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Wall-Diaphragm Connection Assessment Guidelines for URM Buildings

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ABSTRACT: The connections between walls and diaphragm in unreinforced masonry (URM) buildings typically consist of two major parts. The first part is the wall anchorages and the second part is the diaphragm connections. From the NZNSEE (1995) and FEMA (2006) guidelines, the connection strength values given by both guidelines are apparently related only with the possible failures of the masonry wall anchorages, and no procedures are currently available to identify the strength values related to failure of the timber bolted connections. This latter failure mode is important to consider as the minimum strength value that will govern the wall-diaphragm connection capacity can be acquired from the diaphragm connections. To counter this limitation, this study recommends a set of design equations to assess the strength of the timber bolted connections. By using these equations, in addition to the default connection strength values provided by the guidelines, the expected strength level of wall-diaphragm connections can be accurately assessed.

1 INTRODUCTION

Referring to the guidelines published by the New Zealand National Society for Earthquake Engineering (NZNSEE, 1995) and Federal Emergency Management Agency (FEMA, 2006), it can be seen that the wall-diaphragm connections in unreinforced masonry (URM) buildings primarily consist of two major parts. The first part is the wall anchorages, either in the form of through-bolt with external bearing plate to anchor the walls or dowel that drilled into the masonry walls. The second part is the diaphragm connections, typically bolted connections to the timber members of the diaphragms (i.e floor joists or roof rafters). From a review of both guidelines, none of the wall-diaphragm connection details provide guidelines on the steel cleat to timber connection. Important design parameters such as end distance and spacing of the bolts, which significantly affect the timber connection strength, are not provided in detail. The connection strength values given by both guidelines are apparently related only with the possible failures of the masonry wall anchorages, and no procedures available to identify the strength values related with the failures of the timber bolted connections. This later failure mode is important to consider as the minimum strength value that will govern the wall-diaphragm connection capacity can be acquired from the diaphragm connections. Further, the current 2006 NZSEE guidelines were identified to provide only a revision on the connection strength values for wall anchorages.

From a review of published works, no attempt was found to assess the bolted connections applied to the existing timber members of the floor and roof diaphragms in unreinforced masonry buildings. The strength of timber bolted connections is very important to investigate since the brittle failure of the diaphragm connections (i.e. tearing out part of the diaphragm joist) observed in past earthquakes (Blaikie and Spurr 1992). From the published research studies on the timber bolted connections (Quenneville and Mohammad, 2000; Mohammad and Quenneville, 2001; Quenneville et al., 2006; Quenneville and Jensen, 2008), many brittle failure modes were identified such as row shear-out, group tear-out, and splitting. Referring to the current standard for timber structures of New Zealand (NZS 3603: 1993), the design of bolted connections was developed based on a ductile failure. The use of this standard can lead to an inaccurate strength assessment of timber bolted connections in unreinforced masonry buildings. Thus, the bolted connection tests of existing timber in the buildings need to be conducted to verify the use of the design equations of the standard. The use of the European Yield Model developed by Johansen in 1949 (quoted from Blass, 2003) and Row Shear Model (Quenneville, 2009) equations was also validated using the test results obtained. From these, a set of design equations in assessing the timber bolted connections can be recommended, and an accurate wall-diaphragm connection assessment guideline for New Zealand unreinforced masonry buildings can be provided.

2 TIMBER PROPERTIES DETERMINATION

2.1 Moisture Content and Density

To determine the moisture content and density of Matai and Rimu, testing procedures outlined in AS/NZ 1080.1: 1997 and AS/NZ 1080.3: 2000 were followed, respectively. As recommended by the standards, the test pieces were taken from the specimens that were particularly prepared for the bolted connection tests. Thus, both moisture content and density test pieces were prepared after conducting the main bolted connection tests.

For both tests, an average dimension of 50.0 mm (length) × 40.0 mm (width) × 10.0 mm (thickness) tests pieces were prepared and measured with an accuracy of 0.1 mm. Before placing the test pieces into the oven with ± 105°C for drying, weight of each test piece was recorded with an accuracy of 0.01 g. After the first 24 hours of the oven dry process, the weighing and measuring operations were conducted and continued until a consistent weight of the test piece was achieved. Table 1 shows the results of the moisture contents and the density tests conducted.

2.2 Embedding Strength

For wood embedding strength tests, test pieces of a 100 mm (length) × 50 mm (width) × 90 mm (height) rectangular prism that comply with minimum sizes specified in ISO/DIS 10984-2 (2008) were prepared. A half 13 mm diameter hole was fabricated in each test piece for the placement of 12 mm diameter fasteners during testing as shown in Figure 1. The test setup shown in Figure 2 was implemented to avoid the bending of the fastener under test.

By using a 100 kN universal testing machine, a monotonic compression loading was applied to the fastener through the steel loading apparatus at a loading rate of 1 mm per minute up to the maximum strength of the wood. A linear variable differential transducer (LVDT) was used to record the relative displacement of the fastener with respect to the test piece. Load and displacement data for each test piece was recorded by a personal computer that was connected to the data acquisition system. The embedding strength of wood, f_h, was calculated by dividing the maximum load achieved with the multiplication product of the fastener diameter, d, and the test piece width, W. The computed embedding strength values of Matai and Rimu are summarised in Table 2 below.

2.3 Yield Compression Strength

The yield compression strength perpendicular to grain of Matai and Rimu was determined by conducting a pure block test as shown in Figure 3. Dimensions of the tested timber block are 100 mm (length) × 110 mm (width) × 50 mm (height). A uniform rate of loading was applied to the test piece using a compression machine until failure occurred or 20 mm deformation was observed. The load and displacement readings for each test were collected and recorded for further analysis.

According to AS/NZ 4063.1 (2007), the yield compression strength perpendicular to grain of wood is determined using the yield load divided by the compressed wood area. Note that the yield load is evaluated using a 2 mm offset method. Table 3 gives the values of the yield compression strength parallel to grain of Matai and Rimu.

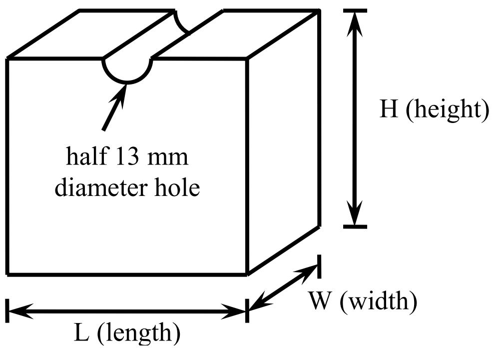


Figure 1: Test piece of the embedding strength tests

Figure 2: Test setup of the embedding strength tests

Table 1: Moisture content and density tests results

Species	Total of test pieces	ρ_{avg} (kg/m^3)	CoV	$\rho_{5^{th}\%}$ * (kg/m^3)	MC_{avg} (%)
Matai	126	589	8.8	504	13.3
Rimu	24	556	11.3	452	12.4

Notes: ρ_{avg} = average density;
CoV = Coefficient of Variations;
$\rho_{5^{th}\%}$ = 5th percentile density;
MC_{avg} = average moisture content;
* Calculated assuming a normal distribution

Table 2: Embedding strength of Matai and Rimu

Species	Total of test pieces	$f_{h,avg}$ (MPa)	CoV	$f_{h,5^{th}\%}$ * (MPa)
Matai	106	69.69	13.8	53.92
Rimu	23	55.66	10.6	45.98

Notes: $f_{h,avg}$ = average embedding strength;
$f_{h,5^{th}\%}$ = 5th percentile embedding strength;
* Calculated assuming a normal distribution

Figure 3: Pure block test setup

Table 3: Compression strength of Matai and Rimu

Species	Total of test pieces	$f_{c,90y,avg}$ (MPa)	CoV	$f_{c,90y,5^{th}\%}$ * (MPa)
Matai	10	14.3	27.5	7.8
Rimu	10	11.2	22.4	7.1

Notes: $f_{c,90y,avg}$ = average compression strength;
$f_{c,90y,5^{th}\%}$ = 5th percentile compression strength;
* Calculated assuming a normal distribution

3 BOLTED CONNECTION TEST DETAILS

An experimental study of bolted connections performed by Abdul Karim et al. (2010) was initiated in an effort to assess the strength of wall-diaphragm connections in New Zealand unreinforced masonry buildings. The study was performed due to an absence of laboratory data on bolted connections for recycled indigenous New Zealand hardwoods, which were typically used to construct the floor and roof diaphragms in New Zealand unreinforced masonry buildings. The study concluded that the current NZS 3603:1993 is far too conservative compared to the actual connection strength and better connection strength predictions can be achieved using the European Yield Model (EYM) and Row Shear Model equations. Thus, the use of the EYM and Row Shear Model equations to design bolted connections in unreinforced masonry buildings is recommended. Details of results and analyses can be found in Abdul Karim et al. (2010).

4 ASSESSMENT GUIDELINES

4.1 Wall Anchorage Capacity

The default strength values of wall anchorages provided in Table 10B.2 of NZSEE (2006) may be adopted for bolts connecting components to masonry.

4.2 Diaphragm Connection Capacity

The capacity of a bolted connection shall be determined using the following equations, whereas the minimum value will govern the connection capacity.

i. Capacity of connection parallel to loading directions (refer to Figure 4)

$$R_{\parallel} = \mathrm{Min}\begin{cases} \text{European Yield Model}\left(R_{EYM}\right) \\ \text{row shear failure of timber}\left(R_{r\,rs}\right) \\ \text{bolt in shear } (V_f^*) \\ \text{steel rod in tension } (N_{rod}^*) \\ \text{steel plate in bearing } (V_{b1}^*) \\ \text{steel plate in tearing out } (V_{b2}^*) \end{cases} \qquad \text{(Eqn. 1)}$$

ii. Capacity of connection perpendicular to loading directions (refer to Figure 5)

$$R_{\perp} = \mathrm{Min}\begin{cases} \text{timber member in bearing } (F_b) \\ \text{washers bearing } (F_c) \\ \text{bolt in tension } (N_{tf}^*) \\ \text{steel rod in shear } (V_{rod}^*) \\ \text{steel plate in bearing } (V_{b1}^*) \\ \text{steel plate in tearing out } (V_{b2}^*) \end{cases} \qquad \text{(Eqn. 2)}$$

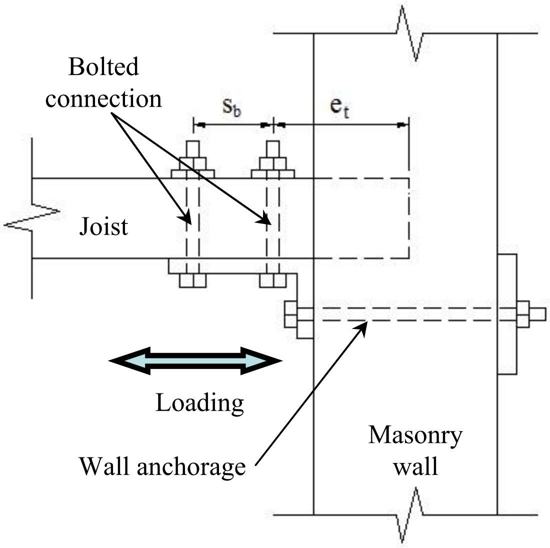


Figure 4: Connection parallel to loading directions

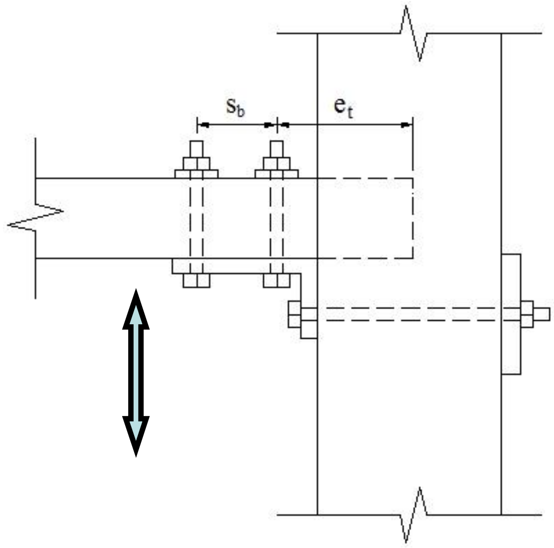


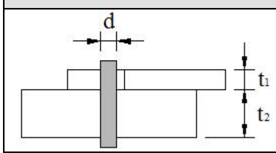
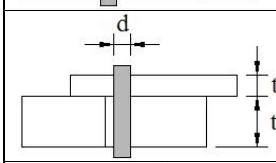
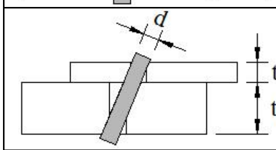
Figure 5: Connection perpendicular to loading directions

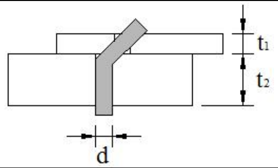
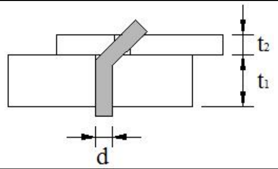
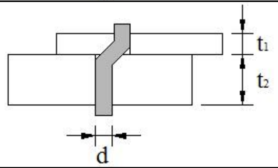
4.2.1 *European Yield Model*

The European Yield Model, 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 4 and 5 show the possible failure modes for single shear and double shear connections, respectively. The capacity, R_{EYM} per fastener per shear plane of a connection shall be calculated using the equations given in each table. The minimum value will govern the connection resistance.

The embedding strength of timber, f_{h2}, was determined experimentally as described in Section 2.2. For the embedding strength of steel plate, f_{h1}, the value is assumed to be equal to $C_1 \cdot \phi_b \cdot 3.2 \cdot f_{up}$, where C_1 is the reduction factor determination of the design bearing strength of the steel plate as per Table 12.9.4.3 of NZS 3404: Part 1: 1997, ϕ_b is the strength reduction factor equal to 0.9 as per Table 3.3(1) of NZS 3404: Part 1: 1997, and f_{up} is the tensile strength of the steel plate. The fastener yield strength, f_{yf}, is obtained from the grade of fasteners. The first figure from the grade indicates the fastener tensile strength, f_{uf},. The second figure specifies the fastener yield strength proportional to the tensile strength in terms of percentage.

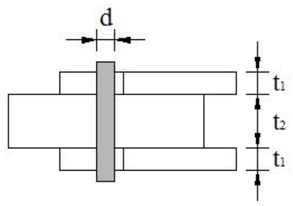
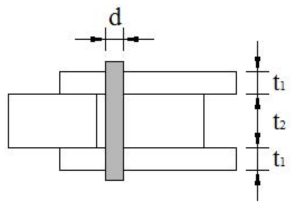
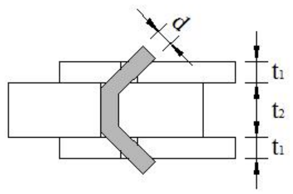
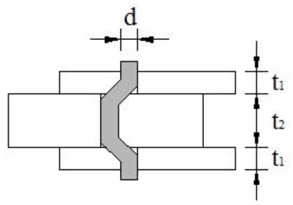
Table 4: Possible failure modes for single shear joint as per European Yield Model

Failure mode	**Resistance, R per fastener per shear plane**
	$R = \phi\, k_1\, k_{12}\, f_{h,1}\, t_1\, d$ (Eqn. 3)
	$R = \phi\, k_1\, k_{12}\, f_{h,1}\, t_2\, d\, \beta$ (Eqn. 4)
	$R = \frac{\phi\, k_1\, k_{12}\, f_{h,1}\, t_1\, d}{1+\beta}\left\{\sqrt{\beta+2\beta^2\left[1+\frac{t_2}{t_1}+\left(\frac{t_2}{t_1}\right)^2\right]+\beta^3\left(\frac{t_2}{t_1}\right)^2}-\beta\left(1+\frac{t_2}{t_1}\right)\right\}$ (Eqn. 5)

	$R=\frac{\phi\, k_1\, k_{12}\, f_{h,1}\, t_1\, d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_y}{f_{h,1}\, t_1^2\, d}}-\beta\right]$ (Eqn. 6)
	$R=\frac{\phi\, k_1\, k_{12}\, f_{h,1}\, t_2\, d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\frac{4\beta(1+2\beta)M_y}{f_{h,1}\, t_2^2\, d}}-\beta\right]$ (Eqn. 7)
	$R=\phi\, k_1\, k_{12}\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_y\, f_{h,1}\, d}$ (Eqn. 8)

Notes:
1. β - ratio of the embedding strengths, $\beta = f_{h,2} / f_{h,1}$.
2. $f_{h,1}$ - embedding strength corresponding to t_1, MPa.
3. $f_{h,2}$ - embedding strength corresponding to t_2, MPa.
4. t_1 and t_2 - timber thickness or fastener penetration of member 1 and 2, mm.
5. d - fastener diameter, mm.
6. M_y is the fastener yield moment, Nmm.
7. ϕ - strength reduction factor (Clause 2.5 of NZS 3603:1993).
8. k_1 - duration of load factor for strength (Table 2.4 of NZS 3603:1993).
9. k_{12} - factor for the design of bolted connections in green timber (Table 4.14 of NZS 3603:1993)

Table 5: Possible failure modes for double shear joint as per EYM

Failure mode	Resistance, R per fastener per shear plane
	$R=\phi\, k_1\, k_{12}\, f_{h,1}\, t_1\, d$ (Eqn. 9)
	$R=\phi\, k_1\, k_{12}\, 0.5\, f_{h,1}\, t_2\, d\, \beta$ (Eqn. 10)
	$R=\frac{\phi\, k_1\, k_{12}\, f_{h,1}\, t_1\, d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_y}{f_{h,1}\, t_1^2\, d}}-\beta\right]$ (Eqn. 11)
	$R=\phi\, k_1\, k_{12}\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_y\, f_{h,1}\, d}$ (Eqn. 12)

4.2.2 *Row Shear Failure of Timber*

Many recent studies (Quenneville and Mohammad 2000; Mohammad and Quenneville 2001; Quenneville and Bickerdike 2006; Quenneville et al. 2006; Quenneville 2009) have identified the row shear failure of connections in timber structures. The observed row shear failure mode is illustrated in Figure 6. An equation to predict the ultimate strength of timber connections based on the row shear failure mode observed during tests was developed by Quenneville (Quenneville and Mohammad, 2000). An optimised Row Shear Model equation to predict the strength of bolted connections for the row shear failure mode when occurring in New Zealand hardwood can be obtained in Abdul Karim, 2010.

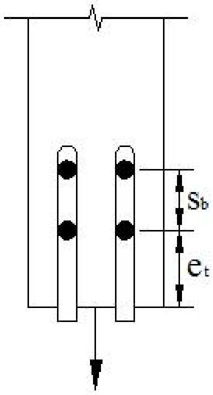


Figure 6: Observed row failure modes for timber bolted connections loaded parallel-to-grain

The capacity of connections due to row shear failure parallel to the timber grain shall be computed using the following equation.

$$R_{r\,rs} = \phi\, k_1\, k_{12}\, RS_{i\,min}\, n_r \qquad \text{(Eqn 13)}$$

where:

n_r = number of rows in the joint as per load component

$RS_{i\,min}$ = minimum (RS_1, RS_2, …, RS_{nr})

The equation given below to determine the shear capacity along two shear planes of fastener row "i", RS_i, shall be used.

$$RS_i = \frac{2(f_v)K_{ls}\ t\ n_{fi}\ a_{cr\,i}}{CF} \qquad \text{(Eqn. 14)}$$

where:

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.

4.2.3 *Timber Member in Bearing*

If the timber members of the diaphragms (i.e. floor joists or roof rafters) are embedded in the masonry walls, the bearing capacity perpendicular to the grain of the timber members on masonry units shall be evaluated using the equation provided below.

$$F_b^* = \phi A_b f_{c,90y} \qquad \text{(Eqn. 15)}$$

A_b is the contact bearing surface area between the timber member and the masonry units, and the strength reduction factor, ϕ, of 0.8 shall be used as per Clause 2.5 of NZS 3603: 1993. The yield compression strength perpendicular to grain of wood, $f_{c,90y}$, was determined experimentally as described in Section 2.3.

4.2.4 *Washers Bearing*

The following equation shall be use to assess the washer bearing capacity perpendicular to the timber grain.

$$F_c^* = \phi A_w f_{c,90y} n_w \qquad \text{(Eqn. 16)}$$

A_w, is determined in Clause 4.4.1.2 of NZS 3603: 1993, n_w is the number of washers, and the strength reduction factor, ϕ, of 0.8 shall be used as given in Clause 2.5 of NZS 3603: 1993.

4.2.5 *Capacity of Other Elements*

The New Zealand Steel Structures Standard (NZS 3404: Part 1: 1997; NZS 3404: Part 2: 1997) shall be used to calculate the shear and tensile capacity of bolts and steel rod, and the capacity of the steel plate in bearing and tearing out.

5 CONCLUSIONS

In addition to the default connection strength values provided by the NZSEE guidelines to assess the wall anchorages, this study provides a set of equations to assess the diaphragm connections. This promotes a complete procedure to assess the strength of the wall-diaphragm connections in New Zealand unreinforced masonry buildings and an inaccurate strength assessment can be avoided.

6 ACKNOWLEDGEMENTS

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