.5 mm for bolts not exceeding 8 mm diameter
- 35 mm x 35 mm x 3 mm for bolts not exceeding 12 mm diameter
- 50 mm x 50 mm x 4 mm for bolts not exceeding 20 mm diameter
- 65 mm x 65 mm x 5 mm for bolts exceeding 20 mm diameter

If round washers are used, they shall be of a thickness and area of not less than those specified above for the equivalent square washer.

Edge and end distances and spacing of bolts in a bolted joint shall be not less than:

As shown in Figure 2-5 with:

\[ a = d_a \left( \frac{n - 4 + r}{r - 1} \right) \]

but not less than 2.4 \( d_a \)

where

- \( n \) = total number of bolts in joint
- \( r \) = number of rows of fasteners across the grain
Table 2-4 – Characteristic strength for a single bolt in dry timber loaded parallel to the grain (Reference: Table 4.9, NZS 3603:1993)

<table>
<thead>
<tr>
<th>Type of joint</th>
<th>Effective timber thickness ((b_{el}))</th>
<th>System characteristic strength (Q_{sku})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Two member</td>
<td>Smaller of (2b_1) and (2b_2)</td>
<td>(Q_k)</td>
</tr>
<tr>
<td>2. Three member</td>
<td>Smaller of (2b_1) and (b_2)</td>
<td>(2Q_k)</td>
</tr>
<tr>
<td>3. Multiple member</td>
<td>(i) Between A and B (\text{Smaller of } b_1 \text{ and } b_2)</td>
<td>(i) (Q_k) (ii) (Q_k) (iii) etc. Total characteristic load = sum of characteristic loads</td>
</tr>
<tr>
<td>4. Alternative steel and timber members</td>
<td>As for types 1, 2 or 3 except that (b_{el}) is based on thickness of timber members only</td>
<td>(1.25 \times \text{value calculated for joint types 1, 2, or 3})</td>
</tr>
</tbody>
</table>

The tensioned bolted joints shall satisfy the following equation

\[ S^* \leq \Phi Q_n \]

where

- \(S^*\) = design load effects on joint
- \(\Phi\) = strength reduction factor

The nominal strength of the joint is given as

\[ Q_n = nKQ_k \]

where

- \(n\) = number of fasteners
- \(Q_k\) = characteristic strength of fasteners as given in Error! Reference source not found. & 2-5
K = product of modification factors listed below:

a) Steel side plate ≥ 3.0 mm thickness 1.5
b) Two shear plane 2

Table 2-5 – Characteristic strength, $Q_k$ (kN) for a single bolt in a two-member joint in dry timber loaded parallel to the grain (Reference: Table 4.10, NZS 3603:1993)

<table>
<thead>
<tr>
<th>Effective Timber Thickness ($b_e$) (mm)</th>
<th>Bolt Shank Diameter (mm)</th>
<th>Timber group</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>J4</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>2.18</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>2.73</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>3.28</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>4.37</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>5.46</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>6.55</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>2.77</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>3.46</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>4.15</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>5.53</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>6.92</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>8.30</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>6.37</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>7.64</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>10.2</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>12.7</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>15.3</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>9.83</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>12.1</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>16.4</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>19.7</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>24.6</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>10.5</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>18.6</td>
<td></td>
</tr>
<tr>
<td>65</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>23.7</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>28.4</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>35.5</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>10.5</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>18.6</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>29.1</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>39.3</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>49.1</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>10.5</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>18.6</td>
<td></td>
</tr>
<tr>
<td>130</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>29.1</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>41.9</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>65.5</td>
<td></td>
</tr>
</tbody>
</table>
2.5.4 Dowels

The tensioned bolted joints shall satisfy the following equation

\[ T^* \leq \Phi R_u \]

The load-carrying capacity \( R_u \) per shear plane is:

\[ R_u = \sqrt{\frac{2}{\Phi}} \sqrt{M_u f_h d} \]

where

- \( M_u \) = dowel bending resistance
- \( f_h \) = timber embedding strength
- \( d \) = dowel diameter

2.6 Fire Severity

When designing structures for fire safety, the fundamental step is to verify that the fire resistance of an element is greater than the severity of the fire that the building element is exposed i.e. fire resistance \( \geq \) fire severity. Buchanan (2001a) gives both definitions of fire resistance and fire severity as the following:

- Fire resistance is the “measure of the ability of the structure to resist collapse, fire spread or other failure during exposure to a fire of specified severity”
- Fire severity is the “measure of the destructive impact of a fire, or the measure of the forces or temperatures which could cause collapse or other failure as a result of the fire”.

Fire severity is often described in terms of a period of exposure to a standard test fire but this is not true for realistic fires which have very distinct characteristics. The most widely used standard time-temperature curve is ISO 834 as shown in the equation below:

\[ T = 345 \log(8t + 1) + T_0 \]

Where
- \( T \) is the temperature \( [\degree C] \)
- \( t \) is the time \( [\text{minutes}] \)
- \( T_0 \) is the ambient temperature \( [\degree C] \)
The severity of a real fire is determined to be significantly worse than the equivalent early stages of the standard ISO 834 time-temperature relationship. Nyman (2001) has highlighted these corresponding factors in the following:

- Thermoplastic materials and modern furnishings where they burn significantly fast as pool fires with higher heat release rates;
- Realistic time-temperature fire histories where some full-scale compartments were burnt and found to fail earlier than that of the standard furnace tests.

Many efforts have been put in by fire engineers and researchers to quantify the severity of a real fire. The concept of equivalent fire severity is the one which is most often used, in which it relates the severity of a real fire to the standard test fire. This concept is of importance allowing designers to use published fire resistance ratings from standard tests with estimates of real fire exposure. There are several approaches that have been used to determine the equivalent fire severity as described in Buchanan (2001a). However among all these methods, the equal area and time-equivalent concepts are of particular significance.

### 2.6.1 Equal Area Concept

The equal area concept was first proposed by Ingberg in 1928, in which he defined the integral or area under the time-temperature curve as the fire severity. Therefore, two fires are considered to have equivalent fire severity if the areas under each curve are equal, above a certain reference temperature. This equal area concept cannot be proved theoretically because the units of area are meaningless. The equal area concept has a problem in that it can give a very poor comparison of heat transfer for fires with distinct shapes of time-temperature curves. At high temperatures, heat transfer from a fire to the surface of a structure is dominated by radiation and due to the fact that radiation is proportional to fourth power of the absolute temperature, taking a direct area under a time-temperature curve cannot truly represent the fire severity. Using such a method would underestimate the severity of a short hot fire and overestimate long cool fire severity.

Due to the theoretically inappropriate usage of the equal area concept in assessing fire severity, Nyman (2001) expanded the concept further to what he called the “radiant exposure area correlation” concept. He proposed using the total energy impinging upon the surface of a
structure to establish the severity of a fire, which is expressed as the area under a plot of the emissive power of the compartment gases against time. The emissive power is given as equation below where the emissivity of the gases is conservatively taken as 1 for simplicity:

\[
\dot{Q}' = \varepsilon \sigma T^4
\]

Where \( \dot{Q}' \) is the radiant heat flux [kW/m²]
\( \varepsilon \) is the emissivity of the gases (1) [-]
\( \sigma \) is the Stefan-Boltzmann constant (5.67×10⁻⁸) [W/m²K⁴]

The area under the emissive power-time curve is therefore expressed mathematically with units of kJ/m² as follows:

\[
\text{Area} = \int_0^t \dot{Q}' \, dt = \varepsilon \sigma \int_0^t T^4 \, dt
\]

The radiant exposure area correlation can be applied to any fire exposure time-temperature profiles. When the radiant exposure concept is applied to the standard ISO 834 curve, the measure of fire severity at any time on the curve can be determined and this measure of fire severity can then be simply equated to a real fire exposure having the same radiant exposure area, which is the equivalent fire severity. For example, if the failure time of an assembly under the exposure of the standard ISO 834 fire is known and a real fire exposure is known or predicted, the failure time of that assembly in real fire exposure can be determined.

There are two assumptions that have been made in this approach in order to correlate the radiation fire severity in each test type; the standard furnace and real test fires. The energy characteristics in both the compartment test fires and the standard furnace test fires are assumed to be identical and also that the convective components of heat transfer to the structure are of equal proportion to the whole energy transfer in both test fires.

2.6.2 Time-Equivalent Concept

The time-equivalent concept uses the most common approach in quantifying the severity of a fire that is by equating the performance of a structure under a real fire exposure in terms of an
equivalent exposure to the standard fire. Buchanan (2001a) includes a number of time-equivalent formulae that have been developed in assessing fire severity and explains in greater details. These time-equivalent formulae are:

- CIB formula, which based on the ventilation parameters of the compartment and the amount of fuel load available for burning;
- Law formula, which based on tests carried out in both small scale and larger scale compartments and the formula is similar to CIB;
- Eurocode formula, which is again similar to CIB but with some modifications; introducing the new compartment lining parameter and the horizontal openings in the roof of the compartment is included into the ventilation factor.
Chapter 3   Design of Connections in Ambient and Fire Condition

3.1 Overview
This chapter describes the overall process of designing connections in structures for ambient conditions and for fire exposure. The design of the connections was based on the tensile member in the bottom chord of a roof and floor trusses. The number of fasteners needed for each joint was determined and the stress in any steel plate used in the connections was checked.

3.2 Structural Design in Fire Conditions
The concept of structural design for fire is similar to structural design for normal temperature conditions. Buchanan (2001a) states that the main differences of fire design compared with cold design are that, at the time of a fire:

- the applied loads are less,
- internal forces may be induced by thermal expansion,
- strengths of materials may be reduced by elevated temperatures,
- cross-sectional areas may be reduced by charring,
- a smaller safety factor can be used, because of the low likelihood of the event of fire,
- deflections are not important (unless they affect strength),
- different failure mechanisms need to be considered.

Not all of the above factors influence the design of the timber connections. The stress in the timber beam increases steadily (under constant load) due to loss of cross section by charring. However, the material strength only decreases very slightly due to elevated temperatures within the beam. Failure occurs when the stress in the member exceeds the material strength.

3.3 Loads for Cold Design
Two assumptions were made in determining the design of the connections. The first assumption is in the size of the timber member to be used. The timber size is determined to be 150 x 63 mm. The side timber member is then determined to be 150 x 45 mm. The second
assumption is in the stress level in the tensile member. The member was assumed to be overdesigned for strength so that the actual load is only 40% of the design strength. (This is an assumption, not based on calculation).

To design the connection, the expected loads, $T^*$ must be compared with the expected strength, $T_n$, of the timber section. The strength reduction factor $\Phi$ is taken as 0.9 for LVL timber, so the design equation becomes

$$T^* \leq \Phi T_n$$

where $T_n = f_t A$, where $A$ is the area of cross section of LVL timber (mm$^2$) and $f_t$ is the characteristic tensile strength of the LVL timber (MPa).

The timber size is chosen as 150 x 63 mm LVL timber. From the Timber Design Guide, the characteristic tensile strength of LVL timber is 26 MPa.

$$T^* \leq 221kN$$

Therefore, the design of the tension strength, 88 kN is based on 40 % of the total strength.

Load combinations for cold conditions:
1.4 $G$ or
1.2 $G + 1.6 \times Q$ or
1.5 $G + 1.2 \times S$

where
$G$ = gravity load
$Q$ = live load
$S$ = snow load

The earthquake and wind load case is typically not critical to timber frame structures since the frames are lightweight.

Table 3-1 summarizes the design loads for roof and floor beams for cold condition.
Table 3-1 – Summaries of load combination for cold condition

<table>
<thead>
<tr>
<th>Load</th>
<th>Roof (kPa)</th>
<th>Floor (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>0.2</td>
<td>0.5</td>
</tr>
<tr>
<td>Q</td>
<td>0</td>
<td>2.5</td>
</tr>
<tr>
<td>S</td>
<td>0.5</td>
<td>0</td>
</tr>
</tbody>
</table>

Cold

<table>
<thead>
<tr>
<th>Cold</th>
<th>Roof (kPa)</th>
<th>Floor (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4 G</td>
<td>0.28</td>
<td>0.70</td>
</tr>
<tr>
<td>1.2 G + 1.6 Q</td>
<td>0.64</td>
<td>4.60</td>
</tr>
<tr>
<td>1.2 G + 1.2 S</td>
<td>0.84</td>
<td>0.60</td>
</tr>
</tbody>
</table>

The maximum load combination for a tensile member in the bottom chord of a roof and floor truss is 0.84 and 4.6 kPa respectively.

3.4 Loads for Fire Testing

The load combinations to be considered for fire conditions from SNZ 1992 are:

G or

G + 0.4 Q

Table 3-2 summarizes the design loads to be considered for roof and floor beams in fire conditions.

Table 3-2 – Summaries of load combination for fire condition

<table>
<thead>
<tr>
<th>Fire</th>
<th>Roof (kPa)</th>
<th>Floor (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>0.2</td>
<td>0.5</td>
</tr>
<tr>
<td>G + 0.4 Q</td>
<td>0.2</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Load Factor | 0.24 | 0.33

The load factor for the roof and floor are found as the ratio of the expected loads on the structure during a fire to the loads that would cause collapse at normal temperatures.
The lower the load factor, the greater the fire resistance because of the large loss in load carrying capacity which can occur before failure occurs.

Since the connections are being designed for a tensile force of 88 kN under ambient conditions, the expected forces in fire conditions will be the load factor multiplied by 88 kN.

Hence, the tensile force in fire $T_{fire}$ would be 21.2 kN and 29.2 kN for the roof and floor member respectively.

### 3.5 Design for Cold Condition

The timber group for the connections design is J4 due to the high tensile strength of LVL timber. The connections are designed to withstand a tensile strength of 88 kN as explained in Section 3.3.

#### 3.5.1 W-W-W Connections

The layout of this type of connection is shown in Figure 3-1. Two types of connectors are used for this joint, i.e. bolts and screws. The outer timber piece is 45mm while the inner timber is 63mm thick.

![Figure 3-1 – Layout of wood – wood – wood connection (W-W-W)](image)

#### 3.5.1.1 Screwed Connections

The screwed joints loaded in tension shall satisfy the following equation

$$S^* \leq \Phi Q_n$$

where

- $S^*$  = design load effects on joint
- $\Phi$  = strength reduction factor
- $Q_n$  = nominal strength of a joint appropriate to the mode of loading
The nominal strength of the joint is given as

\[ Q_n = 2KQ_k \]

where

- \( Q_k \) = strength of the nails
- \( K \) = product of modification factors listed below

The strength reduction factor for a screwed connection is given as 0.8 in NZS 3603: 1993. Type 17, with 150 mm long screws are used in this connection. The characteristic strength for the screws used is found from Table 4.3, NZS 3603: 1993, which is given as 2.118 kN as the screws have shank diameter of around 5 mm.

\[ \therefore \Phi Q_n = 2 \times 0.8 \times (2.118 \text{kN}) = 3.38 \]

No of screws required \( = \frac{T^*}{\Phi Q_n} = \frac{88}{3.38} = 26 \)

### 3.5.1.2 Bolted Connections

The bolted joints loaded in tension shall satisfy the following equation

\[ S^* \leq \Phi Q_n \]

The nominal strength of the joint is given as

\[ Q_n = 2KQ_k \]

The diameter of the bolt used is 12 mm. The strength reduction factor for a bolted connection is given as 0.7 in NZS 3603: 1993. The characteristic strength for the bolts used is found from Table 4.3, NZS 3603: 1993, which is given as 10.5 kN.

\[ \therefore \Phi Q_k = 2 \times 0.7 \times 10.5 \text{kN} = 14.7 \]

No of bolts required \( = \frac{T^*}{\Phi Q_k} = \frac{88}{14.7} = 6 \) bolts
3.5.2 S-W-S Connections

In this particular type of joint, the connections used are nails, screws, and bolts. The layout of the arrangement of the connections is shown in Figure 3-2. The timber group is J4 due to high tensile strength of LVL timber.

![Figure 3-2 – Layout of steel – wood – steel connection (S-W-S)](image)

3.5.2.1 Nailed Connections

The nailed-on plate used is 3mm thick with timber thickness of 63mm. The nailed joints loaded in tension shall satisfy the following equation

\[ S^* \leq \Phi Q_n \]

The nominal strength of the joint is given as

\[ Q_n = K Q_k \]

where K is the product of the factors given below:

a) Steel side plate \( \geq 3.0 \text{ mm thickness} \) 1.5
b) For connections containing 50 or more nails the design strength shall be increased by 1.3

The nails used have nail shank diameter of 3.15 mm. The strength reduction factor \( \Phi \) for nailed connection is given as 0.8 in NZS 3603: 1993. The characteristic strength for the nail used is found from Table 4.3, NZS 3603: 1993, which is given as 0.863 kN.

\[ \Phi Q_k = 0.8 \times (1.5 \times 1.3 \times 0.863 \text{ kN}) = 1.346 \]

No of nails required

\[ \frac{T^*}{\Phi Q_k} = \frac{88}{1.346} = 66 \text{ nails on each side} \]
3.5.2.2 Screwed Connections

The steel plate used in the joint is 3 mm thick. The thickness of the timber used in this connection is 63mm. The screwed joints loaded in tension shall satisfy the following equation

\[ S^* \leq \Phi Q_n \]

The nominal strength of the joint is given as

\[ Q_n = KQ_s \]

a) Steel side plate

1.25

The screws used have a shank diameter of 3.10 mm. The strength reduction factor for a screwed connection is given as 0.8 in NZS 3603: 1993. The characteristic strength for the screws used is found from Table 4.3, NZS 3603: 1993, which is given as 0.96 kN.

\[ \Phi = 0.8 \times (1.25 \times 0.96 kN) = 0.96 \]

No of screws required = \[ \frac{T^*}{\Phi Q_n} = \frac{88}{0.96} = 92 \] screws on each side

3.5.2.3 Bolted Connections

The steel plate used in the joint is 6 mm thick. The thickness of the timber used in this connection is 63mm. The bolted joints loaded in tension shall satisfy the following equation

\[ S^* \leq \Phi Q_n \]

The nominal strength of the joint is given as

\[ Q_n = KQ_s \]

where K is the product of the factors given below:

a) Steel side plate ≥ 3.0 mm thickness

1.5

b) Two shear plane

2

The diameter of the bolt used is 12 mm. The strength reduction factor for a bolted connection is given as 0.7 in NZS 3603: 1993. The characteristic strength for the bolts used is found from Table 4.3, NZS 3603: 1993, which is given as 10.5 kN.
\[ \therefore \Phi Q_n = 0.7 \times (1.5 \times 2 \times 10.5 \text{kN}) = 22.05 \]

No of bolts required \( = \frac{T^*}{\Phi Q_n} = \frac{88}{22.05} = 4 \text{ bolts} \)

### 3.5.3 W-S-W Connections

The steel plate used in the joint is 6 mm thick. The thickness of the timber used in this connection is 45mm on each side of the steel plate.

![Figure 3-3 – Layout of wood – steel – wood connection (W-S-W)](image)

#### 3.5.3.1 Dowelled Connections

The doweled joints loaded in tension shall satisfy the following equation

\[ T^* \leq \Phi R_u \]

The load-carrying capacity \( R_u \) per shear plane is:

\[ R_u = \sqrt{2} \sqrt{M_u f_h d} \]

where

- \( M_u \) = dowel bending resistance
- \( f_h \) = timber embedding strength
- \( d \) = dowel diameter

The dowel diameter used is 5 mm.

Dowel bending resistance is 18.9 Nm (Table 2, Mischler)

Timber embedding strength is 49.8 MPa (Scheibmair, 2003)

\[ \therefore \Phi R_u = 0.9 \times \left( \sqrt{2} \sqrt{0.0189 \times 49.8 \times 1000 \times 5 \times 10^{-3}} \right) = 2.76 \]

Number of dowels required to carry 88 kN load:

No of dowels required \( = \frac{T^*}{R_u} = \frac{88}{2.76} = 32 \)
3.5.3.2 Bolted Connections

The bolted joints loaded in tension shall satisfy the following equation

\[ S^* \leq \Phi Q_n \]

The nominal strength of the joint is given as

\[ Q_n = KQ_s \]

where \( K \) is the product of the factors given below:

a) Alternative steel and timber members \( 1.25 \)
b) Two shear planes \( 2 \)

The strength reduction factor for bolted connections is given as 0.7 in NZS 3603: 1993. The characteristic strength for the bolts used is found from Table 4.3, NZS 3603: 1993, which is given as 10.5 kN. The diameter of the bolt used is 12 mm.

\[ \therefore \Phi Q_n = 0.7 \times (1.25 \times 2 \times 10.5 \text{kN}) = 18.4 \]

No of bolts required \( = \frac{T^*}{\Phi Q_n} = \frac{88}{18.4} = 5 \)

3.5.4 Checking the Steel Stress

These checks are only applicable to S-W-S and W-S-W connections which use steel plates in the joints. The steel plate used in the S-W-S connections is 130 x 3 mm thick for nailed and screwed joints while 130 x 6 mm thick steel is used for bolted joints. The steel plate used in W-S-W connection is 120 x 6 mm thick. The stress of the steel plate is checked to make sure that the plate does not fail before the connections fail.

3.5.4.1 Steel Plate in S-W-S Connections

The tensile stress of the plate must be more or equal to the design tensile strength, which is shown in the formula below.

\[ T^* \leq \Phi T_s \]

where \( T_s = f_y A_s \), \( A_s \) is the area of cross section of steel plate (mm\(^2\)) and \( f_y \) is the characteristic tensile strength of the steel plate (MPa).
For typical mild steel, the tensile yield strength of the plate is 275 MPa. The strength reduction factor $\Phi$ is taken as 0.9 for steel plate.

Nail and screws: $T^* \leq 0.9 \times 275 \text{ MPa} \times (2 \times 130 \text{ mm} \times 3 \text{ mm}) / 1000 = 193 \text{kN}$

Bolts: $T^* \leq 0.9 \times 275 \text{ MPa} \times (2 \times 130 \text{ mm} \times 6 \text{ mm}) / 1000 = 386 \text{kN}$

$T^*$ is 88 kN, therefore the steel plate would not fail under the design tensile load.

### 3.5.4.2 Steel Plate in W-S-W Connections

The same formula is used to check the tensile strength of the steel plate as shown above.

$$T^* \leq \Phi T_s$$

$$\therefore T^* \leq 0.9 \times 275 \text{ MPa} \times (120 \text{ mm} \times 6 \text{ mm}) / 1000 = 178 \text{kN}$$

Since the steel stress, 178 kN is larger than $T^*$ of 88 kN, the steel plate would not fail before the connection fails.

### 3.6 Design for Fire Conditions

The cross section of the LVL timber is checked to determine how long it can withstand the fire. The charring rate for LVL timber is taken to be 0.72 mm/min (Lane et al. 2004). Therefore in a 30 minute fire, the effective cross section of the timber will char around 21.6 mm on each side. Since there are variations in the thickness of the timber used, each connection is analysed separately.

#### 3.6.1 W-W-W and W-S-W Connections

For these connections, only the side timber piece is exposed to the fire. The cross section of the 45 mm LVL timber after 30 minutes of fire will be 23.4 x 106.8 mm.

The tensile stress of LVL member after 30 minutes fire will be $29.2 \text{kN} \times 1000 / 23.4 \times 106.8 = 11.68 \text{MPa}$. This is less than the tensile strength of LVL which is 26 MPa.

The tensile stress of the timber is checked with the tensile load applied during the testing.
Design of Connections in Ambient and Fire Condition

\[ T^* \leq \Phi T_f \]

where \( T_f = f_i A_s \), \( A_s \) is the area of cross section of LVL after 30 minutes exposure to fire (\( \text{mm}^2 \)) and \( f_i \) is the characteristic tensile strength of LVL timber (MPa).

\[ : T^* \leq 1.0 \times 26 \text{MPa} \times \left(23.4 \times 106.8 \text{mm}\right)/1000 = 64.98 \text{kN} \]

A similar check was made of the strength after various lengths of fire exposure giving the relationship shown in Figure 3-4.

The timber piece can withstand up to 60 minutes in the fire as shown on Figure 3-4.

![Figure 3-4 – Tensile strength (kN) vs Time (min) for exposure on one side of 45 mm LVL timber](image)

3.6.2 S-W-S Connections

For this connection, the cross section of the 63 mm LVL timber after 30 minutes of fire will be 19.8 x 106.8 mm.

The tensile stress in the LVL member after 30 minutes fire will be \( 29.2 \text{kN} \times 1000 / \left(23.4 \times 106.8 \text{mm}\right) = 13.81 \text{MPa} \). This is less than the tensile stress of LVL which is 26 MPa.
The tensile stress of the timber is checked with the tensile load applied during the testing.

\[ T^* \leq \Phi T_f \]

where \( T_f = f_i \), \( A_s \) is the area of cross section of LVL after 30 minutes exposure to fire (mm\(^2\)) and \( f_i \) is the characteristic tensile strength of LVL timber (MPa).

\[ : T^* \leq 1.0 \times 26 \text{ MPa} \times (19.8 \times 106.8 \text{ mm})/1000 = 54.98 \text{ kN} \]

A similar process has been used to examine whether the timber can withstand 45 minutes of fire exposure. However, it is calculated that the timber would fail by then.

The timber piece would fail in 43.5 minutes when exposed to the fire as shown on Figure 3-5.

![Figure 3-5 – Tensile strength (kN) vs Time (min) for exposure on both side of 63 mm LVL timber](image)

### 3.6.3 Checking Stress in Heated Steel Plate


\[ \frac{f_y(T)}{f_y(20)} = 1.0 \text{ when } 0 ^\circ \text{C} < T \leq 215 ^\circ \text{C} \]
\[
\frac{902 - T}{690} \quad \text{when} \quad 215^\circ C < T \leq 905^\circ C
\]

where 

\( f_y(T) \) = yield stress of steel at \( T \) °C \\
\( f_y(20) \) = yield stress of steel at 20 °C \\
\( T \) = temperature of the steel in °C

The relationship is shown by Figure 3-6.

![Figure 3-6 – Variation of yield stress of steel with temperature](image)

**3.6.3.1 Steel Plate in Connections**

The tensile stress of the plate must be greater or equal to the design tensile strength, as shown below.

\[ T^* \leq \Phi T_s \]

where \( T_s = f_y A_s \), \( A_s \) is the area of cross section of steel plate (mm\(^2\)) and \( f_y \) is the characteristic tensile strength of the steel plate (MPa).
For typical mild steel, the tensile yield strength of the plate is 275 MPa. The tensile yield strength is decreases as the temperature of steel increases as shown in Figure 3-6. The strength reduction factor Φ is taken as 0.9 for steel plate.

The size of steel plate used in nailed and screwed connections in S-W-S arrangement is 2 x 130 mm x 3 mm thick and for bolted connections is 2 x 130 mm x 6 mm thick. The size of steel plate used in W-S-W arrangement is 120 mm x 6 mm thick.

From Figure 3-7, the steel in W-S-W arrangement and nailed and screwed connection in S-W-S arrangement would fail when the temperature of the steel reaches 800 °C. The steel used in bolted connection for S-W-S arrangement fail around 850 °C.

Figure 3-7 – Tensile Stress of each steel plate (kN) and its variation with temperature
Chapter 4 Preparation of Test Specimens

4.1 Overview
This chapter will provide an overview of all the materials that will be used in the test specimens. In particular, the chapter describes the preparation of the three types of connection being tested, and the fasteners used in each case.

4.2 Laminated Veneer Lumber (LVL)
In this research, Carter Holt Harvey (CHH) supplied all the LVL directly from the Marsden Point factory or from the Distribution Centre in Auckland. The LVL used for all the test is 45 or 63 mm thick which is widely used in New Zealand construction industry. The width of the LVL is 150 mm, a stock size produced by the manufacturer.

HYSPAN LVL is constructed from 3 mm thick laminates of radiate pine, glued together with a phenolic adhesive to form a continuous billet. This billet can be varied in thickness and can be purchased in lengths up to 12 metres.

4.3 Description of Fasteners

4.3.1 Fasteners
The type of fasteners that are being used in the research can be classified into several main groups:

- Bolts
- Nails
- Screws
- Self-drilling Dowels

Each of these types of fasteners is discussed in the following sections.

4.3.2 Bolts
During the testing, the bolts used in the connections joints were M12 bolts. All the bolts used were purchased from Fastenings Supplies of Christchurch.
4.3.3 Nails

The nails used in the testing were Lumberlok 30mm x 3.15 diameter stainless steel which have a characteristic load of 1.0 kN and serviceability load of 0.69 kN.

4.3.4 Screws

The screws used in the experiments are 3.10 diameter with a characteristic load of 0.96kN.

4.3.5 Self – drilling Dowels

A 93 mm long and 5 mm diameter dowel was imported from SFS in Switzerland for this project. The dowel has a thread at the head end of the round pin to enable the dowel to be driven into the timber member and ensures dowel fixation in the timber member. The opposite end of the dowel contains a small flat blade with sharpened edges functioning as a drilling tip.

4.4 Description of Steel Plates

The steel plates used in the tests were 130 x 6 mm thick. The nailed-on plates which were used in nailed and screwed connections in S-W-S joints are 3mm thick.

4.5 Preparation of Test Specimens

All test specimens used in this research were produced from LVL block of 45 or 63 mm thick by 150 mm wide Hyspan LVL. A table saw was used to cut the specimens to the required width and length. Details for the different connection tested are given in the following sections.

4.5.1 W-W-W Connections

For this arrangement of connections, two types of fasteners were used: 12mm Ø 200mm long bolts and Type 17 Gauge 14 wood screws. Multiple tests were done to determine the strength of connections with the different fasteners used.

The LVL members used to construct each connection were cut from lengths of Hyspan LVL. The central member was cut from 63 x 150 mm LVL while the other members were cut from 45 x 150 mm LVL. All three members were cut to a length of 280mm. Special attention was given to having the edge distance for each arrangement comply with the code requirement as shown in Figure 2-4. The connections were fabricated as shown in Figure 4-1 and Figure 4-2
and were tested in an Instron Testing Machine for the cold tests. Two potentiometers were fixed at both sides of the specimens and the deflection was measured as the load was applied. Two identical replications of each joint type were fabricated.

Figure 4-1 – Arrangement of bolted connections for W-W-W cold tests
For the furnace tests, almost the same arrangements were used except that instead of a single connection being tested at a time as in the cold tests, two connections as in normal construction were tested as shown on Figure 4-3 and Figure 4-4 below. The specimens were placed inside the furnace and loaded to 29.2 kN load as determined in Section 3.4. During the experiments, the deflections of the connections were measured by fixing potentiometers on the sides of the specimens. The temperatures were measured at three locations inside all the specimens:

- the temperature of the fasteners inside the timber,
- temperature between the two piece of timbers,
- surface temperature during the whole duration of the experiments
4.5.2 S-W-S Connections

For this arrangement of connections, three types of fasteners were used: 12mm Ø 100mm long bolts, Lumberlok nails and 3.10mm diameter screws. Multiple tests were done to determine the strength of connections using the different fasteners.
Preparation of Test Specimens

The LVL members used to construct each connection were cut from lengths of Hyspan LVL. The length of the LVL is shown figures shown in Figure 4-5 below and had a cross section of 63 x 150 mm and length of 200 mm long in Hyspan LVL.

![Figure 4-5 – Arrangement of bolted connections for S-W-S cold tests](image)

![Figure 4-6 – Arrangement of nailed connections for S-W-S cold tests](image)
Preparation of Test Specimens

Figure 4-7 Arrangement of screwed connections for S-W-S cold tests

The mild steel plate used in the bolted connections was cut to a length as shown in Figure 4-5 to ensure ample length protruding beyond the end of the LVL members. This allowed a small piece of LVL to be fixed on the ends to prevent the steel plate from buckling outwards or inwards. For both nailed and screwed connections, nail-on plates were used. Similar methods were used to prevent buckling of the steel plates. The nail-on plates were pre-drilled, so the arrangement of the nails and screws would follow the drilled holes on the plates.

The connections were fabricated as shown in Figure 4-5, Figure 4-6 and Figure 4-7 and were tested in an Instron Testing Machine for the cold tests. Two potentiometers were fixed at both sides of the specimens and the deflection was measured as the load was applied. Two replications of each joint type were fabricated.

For the furnace test, almost the same arrangement was used except that instead of a single connection being tested as in the cold tests; two connections as in construction were tested as shown on Figure 4-8, Figure 4-9 and Figure 4-10 below. The specimens were loaded inside the furnace to 29.2kN. During the experiments, the deflections of the connections were
Preparation of Test Specimens

measured by fixing potentiometers on the sides of the specimens. The temperatures were measured at three locations inside all the specimens:

- the temperature of the fasteners inside the timber,
- temperature between the steel plates and timber,
- surface temperature during the whole duration of the experiments

Figure 4-8 – Arrangement of bolted connections for S-W-S furnace tests

Figure 4-9 – Arrangement of nailed connections for S-W-S furnace tests
4.5.3 W-S-W Connections

For this arrangement of connection, two types of fasteners were used: 12 mm Ø 120mm long bolts and self-drilling dowels from SFS. Multiple tests were done to determine the strength of connections using the different fasteners.

The LVL members used to construct each connection were cut from lengths of Hyspan LVL. The length of the LVL is according to figures shown below with cross section of 45 x 150 mm in Hyspan timber. The steel plate is sandwiched in between two pieces of timber for ease of construction, instead of cutting through the timber pieces. Two replications of each joint type were made.

From Figure 4-11 and Figure 4-12, the fasteners that were really being tested were on the right side. The left hand side connections were just merely acts as a stabilizer; by holding the steel into place and prevent it from buckling.
Preparation of Test Specimens

Figure 4-11 – Arrangement of bolted connections for W-S-W cold tests

Figure 4-12 – Arrangement of doweled connections for W-S-W cold tests
For the furnace test, the test specimens shown on Figure 4-13 and Figure 4-14 were tested. The specimens were loaded inside the furnace to 29.2kN. During the experiments, the deflections of the connections were measured by fixing potentiometers on the sides of the specimens. The temperatures were measured at three locations inside all the specimens:

- the temperature of the fasteners inside the timber,
- temperature between the steel plates and timber,
- surface temperature during the whole duration of the experiments.

Figure 4-13 – Arrangement of bolted connections for W-S-W furnace tests
4.6 Heating Regime

During the fire tests, the temperature in the furnace could not follow the standard time-temperature curve in accordance with ISO 834 due to the different heating method used. The heating in the ISO 834 is done using the gas burner while the heating in the furnace for this research is by electrical coil.

The temperature that was achieved in the furnace is shown in Figure 4-15 with comparison with the ISO 834 curve. It can be seen in the figure that the fire curve achieved from the furnace could not achieve the rise in the temperature in the early stages of the fire. However, after 5 minutes, there is some agreement in the temperature.

Due to the difference in the time-temperature curve that was achieved in the furnace, some correlation has to be done to relate the temperature to the ISO 834 curve.
Figure 4-15 – Comparison between the ISO 834 fire curve and fire curve achieved from furnace in research
5.1 Overview

Cold testing was carried out to compare the performance of different fasteners in different connection arrangements under normal operating temperatures. This chapter describes the results from the tests.

5.2 W-W-W Connections

In W-W-W connections, two types of fasteners are used: 12mm bolts and Type 17 – Gauge 14 screws. The two types of connections are loaded using an Instron Testing machine to find out the ultimate load.

5.2.1 Bolted Connections

The failure observed in all the tests in this series occurred in the timber. There were no failures of the bolts although they were seen to be slightly bent at the end of the tests. Tests 1 and 2 are similar and Tests 3 and 4 are similar. Tests 1 and 2 shown on Figure 5-1 were bolted connections with normal round washers (Figure 5-2) which do not meet the requirements of the Timber Standard while tests 3 and 4 used 30mm square washers as required by the Standard (Figure 5-3).

From the results, there is not much difference between the cases where normal round washers and standard required washers were used as both of the ultimate loads are greater than the design load of 88 kN. The bolted connections using normal round washers failed around 180 kN while the connections using the standard required washers failed at around 200 kN.
The failure of the LVL timber occurs by splitting at the locations of the bolts and was parallel to the grain of the LVL as shown in Figure 5-2. Meanwhile the bolts were bent in the test as shown in Figure 5-3 but still maintained their initial strength. This failure mode suggests that there was sufficient strength in the steel bolts; but that the timber was crushed by the steel bolts causing splitting to occur along the direction parallel to grain.
Cold Testing Results

To minimize the splitting of the timber, the end distance of the connections can be increased. However, since the ultimate load of the connections already exceeds the design load, it was not necessary to do so. A similar failure mode was seen with connections using bigger washers as required by the Standard.
5.2.2 Screwed Connections

The ultimate load achieved by the screwed connections in the W-W-W arrangement is 250 kN which is higher than the design load as shown in Figure 5-4. The screwed connections achieved more than 180% greater strength than anticipated. The failures observed during the tests were by the failure of the timber at the location where the screws are bent as the load is applied. Subsequently, failure lines could be seen along the timber perpendicular to the screws. The splitting of the timber then causes the connections to fail. As shown in Figure 5-5, the screws bent and broke during the test and two plastic hinges can be seen. These plastic hinges are located at the interfaces of the timber members. The screws were broken at very large displacements. This breakage could have occurred when the maximum load of the screws were reached during the tests.
Cold Testing Results

Figure 5-4 – Strength of the screwed connection in W-W-W cold tests

Figure 5-5 – Bending and fracture of screws as seen after the test
5.2.3 Comparison between bolted and screwed fasteners

Figure 5-6 shows the average results from the bolted and screwed connections tested. It can be seen that the experimental results for the two types of fasteners used have higher ultimate loads than the design load as calculated by NZS3603:1993. It can also be seen that the ultimate loads for screwed connections are relatively higher than the ultimate loads achieved by the bolted connections. The screwed connection is also stiffer than the bolted connection.

![Comparison of the bolted and screwed connections](image)

The ultimate load achieved by the screwed connection is 250kN while the bolted connection achieved 205kN. That is an 18% difference. When tested, the screwed connections last longer as failure needs to occur at several places before total failure occurs. Even when failure occurs on one or two screws, other screws in the connections have the ability to carry the applied load. Meanwhile, for bolted connections the load suddenly dropped immediately after failure as there are only three places in the bolted connections required to fail as shown in Figure 5-2.

It can also be seen that when the screwed connections reach the ultimate load, the displacement is around 7mm while it is nearer 14mm for bolted connections. The screws tend to hold the timber pieces tightly together and prevent movement. For bolted connections, the holes are pre-drilled first before the bolts are put into place. This results in some spacing
between the bolts and the holes. When the bolted connections were loaded, the movement in
the first few minutes is the movement between the bolts and the drilled holes.

In conclusion, screwed connections in a W-W-W arrangement are better than bolted
connections in terms of strength, displacement and most importantly aesthetics.

5.3 S-W-S Connections

In S-W-S connections, three types of fasteners are used: 12mm bolts, lumberlok nails and
3.10mm diameter screws. All three types of fasteners are loaded to find out the ultimate load.

5.3.1 Bolts

Tests 1 and 2 shown in Figure 5-7 were bolted connections with 50% of the edge distance
recommended by NZS3603:1993 while tests 3 and 4 used the edge distance required by the
Standard. From the results, there is 15% difference of ultimate load between the 50% and
100% edge distance recommended by the Standard. However, all ultimate loads were
considerably more than the design load of 88 kN.

![Figure 5-7 – Strength of bolted connections in S-W-S arrangement](image_url)
The bolted connections using 50% edge distance failed around 140 kN while one of the connections using the standard required edge distance failed at around 160 kN.

The failure in all four cases was in the LVL timber where splitting occurs at the locations of the bolts and the splits were parallel to the grain of the timber. Meanwhile the bolts were bent in the test. This failure mode suggest that there was sufficient strength in the steel rod, but that the timber was crushed by the steel bolts causing splitting in the direction perpendicular to the steel rod.

5.3.2 Nails

For the nailed connections, the ‘design’ load was reduced to 44 kN which is half of the value that was calculated in 3.5.2.1 as initially designed. This is because during the compression test, the steel plates buckled which caused the premature failure. This does not give an accurate prediction of the strength of the nails. Therefore, the number of nails was reduced to half of the numbers needed and the applied load was reduced to 44 kN.

![Figure 5-8 – Strength of nailed connections in S-W-S arrangement](image)

From Figure 5-8, the ultimate load of the nailed connections is 90 kN which is more double the design load. This is very promising considering that this type of connection is used widely for roof construction in New Zealand. It can be seen that for both tests, the failure load is
Cold Testing Results

almost identical. The failure mode seen in this connection is the nails are being pulled out as the load was applied. The movement of the steel plates causes the nails to lose their grip in the timber and failure occurs almost immediately as shown in Figure 5-8. The bending of the nails can be seen in Figure 5-9.

![Figure 5-9 – Bending of nails](image)

### 5.3.3 Screws

For screwed connections, the design load was 44 kN, being is the same as the nailed connections on account of using the same thickness of the nailed-on plates. Again, the numbers of screws were halved as was the case for the nailed connections.

![Figure 5-10 – Strength of screwed connections in S-W-S arrangement](image)
Cold Testing Results

As seen in the load displacement results shown in Figure 5-10, the screwed connections performed very poorly in the compression tests. The average failure load of 43.2kN is lower than the design load. The cap of the screws came off easily during the test which is defined as failure area of the screws as the screws can no longer withstand load. This could be due to the process of manufacturing of the screws where a groove is cut on a piece of tiny rod to make it into a screw. The weakest point of the screw is at the cap as that is where the cutting of grooves ends, thus creating the weakest point where the load is concentrated at the area. Therefore, the cap of the screws just could not withstand the load applied.

As the connection performed poorly in the cold tests, it was decided that a furnace or fire testing would not be of benefit.

5.3.4 Comparison between bolted, nailed and screwed fasteners

It can be seen from Figure 5-11 and Figure 5-12 that the experimental results for bolted and nailed connections have higher ultimate loads than the design load as calculated to NZS3603:1993. For screwed connections, the ultimate load is lower than the design load which is not acceptable.
The ultimate load was reached in the bolted connections when the displacement of the connections is near 4mm. When they failed, the connections suddenly lose the strength and the load reduced steeply as shown in Figure 5-11. It is the same case as the nailed and screwed connections as shown in Figure 5-12 where the load just fell suddenly when failure occurred.

In conclusion, bolted and nailed connections in S-W-S arrangement are better than screwed connections in terms of strength and displacement.

**5.4 W-S-W Connections**

In W-S-W connections, two types of fasteners are used: 12mm bolts and self-drilling dowels. Each connection with different fasteners was loaded in the Instron Testing Machine to find out the ultimate load.

**5.4.1 Bolts**

Test 1 on Figure 5-13 is for a bolted connection with 50% of the edge distance recommended by NZS3603:1993 while test 2 and test 3 used the edge distance required by the Standard. From the results, there seems to be no difference in the ultimate load between the 50% and 100% edge distance recommended by the Standard.
The failure is in the LVL timber where splitting occurs along the locations of the bolts and the splits were perpendicular to the grain of the timber. The bolts were bent in the test but still maintained their full strength. This failure mode suggested that there was sufficient strength in the steel rod, but that the timber was crushed by the steel rod causing splitting to occur along the direction perpendicular to the steel rod.

**5.4.2 Self-drilling Dowels**

The ultimate load achieved by the self-drilling doweled connections in W-S-W arrangement is 250 kN which is much higher than the design load. The screwed connections achieved 180% more strength than anticipated.
The connections clearly failed by a longitudinal crack on the inner side of the two LVL members. The crack then elongated through the thickness of the members to the external surface of timber member. As the load was increased, the cracks elongated along the length of the joint. The cracking resulted in a slow and predictable failure with small but not catastrophic loss of load carrying ability as shown in Figure 5-14. This is a very remarkable performance as the large cracks were appearing on the surface of the timber piece but the load carrying capacity is not lost.
Longitudinal cracking occurred along both outer edge rows of dowels as shown in Figure 5-15. As the outer rows of dowels lose their ability to carry substantial loads, the inner dowels attract their full load, resulting in a load plateau being established (Scheibmair 2003).

5.4.3 Comparison between bolted and self-drilling doweled connections

It can be seen from Figure 5-16 that the experimental results for the two types of fasteners used have higher ultimate loads than the design load as calculated by NZS3603:1993. The ultimate loads for self-drilling doweled connections are relatively higher than the ultimate loads achieved by the bolted connections.

The ultimate load achieved by the screwed connection is 250kN while the bolted connection achieved 215kN. That is approximately a 14% difference. When tested, the doweled connections last longer as failure needs to occur at several places before total failure occurs.
It can also be observed from the figure that the self-drilling dowels achieve the ultimate load faster than the bolted connections. When the dowelled connections reach ultimate load, the displacement is around 3mm while it is nearer 10mm for bolted connections. The dowels tend to hold the timber pieces tightly and prevent movement. This is because the dowels are drilled into the timber piece and are tight. For a bolted connection, holes are pre-drilled first before the bolts are put into place. This results in some gaps between the bolts and the holes. When the connections are loaded, the initial movement will be a result of the gap closures while subsequent displacements will be result of the interaction between the bolts and the LVL. The bolts are then crushed into the LVL during the tests which will increase the displacement of bolted connections slightly as there is a movement by the bolts to minimize the holes.

In conclusion, self-drilling dowels connections in W-S-W arrangement are better than bolted connections in terms of strength and displacement.
Chapter 6 Furnace Testing Results

6.1 Overview
The objective of this phase of testing was to measure the fire resistance of the connections tested in cold conditions using LVL timber. This was done by testing the specimens at a constant tensile load while being heated in a furnace until failure.

6.2 Testing Frame
A testing frame was constructed for the furnace tests. Load was applied to the frame using a manual pump, which fed the hydraulic ram attached to the holder for testing the specimen. The load applied was constant throughout the tests. The furnace is shown in Figure 6-1. The load applied was 29.2 kN which is the fire design load.

Figure 6-1 – The furnace that was constructed for furnace testing
The test specimens were positioned in the furnace and the air supply was regulated such that the heated specimen had approximately uniform charring in all four surfaces as shown in Figure 6-2.

![Figure 6-2 – The furnace with specimens inside](image)

### 6.3 Charring Tests

Two charring tests were carried out to investigate the consistency of heating in the furnace. These tests consisted of loading the furnace with 63 x 150 mm specimens but without any tensile load applied. These specimens were heated in the furnace for 20 minutes. Following the tests, the specimens were removed from the furnace flames, extinguished and allowed to cool.

For observation purposes, these test specimens were then cut open to expose the cross section. The layer of char was then removed to expose the remaining section for measurement.
The average charring rate achieved in the furnace is 0.67 mm/min which was slightly lower than that achieved by Lane et al. (2004) in a pilot test furnace heated in accordance with ISO 834, where an average charring rate of 0.72 mm/min was achieved for LVL timber.

The measured charring rate gave an indication that the furnace was less severe than the ISO 834 heating regime. A scaling factor was used to convert the results of the furnace testing to those achieved in a standard furnace test based on the depth of charring achieved and temperature impinging on the timber surface.

6.4 Results

During testing of the specimens, a slow decrease in load was observed with increasing temperature. Some of this was attributed to a slight leak in the hydraulic pump but most it is due to the expansion of the specimens as measured by the potentiometer. To maintain the same constant load for the whole duration of the experiment, the pump was used to keep the hydraulic ram applying load.

The displacements of the specimens were measured and temperature was measured on the timber surfaces and fasteners as described in Chapter 4.

After the experiments were started, it took all specimens between 2 and 5 minutes to heat sufficiently to begin charring. The onset of charring was only evidenced as smoke began to be expelled from the furnace. This was because the surfaces of the test specimens were not visible from the outside of the furnace. This led to a period where the test specimens were charring and building up pyrolyzates within the furnace, but prior to flaming occurring (Harris, 2004). After the pyrolyzates built up for a few more minutes, the gases reached their

---

Table 6-1 – Char depths for charring specimens

<table>
<thead>
<tr>
<th>Exposure (min)</th>
<th>Time to charring (min)</th>
<th>Time to Ignition (min)</th>
<th>Depth of Char (mm)</th>
<th>Average Char Rate (mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>3</td>
<td>4</td>
<td>11</td>
<td>0.65</td>
</tr>
<tr>
<td>20</td>
<td>2.5</td>
<td>4</td>
<td>12</td>
<td>0.69</td>
</tr>
<tr>
<td>Overall average</td>
<td></td>
<td></td>
<td></td>
<td>0.67</td>
</tr>
</tbody>
</table>

The measured charring rate gave an indication that the furnace was less severe than the ISO 834 heating regime. A scaling factor was used to convert the results of the furnace testing to those achieved in a standard furnace test based on the depth of charring achieved and temperature impinging on the timber surface.

6.4 Results

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unpiloted ignition temperature and ignited. From the point of ignition, there were flames on all surfaces of the test specimens until the conclusion of the test.

When the test specimens had failed, the furnace was switched off and the test specimens were quickly removed from the furnace. The flames were then quickly extinguished and the specimens were cooled with water to prevent further charring.

6.4.1 Time to Failure

The experiments started when a load of 29.2 kN was applied. The failure time was the time at which the test load was not able to be sustained by the test specimens.

<table>
<thead>
<tr>
<th>Connections</th>
<th>Sample</th>
<th>Time to Charring (min)</th>
<th>Time to Ignition (min)</th>
<th>Time to Failure (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts 1</td>
<td>3</td>
<td>4.9</td>
<td>23.2</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>2.8</td>
<td>4</td>
<td>21.7</td>
</tr>
<tr>
<td></td>
<td>Screws 1</td>
<td>3</td>
<td>4.5</td>
<td>32.7</td>
</tr>
<tr>
<td></td>
<td>Screws 2</td>
<td>3.25</td>
<td>4.7</td>
<td>33.9</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Bolts 1</td>
<td>3</td>
<td>4.5</td>
<td>11.5</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>3.5</td>
<td>4.5</td>
<td>11.3</td>
</tr>
<tr>
<td></td>
<td>Nails 1</td>
<td>3.5</td>
<td>4.45</td>
<td>10.5</td>
</tr>
<tr>
<td></td>
<td>Nails 2</td>
<td>3.3</td>
<td>4.6</td>
<td>10.8</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts 1</td>
<td>3</td>
<td>4.3</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>3.2</td>
<td>4.3</td>
<td>19.5</td>
</tr>
<tr>
<td></td>
<td>Dowels 1</td>
<td>2.85</td>
<td>4.7</td>
<td>27.5</td>
</tr>
<tr>
<td></td>
<td>Dowels 2</td>
<td>3</td>
<td>4.5</td>
<td>28</td>
</tr>
</tbody>
</table>

6.5 Prediction of Fire Resistance

As the tests specimens were not exposed to the standard ISO 834 fire in the furnace, the time of failure obtained from these tests was not the fire resistance of the connections. To convert from the time of failure in the furnace to an estimated fire resistance, the fire severity on the surface of the timber piece and the rate of char were analysed to correlate to the fire resistance of connections in ISO 834 fire.
6.5.1 Fire Resistance and Rate of Charring

During a fire a layer of char forms over the surface of unburnt timber which then shrinks and burns away after a period of time. The fire resistance of the cross section are based on the residual cross section after charring. The base of the char layer is at approximately 300°C, with a heated layer about 35mm thick below the char front. The part of this layer above 200°C is known as the pyrolysis zone which is undergoing thermal decomposition into gaseous pyrolysis products, accompanied by loss of weight, loss of strength and discolouration (Buchanan, A.H. 2005).

To measure the charring rate of the timber, the thickness of timber which has been converted to char was required. This thickness was best measured by removing the char layer after testing and measuring the remaining section to calculate the amount of timber which has been converted to char.

<table>
<thead>
<tr>
<th>Connections</th>
<th>Sample</th>
<th>Time to Failure (min)</th>
<th>Depth of Char (mm)</th>
<th>Average Char Rate (mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts 1</td>
<td>23.2</td>
<td>15</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>21.7</td>
<td>15</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>Screws 1</td>
<td>32.7</td>
<td>23</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>Screws 2</td>
<td>33.9</td>
<td>24</td>
<td>0.71</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Bolts 1</td>
<td>11.5</td>
<td>7</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>11.3</td>
<td>6</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>Nails 1</td>
<td>10.5</td>
<td>7</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>Nails 2</td>
<td>10.8</td>
<td>7</td>
<td>0.65</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts 1</td>
<td>19</td>
<td>11</td>
<td>0.58</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>19.5</td>
<td>12</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>Dowels 1</td>
<td>27.5</td>
<td>19</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>Dowels 2</td>
<td>28</td>
<td>19.5</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Mean char rate (mm/min) 0.65

The char rates tabulated in Table 6-3 are the average char over the duration of the test, which is the observed depth of char divided by the duration of exposure. This treats the period at the
beginning of the test before the onset of char as part of the charring rate. This was done to be consistent with the testing procedure of Lane et al. (2004) the source of the standard furnace charring data to be used for comparison (Harris, 2004).

It can be seen from Table 6-3 that the charring rates for the test specimens in the furnace range from 0.53 mm/min up to 0.70 mm/min, with a mean value of 0.65 mm/min. This was slightly lower than the average charring rate observed in the ISO 834 furnace by Lane et al. (2004) of 0.72 mm/min.

Harris (2004) developed a formula to convert the time in the custom-built furnace to time in the standard furnace.

\[
    t_{ISO} = \frac{c_{ISO}}{c_{cust}} t_{cust}
\]

where

- \( t_{ISO} \) Fire resistance time in ISO furnace (minutes)
- \( t_{cust} \) Time exposed to the custom furnace
- \( c_{ISO} \) Char rate recorded in the ISO furnace (0.71 mm/min)
- \( c_{cust} \) Char rate recorded in the custom furnace (mm/min)

The durations in the custom-furnace were converted to expected durations in the ISO furnace, or the fire resistance. The comparison between the durations in the custom-furnace and the fire resistance is shown in Figure 6-3.
From these comparisons, the results in Table 6-3 can be recalculated with addition of the fire resistance of the test specimens. This is shown in Table 6-4.

![Figure 6-3 – Comparison of recorded furnace exposures with fire resistance](image)

**Table 6-4 – Calculated fire resistance using the charring rate method**

<table>
<thead>
<tr>
<th>Connections</th>
<th>Sample</th>
<th>Time to Failure (min)</th>
<th>Depth of Char (mm)</th>
<th>Average Char Rate (mm/min)</th>
<th>Calculated Fire Resistance (mins)</th>
<th>Average Fire Resistance (mins)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts 1</td>
<td>23.2</td>
<td>15</td>
<td>0.65</td>
<td>21.2</td>
<td>21.2</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>21.7</td>
<td>15</td>
<td>0.69</td>
<td>21.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Screws 1</td>
<td>32.7</td>
<td>23</td>
<td>0.70</td>
<td>32.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Screws 2</td>
<td>33.9</td>
<td>24</td>
<td>0.71</td>
<td>33.9</td>
<td>33.1</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Bolts 1</td>
<td>11.5</td>
<td>7</td>
<td>0.61</td>
<td>9.9</td>
<td>9.9</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>11.3</td>
<td>6</td>
<td>0.53</td>
<td>8.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Nails 1</td>
<td>10.5</td>
<td>7</td>
<td>0.67</td>
<td>9.9</td>
<td>9.9</td>
</tr>
<tr>
<td></td>
<td>Nails 2</td>
<td>10.8</td>
<td>7</td>
<td>0.65</td>
<td>9.9</td>
<td></td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts 1</td>
<td>19</td>
<td>11</td>
<td>0.58</td>
<td>15.5</td>
<td>15.5</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>19.5</td>
<td>12</td>
<td>0.62</td>
<td>17.0</td>
<td>16.3</td>
</tr>
</tbody>
</table>
### 6.5.2 Fire Resistance and Fire Severity Correlation

The fire severity for each type of connections was plotted against exposure time and shown in Figure 6-4 to Figure 6-17. All these figures showed a similar trend with the fire severity increasing as the exposure time increased but eventually reaching a limiting value.

### 6.5.3 W-W-W

#### 6.5.3.1 Bolted Connections

The correlation between fire severity and exposure time for bolted connections in the W-W-W arrangement is shown in Figure 6-4. The figure shows the average value from Bolt 1 and Bolt 2 in Table 6.4. The fire severity curve for the connections is relatively similar in shape with the fire severity curve of the ISO 834 fire curve. The connections initially failed at 22.5 minutes in the tests but after correlation with the fire severity curve for the standard fire, the failure time is 20.5 minutes.

![Figure 6-4 – Correlation between time of exposure and fire severity for bolted connection in W-W-W](image-url)
The failure of the LVL timber in furnace tests occurs by splitting at the locations the bolts and was along the grain of the timber as shown in Figure 6-5. Meanwhile there is no bending of the bolts observed. The bolts can be seen to have charred through the timber and the resulting elongation of the bolt holes caused the connections to fail.

6.5.3.2 Screwed Connections

The correlation between fire severity and exposure time for screwed connections in the W-W-W arrangement is shown in Figure 6-6. The average result of the 2 replications of the connection is illustrated in figure. The fire severity curve for the connections is very close to the fire severity curve of the ISO 834 fire curve. The connections initially failed in 33.3 minutes in the tests but after correlation with the fire severity curve for the standard fire, the failure time is 30.3 minutes.
The failure of the screwed connection in LVL timber in furnace tests occurs by splitting at the locations the screws. The side piece of the connections fail due to the loss of the timber piece as it was burnt in the furnace as shown in Figure 6-7. The wood around the screws burnt and the charring around the screws is more severe.

The condition of the screwed connections before and after the furnace test is shown in Figure 6-8.
6.5.4 S-W-S

6.5.4.1 Bolted Connections

The correlation between fire severity and exposure time for the bolted connections in the S-W-S arrangement is shown in Figure 6-9. The figure illustrated the average results from the two replication tests. The fire severity curve for the connections differs from the fire severity curve of the ISO 834 fire curve. The connections initially failed in 11.4 minutes in the tests but after correlation with the fire severity curve for the standard fire, the failure time is 8.8 minutes.
The failure mode is similar to that for the bolted connections in the W-W-W arrangement. The failure of the LVL timber in the furnace tests occurs by splitting at the locations of the bolts out to the end and was parallel to the grain of the timber as shown in Figure 6-10. Large displacement could be seen in the connections after the furnace test as shown in Figure 6-11 where a large gap could be seen between the ends of the two pieces of LVL timber.
6.5.4.2 Nailed Connections

The correlation between fire severity and exposure time for the nailed connections in S-W-S arrangement is shown in Figure 6-12. The connections initially failed in 10.7 minutes in the
Furnace Testing Results

tests but after correlation with the fire severity curve for the standard fire, the failure time is 9 minutes.

Figure 6-12 – Correlation between time of exposure and fire severity for nailed connection in S-W-S arrangement

The failure of the nailed connection in LVL timber in the furnace tests occurs by nail deformation and timber embedment failure as shown in Figure 6-13.

Figure 6-13 – Before and after condition of nailed connection in S-W-S arrangement in furnace tests
6.5.5 W-S-W

6.5.5.1 Bolted Connection

The correlation between fire severity and exposure time for the screwed connections in the W-S-W arrangement is shown in Figure 6-14. The fire severity curve for the connections is quite similar to the fire severity curve of the ISO 834 fire. The connections initially failed in 19.3 minutes in the tests but after correlation with the fire severity curve for the standard fire, the failure time is 16.5 minutes.

The failure mode is similar to that for the bolted connections in the W-W-W and S-W-S arrangement. The failure of the LVL timber in the furnace tests occurs by splitting at the locations of the bolts out to the end and was parallel to the grain of the timber. Large displacements could be seen due to the burning of the timber by the hot bolts in the furnace as shown in Figure 6-15.

![Figure 6-14 – Correlation between time of exposure and fire severity for bolted connection in W-S-W arrangement](image-url)
6.5.5.2 Self-drilling Doweled Connection

The correlation between fire severity and exposure time for the bolted connections in the W-S-W arrangement is shown in Figure 6-17. The fire severity curve for the connections is very close to the fire severity curve of the ISO 834 fire curve. The connections initially failed in

Figure 6-15 – Failure mode of the bolted connections in W-S-W arrangement

Figure 6-16 – Before and after condition of bolted connection in W-S-W arrangement in furnace tests
27.8 minutes in the tests but after correlation with the fire severity curve for the standard fire, the failure time is 25.8 minutes.

![Correlation between time of exposure and fire severity for self-drilling doweled connection in W-S-W](image)

**Figure 6-17 – Correlation between time of exposure and fire severity for self-drilling doweled connection in W-S-W**

The failure of the screwed connection in LVL timber in the furnace tests occurs by splitting at the locations of the dowels. The side piece of the connections fail due to the loss of the timber as it was burnt in the furnace as shown in Figure 6-18.

The condition of the screwed connections after the furnace test is shown in Figure 6-19.
Figure 6-18 – Failure mode of self-drilling doweled connection in W-S-W arrangement

Figure 6-19 – Conditions of self-drilling doweled connections in W-S-W arrangement after furnace test
6.5.6 Summary of Fire Resistance and Fire Severity

Comparisons were made between the time of failure and the fire resistance found using the relationship between fire resistance and fire severity. The summary of the fire resistance is shown in Table 6-5. The time to failure is the duration in the custom-furnace.

<table>
<thead>
<tr>
<th>Connections</th>
<th>Fasteners</th>
<th>Time to Failure (mins)</th>
<th>Fire Resistance (mins)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts</td>
<td>22.5</td>
<td>20.5</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td>33.3</td>
<td>30.3</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Bolts</td>
<td>11.4</td>
<td>8.8</td>
</tr>
<tr>
<td></td>
<td>Nails</td>
<td>10.7</td>
<td>9</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts</td>
<td>19.3</td>
<td>16.5</td>
</tr>
<tr>
<td></td>
<td>Dowels</td>
<td>27.8</td>
<td>25.8</td>
</tr>
</tbody>
</table>

6.6 Comparison between Fire Resistance from Charring Rate and Fire Severity

The comparison between the fire resistance found using the charring rate and the fire severity correlation is shown in Table 6-6. This comparison was made for all test specimens tested in the custom furnace.

<table>
<thead>
<tr>
<th>Connections</th>
<th>Fasteners</th>
<th>Time to Failure (mins)</th>
<th>Fire Resistance from Charring Rate (mins)</th>
<th>Fire Resistance from Fire Severity Correlation (mins)</th>
<th>Difference in Fire Resistance (mins)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts</td>
<td>22.5</td>
<td>21.2</td>
<td>20.5</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td>33.3</td>
<td>33.1</td>
<td>30.3</td>
<td>2.8</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Bolts</td>
<td>11.4</td>
<td>9.2</td>
<td>8.8</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>Nails</td>
<td>10.7</td>
<td>9.9</td>
<td>9</td>
<td>0.9</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts</td>
<td>19.3</td>
<td>16.3</td>
<td>16.5</td>
<td>-0.2</td>
</tr>
<tr>
<td></td>
<td>Dowels</td>
<td>27.8</td>
<td>27.2</td>
<td>25.8</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Table 6-6 shows that there is a slight difference in fire resistance of the connections between the fire resistance found from the charring rate and the fire resistance found using the fire
severity correlation. The difference in fire resistance for every connection is within a range of 2 minutes except for the case of screwed connections in W-W-W arrangement.

From Table 6-6, the fire resistance from the charring rate seems to overpredict the fire resistance. This may be caused during the experiments when the charring around the timber cross section was removed and the residual cross section was measured. The measured residual cross section is very sensitive as 1mm difference will cause around 1.5 minutes difference in fire resistance. Therefore, the method and process must be carried properly and precisely to ensure that the residual cross section is measured accurately.

6.7 Fire protection

In this section, intumescent materials were used to cover the steel plates used in the experiments. Intumescent materials were used in the S-W-S and W-S-W tests as only those specimens used steel plates. In the S-W-S specimens, the steel plates were painted with intumescent paint. In the W-S-W specimens the gap between the steel plates and the timber piece due to the difference in width was filled up with intumescent sealants to protect the steel plates from fire exposure.

6.7.1 Fire Protection in S-W-S

Figure 6-20 and Figure 6-21 show the fire severity correlation for unprotected and protected connections i.e. with or without application of intumescent paint on the external surfaces of the steel plates. The condition of the connections before and after application of the intumescent paint can be seen in Figure 6-22.
For the bolted connections, there is a great increase of fire resistance due to the application of the intumescent paints. The fire resistance before the application of protection is 8.8 minutes and after the application, the connections failed at 18.7 minutes. There is a 112% of increase in fire resistance.
Figure 6-21 – Comparison of fire severity between protected and unprotected nailed connections in S-W-S

For the nailed connections, there is also a great increase of fire resistance due to the application of the intumescent paints. The fire resistance after the furnace tests shows that the fire resistance increased from 9 minutes to 17.2 minutes after the application of protection. There is a 91% of increase in fire resistance.

Figure 6-22 – Before and after condition of S-W-S arrangement in furnace test with application of intumescent paints
6.7.2 Fire Protection in W-S-W

Figure 6-23 and Figure 6-24 show the fire severity correlation for unprotected and protected connections. The condition of the connections before and after application of the intumescent paint can be seen in Figure 6-25.

![Diagram showing comparison of fire severity between protected and unprotected bolted connections in W-S-W](image)

**Figure 6-23 – Comparison of fire severity between protected and unprotected bolted connections in W-S-W**

For the bolted connections, there is an increase of fire resistance due to the use of the intumescent material. The fire resistance before the application of protection is 16.5 minutes and after the application, the connections failed at 19.7 minutes. There is a 19% of increase in fire resistance.
Figure 6-24 – Comparison of fire severity between protected and unprotected self-drilling doweled connections in W-S-W

For the self-drilling doweled connections, there is also an increase of fire resistance due to the application of the intumescent materials. The fire resistance after the furnace tests shows that the fire resistance increased from 25.8 minutes to 29 minutes after the application of protection. There is a 12.4% increase in fire resistance.

Figure 6-25 – Before and after condition of W-S-W arrangement in furnace test with application of intumescent paints
6.8 Conclusions from Furnace Testing

The comparison between fire resistance of unprotected and protected connections found using fire severity correlation is shown in Table 6-7. This comparison was made for all test specimens tested in the custom furnace.

<table>
<thead>
<tr>
<th>Connections</th>
<th>Fasteners</th>
<th>Fire Resistance (min)</th>
<th>Increased in Performance %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unprotected Connections</td>
<td>Protected Connections</td>
<td></td>
</tr>
<tr>
<td>S-W-S</td>
<td>Bolts</td>
<td>8.8</td>
<td>18.7</td>
</tr>
<tr>
<td></td>
<td>Nails</td>
<td>9</td>
<td>17.2</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts</td>
<td>16.5</td>
<td>19.7</td>
</tr>
<tr>
<td></td>
<td>Dowels</td>
<td>25.8</td>
<td>29</td>
</tr>
</tbody>
</table>

Table 6-7 shows that the S-W-S connections have a dramatic increase in fire resistance after the application of intumescent paint. The increase of performance ranges from 90% to 112%. Meanwhile, the intumescent materials have no significant influence on the W-S-W joint arrangement where there is only an increase of performance in the ranges of 12% to 19%. This is largely due to the fact that the steel plates are protected by the timber members and only the edges of the steel are exposed to the fire.
Chapter 7 Discussions

7.1 Overview

This section discusses the results achieved and comparison with some tests done previously.

7.2 Temperatures in Joints

The temperature in the joints was recorded. It was measured on the fastener surface and between the side and central member. Examples of measured temperatures are shown as a function of time in Figure 7-1.

![Temperature measured on the bolt surface in S-W-S as a function of time](image)

The figure shows that the temperature in the bolt in an S-W-S joint rise as the time increased. It is also clear that the heat flux through the bolt results in a temperature rise in the wood close to the bolt. However, it is obvious that the bolt temperature in the central member for the unprotected connection is higher than the bolt temperature for the protected connection. The intumescent material used on the surface of the side steel plate acts as insulation to prevent rapid temperature rise in the steel plate and the bolts. The rapid temperature rise in the bolt in an unprotected connection causes the connection to fail earlier as shown in the figure.
The timber around the hot bolts was burnt. The heat was conducted through the bolts to the centre of the timber.

Direct comparison between different types of fasteners could not be done due to the diverse temperatures impinging on the surface of the connections. The temperature in the bolts was higher than the temperature in the nails, screws and self-drilling dowels. The bolt which has large cross section tends to conduct more heat to inside the timber and play a major influence on the failure of the connections.

Besides, the S-W-S connections also failed in the early stages due to the high ratio of heated perimeter to cross sectional area for the exposed steel plates. For this reason, connections with embedded steel plates as in the case of W-S-W tend to have much better fire resistance than connections with exterior steel plates exposed to the fire.

### 7.3 Fire Resistance

König (2005) states that for connections with side members of wood, i.e. the W-W-W and W-S-W connections, fire resistance durations of 15 minutes (eg. nails, screws, bolts) or 20 minutes (dowels) is achievable. This can be seen to be in line in this research where the fire resistance for those connections is shown in Table 7-1. For screw and bolt fasteners, the fire resistance is more than 15 minutes and the dowels fail at more than 20 minutes.

<table>
<thead>
<tr>
<th>Connections</th>
<th>Fasteners</th>
<th>Fire Resistance (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts</td>
<td>20.9</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td>31.7</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts</td>
<td>16.5</td>
</tr>
<tr>
<td></td>
<td>Dowels</td>
<td>25.8</td>
</tr>
</tbody>
</table>

When a fire resistance of 30 minutes is required, the thickness and width of the side members and the end and edge distance to fasteners should be increased, except in the case of bolted connections, since the bolt heads give rise to an increased heat flux into the timber members. Eurocode 5 – “Design of timber structures” suggests that for connections with bolts and any other of the above mentioned fasteners, for fire resistances up to 60 minutes, the connection
Discussions

should be protected by an additional layer of timber, wood-based panel or gypsum plasterboard. Alternatively, the fasteners should be protected by bonded-in plugs.

Erchinger et al.(2005) show that a cross section of 200 x 200mm can achieve more than 30 minutes fire resistance. However, in this research the timber size used is 2 x 45 x 150 mm which fails in 25.8 minutes. The fire resistance will increase if similar cross sections are used. Frangi and Mischler (2004) performed an extensive study into the fire tests on dowel-type fasteners in timber connections. When the timber dimensions were increased from 200 x 200 mm to 280mm x 280mm, the failure time increased from 30 minutes to more than 70 minutes. The observed increased fire resistance corresponds to an improvement of about 1 minute for 1mm of increased timber cover of the connection. To achieve 30 minutes fire rating in LVL timber, the timber size used should be 100 x 150mm with slotted-in steel plates.

Frangi and Mischler (2004) also performed some fire tests with nails with steel side plates. The failure time achieved was 11.8 minutes where the failure mode is nail deformations and timber embedment failure. In this research, the failure time achieved was 9 minutes which is around 31% lower. However, the length of nails used by Frangi and Mischler is unknown and the diameter was 4mm. The diameter used in this research was 3.1mm which may cause the difference in the failure time.

7.4 Effect of Intumescent Materials on Fire Resistance

The application of intumescent materials on the S-W-S and W-S-W connections showed increased fire resistance. As shown in Table 6-7, the failure times of the nailed connections and bolted connections in S-W-S arrangement were 9 and 8.8 minutes. Thus by protecting the connections, the fire resistance was increased by about 8.2 and 9.9 minutes.

The intumescent material protects the steel plates from fire exposure which will reduce the heat transfer from the fire to the fasteners. This will reduce the heating around the fasteners and increase the fire resistance of connections.

Research done by Frangi and Mischler (2004) tested nailed steel side plate connections protected by a 2mm thick fire-proof coat. The failure times of the connections were increased from 11.5 to 28.5 minutes after protection of the connections. Although that kind of big
increase is not seen in this research, intumescent coating did increase the failure time around 91% to 112%. Better application and use of the intumescent coating will further increase the fire resistance of the connections.

7.5 Effect of Other Parameters on Fire Resistance

From research done by Erchinger et al. (2005), a reduction of the load level from $0.3F_u$ ($0.3 \times$ ultimate load) to $0.15 F_u$ led to an increased failure time of about 3 to 8 minutes. This slight increase in fire resistance seems not to be a relevant parameter to increase the fire resistance.

The increase of timber cover of the connections as well as the end distance will result in a much higher fire resistance. As tests done by Erchinger et al. (2005) show, the increase of timber cover of the slotted-in steel plates as well as an increase in the end distance by 40mm, the fire resistance of the connection reached more than 70 minutes. From a fire design point of view these modifications were very favourable in order to increase significantly the fire resistance of the connections.

Besides that, the connections with dowels protected by timber boards or gypsum plasterboards showed failure times of around 60 minutes (Frangi and Mischler, 2004). From tests done by Erchinger et al. (2005), protection of connections led to increased fire resistance of about 25 minutes after used of timber boards or gypsum plasterboards. This corresponds to an improvement of fire resistance of about 1mm for 1mm timber protection.
Chapter 8 Conclusions and Recommendations

8.1 Conclusions

During this research testing of the strength of connections in LVL timber was carried out using different types of fasteners in different connection arrangement. The types of fasteners used were Type 17 Gauge 14 150mm long wood screws, 12mm diameter galvanized bolts, 3.15mm diameter screws, Lumberlok nails and self-drilling dowels. The arrangements of the connections are shown below:

![Connection Arrangements](image)

Compression testing was initially performed at ambient temperature to compare the performance of these connections and fasteners at normal temperature. Then specimens were tested in tension in a custom-built furnace to investigate their fire resistance while carrying load. Finally, some connections were protected using intumescent material and tested in the furnace. From these tests, a number of conclusions can be drawn:

- Of the three types of connection arrangements, both the W-W-W and W-S-W connections were found to have similar strength at ambient temperature. The fasteners used in the S-W-S connections were found to have lower ultimate strength than that achieved for the two W-W-W and W-S-W connections. The summary of the tests at ambient temperature are shown below:
Conclusions

<table>
<thead>
<tr>
<th>Sample</th>
<th>Fasteners Type</th>
<th>Designed Load (kN)</th>
<th>Failure Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts</td>
<td>88</td>
<td>200.22</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td>88</td>
<td>255.66</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Nails</td>
<td>44</td>
<td>91.5</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td>44</td>
<td>43.2</td>
</tr>
<tr>
<td></td>
<td>Bolts</td>
<td>88</td>
<td>161.6</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts</td>
<td>88</td>
<td>216.07</td>
</tr>
<tr>
<td></td>
<td>Self-drilling dowels</td>
<td>88</td>
<td>249.9</td>
</tr>
</tbody>
</table>

- The failure mode observed at the location along the fasteners where the LVL timber was a split parallel to the direction of the fasteners. A greater edge distance could be used to increase the failure load. In this case, it is unnecessary as the failure loads in all experiments are 100 – 200% higher than the design load which can be considered successful and satisfying except for the screwed connection S-W-S joint.

- Small screws like the 3.15mm diameter used in this research are not recommended to be used in timber connections as the failure load for these connections is much lower than the design load. The cap of the screws came off easily during the test which is defined as failure of the screws as the screws can no longer withstand load. This is due to the process of manufacturing of the screws where a groove was cut on a piece of tiny rod to make it into a screw. The weakest point of the screw is at the cap as that is where the cutting of groves ends, thus creating the weakest point where the load is concentrated at the area.

- During furnace testing, the connections with large sections of wood last longer, such as the case of W-W-W and W-S-W connections. The S-W-S connection which has a large area of steel exposed to the fire failed very early in the tests due to the rapid charring of the wood due to the heat transfer from the fire to the fasteners and the timber area around the fasteners. The fasteners would lose grip and cannot hold the timber together. The fasteners then burnt through the timber and cause the edge distance to be smaller and smaller. In the end, failure occurs as splitting occurs along the edge of the timber.
Conclusions

- For the W-S-W joint, one specimen failed because of the loss of wood due to severe charring. The timber snapped because it could no longer withstand the load due to the small area of timber left. The fasteners were also close to failure as splits could be seen along the locations of the fasteners.

- The fire resistance of the connections was found by calibrating the char depths recorded in this research with results by Lane et al. (2004), who burnt LVL in the standard furnace using an ISO fire. Also, the fire resistance of connections was found using the “radiant exposure area correlation” concept suggested by Nyman (2001). The measure of fire severity at any time on the curve can be determined and this measure of fire severity can then simply equated to a real fire exposure having the same radiant exposure area, which is the equivalent fire severity.

- Bolted and nailed S-W-S connections failed very early in the experiments. The failure occurred after only 10 minutes. This is very alarming as these connections are used widely in the industry and could cause buildings to collapse if the structures with similar connections are exposed directly to fire.

- Intumescent materials applied on the connections tend to increase the fire resistance of the connections. Only the S-W-S and W-S-W connections required testing with intumescent protection. The summary of the failure time between unprotected and protected connections are shown below:

<table>
<thead>
<tr>
<th>Connections</th>
<th>Fasteners</th>
<th>Fire Resistance (min)</th>
<th>Increased in Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Unprotected Connections</td>
<td>Protected Connections</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Bolts</td>
<td>8.8</td>
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<td>17.2</td>
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<tr>
<td>W-S-W</td>
<td>Bolts</td>
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<td>19.7</td>
</tr>
<tr>
<td></td>
<td>Dowels</td>
<td>25.8</td>
<td>29</td>
</tr>
</tbody>
</table>
Conclusions

- Fire resistance ratings of 30 minutes are feasible for the W-S-W connection with self-drilling dowels but more tests should be carried out and better application of the intumescent materials needs to be observed.

- Fire resistance ratings of 60 minutes or more for all connections would either require an impractical thickness of timber for insulation or protection with other materials such as gypsum plaster board.

8.2 Recommendations for Further Research

- Measuring the temperature in the timber to determine at what temperature the connections start to fail. It is suggested that failure of connections occurs when the wood reaches a temperature of $80^\circ$C as this is the "glass transition temperature" of lignin. Lignin is the natural glue which holds the wood fibres together.

- Further assessment using other types of materials like gypsum or plasterboard to protect the connections to compare the difference in fire resistance.

- Performing fire tests using powder coating nail-on plates in the connections. Further assessment can be done to see to what extent the powder coating nail-on plates has on the fire resistance of the connections.

- In the furnace tests, it has been observed that the hot steel bolts were seen to have charred through the connections. Further investigation could look into temperature profile of the steel bolts and investigate the charring rate around the bolts and how significant the charring is on the fire resistance of the connections.
Chapter 9  References


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Appendix A Summary Cold Testing Results

<table>
<thead>
<tr>
<th>Connections</th>
<th>Sample</th>
<th>Designed Load (kN)</th>
<th>Failure Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts 1</td>
<td>88</td>
<td>200.22</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td>88</td>
<td>255.66</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Nails</td>
<td>44</td>
<td>91.5</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td>44</td>
<td>43.2</td>
</tr>
<tr>
<td></td>
<td>Bolts</td>
<td>88</td>
<td>161.6</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts</td>
<td>88</td>
<td>216.07</td>
</tr>
<tr>
<td></td>
<td>Self-drilling</td>
<td>88</td>
<td>249.9</td>
</tr>
<tr>
<td></td>
<td>dowels</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Appendix B Summary of Furnace Testing Results

<table>
<thead>
<tr>
<th>Connections</th>
<th>Sample</th>
<th>Time to Charring (min)</th>
<th>Time to Ignition (min)</th>
<th>Time to Failure (min)</th>
<th>Depth of Char (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-W-W</td>
<td>Bolts 1</td>
<td>3</td>
<td>4.9</td>
<td>23.2</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>2.8</td>
<td>4</td>
<td>21.7</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Screws 1</td>
<td>3</td>
<td>4.5</td>
<td>32.7</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td>Screws 2</td>
<td>3.25</td>
<td>4.7</td>
<td>33.9</td>
<td>24</td>
</tr>
<tr>
<td>S-W-S</td>
<td>Bolts 1</td>
<td>3</td>
<td>4.5</td>
<td>11.5</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
<td>3.5</td>
<td>4.5</td>
<td>11.3</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Nails 1</td>
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<td>4.45</td>
<td>10.5</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Nails 2</td>
<td>3.3</td>
<td>4.6</td>
<td>10.8</td>
<td>7</td>
</tr>
<tr>
<td>W-S-W</td>
<td>Bolts 1</td>
<td>3</td>
<td>4.3</td>
<td>19</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>Bolts 2</td>
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<td>Dowels 1</td>
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<tr>
<td></td>
<td>Dowels 2</td>
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<td>4.5</td>
<td>28</td>
<td>19.5</td>
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