# Assessing the Bolted Connection Strength of New Zealand Hardwood

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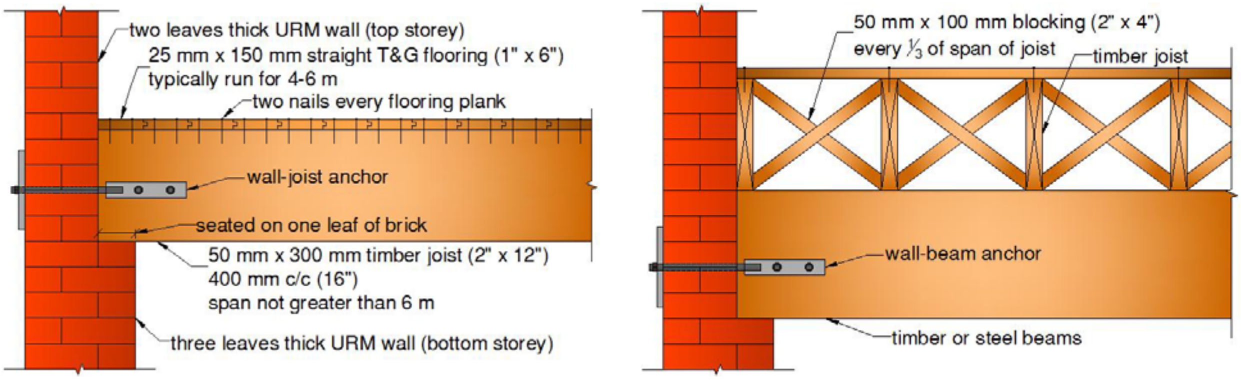
**ABSTRACT:** From published literature, it was found that through-bolt connections were typically applied as a retrofit technique to most New Zealand unreinforced masonry (URM) buildings following the 1931 Hawke's Bay earthquake. As connection failure by tearing out part of the diaphragm joist was observed in past earthquakes due to lateral earthquake loading, the strength of the bolted connection in existing indigenous New Zealand timber joists needs to be assessed. The main objectives of this study were to evaluate the strength and to identify the possible failure modes of bolted connections in New Zealand hardwood. Bolted connection tests loaded parallel-to-grain were performed using recycled native New Zealand Matai and Rimu hardwoods because the timber diaphragms in URM buildings are typically constructed using such wood species. From the experimental study, it was observed that the timber bolted connection can fail in either ductile or brittle modes. The test results obtained were compared with the European Yield Model (EYM), the New Zealand timber code (NZS 3603:1993), and a proposed set of equations (Quenneville 2009) in order to evaluate the applicability of those equations in predicting bolted connection strength for New Zealand hardwood. It was found that the EYM equations provide better predictions than the NZS 3603:1993 when compared to the actual capacity. However, the EYM predictions are only good in estimating the strength of timber bolted connections that fail under ductile mode. For the connections that fail exhibiting the brittle mode, the proposed row shear equation by Quenneville was found to give better strength estimation.

## 1 INTRODUCTION

Unreinforced masonry (URM) buildings are typically the class of structures with the highest risk of failure during an earthquake, and the requirement to seismically upgrade these earthquake damage-prone buildings in New Zealand was mandated by The Building Act 2004 (DBH 2004). Importantly, these URM buildings form a significant percentage of New Zealand's building stock and represent the predominant national architectural heritage (Russell and Ingham 2008). Most URM buildings in New Zealand consist of solid URM bearing walls and flexible timber diaphragms (floor and roof), with the wall thickness configuration over the height of the building typically reduced by a single leaf at each storey height in order to support the diaphragm. The most common diaphragm seating method was to bear the joists and transverse beams on a single brick width without embedment.

No connections between URM walls and diaphragm were identified in URM buildings constructed before the 1931 Hawke's Bay earthquake and most out-of-plane wall failures were related to the absence of anchorage between the walls and diaphragms. Following the 1931 Hawke's Bay earthquake, most URM buildings were seismically retrofitted, which included the installation of wall-floor and wall-roof connections (Blaikie and Spurr 1992). Most wall-diaphragm connections that were installed as seismic retrofits were through-bolt anchors, used in conjunction with a steel bearing plate located on the exterior of the building and a bolted connection on the timber diaphragm joist. Typical wall-diaphragm connection details can be found in Abdul Karim et al. (2009). Figure 1 shows typical diaphragm details determined for existing New Zealand URM buildings.

As connection failure by tearing out part of the diaphragm joist was observed in past earthquakes due to lateral earthquake loading (Blaikie and Spurr 1992), the strength of the bolted connections in existing indigenous New Zealand timber joists requires assessment as they need to be properly detailed. The main objectives of this study were to evaluate the strength and to verify the possible failure modes of the bolted connections in New Zealand hardwood. Based on experimental data obtained, a set of design equations for predicting the strength of timber bolted connections is recommended. Thus, detailed seismic assessment and retrofit solution procedures for wall-diaphragm connections in existing New Zealand URM buildings can be provided. The timber bolted connection tests conducted at the University of Auckland using recycled native New Zealand hardwood such as Matai and Rimu are described, with the experimental results compared to several strength prediction equations in order to evaluate their potential for calculating the capacity of bolted connections, specifically for New Zealand hardwood.




a) Joist perpendicular to URM wall　　　　b) Joist parallel to URM wall

Figure 1: Typical diaphragm details (Abdul Karim et al. 2009)

## 2 DESIGN EQUATIONS FOR TIMBER BOLTED CONNECTIONS

There is agreement, in principle, within the international timber engineering community that criteria in timber design standards for determining the capacity of bolted connections should be based on recognised mechanics models that are capable of identifying each possible mode of failure (Quenneville 2009). Failure modes to be considered are the ductile ‘bearing failures’ and the brittle ‘fracturing failures’ in wood, where the mode with the lowest estimated capacity will govern performance (Quenneville et al. 2006). This section describes the design equations that are currently available to predict the capacity of timber bolted connections.

### 2.1 New Zealand Timber Structures Standard (NZS 3603:1993)

For the purpose of timber bolted connection design, which is associated with a ductile failure mode, Clause 4.4 of NZS 3603:1993 can be used. The characteristic strength of a bolt loaded parallel to the grain in dry timber, $Q_{kl}$ for a bolt in single shear shall be the lesser of:

$$k_{11}\ f_{cj}\ d_a^2 \qquad \text{(Eqn. 1)}$$

or

$$0.5\ b_e\ f_{cj}\ d_a \qquad \text{(Eqn. 2)}$$

where

$k_{11}$ = bolt bearing stress factor (Table 4.8 of NZS 3603:1993)

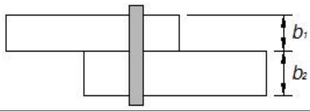
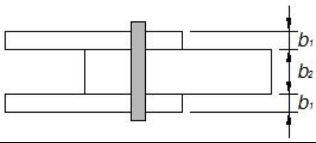
$f_{cj}$ = characteristic bolt bearing stress parallel to the grain, MPa (Table 4.8 of NZS 3603:1993)

$d_a$ = diameter of a fastener, mm

$b_e$ = effective timber thickness in a bolted connection, mm (Table 4.9 of NZS 3603:1993).

Thus, the strength of a laterally loaded bolted connection, N* for two-member and three-member connection types is given in Table 1. In these equations, the connection behaviour is assumed to be ductile. A brittle failure is assumed to occur only for connections with four fasteners or more, taken into account with a value of $k_{13}$ less than 1.

**Table 1. Bolted connection strength for a single bolt in dry timber loaded parallel to the grain.**

| Type of connection | Bolted connection strength, N* | |
|---|---|---|
| a) Two-members   | $N_1^* = \phi\, n\, k_1\, k_{12}\, k_{13}\, (k_{11}\, f_{cj}\, d_a^2)$ <br> $N_2^* = \phi\, n\, k_1\, k_{12}\, k_{13}\, (0.5\, b_e\, f_{cj}\, d_a)$ | (Eqn. 3a) <br> (Eqn. 3b) |
| b) Three-members   | $N_1^* = \phi\, n\, k_1\, k_{12}\, k_{13}\, (2\, k_{11}\, f_{cj}\, d_a^2)$ <br> $N_2^* = \phi\, n\, k_1\, k_{12}\, k_{13}\, (b_e\, f_{cj}\, d_a)$ | (Eqn. 4a) <br> (Eqn. 4b) |
| **Notes:**<br>1. N* is the design load effects on connection produced by strength limit state loads, N.<br>2. $\phi$ is the strength reduction factor (Clause 2.5 of NZS 3603:1993).<br>3. n is the number of fasteners.<br>4. $k_1$ is the duration of load factor for strength (Table 2.4 of NZS 3603:1993).<br>5. $k_{12}$ is the factor for the design of bolted connections in green timber (Table 4.14 of NZS 3603:1993).<br>6. $k_{13}$ is the factor for the design of multiple-bolt connections (Table 4.15 of NZS 3603:1993). | | |

## 2.2 European Yield Model

The European Yield Model (EYM), which considers ductile failure modes of bolted connections, is associated with the Johansen's theory. This theory is based on the assumption that the materials (i.e. timber under embedding stresses and fastener under bending action) behave as rigid-plastic (Blass 2003). Tables 2 and 3 show the possible failure modes for double shear and single shear connections, respectively. The resistance, R per fastener per shear plane of a connection can be calculated using equations given in each table. The minimum value will govern the connection resistance.

**Table 2. Possible failure modes for double shear joint.**

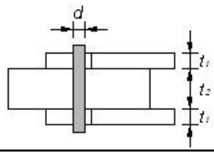
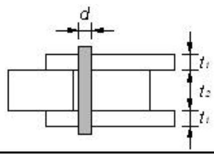
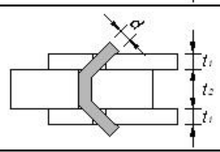
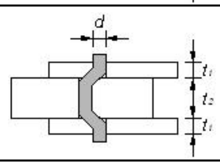
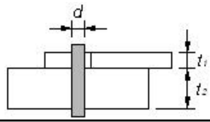
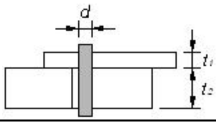
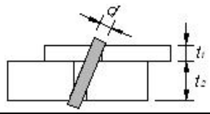
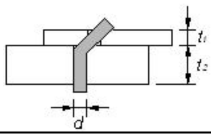
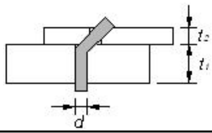
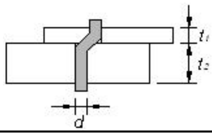
| Failure mode | Resistance, R, per fastener per shear plane | |
|---|---|---|
|   | $R = f_{h,1}\, t_1\, d$ | (Eqn. 5) |
|   | $R = 0.5\, f_{h,1}\, t_2\, d\, \beta$ | (Eqn. 6) |
|   | $R = \dfrac{f_{h,1}\, t_1\, d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_y}{f_{h,1}\, t_1^2\, d}}-\beta\right]$ | (Eqn. 7) |
|   | $R = \sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_y\, f_{h,1}\, d}$ | (Eqn. 8) |

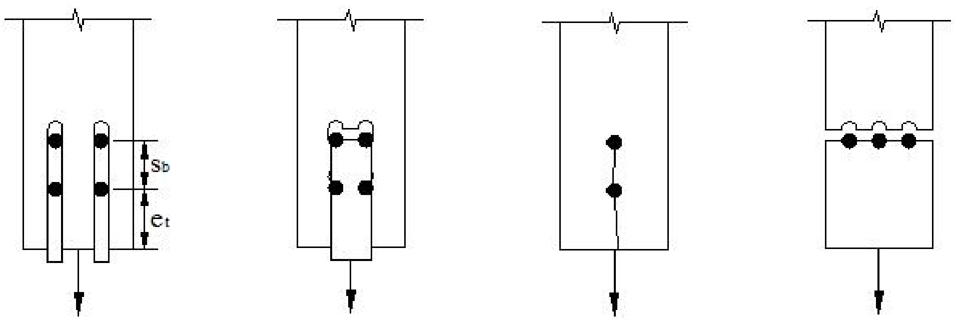
**Table 3. Possible failure modes for single shear joint.**

| Failure mode | Resistance, R per fastener per shear plane |
|---|---|
|  | $R = f_{h,1}\ t_1\ d$ (Eqn. 9) |
|  | $R = f_{h,1}\ t_2\ d\ \beta$ (Eqn. 10) |
|  | $R = \frac{f_{h,1}\ t_1\ d}{1+\beta}\left\{\sqrt{\beta+2\beta^2\left[1+\frac{t_2}{t_1}+\left(\frac{t_2}{t_1}\right)^2\right]+\beta^3\left(\frac{t_2}{t_1}\right)^2}-\beta\left(1+\frac{t_2}{t_1}\right)\right\}$ (Eqn. 11) |
|  | $R = \frac{f_{h,1}\ t_1\ d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_y}{f_{h,1}\ t_1^2\ d}}-\beta\right]$ (Eqn. 12) |
|  | $R = \frac{f_{h,1}\ t_2\ d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\frac{4\beta(1+2\beta)M_y}{f_{h,1}\ t_2^2\ d}}-\beta\right]$ (Eqn. 13) |
|  | $R = \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_y\ f_{h,1}\ d}$ (Eqn. 14) |

**Notes:**
1. $\beta$ is the ratio of the embedding strengths, $\beta = f_{h,2} / f_{h,1}$.
2. $f_{h,1}$ is the embedding strength corresponding to $t_1$, MPa.
3. $f_{h,2}$ is the embedding strength corresponding to $t_2$, MPa.
4. $t_1$ and $t_2$ is the timber thickness or fastener penetration of member 1 and 2, mm.
5. d is the fastener diameter, mm.
6. $M_y$ is the fastener yield moment, Nmm.

## 2.3 Brittle failure model

Many recent studies (Quenneville and Mohammad 2000; Mohammad and Quenneville 2001; Quenneville and Bickerdike 2006; Quenneville et al. 2006; Quenneville 2009) have identified the brittle failure of connections in timber structures. The observed brittle failure modes are illustrated in Figure 2. A set of equations to predict the ultimate strength of timber connections based on each brittle failure mode observed during tests was developed by Quenneville (Quenneville and Mohammad 2000). The details of all equations can be found in Quenneville (2009). However, only the row shear equation is used in this study and is described below.




a) row shear b) group tear-out c) splitting d) net tension

Figure 2: Observed brittle failure modes for timber bolted connections loaded parallel-to-grain

The row shear design capacity of a group of dowel fasteners is given by:

$$R_{r\ rs} = \phi\ k_1\ k_{12}\ RS_{i\ min}\ n_r \quad \text{(Eqn. 15)}$$

where:

| | |
|---|---|
| $\phi$ | = strength reduction factor |
| $k_1$ | = duration of load factor for strength |
| $k_{12}$ | = factor for the design of bolted connections in green timber |
| $n_r$ | = number of rows in the joint as per load component |
| $RS_{i\,min}$ | = minimum ($RS_1$, $RS_2$, …, $RS_{nr}$) |
| $RS_i$ | = shear capacity along two shear planes of fastener row "i", in N |
| | $= \dfrac{2(f_v)K_{ls}\ t\ n_{fi}\ a_{cr\,i}}{CF}$ |
| $f_v$ | = member shear strength, MPa, equal to 21.9 $G^{1.13}$ |
| G | = relative density of timber for the oven dry condition |
| $K_{ls}$ | = factor for member loaded surfaces (0.65 for side member, 1 for internal member) |
| t | = member thickness, mm |
| $n_{fi}$ | = number of fasteners in row "i" |
| $a_{cr\,i}$ | = minimum of $e_t$ and $s_b$ for row "i" (see Figure 2a), mm |
| CF | = calibration factor. |

## 3 TIMBER BOLTED CONNECTION TESTS

### 3.1 Specimen configurations

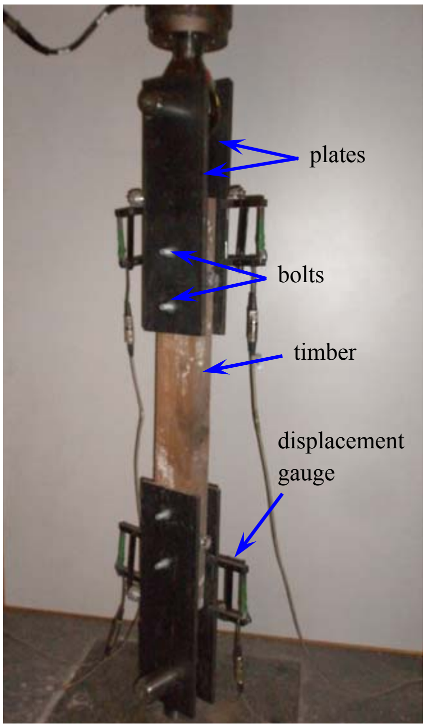



Figure 3: Typical specimen of bolted timber connection loaded parallel to the grain in testing apparatus

In the laboratory tests, recycled native New Zealand hardwood such as Matai and Rimu were used because the floor joists in URM buildings are typically constructed using such wood species. All specimens consisted of three-member connections with two steel side plates sandwiching a timber centre piece as shown in Figure 3. The cross section of each timber specimen was 50 mm (thickness) × 100 mm (width) and had a moisture content of 13% at the time of testing. 12 mm diameter (d) bolts of 4.8 grade ($f_y$ = 320 MPa) were used in all specimens. The steel side plates used were 10 mm thick with an ultimate tensile strength, $f_u$, of 400 MPa.

Eleven groups of specimens were tested, where each group consisted of at least ten replicates. Groups 2 and 3 were tested with both Matai and Rimu wood species and other groups were tested with Matai wood only due to the limited availability of Rimu wood specimens. All groups had a single row ($n_r$), but varied with number of fasteners ($n_f$) and end distance ($e_t$). The number of fasteners varied from 1 to 4. The majority of connections with 2 fasteners had a 100 mm bolt spacing ($s_b$) except for group 9 with a 150 mm bolt spacing. Groups with a 50 mm end distance were designed in order to maximise the number of observations on the row shear failure mechanisms, to determine the calibration factor of equation 15 for New Zealand hardwood. Details of the specimen configuration are given in Table 4.

**Table 4. Specimen configuration and summary of test results.**

| Group | d (mm) | $e_t$ (mm) | $s_b$ (mm) | $s_r$ (mm) | $n_r$ | $n_f$ | Species | Cross section (mm) | Experimental $R_{avg}$ (kN) | Experimental COV (%) | Experimental $R_{5th\%}$ (kN) | Observed failure mode |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 12 | 200 | 100 | – | 1 | 2 | Matai | 50 x 100 | 75 | 8 | 66 | B |
| 2 | 12 | 150 | 100 | – | 1 | 2 | Matai | 50 x 100 | 75 | 11 | 62 | B |
| 2R | 12 | 150 | 100 | – | 1 | 2 | Rimu | 50 x 100 | 66 | 7 | 59 | B |
| 3 | 12 | 100 | 100 | – | 1 | 2 | Matai | 50 x 100 | 75 | 13 | 58 | B |
| 3R | 12 | 100 | 100 | – | 1 | 2 | Rimu | 50 x 100 | 66 | 14 | 50 | B |
| 4 | 12 | 50 | 100 | – | 1 | 2 | Matai | 50 x 100 | 40 | 22 | 25 | RS |
| 5 | 12 | 200 | – | – | 1 | 1 | Matai | 50 x 100 | 44 | 12 | 35 | B |
| 6 | 12 | 150 | – | – | 1 | 1 | Matai | 50 x 100 | 40 | 11 | 32 | B |
| 7 | 12 | 100 | – | – | 1 | 1 | Matai | 50 x 100 | 41 | 16 | 30 | B |
| 8 | 12 | 50 | – | – | 1 | 1 | Matai | 50 x 100 | 16 | 11 | 13 | RS |
| 9 | 12 | 50 | 150 | – | 1 | 2 | Matai | 50 x 100 | 32 | 26 | 18 | RS |
| 10 | 12 | 50 | 50 | – | 1 | 3 | Matai | 50 x 100 | 43 | 35 | 18 | RS |
| 11 | 12 | 50 | 50 | – | 1 | 4 | Matai | 50 x 100 | 54 | 35 | 23 | RS |

**Notes:** B = Bearing failure; RS = Row Shear failure

### 3.2 Test setup

All specimens were loaded in tension parallel-to-grain and were fabricated with an identical connection configuration at each end. All fasteners were finger tight to allow self-alignment, and a monotonic tension load was applied through the side steel plates using an MTS loading system. Each specimen was tested at a displacement-control rate of 1 mm/min until failure, when the load dropped with no recovery. Both ends were monitored for load and slip, and the ultimate load that was recorded was for the extremity that failed. Two displacement gauges were used to measure the slip of the wood internal member with reference to the side steel plate at each extremity. Each load-slip data was collected by a data acquisition system and recorded on a personal computer. Figure 3 shows a typical specimen in the testing frame.

## 4 RESULTS AND DISCUSSION

### 4.1 General

Each specimen was loaded in tension up to the ultimate capacity of one of the two extremities of the connections. The load-slip curve of each specimen was plotted and the ultimate load and the type of failure were recorded. The experimental results of the eleven groups tested are listed in Table 4. The average experimental values, $R_{avg}$, were determined and the lower $5^{th}$ percentile strength of the experimental results, $R_{5th\%}$, was calculated assuming a normal distribution. The predominant modes of failure observed after testing are listed in Table 4, where B and RS are designated for bearing failure and row shear, respectively.

The calculated strength values (i.e. predictions) for each group of connections are tabulated in Table 5 for comparison. The NZS 3603:1993 values represent the strength of laterally loaded three-member type bolted connections that were calculated using equations 4a and 4b. Equations 5 to 8 were used to predict the strength of the connections based on the EYM. Predictions using the row shear equation (Eqn. 15) proposed by Quenneville for connections that fail in a brittle mode are also given. All strength values were calculated for short term duration of load (i.e. seismic loading).

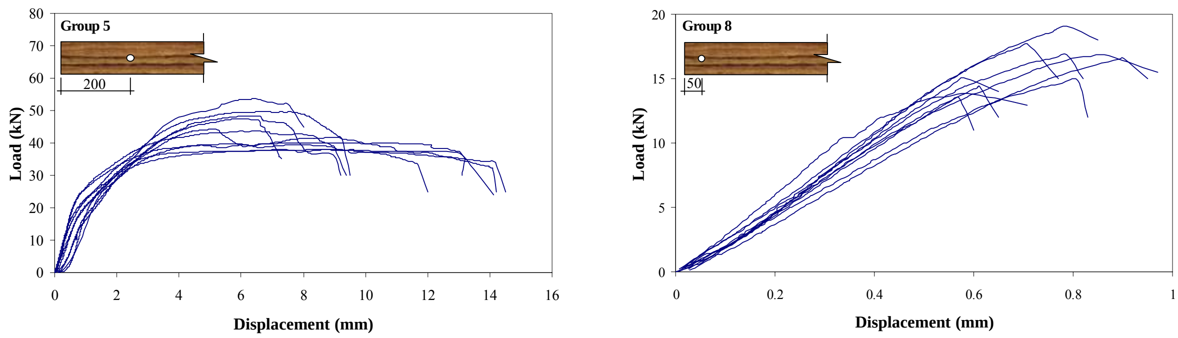
### 4.2 Failure modes

Two dominant modes of failures were observed during the connection tests, which were bearing and row shear failure. All groups fabricated with an end distance of 100 mm or more failed primarily in a ductile mode (i.e. bearing) until a secondary brittle failure such as splitting or row shear caused the load to drop suddenly as illustrated in Figure 4a. The dominant final mode of failure in most specimens for groups 1, 2, 5 and 6 ($e_t \geq 150$ mm) was splitting, where few specimens failed in row shear. However, the final failure mode of row shear was observed in groups 3 and 7 ($e_t = 100$ mm). Yielding of the fasteners was also observed in these groups of connections. As expected, groups of specimens with $e_t$ equal to 50 mm primarily failed in row shear and very few failed in splitting. This brittle failure mode is associated to a load drop at a low displacement value as shown in Figure 4b. From both observations, one can see that the mode of failure is affected by the end distance of the connections. Figures 4a and 4b show the typical load-displacement curves for all specimens in groups 5 and 8 that exhibited ductile and brittle behaviour, respectively.

In general, the ultimate strength was also considerably affected by decreasing the end distance from 200 mm to 50 mm. The effect of the end distance on the ultimate strength is best described by comparing the 5$^{th}$ percentile experimental values of group 1 with group 4, as well as group 5 with group 8. For group 4, the ultimate strength was about 0.38 of the strength of group 1. The ultimate strength of group 8 was also lower by a factor of 0.37 compared to group 5. This is consistent with other experimental data available in the literature (Quenneville and Mohammad 2000; Mohammad and Quenneville 2001).

**Table 5. Comparison between test results and predictions.**

| | Experimental | | | NZS 3603:1993 | | | European Yield Model (EYM) | | | | | Row shear |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Group | $R_{avg}$ (kN) | COV (%) | $R_{5th\%}$ (kN) | $N_1$* (kN) | $N_2$* (kN) | $N_{min}$ (kN) | $R_{Eqn.5}$ (kN) | $R_{Eqn.6}$ (kN) | $R_{Eqn.7}$ (kN) | $R_{Eqn.8}$ (kN) | $R_{min}$ (kN) | $R_{r\,rs}$ (kN) |
| 1 | 75 | 8 | 66 | 42 | 43 | 42 | 576 | 65 | 84 | 60 | 60 | 50 |
| 2 | 75 | 11 | 62 | 42 | 43 | 42 | 576 | 65 | 84 | 60 | 60 | 50 |
| 2R | 66 | 7 | 59 | 42 | 43 | 42 | 576 | 55 | 78 | 56 | 55 | 45 |
| 3 | 75 | 13 | 58 | 42 | 43 | 42 | 576 | 65 | 84 | 60 | 60 | 50 |
| 3R | 66 | 14 | 50 | 42 | 43 | 42 | 576 | 55 | 78 | 56 | 55 | 45 |
| 4 | 40 | 22 | 25 | 42 | 43 | 42 | 576 | 65 | 84 | 60 | 60 | 25 |
| 5 | 44 | 12 | 35 | 21 | 22 | 21 | 288 | 32 | 42 | 30 | 30 | 50 |
| 6 | 40 | 11 | 32 | 21 | 22 | 21 | 288 | 32 | 42 | 30 | 30 | 38 |
| 7 | 41 | 16 | 30 | 21 | 22 | 21 | 288 | 32 | 42 | 30 | 30 | 25 |
| 8 | 16 | 11 | 13 | 21 | 22 | 21 | 288 | 32 | 42 | 30 | 30 | 13 |
| 9 | 32 | 26 | 18 | 42 | 43 | 42 | 576 | 65 | 84 | 60 | 60 | 25 |
| 10 | 43 | 35 | 18 | 62 | 65 | 62 | 864 | 97 | 126 | 91 | 91 | 38 |
| 11 | 54 | 35 | 23 | 83 | 87 | 83 | 1152 | 130 | 168 | 121 | 121 | 50 |




a) connections exhibiting ductile behaviour

b) connections exhibiting brittle behaviour

Figure 4: Typical load-slip curves

## 4.3 Calibration factor

In order to determine the calibration factor of equation 15 for New Zealand hardwood, groups 4, 8, 9, 10 and 11 were analysed. As mentioned previously, these groups, which had an end distance of 50 mm, were purposely designed to maximise the number of observations of row shear failure.

The average strength values of row shear failure (Eqn. 15), $R_{r\ rs,\ avg}$, for each group were determined. These values were then plotted against the average experimental results (see Figure 5). The calibration factor was determined when the linear best fitted line of the prediction values (G4, G8, G9, G10, G11) matched the linear 'one-to-one ratio' line. Figure 5 shows that the linear best fit line of the prediction values is perfectly matching the aforementioned line. Thus, a calibration factor of 4 was found in this analysis.

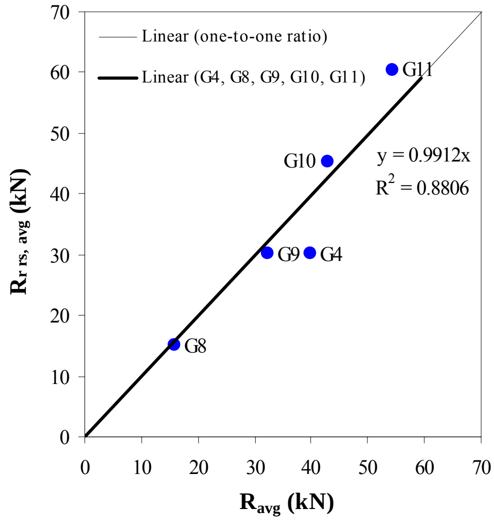



Figure 5: Predictions vs. test results

## 4.4 Predictions vs experimental

In order to evaluate the capability of the current strength prediction equations (i.e. NZS 3603:1993, EYM, and $R_{r\ rs}$) to estimate the capacity of bolted connections for New Zealand hardwood, a graph showing the effectiveness of those predictions versus the test results is presented in Figure 6. Any prediction values plotted below the 45° line are considered to be conservative. For comparison purposes, Table 6 summarises the calculated ratios between the predictions and test results.

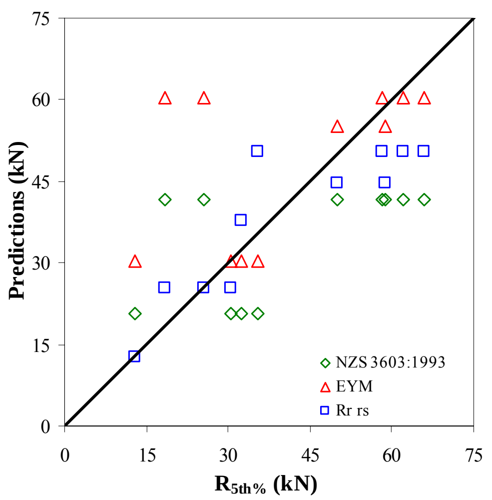



Figure 6: Comparison between test results and predictions from NZS 3603:1993, EYM or $R_{r\ rs}$

**Table 6. Ratios between predictions and test results.**

| Group | $R_{5th\%}$ (kN) | $N_{min}$ (kN) | $R_{min}$ (kN) | $R_{r\ rs}$ (kN) | $N_{min}/R_{5th\%}$ | $R_{min}/R_{5th\%}$ | $R_{r\ rs}/R_{5th\%}$ |
|---|---|---|---|---|---|---|---|
| 1 | 66 | 42 | 60 | 50 | 0.63 | 0.92 | 0.77 |
| 2 | 62 | 42 | 60 | 50 | 0.67 | 0.98 | 0.81 |
| 2R | 59 | 42 | 56 | 45 | 0.71 | 0.94 | 0.76 |
| 3 | 58 | 42 | 60 | 50 | 0.72 | 1.04 | 0.87 |
| 3R | 50 | 42 | 56 | 45 | 0.83 | 1.10 | 0.89 |
| 4 | 25 | 42 | 60 | 25 | 1.63 | 2.37 | 0.99 |
| 5 | 35 | 21 | 30 | 50 | 0.59 | 0.85 | 1.42 |
| 6 | 32 | 21 | 30 | 38 | 0.64 | 0.93 | 1.17 |
| 7 | 30 | 21 | 30 | 25 | 0.68 | 0.99 | 0.83 |
| 8 | 13 | 21 | 30 | 13 | 1.61 | 2.34 | 0.98 |
| 9 | 18 | 42 | 60 | 25 | 2.27 | 3.30 | 1.38 |

By referring to groups that failed in bearing, predictions using the current NZS 3603:1993 were found to be too conservative compared to the lower 5$^{th}$ percentile of the experimental results. The design values provided by the NZS 3603:1993 would make the choice of bolted connections impractical. Excluding groups 4, 8 and 9 for which the connections failed in row shear, the ratio of the timber standard values to the test results varies between 0.59 and 0.83, with an average of 0.68. Better prediction values were obtained using the EYM equations with the same ratio ranges from 0.85 to 1.10 and an average of 0.97. Using the row shear equation, an acceptable estimation of strength was found for groups 1, 2, 2R, 3, 3R, and 7, but strength was overestimated for groups 5 and 6 with differences of 42% and 17%, respectively.

A similar comparison was performed for the groups of connections that failed in row shear (groups 4, 8, and 9) as shown in Table 6. Both the EYM and NZS 3603:1993 were clearly over predicting the strength (i.e. unsafe) with an unacceptable percentage of error. However, good prediction values were obtained using the row shear equation, especially for groups 4 and 8 with ratios of 0.99 and 0.98, respectively. The strength of Group 9 was over predicted with a ratio of 1.38. The use of the EYM equation in combination with the one for row shear would thus form a better set of equations to design bolted connections in URM buildings.

## 5 CONCLUSIONS

Based on the experimental study conducted, the following conclusions can be drawn:

1. The strength and possible failure modes of New Zealand hardwood bolted connections were successfully assessed. Both strength and failure modes were found to be significantly affected by the end distance, $e_t$, of the connections, with a greater end distance producing a stronger connection.
2. A calibration factor of 4 was obtained and applied in equation 15 in order to predict the strength of bolted connections for the row shear failure mode when occurring in New Zealand hardwood.
3. NZS 3603:1993 is far too conservative compared to the 5$^{th}$ percentile of the actual strength with a ratio as low as 0.59. The design values provided by the timber standard would make the choice of bolted timber connections impractical. Better strength predictions were achieved using the European Yield Model and row shear equations for bearing and row shear failures, respectively. Thus, use of the EYM and row shear equations to design bolted connections in unreinforced masonry buildings is recommended.

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