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Design Optimization of the Bolted Connection Loaded Parallel to the Timber Grain for Masonry Building Retrofits

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Abstract—The experimental results of timber bolted connection tests for the purpose of optimizing the use of current design standard are presented. The test was conducted to investigate the structural performance of bolted connections loaded parallel to the timber grain. Both ductile and brittle failure modes were investigated to identify the governing parameters that affect the types of failures. The Meraka hardwood was chosen because it was found to be commonly used in the construction of the structural components of floor and roof diaphragms in Malaysia unreinforced masonry (URM) buildings. From this study, a wood database can be established for assisting the design engineers in developing the retrofitting technique of the building, especially the timber diaphragm joint part of the wall-diaphragm connections. Eighteen characteristics of steel-wood-steel (SWS) with a single row bolted joints were tested in tension, whereas ten specimens were prepared for each connection group. From the results obtained, it can be observed that the current timber design code is far too conservative compared to the optimized design proposed in this paper.

Keywords—bolted timber connection, optimized design, retrofitting technique, SWS joint.

I. Introduction

Unreinforced masonry (URM) building constructions can be found in Malaysia as early as 1650, during the European and British colonization, which is the Stadthuys in Malacca [1] that was used as the official residence of the Dutch Governors. Now, it is the most popular landmarks that the tourists never miss to visit. According to Ho et al. [2], many unreinforced masonry buildings still exist in town area, that being used as residential from one to two-storey heights. There are also many of these buildings being operated as business or commercial purposes in Melaka town [3]. Similarly, the unreinforced masonry buildings can be found in other states in West and in East of Malaysia that were built between 1800 and 1948 [4]. Based on the data reported by Kamal et al. [4], in Sarawak alone, the total of unreinforced masonry buildings is 1,010. From the author's observations in the Kuching town area, there are many of them still exist and are of commercial use. A retrofit implementation of wall-diaphragm connections can be seen clearly from the visible wall anchors (double-C-shape) on the exterior masonry walls of the buildings. Most of the anchors were applied to laterally restrain the wall at the roof level with a wide spacing. Very few unreinforced masonry buildings were observed to have the wall anchors at both floor and roof levels, where the majority of the buildings found to have the wall anchors the at roof level only. The diaphragm connection details were unable to be identified as the underneath of the roof and floor diaphragms was covered by the ceiling panels. However, the author strongly believed that similar bolted connection features are applied to the floor timber joists or roof timber rafters as mentioned in published literature [5, 6, 7] due to the similarity of the local building characteristics in comparison to other countries such as the United States and New Zealand.

The lack of connections between masonry walls and timber diaphragms in unreinforced masonry buildings has long been identified causing the out-of-plane failures of masonry walls, gables and parapets during earthquakes. Bruneau [8, 9] reported that many out-of-plane wall failures were observed in the 1989 Loma Prieta Earthquake are due to a total absence of wall-diaphragm connections, which caused the walls to act as a cantilever over the building height. In the 1931 Hawke's Bay Earthquake, Blaikie and Spurr [10] stated that the catastrophic damage and collapse of numerous unreinforced masonry buildings in both Napier and Hasting towns are due to the non-presence of connections between walls and diaphragms. Griffith [11] identified that the lack of connections between gables and the supporting roof structures is the reason for many gable-end failures occurred in the 1989 Newcastle Earthquake. From the article written by Evans [12], one can see that the collapse of the masonry walls in the 2007 Gisborne Earthquake is due to the non-presence of connections between the wall and diaphragms, which causing the adjacent building severely damaged by the fell brick walls. In the 2010 Darfield Earthquake, Dizhur et al. [13] identified that many unreinforced masonry buildings suffered gable end wall collapsed due to insufficient or no connections provided between walls and roof diaphragms, which caused further damage to the neighboring buildings due to either single bricks or wall sections falling through the roofs. Dizhur et al. [14] also reported that many out-of-plane masonry wall failures observed in the 2011 Christchurch Earthquake, either one-way or two-way bending due to insufficient or absence of wall-diaphragm connections to resist the lateral earthquake loading.

The reasons for initiating this study are due to the occurrence of recent earthquake activities in Malaysia and the similarity of Malaysia unreinforced masonry building characteristics compared to other countries. Similar building characteristics, due to the similar method of constructions used, can obviously cause a major destruction to the unreinforced masonry buildings as they are well known to perform poorly during earthquakes when referring to their past structural performances in previous earthquakes in other seismic-prone countries. Additionally, this type of buildings is still being used for commercial purposes, which is mainly found in town areas and saturated with the crowds. The collapse of these buildings when subjected to earthquake loadings can cause very serious injuries or casualties. Referring to the current Malaysian Timber code of practice for structural use of timber [15], the design of bolted connections was developed based on a ductile failure mode. The use of this code can lead to an inaccurate in designing the strength of timber bolted connections, because many published research works have identified brittle failure modes such as row shear, group tear-out, splitting and tension [7, 16, 17, 18, 19, 20, 21]. In order to validate the effectiveness of the timber code in designing the bolted connections in local hardwoods, an experimental investigation is required to be performed due to the lack of connections data on the Meraka hardwood. The results obtained can also be used to evaluate other design equations such as European Yield Model and Row Shear Model, so that a set of optimized design equations can be recommended. It is believed that, a continuance of this experimental work on local hardwoods can establish a comprehensive wood database that can specifically assist the Malaysia design engineers in developing a strength-effective retrofitting technique of the unreinforced masonry buildings. By matching the wood species and their mechanical properties obtained from the wood database, it can also be utilized by the engineering practitioners of other countries for the same purpose. It should be noted as well that, by providing a strength-optimized design of bolted timber joints, a cost-efficient design output can be achieved due to the less fabrications of the steel bolts are required (i.e., either in number of bolts or in diameter of bolts).

II. Design Equations for Timber Bolted Connections

Referring to the criteria in the timber standards to determine the capacity of bolted joints agreed by the international timber engineering community [22], the models proposed should be capable of identifying each possible failure mechanism. In other words, the wood failure modes of ductile (wood fails in bearing) and brittle (wood fails in fracture) must both be included in the design consideration. The governing capacity of the connection should be taken from the lowest magnitude of strength exhibits between the ductile and brittle wood failures. In order to review the existing design equations that are currently available to estimate the capacity of bolted timber joints loaded parallel to the timber grain, this section provides the detail descriptions of the Malaysia timber code [15]; followed by the European Yield Model (EYM); and the brittle failure models developed by Quenneville et al. [16].

A. *Malaysia Timber Standard*

For determining the permissible load (F_{adm}) of a laterally loaded bolt system, Section 11.2.3 of MS 544: Part 5 [23] can be utilized. The equation is shown below:

$$F_{adm} = k_1 k_2 k_{16} k_{17} F \tag{1}$$

where

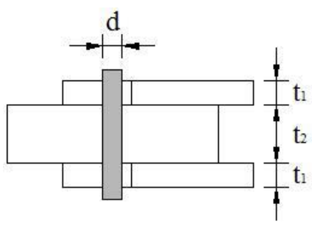
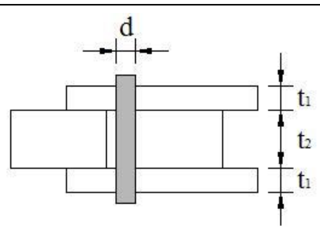
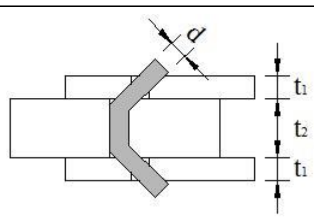
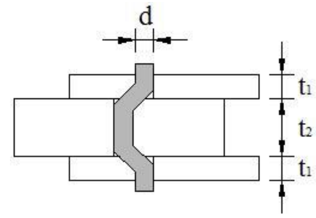
- k_1 = the factor for duration of load, refer to Table 4 of [23];
- k_2 = 1.0 for dry timber or 0.7 for wet timber;
- k_{16} = 1.25 for bolts that transfer load through metal side plates of adequate strength and the bolts are a close fit to the holes in these plates provided that $b/d > 5$ (where b denotes the effective timber thickness and d is the bolt diameter) or 1.0 otherwise;
- k_{17} = factor for multiple bolted joint, see Table 15 of [23];
- F = basic working load as derived in Section 11.2.2 of [23].

The values of the basic working load, *F*, for a selection of bolt diameter size, d, and effective timber thickness, *b*, can be taken from Table 12 of [23] considering for a single bolt bearing parallel to the timber grain acting in single shear. From this, one can see that the timber standard assumes only the ductile failure modes of the bolted joints to be occurring. The controlling parameters of the brittle failure occurrences, which can be found in many published works [7, 16, 17, 18, 20, 22], cannot be found in the standard. Instead, the standard introduces only k_{17}, which is the value is assumed to be less than one for connections with more than four bolts taken from Table 12 of the timber code. This can lead to a very conservative design output as highlighted in [24].

B. European Yield Model

A theory to predict the bolted connection capacity of ductile failure mode was first developed by Johansen in 1949 [25, 26]. This Johansen's theory, also known as the European Yield Model (EYM), assumes a rigid-plastic behavior for both woods and dowel fasteners and prevents the brittle wood failures such as group tear-out or row shear out or splitting. This means that the theory considers the timber to be under embedding stresses due to the contact between the wood surface and the bolt diameter. Also, once the embedding stress surpasses the bending capacity of the bolts, the dowel fasteners are under bending stresses. The connection capacity, typically designated as R, per fastener per shear plane is given in Table I. The equations are representing the four possible modes of failures for a double shear connection type as illustrated in the table.

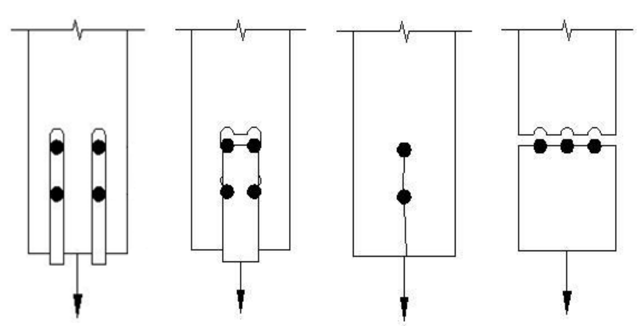
Table I. Possible failure mode and resistance, R, per fastener per shear plane for double shear bolted timber joint

No.	Failure mode and resistance, R
1	 $R = f_{h,1} t_1 d$ (2)
2	 $R = 0.5 f_{h,1} t_2 d\beta$ (3)
3	 $R = \frac{f_{h,1} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_y}{f_{h,1} t_1^2 d}}-\beta\right]$ (4)
4	 $R = \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_y f_{h,1} d}$ (5)

Notes:
- β is the ratio of the embedding strengths, $\beta = f_{h,2}/f_{h,1}$
- $f_{h,1}$ is the embedding strength corresponding to t_1, in MPa
- $f_{h,2}$ is the embedding strength corresponding to t_2, in MPa
- t_1 and t_2 is the timber thickness or fastener penetration of member 1 and 2, in mm
- d is the fastener diameter, in mm
- M_y is the fastener yield moment, in mm, $M_y = (1/6) f_y d^3$
- f_y is the fasterner yield strength, in MPa

C. Row Shear Model

In reference to a number of published literatures on the identification of brittle failures in experimental studies of wood joints [7, 16, 17, 18, 19, 20, 22, 24], it can be observed that group tear-out, row shear, and splitting wood failures are the most common types of brittle modes identified. Illustrations of those failures in bolted timber connection, loaded parallel to the wood grain, including the net tension of woods, are shown in Fig. 1. A set of equations to embody the aforesaid types of brittle failures was developed by Quenneville [16]. The equations and their detailed descriptions of parameters to predict the bolted connection capacity can be found in [22]. The readers of this paper should be noted that, because of the test specimens of this present works were considered only wood connections with single row of fasteners. Therefore, the critical mode of brittle failure was identified to be the failure row shear model (RSM). Due to this reason, only the row shear equation details are provided herein.



(a) (b) (c) (d)

Figure 1. Possible modes of brittle failures in bolted wood joints (a) row shear; (b) group tear-out; (c) splitting; and (d) net tension [22]

The following equation given below can be used to estimate the connection capacity of a single bolt or a group of bolts that fail in row shear of its wood component. In this study, the calibration factor was determined for the purpose of design optimization, which is further discussed in Section V of this paper. The row shear equation is given as:

$$R_{r\,rs} = RS_{i\,\min}\, n_r \tag{1}$$

where

- n_r = number of rows in the joint as per load component;
- $RS_{i\,\min}$ = minimum (RS_i , RS_2 , …, RS_{nr}), in N;
- RS_i = shear capacity along two shear planes of fastener row "i", in N, $RS_i = \dfrac{2(f_v)K_{ls}\, t\, n_{f\,i} a_{cr\,i}}{CF}$;
- f_v = member shear strength, in MPa, equal to $17.8G^{1.24}$;
- G = 5th percentile relative density of timber in the oven dry condition;
- K_{ls} = factor for member loaded surfaces (0.65 for side member, 1 for internal member);
- t = member thickness, in mm;
- $n_{f\,i}$ = number of fasteners in row "i";
- $a_{cr\,i}$ = minimum of et and sb for row "i" (see Fig. 2), in mm; and
- CF = calibration factor.

From the Wood Handbook [27], the correlation between the shear strength and the wood specific gravity can be identified. Thus, in this paper, an expression of $f_v = 17.8G^{1.24}$ is adopted in estimating the Meraka hardwood shear strength value.

III. Materials and Basic Properties

A. Materials

Due to the purpose of this research study was to develop a strength-effective retrofitting technique for the unreinforced masonry buildings, a reliable selection of wood species of the structural components used (i.e., rafters and joists) to construct the roof and floor diaphragms of the buildings is crucial. From [15], the strength of Meraka hardwoods is categorized under the third group (SG3). Referring to the strength group of timber and their applications published by the Malaysian Timber Industry Board [28], SG3 is suitable to be used as structural components like beams, joists, and roof rafters. From this, one can see that the Meraka hardwood can be considered to be one of the most commonly hardwood species used for the floor and roof diaphragms constructions in Malaysia unreinforced masonry buildings. Due to these findings, the Meraka hardwood was selected for the bolted connection tests. The cross section of the solid timber planks used in the preparation of the bolted connection specimens was 50 mm (width) × 100 mm (height) as per recommended by the MS544-2 [15] on the typical sizes of Malaysia structural timbers. In addition, the botanical name of Meraka hardwood is *Shorea Albida*. According to Malaysia Timber Council [29], in the Peninsular Malaysia, Meraka is also known as Red Balau. But, in Sarawak, its common trade name called Alan Batu [30].

Steel materials used in the bolted connection tests were 13 mm diameter of bolts (d_f) with 85 mm length of the bolt shank, and steel plates of 15 mm thick. The length of such bolt shank was used to provide a sufficient length of contact between the bolt shank and wood as well as the two sides of steel plates used to test a double shear connection type. Thus, to avoid the bolt threaded part to be in contact with the wood component. The bolts used were mild steel with a grade of 4.6. To have rigid steel plates, at least a 15 mm thick was used to ensure that the steel plates are intact and do not fail in bearing during testing. So, the connection strength of both wood and bolts can be evaluated accurately. The plates used also a mild steel with an ultimate tensile strength (f_{up}) of 400 MPa.

B. Basic Properties

From the EYM equations, it can be seen that the wood embedding strength value is an essential parameter to be determined for predicting the connection capacity. The embedding strength values of the Meraka hardwood were identified by conducting the embedding strength tests that comply with ISO/DIS 10984-2 [31]. Referring to the row shear equation, the density value of the wood is important for the calculation of the wood shear strength. The Meraka hardwood density values were established by performing the test procedure as per described in [32]. It is well known that the moisture content of timber can significantly affect the capacity of bolted timber connection. A moisture content monitoring on the Meraka hardwood was done by executing the AS/NZ 1080.1 [33] standard experimental method. One should note that all standards recommend that the test specimens must be extracted from the test specimens of the bolted connection tests. From Tables II and III, the values of density & moisture content and embedding strength were tabulated, respectively. By assuming a normal data distribution of the results obtained the fifth percentile values were determined for further use in the calculation of strength prediction values of the bolted wood joints. One should be noted that the fifth percentile values were used to calculate the connection design strength values because the strength values provided by the design equations were based on an estimation of the characteristic strength that is liable to come on bolted joints during its lifetime. In other words, the variation in the strength is assumed to follow a normal distribution curve, which is typically known as the 5th percentile strength value in timber design code. The average moisture content of wood specimens was found equal to 18%, thus, can be considered as dry timber due to in compliance with less than 19% [15]. Satisfactorily equivalent density results were also achieved in comparison with the 732 kg/m^3 at 18% moisture content data published by Forest Department Sarawak [30].

Table II. Moisture content and dry density of Meraka [24]

Species	Total of test pieces	ρ_{avg} (kg/m^3)	CoV	$\rho_{5th\%}$ (kg/m^3)	MC_{avg} (%)
Meraka	180	696	15.0	524	18

Notes:
- ρ_{avg} = average dry density;
- CoV = coefficient of variations;
- $\rho_{5th\%}$ = 5th percentile dry density; and
- MC_{avg} = coefficient of variations.

Table III. Embedding Strength of Meraka [34]

Species	Total of test pieces	$f_{h,avg}$ (MPa)	CoV	$f_{h,5th\%}$ (MPa)
Meraka	158	45.3	15.0	524

Notes:
- $f_{h,avg}$ = average embedding strength;
- CoV = coefficient of variations; and
- $f_{h,5th\%}$ = 5th percentile embedding strength.

IV. Bolted Timber Connection Tests

A. Test specimen configurations

The configuration of all bolted connection test specimens was steel-wood-steel (SWS) or double shear joints for avoiding the effect of eccentricity when the connections were tensile loaded parallel to grain. Eight groups of connections with different configuration details were prepared to investigate the ductile failure modes. To examine the brittle failure modes of the Meraka hardwood, ten different groups of connections were fabricated. The list of the configuration details for each connection group tested are given in Table IV. All groups consisted of a minimum of ten duplicates for further statistical data analysis purposes.

Every specimen of the bolted connection testing was prepared to have an identical connection detail at both extremities, thus, any extremity failed shall governs the capacity of the connections. To avoid both end connections become unity, a 400 mm distance between connections was fabricated in each bolted connection specimen. The 400 mm distance used was in accordance with the minimum distance of 30d stated in [31], whereas d is the bolt shank diameter. The reader should be noted that all connection groups were designed to have only one row of fasteners, $n_r = 1$. This was done to comply with the minimum size of a (see Fig. 2), which is equal to 3d as per [31]. The number of fasteners, n_f, for all connection groups were varies from one to three bolts.

In order to evaluate the strength of connections that fail in ductile mode (i.e., bearing failure of timber component and bending failure of fasteners), the groups of connections with $e_t > 50$ mm (Groups 1, 2, 3, 4, 6, 7, 8, and 9) were fabricated. Besides that, the groups of connections with $e_t = 50$ mm (Group 5, and Groups 10-18) were prepared to assess the strength of connections that fail in brittle mode (i.e., shear failure of timber element). The latter connection groups are important in the calibration factor determination of the row shear equation for achieving an optimized design of the bolted connection capacity in the Meraka hardwood.

The bolt spacing, s_b, is also one of the most important parameters that controlling the failure modes in bolted wood joints. In the row shear model equation, it can be seen that the $a_{cr\ i}$ is taken as the minimum between e_t and s_b. This tells us that the row shear failure can be occurred not only at the end distance, but as well as possible to be observed at the spacing between bolts. To have a better view on the configuration details of the bolted timber connection specimens, the illustrations of all parameters are drawn in Fig. 2.

Table IV. Specimen configuration details

Group	d (mm)	e_t (mm)	s_b (mm)	n_r	n_f
1	13	150	-	1	1
2	13	125	-	1	1
3	13	100	-	1	1
4	13	75	-	1	1
5	13	50	-	1	1
6	13	150	100	1	2
7	13	125	100	1	2
8	13	100	100	1	2
9	13	75	100	1	2
10	13	50	100	1	2
11	13	150	50	1	2
12	13	125	50	1	2
13	13	100	50	1	2
14	13	75	50	1	2
15	13	50	50	1	2
16	13	100	50	1	3
17	13	75	50	1	3
18	13	50	50	1	3

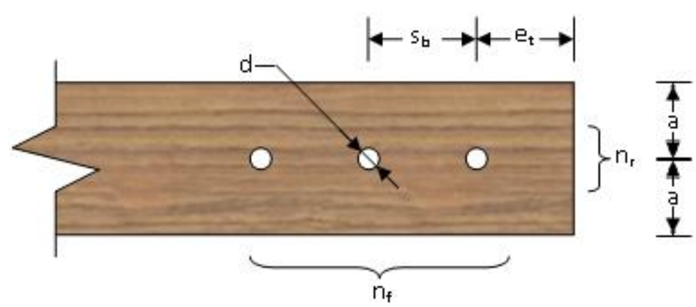


Figure 2. Illustrations of the variables used

B. Test Setup and Procedures

All specimens of bolted timber connections were loaded in tension using Shimadzu 300kN Universal Testing Machine, which the direction of the loading was parallel to the timber grain. The typical setup of the bolted connection testing can be found in [24]. The loading applied was a monotonic or static tension load at 1 mm per minute rate of displacement-control

[35]. In order to avoid a development of friction force between the two side steel plates and the wood specimen at the centre, also between the steel plate and bolt head or nut, the bolts were only tightened up using force of fingers. This was also done to allow the test piece to be self-aligned during the application of loading. During testing, the load and displacement readings were transferred by the data logger to a computer unit for recording. Each connection specimen was loaded until it fails, which can be identified from the graph of load versus displacement displayed by the computer monitor. The failure modes observed in the timber specimens were clearly marked and recorded.

V. Bolted Timber Connection Test Results and Discussion

A. Previous Works Related to the Present Study

The structural performance of the bolted connections for Meraka hardwood was investigated by Abdul Karim et al. [34]. The study was performed to identify the failure mode of the hardwood in ductile behavior. All test specimens were fabricated with a connection configuration of more than 50 mm of end distance and bolts spacing. From the published experimental work, it was found out that the connection configuration details induce a ductile failure mode of bearing in the wood component. The efficacy of both MS544-5 [15] and EYM equations to predict the bolted timber connection strength was evaluated. The strength predictions of the timber code were verified to be 38% efficient in comparison to the experimental data, whereas the EYM shown a better strength prediction of 81%.

The evaluation of the Meraka hardwood was extended by Abdul Karim et al. [24] to determine the possible failure modes in brittle behavior. In the study, the specimens of bolted wood joints were prepared with an end distance or spacing between bolts of 50 mm. Due to this connection configuration detail, the brittle failure mode of row shear was predominantly observed in the wood pieces. A comparison between the prediction equations (i.e., MS544-5 and RSM) and the test results was done to measure the strength prediction capability of the currently available design equations. The MS544-5 was identified with 41% of accuracy in predicting the bolted timber joint capacity, meanwhile the RSM provides a better capacity prediction of 79%.

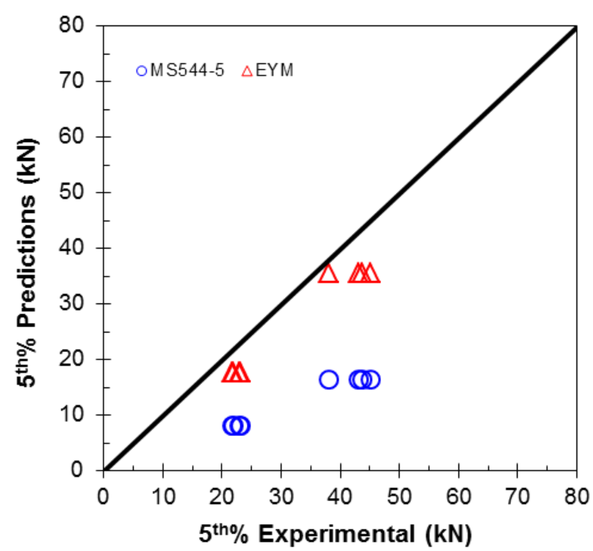
For the ease of all readers, the comparisons made between the strength predictions of the current design equations and the experimental data published in both Abdul Karim et al. [34] and Abdul Karim et al. [24] are shown in Table V and Table VI for ductile and brittle failure modes, respectively. In order to see the efficiency of the EYM and RSM equations in predicting the bolted connection capacity, from the comparison findings, their graphical data distribution is plotted in Fig. 3.

Table V. Prediction and experimental of Meraka hardwood failed in bearing [34]

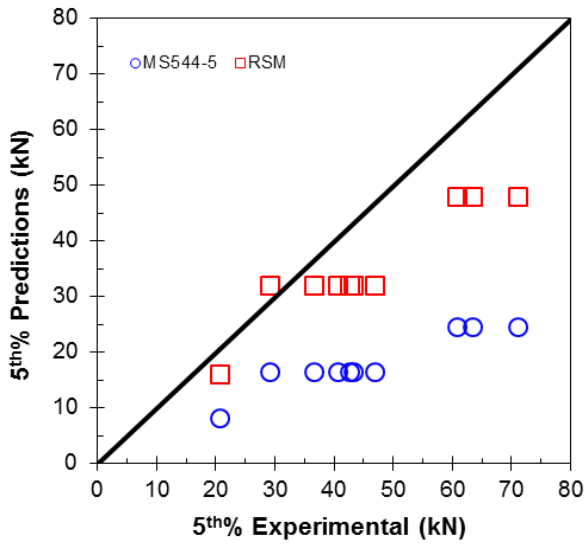
	Predictions		Experimental			Ratio	
	EYM	MS544	R_{avg}	COV	$R_{5th\%}$	EYM	MS544
Group	(kN)	(kN)	(kN)	(%)	(kN)	$R_{5th\%}$	$R_{5th\%}$
1	18	8.2	32	20	22	0.82	0.38
2	18	8.2	30	16	22	0.81	0.37
3	18	8.2	31	16	23	0.77	0.35
4	18	8.2	30	14	23	0.77	0.36
6	35	16.4	46	10	38	0.93	0.43
7	35	16.4	53	12	43	0.82	0.38
8	35	16.4	57	13	45	0.79	0.36
9	35	16.4	56	13	44	0.81	0.38

Table VI. Prediction and experimental of Meraka hardwood failed in row shear [24]

	Predictions		Experimental			Ratio	
	RSM	MS544	R_{avg}	COV	$R_{5th\%}$	RSM	MS544
Group	(kN)	(kN)	(kN)	(%)	(kN)	$R_{5th\%}$	$R_{5th\%}$
5	16	8.2	26	11	21	0.77	0.40
10	32	16.4	53	11	43	0.74	0.38
11	32	16.4	53	14	41	0.78	0.40
12	32	16.4	47	23	29	1.10	0.56
13	32	16.4	50	9	43	0.75	0.38
14	32	16.4	55	9	47	0.68	0.35
15	32	16.4	48	14	37	0.87	0.45
16	48	24.6	79	6	71	0.67	0.35
17	48	24.6	71	7	63	0.75	0.39
18	48	24.6	65	4	60	0.79	0.40



(a) EYM [34]



(b) RSM [24]

Figure 3. EYM and RSM accuracy evaluations in bolted connection strength predictions

The reader of this paper should be noted that, the detailed descriptions of the observed ductile and brittle failure modes can be found in the above-mentioned published literatures of [34] and [24], respectively. Referring to the latter published work, parameters that can affect the ultimate capacity of connections were also discussed in details. Thus, this present study was reported to disclose the details of the calibration factor determination of the RSM equation for achieving the design strength optimization of bolted connections in particular to the Meraka hardwood.

B. *Determination of RSM Calibration Factor*

The groups of connections involved in the analysis of RSM calibration factor identification were the one that failed in row shear, which were G5, G10, G11, G12, G13, G14, G15, G16, G17, G18. All of these connection groups were fabricated with end distance or bolt spacing of 50 mm purposely for inducing the row shear failure occurrences on the test specimens.

Initially, a guess value of calibration factor was used to predict the average strength values of those connection groups using the RSM equation. The strength values predicted by the RSM using the guess value of the calibration factor were then plotted against the average experimental data and a linear regression line for the plotted data was established. A forty-five-degree linear line, which is represented as a reference line of one-to-one ratio, was also drawn for comparison with the linear regression line. By guessing several values of the calibration factor, the linear regression line that approximately matched with the one-to-one ratio reference line can be found. From these iterations, a calibration factor of 2.5 was found to be appropriate as the plotted data are mostly below the reference line as shown in Fig. 4. The RSM calibration factor nominated in the regression model has resulted a good coefficient of determination, R^2, value of 0.99 for timber connection statistical analyses. One must note that this calibration factor can be used to evaluate the strength prediction capability of the RSM for the comparison with the experimental data presented in the previous Section V (A). Thus, the design optimization of the bolted connection for Meraka hardwood using the RSM can be achieved.

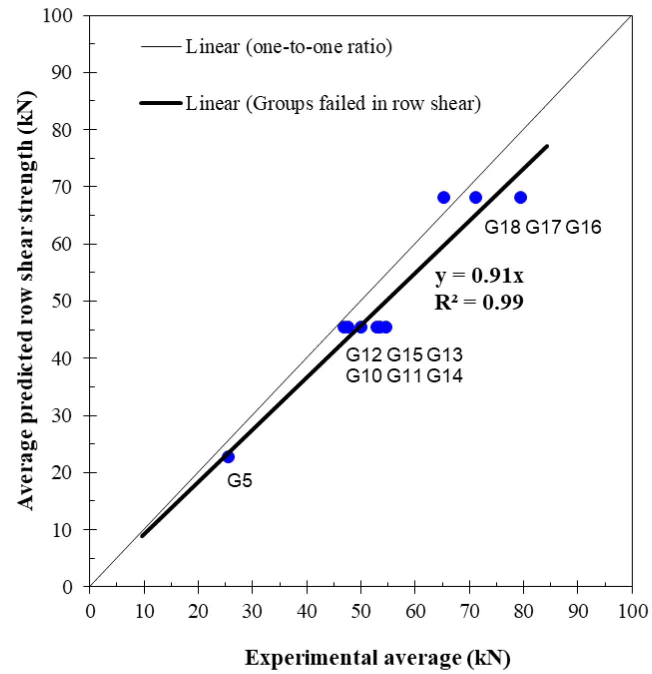


Figure 4. RSM calibration factor determination using the best-fitted technique.

VI. Conclusion

By conducting the bolted timber connection and the embedding strength tests, the experimental data can be utilized to determine the parameters of both Row Shear Model and European Yield Model equations, respectively. The embedding strength of the Meraka hardwood species provided herein, which are not available in the existing Malaysia timber standard, can be used to obtain an optimized strength design of bolted joints for the EYM that representing the ductile mode of failure. The calibration factor of the Meraka hardwood species found from the regression model can be applied in the RSM that embodying the brittle failure mode to achieve optimization of design in the bolted connection capacity. Both EYM and RSM establish a percentage of design optimization up to 81% and 79%, respectively. Thus, the combination of EYM and RSM equations is recommended to be included in the [15] design procedures of the bolted timber connections, so that a strength-effective retrofitting technique of unreinforced masonry buildings can be developed.

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REFERENCES

[1] Muhammad Talpur, M. A. Melaka Review: Tourism - Places of Interest - The Stadthuys. http://www.melakareview.com/Home/travel/places-of-interest/the-stadthuys [Retrieved on 01/01/17] (1990)

[2] Ho, K. C., Hasan, A. S. & M Noordin, N. An Influence of Colonial Architecture to Building Styles and Motifs in Colonial Cities in Malaysia. 8th International Conference of the Asian Planning Schools Association. Penang, Malaysia (2005).

[3] Lee, W. H. The History of Melaka's Urban Morphology. https://www.researchgate.net/publication/301543855 [Retrieved on 01/01/17]. (2016)

[4] Kamal, K. S., Ab Wahab, L. & Ahmad, A. G. Pilot Survey on the Conservation of Historical Buildings in Malaysia. 2nd International Conference on Built Environment in Developing Countries. Penang, Malaysia (2008).

[5] Federal Emergency Management Agency. Techniques for the Seismic Rehabilitation of Existing Buildings. FEMA 547, Federal Emergency Management Agency, Washington, D.C. (2006).

[6] New Zealand National Society for Earthquake Engineering. Draft Guidelines for Assessing and Strengthening Earthquake Risk Buildings, New Zealand National Society for Earthquake Engineering, Wellington, New Zealand (1995).

[7] Abdul Karim, A. R. Seismic Assessment of Wall-Diaphragm Connections in New Zealand Unreinforced Masonry Buildings. PhD thesis The University of Auckland, New Zealand, 2012.

[8] Bruneau, M. State-of-the-art report on seismic performance of unreinforced masonry buildings. Journal of Structural Engineering 120(1) (1994a) 230-251.

[9] Bruneau, M. Seismic evaluation of unreinforced masonry buildings – a state-of-the-art report. Canadian Journal of Civil Engineering 21(2) (1994b) 512-539.

[10] Blaikie, E. L. & Spurr, D. D. Earthquake vulnerability of existing unreinforced masonry buildings. Works Consultancy Services, Wellington, New Zealand (1992).

[11] Griffith, M. C. Performance of Unreinforced Masonry Buildings during the Newcastle Earthquake, Australia. Research Report No. R86, The University of Adelaide, Adelaide, Australia (1991).

[12] Evans, N. The earthquake's impacts on buildings and infrastructure. In Shaken Up, proceedings of a workshop on Recovery following the Gisborne Earthquake (Vol. 7) (2009).

[13] Dizhur, D., Lumantarna, R., Ismail, N., Ingham, J. M. & Knox, C. Performance of unreinforced and retrofitted masonry buildings during the 2010 Darfield earthquake. Bulletin of the New Zealand Society for Earthquake Engineering 43(4): 321 (2010).

[14] Dizhur, D., Ingham, J., Moon, L., Griffith, M., Schultz, A., Senaldi, I., Magenes, G., Dickie, J., Lissel, S., Centeno, J., Ventura, C., Leite, J. & Lourenco, P. Performance of masonry buildings and churches in the 22 February 2011 Christchurch earthquake. Bulletin of the New Zealand Society for Earthquake Engineering 44(4) (2011) 279-296.

[15] Malaysian Standard. MS 544: Part 5: 2001 Code of Practice for Structural Use of Timber: Part 5: Timber Joints. Department of Standards Malaysia (2001).

[16] Quenneville, J. H. P. & Mohammad, M. On the Failure Modes and Strength of Steel-Wood-Steel Bolted Timber Connections Loaded Parallel-To-Grain. Canadian Journal of Civil Engineering 27 (2000) 761-773.

[17] Mohammad, M., and Quenneville, J. H. P. Bolted Wood-Steel and Wood-Steel-Wood Connections: Verification of a New Design Approach. Canadian Journal of Civil Engineering 28 (2001) 254-263.

[18] Quenneville, J. H. P., Smith, I., Aziz, A., Snow, M. & Ing, H. C. Generalised Canadian Approach for Design of Connections with Dowel Fasteners. CIB-W18 meeting Proceedings, Florence, Italy, paper CIB-W18/39-7-6 (2006).

[19] Quenneville, J. H. P. & Jensen, J. Validation of the Canadian Bolted Connection Design Proposal. CIB-W18 meeting Proceedings (2008).

[20] Abdul Karim, A. R., Quenneville, P., M.Sa'don, N. & Ingham, J. Assessment Guidelines of Wall-Diaphragm Connections for Masonry Buildings. In: Yazdani, S. and Singh, A. (Ed.). New Developments in Structural Engineering & Construction. Research Publishing, Singapore (2013).

[21] Cabrero, J. M. & Yurrita, M. Performance assessment of existing models to predict brittle failure modes of steel-to-timber connections loaded parallel-to-grain with dowel-type fasteners, Engineering Structures 171 (2018) 895-910.

[22] Quenneville, P. Design of Bolted Connections: A Comparison of a Proposal and Various Existing Standards. Journal of the Structural Engineering Society (SESOC) New Zealand Inc. 22(2) (2009) 57-62.

[23] Malaysian Standard 2001. MS 544: Part 5: Code of Practice for Structural Use of Timber: Part 5: Timber Joints. Department of Standards Malaysia (2001).

[24] Abdul Karim, A. R., Quenneville, P., & Sa'don, N. M. (2021). Shear failure of the Meraka hardwood in bolted connections loaded parallel to the timber grain. In IOP Conference Series: Materials Science and Engineering (Vol. 1101, No. 1, p. 012005). IOP Publishing.

[25] Blass, H. J. Joints with Dowel-type Fasteners. In: Thelandersson, S. and Larsen, H. J. (Ed.). Timber Engineering. John Wiley & Sons Ltd., England, 2003, Pp. 315-331

[26] Ballerini, M., Mares, F. & Sommavilla, D. Dowel Timber Connections with Two Shear Planes Loaded Parallel-to-the-Grain: Reliability of the New European Design Code

by Means Experimental Tests. 9th World Conference on Timber Engineering 2006, Portland, Oregon, USA (2006).

[27] Forest Products Laboratory. Wood Handbook: Wood as an Engineering Material." Forest Service U.S. Department of Agricultural, Washington, D.C (1987)

[28] Malaysian Timber Industry Board. Wood Wise: Strength Groups of Timber and Their Applications. http://www.mtib.gov.my/index.php [Retrieved on 29/04/13] (2013)

[29] Malaysia Timber Council. MTC Wood Wizard: Malaysian Timbers at Your Fingertips. http://www.mtc.com.my/resources-WoodWizard.php [Retrieved on 31/01/13] (2013)

[30] Forest Department Sarawak. Handbook of Some Sarawak Timbers. Forest Department Sarawak, Kuching, Sarawak, Malaysia (1999).

[31] International Organization for Standardization. Draft International Standard ISO/DIS 10984-2 Timber structures - Dowel-type fasteners - Part 2: Determination of embedding strength and foundation values. International Organization for Standardization, Geneva, Switzerland (2008).

[32] Australian/New Zealand Standard. AS/NZS 1080.3: 2000 Timber - Methods of Test - Method 3: Density. Standards New Zealand (electronic copy) (2000).

[33] Australian/New Zealand Standard. AS/NZS 1080.1: 1997 Timber - Methods of Test - Method 1: Moisture content. Standards New Zealand (electronic copy) (1997).

[34] Abdul Karim, A. R., Quenneville, P., M.Sa'don, N. & Yusof, M. Investigating the Meraka Hardwood Failure in Bolted Connections Parallel to the Timber Grain. International Journal of Engineering & Technology 7(3.18) (2018) 62-65.

[35] International Organization for Standardization (1983). International Standard ISO 6891 Timber structures – Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics. International Organization for Standardization, Geneva, Switzerland.

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