M-θ Curve of Timber Connection with Various Bolt Arrangements Under Monotonic Loading

Ali Awaludin [*], Watanachai Smittakorn [**], Toshiro HAYASHIKAWA [***] and Takuro HIRAI [****]

* Graduate Student, Graduate School of Engineering, Hokkaido University, Nishi 8 Kita 13 Kita-ku, Sapporo 060-8628
** Ph.D., Lecturer, Dept. of Civil Eng., Chulalongkorn University, Phayathai Rd Pathumwan, Bangkok 10330
*** Dr. of Eng., Professor, Dept. of Civil Eng., Hokkaido University, Nishi 8 Kita 13 Kita-ku, Sapporo 060-8628
**** Ph.D., Professor, Dept. of Timber Eng., Hokkaido University, Nishi 9 Kita 9 Kita-ku, Sapporo 060-8589

Moment resistance of timber joint is semi-empirically analyzed by considering the experimental results of single-bolt connection and the energy conservation principle. In the analysis, experimental load-slip response of single-bolt connection is approximated by exponential and bi-linear models. Verification of the proposed models is carried out through full scale test of semi-rigid joints that monotonically loaded by a pure moment. Wood specimens of *Shorea obtusa* and three different bolt arrangements are implemented in both steel-to-wood and wood-to-wood connections. Experimental results show bolt arrangement that has long distance along the grain between bolts gives high moment capacity, high ductility, and high rotational stiffness. Wood-to-wood connections have higher moment resistance and higher ultimate joint rotation than steel-to-wood connections, since wooden plates as side members in wood-to-wood connection are less stiff than steel plates and they allow more load redistribution among bolts during inelastic stage.

Key Words: Bolt arrangement, load redistribution, moment-rotation curve, timber connection

1. Introduction

Wood has been known to have high strength-to-weight ratio that leads to a rapid development of light-weight structures all over the world. Well designed wood structures can perform satisfactorily under service loading or even under seismic/dynamic actions. In contrast, poorly designed timber structures may exhibit catastrophic brittle-failure which is initiated mostly in the connection area. So it is generally accepted that joints or connections are the weak part of timber structures. Some studies, moreover, showed that the assessment of timber building damages after extreme wind or earthquake often pointed out the inadequate connection as the primary cause[1), 2)]. Ductile connection which is essentially required during seismic events could only be provided by detail design of steel fasteners to wood member such as in bolted timber connections. The use of a large number of small-diameter bolts results in a more ductile response than a small number of large diameter bolts[3)].

Connections which are mainly subjected to moment loads known as moment-resisting connections are commonly used to join members of most engineered timber structures such as arches or semi-rigid frames. The rotational stiffness of these connections, which is often modeled by springs, is flexible and lies somewhere between those of fully pinned and fully fixed. When these connections are subjected to applied moments, the mechanically fastened members rotate relative to each other and thus contributing additional deformations. Therefore, the infinite rotational stiffness assumption as adopted in rigid joint of most structural analyses is unrealistic and underestimates the overall structure deformation[4)].

The rotation induced by applied moments has the effect of redistributing member end forces through fastener load-slip actions. Fastener load increases as the joint rotation develops until the equilibrium condition between the applied moment and the internal moment developed by load-slip actions is reached. The moment-rotation curve of bolted timber connection has, in general, small rotational stiffness at initial stage which is mainly originated from the member friction. In this rotation level, the fasteners displace freely within the gap created due to different diameter of fastener and pre-drilled hole. In connection with several fasteners, widely called multiple-bolt connection, most fasteners in the group resist lateral load in inclined angles to the wood grain. These inclined angles may vary from one fastener to another according to fasteners arrangement. Therefore, it is well understood that the moment resistance is greatly affected by fastener arrangement[5]. Load distribution among fastener in multiple-bolt connection is highly indeterminate even if the moment load is acting on the center of fastener configuration[6-7].

This study investigates the moment-rotation (M-θ) curve of timber joint using different bolt arrangements and different materials of side members subjected to monotonic pure moment. In particular, the purpose of this study is to identify the important characteristic of bolt arrangement that governs joint capacity and ductility as well. A tropical hardwood species, *Shorea obtusa*, is used in the experimental program. Moment resistance is analyzed semi-empirically based on the experimental results of single-bolt connection and the energy conservation principle. Failure of wood main member (crushing followed by splitting) is selected because this failure mode is often found in practice. Findings in this study are expected to enrich the practical considerations of structural engineers especially when they deal with hardwoods.

2. Moment Resistance Analysis

A moment-resisting connection subjected to an applied moment as shown in Fig. 1 often demonstrates a nonlinear moment-rotation (M-θ) response. This is completely affected by some material and geometric nonlinearities inherent in the connection. The applied moment will generate a lateral force on each fastener with the force being at a right angle to the line connecting the fastener and the center of fastener group. All fasteners displace to their new positions as marked by the dotted-circle by the same amount of rotation angle to the center of the fastener group when the rigid-plate assumption is utilized (see Fig. 1). Fastener displacement due to applied moment represents the relative movement between main and side members.

Moment resistance of connection can be derived from the energy conservation principle when the energy loss off the system (e.g.: friction between the adjacent surfaces) is assumed to be small and can be ignored[8-9]. This principle

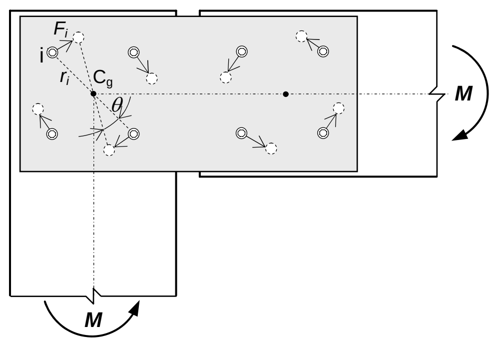


Fig.1 Example of moment-resisting connection

stated that the external work done by applied bending moment is equal to the internal work done by the fasteners:

$$\text{External work } (W_E) = \text{Internal work } (W_I) \qquad (1)$$

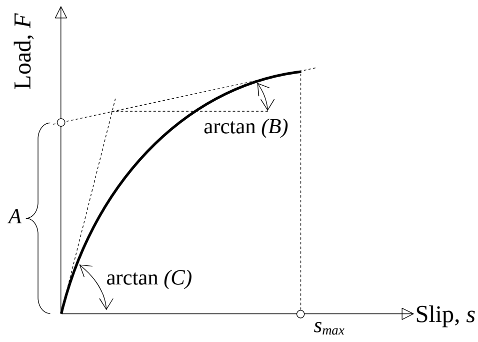
Due to a non-linear relationship between applied bending moment and joint rotation (θ) as previously stated, the external work is then expressed by integration of the total work through Eq. 2. The internal work is obtained by summing the product of the force acting on each fastener (F) and its corresponding slip (s) which is also non-linear as expressed in Eq. 3. Introducing Eq. 2 and Eq. 3 into Eq. 1, finally moment resistance of connection can be obtained from Eq. 4.

$$W_E = \int_0^{\theta_{\max}} M d\theta \qquad (2)$$

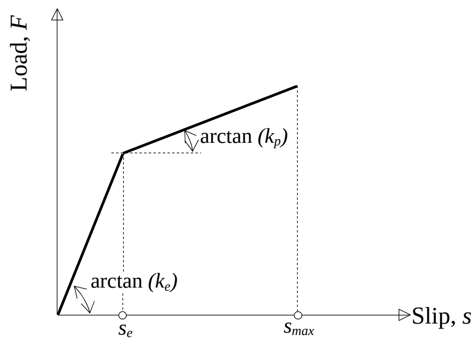
$$W_I = \sum_{bolts} \left(\int_0^{s_{\max}} F ds \right) \qquad (3)$$

$$M = \frac{d}{d\theta} \sum_{bolts} \left(\int_0^{s_{\max}} F ds \right) \qquad (4)$$

The solution of Eq. 4 definitely depends on the empirical model of lateral load-slip relationships. The load-slip curve does not implicitly inform the constitutive equation of wood material but describes the mechanical behavior of wood under a lateral load exerted by a fastener. Instead of using logarithmic model[9], the exponential model known as Foschi's model[10] and the bi-linear model as illustrated in Fig. 2 are investigated in this study. The exponential model is selected since it was frequently used by many previous researchers, while the bi-linear load-slip curve is chosen due to its simplicity.



(a) Exponential load-slip relationship



(b) Bi-linear load-slip relationship

Fig.2 Empirical models of load-slip relationship

The relationship between the lateral force and its slip up to maximum load according to the exponential model can be expressed by

$$F = (A + Bs)\left(1 - e^{-\frac{C}{A}s}\right) \tag{5}$$

where A, B, and C are empirical constants (see Fig. 2a) obtained from test results of single-bolt connection for loading direction parallel to the wood grain. Using the bi-linear model, the load-slip relationship can be described as follows

$$F = k_{e\alpha} s\,; \quad s \le s_e \tag{6.a}$$

$$F = k_{e\alpha} s_e + k_{p\alpha}\left(s - s_e\right); \quad s > s_e \tag{6.b}$$

where parameter constants (s_e is elastic slip, $k_{e\alpha}$ and $k_{p\alpha}$ are elastic and plastic slip moduli at loading direction α to the wood grain, respectively.) are determined from the test results of single-bolt connections.

For small angle of rotation, fastener slip can be expressed as in Eq. 7 where r is distance of fastener to the rotation center of the group of fasteners and is assumed to be unchanged during rotation[9), 11)]. In the equation, tan (θ) is replaced by θ in unit of radian resulting a negligible error for angle of rotation up to 0.087 rad (equals to 5 degree).

$$s = r\tan(\theta) \cong r\theta \tag{7}$$

$$M = \sum_{bolts} r_i\left(A + Br_i\theta\right)\left(1 - e^{-\frac{C}{A}r_i\theta}\right) \tag{8}$$

$$M = \theta\sum_{bolts}\left(k_{e\alpha} r_i^2\right) \tag{9.a}$$

$$M = \sum_{bolts}\left(k_{e\alpha} - k_{p\alpha}\right)s_e r_i + \theta\sum_{bolts} k_{p\alpha} r_i^2 \tag{9.b}$$

By substitution of Eq. 5 ~ Eq. 7 into Eq. 4, moment resistance of exponential and bi-linear models can be obtained from Eq. 8 and Eq. 9, respectively. When the parameter constants of Eq. 8 and Eq. 9 are given, the moment-rotation response of an arbitrary fastener arrangement can be generated.

As an orthotropic material, wood exhibits different load-slip responses according to the angle of fastener load to the wood grain. When the load angle is parallel to the grain, the maximum load that can be sustained is significantly higher compared with the maximum load of loading direction perpendicular to the grain. This behavior is also well observed in the case of slip modulus (the slope of the curve). In general, the lateral load at direction α to the wood grain (F_α) can be expressed by Eq. 10, while the elastic and plastic slip moduli at direction α to the wood grain ($k_{e\alpha}$ and $k_{p\alpha}$) is expressed by Eq. 11 and Eq. 12, respectively.

$$F_\alpha = \frac{(A + Bs)\left(1 - \exp(-\frac{C}{A}s)\right)}{\left(a_1 - a_2\exp(-\frac{a_3}{a_2}s)\right)\sin^m\alpha + \cos^m\alpha} \tag{10}$$

$$k_{e\alpha} = \frac{k_{e//}k_{e\perp}}{(k_{e//}\sin^m\alpha + k_{e\perp}\cos^m\alpha)} \tag{11}$$

$$k_{p\alpha} = \frac{k_{p//} + k_{p\perp}}{2} \tag{12}$$

Where a_1, a_2, a_3, and m are constants obtained from a series of single-bolt connection test results[12)]. The subscripts // and $\perp$ in the above equations denote the loading direction of parallel and perpendicular to the wood grain, respectively.

The *National Design and Specification for Wood Construction* (NDS)[13], in particular, determines m equals to 2.0.

3. Experimental Verification

Wood specimens of 34 mm thickness are used both as main member and side members of wood-to-wood (WW) connections. Steel-to-wood (SW) connections are made of wood specimen of 34 mm thickness and two steel plates of 4 mm thickness. The bolt of 12.4 mm of diameter is inserted into a 13 mm pre-drilled hole so that the clearance or gap between bolt and wood member is not higher than 1.0 mm[14]. The diameter of bolt and the thickness of wood member are designed according to the European Yield Model[15] to give the desired failure mode, crushing and followed by splitting of wood main member. Only hand tightening is applied to the bolt in the connection specimens before tested.

Wood specimens are constructed from *Shorea obtusa*, and are cut so as to avoid localized defects in the wood as much as possible. Wood specimens are conditioned to the equilibrium moisture content before tested. Connection specimens are tested under four-point bending test as illustrated in Fig. 3 where the clear-span length between two supports and the height of tested specimen are 2750 mm and 200 mm, respectively. The applied load is distributed through a steel plate which has a width equal to the thickness of main member.

Each tested specimens consists of two symmetrical joints which are expected to perform in the similar manner during test. Three bolt arrangements shown in Fig. 4 are applied to both SW and WW connections because they are used commonly in practice. These three bolt arrangements have similar characteristics: the number of bolts; and the overlapping-area. Geometry requirements of connection are designed according to EUROCODE 5[14].

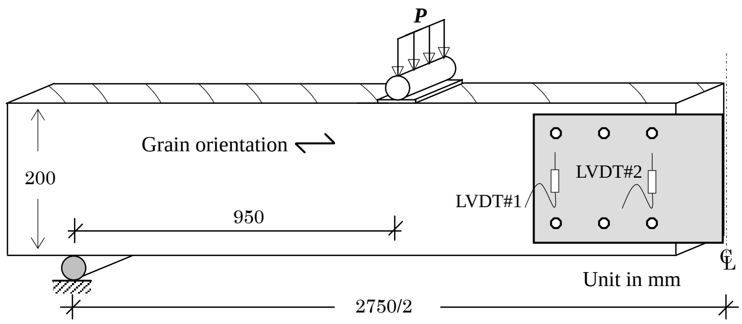


Fig.3 Simplified test set-up (half of symmetry)

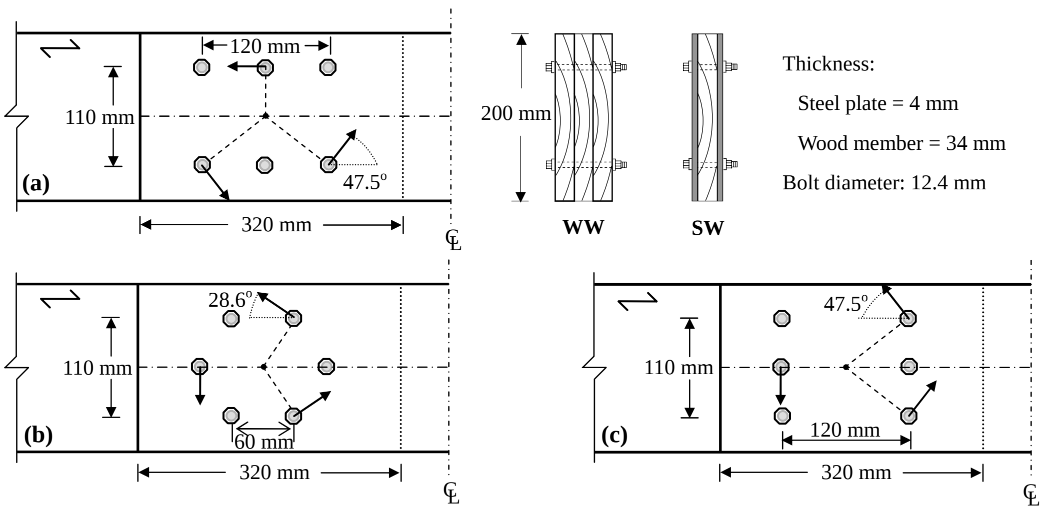


Fig.4 Fastener arrangements: (a) 6H bolt arrangement; (b) 6C bolt arrangement; and (c) 6V bolt arrangement

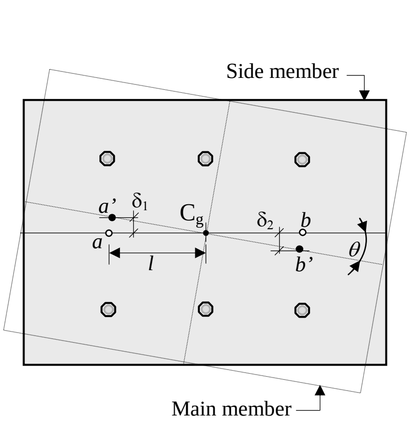


Fig.5 Schematic to measure the rotation of joint

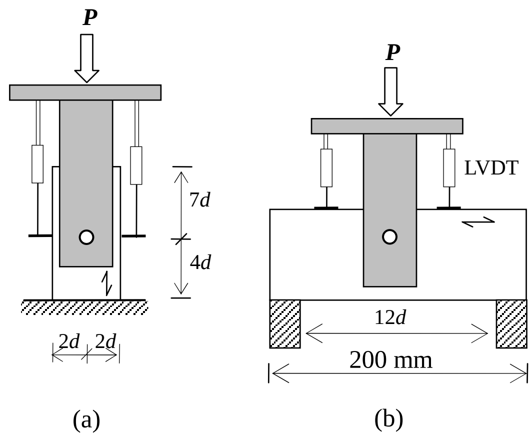


Fig.6 Single bolt connection test setup: (a) loading parallel to the wood grain; and (b) loading perpendicular to the wood grain. *d* is bolt diameter

The joint rotation is measured by considering the relative deformation between side member and main member which is recorded by linear variable differential transducers (LVDT's), 1 and 2, as can be seen in Fig. 3 and Fig. 5. LVDT 1 measures the movement of *a* to *a*', while LVDT 2 records the movement of *b* to *b*'. Due to small deformation assumption, the joint rotation is sufficiently calculated by averaging the rotation obtained from the two LVDT's as

$$\theta \cong \frac{1}{2}\left\{\left(\frac{a-a'}{l}\right)+\left(\frac{b-b'}{l}\right)\right\}=\frac{\delta_1+\delta_2}{2l} \qquad (13)$$

where δ_1 and δ_2 are relative displacements of two measurement points of distance 2l which rigidly attached to side members but freely connected to main member. The applied load and displacement data are measured continuously by a data acquisition system and saved on a personal computer. The test is stopped when the specimens show brittle failure due to wood splitting or significant decrease of applied load.

Single-bolt connection tests are carried out besides moment-resisting connection tests in order to define the parameter constants expressed in Eq. 6 and Eq. 10. In this test, a monotonic compressive load is applied using a displacement control rate of 1.2 mm/min. The diameter of the bolt and the lead-hole are similar to those used in moment-resisting connection tests. Wood samples used in single-bolt connection test are taken from the wood specimens used in moment-resisting connection test. Figure 6 show a single-bolt connection in the test setup of loading parallel and perpendicular to the wood grain. The wood thickness and bolt diameter (*d*) are 34 mm and 12.4 mm, respectively. The joint configurations are based on NDS[13)] requirements, and the test setup are similar to previous study[16)]. The LVDT measures the relative displacement between the side member (steel plate for SW connection or wood member for WW connection) and the wood main member. The actual deflection is neither needed nor measured. Bearing properties of S*horea obtusa* under a bolt for five different loading directions to the grain were reported previously by the authors[17)]. When the maximum embedment of bolt into wood member was determined by first crack/splitting, the maximum embedment for any loading angles to the wood grain could be well replaced with the average value between the maximum embedment of loading parallel and perpendicular to the grain.

4. Results and Discussions

The exponential load-slip model (Eq. 5) with parameter constants listed in Table 1 successfully fitted experimental load-slip curves of single-bolt connections for loading directions parallel and perpendicular to the grain. Utilizing those constants, load-slip curve of any load angles to the grain can be generated by Eq. 10. In particular, load-slip curves for load angles 0°, 28.6°, 47.5° and 90° to the wood grain are analyzed and presented in Fig. 7; these load angles are obtained due to joint configuration illustrated in Fig. 4. Experimental load-slip curves of single bolted connections are also approximated by bi-linear load-slip model. The characteristics of the bi-linear model best fitted with the test results are summarized in Table 2. The single-bolt connections of loading direction parallel to the wood grain failed due to crack of wood main member after reaching the

Table 1 Parameter constants of exponential model

	SW	*WW*		*SW*	*WW*
A (kN)	18	15.5	a_1	1.368	1.300
B (kN/mm)	1.4	4.10	a_2	-1.380	-0.640
C (kN/mm)	29.4	33.2	a_3	-1.063	-0.623
s_{max} (mm)	2.28	3.22	*m*	2.0	2.2

Table 2 Parameter constants of bi-linear model

	SW	*WW*		*SW*	*WW*
$k_{e//}$ (kN/mm)	19.7	19.8	*m*	2.0	2.2
$k_{e\perp}$ (kN/mm)	7.9	10.1	s_e (mm)	0.70	1.23
$k_{p//}$ (kN/mm)	1.5	1.6	s_{max} (mm)	2.28	3.22
$k_{p\perp}$ (kN/mm)	2.2	2.9	-	-	-

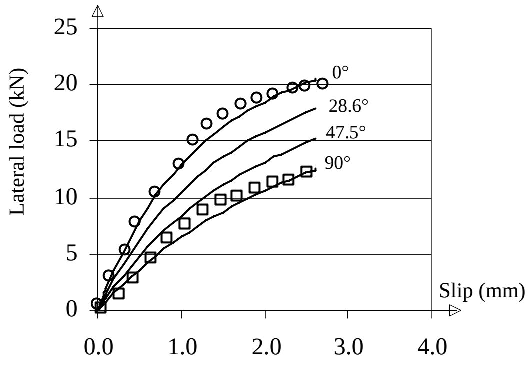
Table 3 Moment resistances of connections

	Maximum moment resistance (kNm)				
	M_e	M_p^{e}	M_p^{b}	M_e/M_p^{e}	M_e/M_p^{b}
SW6H	3.61	7.34	5.29	0.50	0.68
SW6C	4.11	6.30	4.71	0.65	0.87
SW6V	4.40	6.64	4.50	0.66	0.98
WW6H	7.40	9.17	10.50	0.81	0.70
WW6C	8.08	10.14	8.65	0.80	0.93
WW6V	9.62	9.43	9.05	1.02	1.06

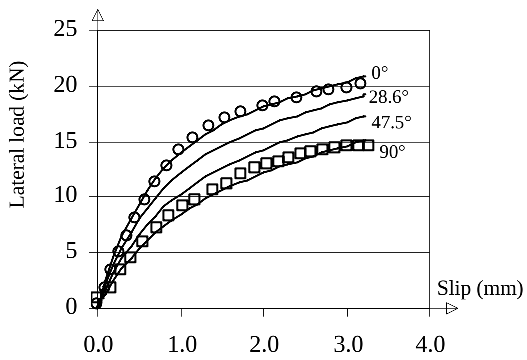
M_e : Experimental moment resistance

M_p^{e} : Predicted moment resistance of exponential model

M_p^{b} : Predicted moment resistance of bi-linear model



(a) Steel-to-wood connection



(b) Wood-to-wood connection

Fig.7 Simulated load-slip curves of single bolted connection using Exponential model

maximum slip, while the connections of loading perpendicular to the wood grain show load increase even after splitting occurred. Since splitting is potential failure mode of multiple-bolt timber joints, the maximum slip of loading perpendicular to the wood grain therefore is determined based on the slip at first crack/splitting of wood main member. The maximum slip for other loading angles to the wood grain is set equal to the average maximum slip of direction parallel and perpendicular to the wood grain. Steel-to-wood connection is found to have less maximum joint slip than the wood-to-wood connection in both loading directions to the wood grain.

Since the load-slip curve does not display significant “yielding” at the ultimate load, it is considered to be unacceptable to estimate the maximum moment resistance by summing the ultimate moment contributed by each fastener. In this analysis, therefore, the maximum moment resistance is obtained by applying Eq. 8 for the exponential model or by Eq. 9 for the bi-linear model and assuming that the maximum fastener slip of either one or more fasteners is exceeded. In other words, the maximum moment resistance occurs when the outermost-fasteners reach their ultimate strength. After this loading stage, the strength and the stiffness of connection decrease significantly.

Experimental and predicted moment resistances for both SW and WW connections are summarized in Table 3. Table 3 shows the predicted moment resistances of exponential and bi-linear models are higher than those of experimental results except for WW connection of fastener arrangement 6V (specimen WW6V, see Fig. 4). The average ratios between moment resistance of experiment and prediction are found as 0.74 and 0.87 for the case of exponential and bi-linear models, respectively. Single-bolt connection has sufficient end margins so that the bearing-strength of wood can be fully utilized. In multiple-bolt connection, only few fasteners might reach its ultimate loads; this is known as group efficiency. Moreover, due to non-uniform load distribution among fasteners caused by lead-hole presence and random lead-hole clearance, the stress concentration exists and reduces the overall connection strength.

Wood-to-wood connections show more capacity than steel-to-wood connections significantly, almost two times, as it can be seen in Table 3. Wooden plates as side members in WW connections are more flexible than steel plates. This behavior consequently allows more load redistribution among bolts during inelastic stage. This great increase, however, is compensated by large joint rotation/deformation. Excessive joint deformation in certain timber structures can be considered as an undesirable mechanism or even failure because it may initiate damages to other non-structural or structural members. For instance, deflection of floor system of semi-rigid frames and deflection of super-structure of arch wooden bridges are essentially needed to be kept relatively small or within elastic limit to provide comfort and to ensure the wearing/surface courses integrity. Moreover, the deformation of timber joints is not only determined by the magnitude of load/actions, but also controlled some other factors such loading-duration and mechanosoprtive creep (deformation of wood member due to temperature and humidity changes). Since the final joint deformation will be influenced by the stress level during the instantaneous deformation, a design which keeps this joint deformation within elastic limits is therefore recommended.

Since the slenderness ratio of fastener (the unsupported length of fastener which is equal to the thickness of wood main member divided by diameter of fastener) is very small, 34/12.4 = 2.7, the fastener is hardly to be bent. Therefore, the connection failure is dominantly controlled by wood strength characteristics rather than the bending of fastener. Crushing of some wood fiber due to compressive loads exerted by fastener before splitting failure take places might be observed in all tested connection. Fiber crushing is caused by the wood bearing yielding, while splitting is governed by fibers slip along the grain and it occurs when fasteners in a row are closely placed.

It seems that fastener arrangement 6C would give the highest moment resistance due to the distances of all fastener to center group are equals, all fastener reach the ultimate loads at the same time, test results indicate that fastener arrangement 6V yields the highest experimental moment resistance both SW and WW connections. This is potentially caused by long distance along the grain between bolts that prevents wood splitting at low capacity. It is observed from the experiment that after crack initiation, connection of fastener arrangement 6V is still capable to carry more loads while deforming a large joint rotation. On the other hand, fastener arrangement 6H gives the lowest experimental moment resistance both in SW and WW connections.

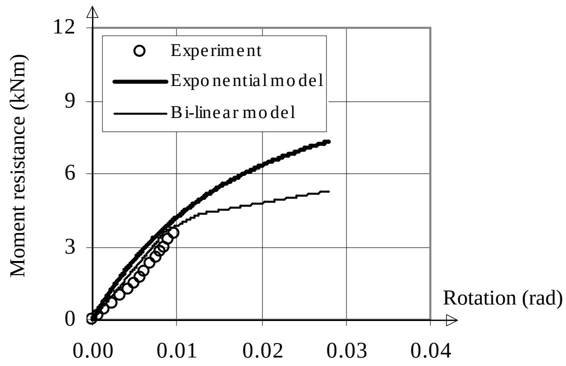
Table 4 Engineering parameters of M-θ curves

	Stiffness (kNm/rad)		Rotation (rad)		μ
	k_{re}	k_{rp}	θ_{Yield}	θ_{Max}	
SW6H	369	NA	0.009	0.009	1.0
SW6C	299	26	0.012	0.021	1.7
SW6V	419	68	0.007	0.029	4.1
WW6H	454	160	0.015	0.017	1.1
WW6C	450	122	0.015	0.025	1.7
WW6V	485	133	0.011	0.043	3.9

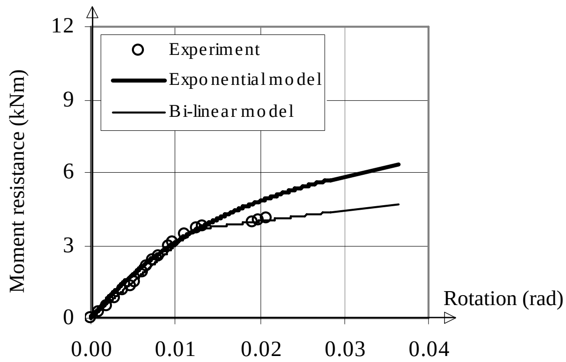
k_{re} : Elastic rotational stiffness (initial tangent)
k_{rp} : Plastic rotational stiffness (final tangent)
NA : Not available
θ_{Yield} : Rotation at yield point
θ_{Max} : Rotation at maximum moment resistance
μ : $\theta_{Max}/\theta_{Yield}$

Experimental moment-rotation curve is identified with some engineering parameters: “yield” rotation, maximum rotation, ductility, and rotational stiffness (the slope of curve) as illustrated in Fig. 9a. In this experiment, maximum joint rotation is also the same as joint rotation at maximum moment resistance since all tested connections failed after reaching the maximum capacity. Connection ductility (μ) is obtained by dividing the maximum rotation (rotation at maximum moment resistance) with “yield” rotation, rotation at intersection point between the initial tangent and the final tangent lines of the curve. Those engineering parameters of SW connections and WW connections obtained from experiment are summarized in Table 4.

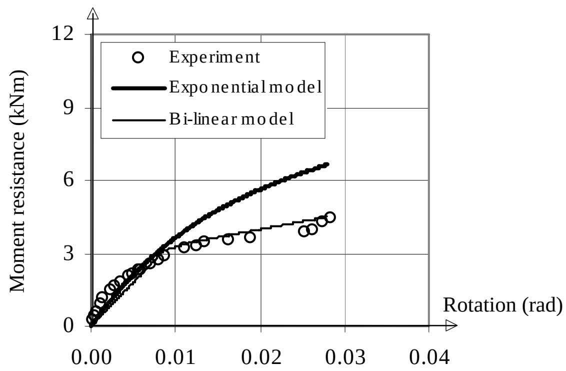
Experimental and predicted moment-rotation curves of steel-to-wood and wood-to-wood connections are presented in Figs. 8 and 9, respectively, where the joint rotation is analyzed by Eq. 12. In both steel-to-wood and wood-to-wood connections, bolt arrangement 6V has the highest rotational stiffness, the highest ductility, and the highest maximum joint rotation among three bolt arrangements. The connection ductility of bolt arrangement 6V is around four, while the ductility of other arrangements is less than two. Although ductility properties obtained under monotonic and cyclic loading tests might be different, ductile joints such as connection of bolt arrangement 6V is preferable in earthquake-prone areas to avoid the catastrophic failure and to increase energy dissipation



(a) Bolt arrangement 6H

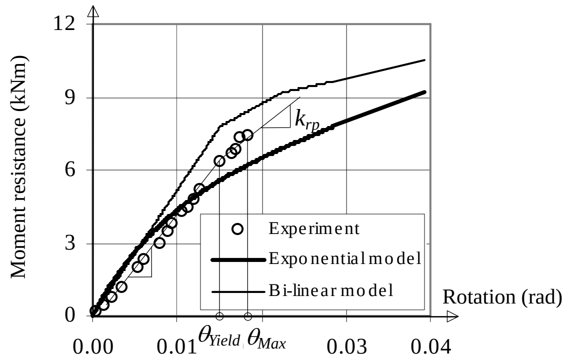


(b) Bolt arrangement 6C

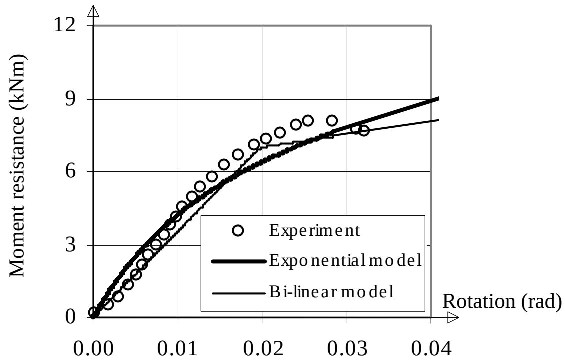


(c) Bolt arrangement 6V

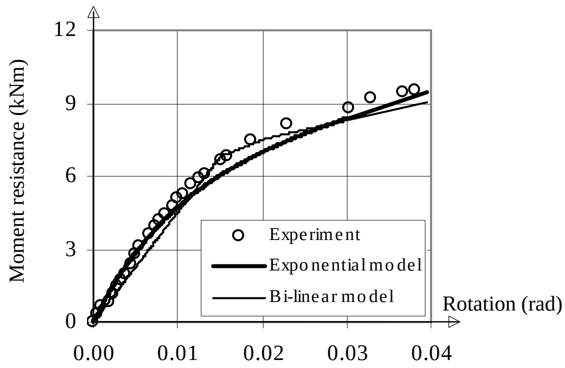
Fig.8 M-θ curves of SW connections



(a) Bolt arrangement 6H



(b) Bolt arrangement 6C



(c) Bolt arrangement 6V

Fig.9 M-θ curves of WW connections

during seismic actions. A more ductile timber joint could be obtained when bending failure of fasteners is selected as the mode of failure that can be done by enlarging the fastener's slenderness ratio. In this failure mode the amount of energy dissipation increases due to the formation of plastic hinges along the fastener length.

Brittle failure is observed in bolt configuration 6H at low joint rotation and at low moment capacity, before reaching the plastic stage. Cracks or splitting that initiate near the fastener hole can easily propagate to the closer fastener hole within fastener row. In nature, crack propagation could not be appropriately described by single-bolt connection test so that smaller joint deformation is always observed in multiple-bolt connection than joint deformation of single bolt connection. Moreover, hardwood species such as used in this study have higher tendency to fail in splitting mechanism rather than softwood species[18]. Currently, fracture mechanics has gained popularity as the most appropriate means of analysis of timber bolted connections that fail due to splitting[19 -20]. Within the same fastener arrangement, both steel-to-wood and wood-to-wood connections have almost equal ductility as informed by

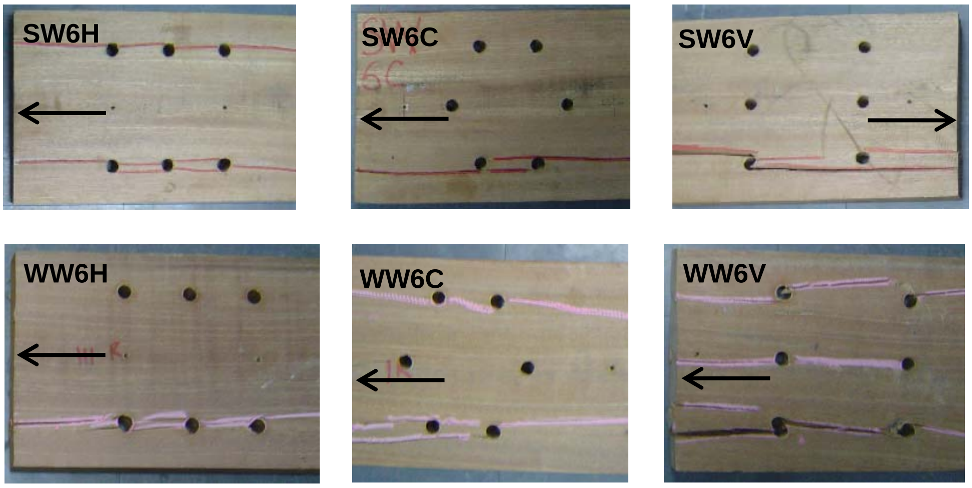


Fig.10 Splitting failure of wood main member (→: End member direction)

Table 4 because of the similar failure mode, crushing followed by splitting of wood main member as shown in Fig. 10. Wood-to-wood connections, however, have higher maximum joint rotation than steel-to-wood connections as previously stated since wooden side members in wood-to-wood connection behave with lower degree of restraining than steel plate in steel-to-wood connection.

The predicted moment-rotation curves developed based on the test results of single-bolt connection are presented in Figs. 8 and 9. It is shown that experimental curves have smaller joint rotation than predictions. Multiple-fastener connections yield less deformation/rotation than single-bolt connections. This finding is compatible with test results of multiple-fastener connections under axial load[21). Steel-to-wood and wood-to-wood connections of fastener arrangement 6V are able to deform in more ductile manner than other fastener arrangements. As a result, the experimental joint rotation of fastener arrangement 6V is large, and it almost equals the predictions.

Predicted curves show good agreement with the experimental curves for all fastener arrangements especially at small angle of rotation, elastic range. For large angle of joint rotation, however, the predicted curves do not agree very well with the experimental curves. The potential sources of this deviation are: 1) fastener slips used in the predicted curve is derived from test results of single-bolt connection which always higher than the actual fastener slips of multiple-bolt connection; and 2) in multiple-bolt connection, splitting may occur before the outermost-fasteners reach their ultimate strength as predicted by single-bolt connection test. When splitting occurs, fasteners that are located close to splitting-lines contribute less forces or even zero. Figs. 8 and 9 show the moment-rotation curves obtained based on bi-linear load-slip response of single-bolt connection have good agreement with the experimental results, just as the curves developed according to the exponential model.

5. Conclusions

Fastener arrangement is one of the most important parameters that govern the rotational stiffness and ductility of timber bolted connection. In this study, three different bolt arrangements having similar number of fasteners and overlapping-area are investigated regarding to ultimate moment resistance and ductility as well. On the basis of the results presented, it is rational to conclude that fastener arrangement 6V, which has long distance along the grain between bolts, gives the highest capacity among the three fastener arrangements considered in this study. Moreover, this fastener arrangement has the highest rotational stiffness, ductility, and ultimate rotation. On the other hand, fastener arrangement that has short distance along the grain between fasteners yields poor moment resistance and ductility.

Wood-to-wood connections show more capacity and higher ultimate rotation than steel-to-wood connections, since wooden plates as side members in wood-to-wood connection are less stiff than steel plates and allow more load redistribution among fasteners during inelastic stage. In this study, the moment capacity of wood-to-wood connections is almost two times of the moment resistance of steel-to-wood connections. Analyzed moment-rotation curves based on test results of single-bolt connection and the principle of energy conservation show good agreement with experimental results especially for small angle of joint rotation. For large joint of rotation, wood splitting that generally governs the strength characteristics of hardwood species is necessary to take into consideration for better prediction. In the case of steel-to-wood connections there is a deviation between the experimental moment-rotation curve and the curve based on the exponential model. The complexity of moment resistance formulation depends on the selected function to simulate the load-slip relationship of single-bolt connection. In spite of its simple formulation, the bi-linear load-slip relationship shows good agreement with the experimental results, just as the exponential model.

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