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Study on Utilization of LVL Sengon (Paraserianthes falcataria) for Three-Hinged Gable Frame Structures

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Abstract

This study focuses on the utilization of non-prismatic LVL members of wood species Sengon (*Paraserianthes falcataria*) for three-hinged gable frame structures. This wood species matures in 6 to 8 years, and the innovative application as LVL product for these structures is evaluated. A full-scale model of a beam-column connection is produced and tested to validate the moment-rotation response predicted by the numerical study using ABAQUS. The FEM results showed a linear-elastic moment-rotation curve response up to a joint rotation of 0.015 radians which is in very good agreement with the experiment. This validated FE model for the beam-column joint was further utilized to generate predictions for the moment-rotation relation using different bolt diameters and configurations. The last part of this study presents an evaluation of the maximum load bearing capacity of three-hinged gable frame timber structures considering a rigid and semi-rigid beam-column joint model. If the load carrying capacity is governed by the yielding of the bolt, the gable frame structure with the rigid beam-column joint overestimates the load bearing capacity by 17% to 25%.

Keywords: bolted connection, finite element model, gable frame, LVL Paraserianthes falcataria

1. Introduction

Laminated Veneer Lumber or LVL is the well-known type of structural composite lumber (SCL) which is produced by bonding layers of wood veneers under proper pressure and temperature [1, 2]. In general, from sheets of LVL different shaped products can be cut such as beams, columns, shear wall framing studs, chords of trusses, and flanges of pre-fabricated I-joists. Since LVL is manufactured with a fairly homogeneous quality with a minimum number of defects or a uniform distribution of small defects, the mechanical properties of the final product can be predicted more accurately than normal sawn timber. LVL is one of the strongest wood-based construction materials relative to its density [3] and has the great advantage of using wood resources efficiently. The production of LVL in Indonesia utilizing lower grade timber called Sengon (*Paraserianthes falcataria*) has started a couple years ago with limited technical information for structural design. Sengon, matures in 6 to 8 years and belongs to the fast-growing timber species planted widely in artificial forests as an effective means to offset green house gasses. LVL of Sengon is considered as a sustainable innovative structural material.

In timber constructions, connections play a significant role, governing both the static and dynamic performance of the overall structure [4, 5]. Modern timber buildings use various types of metal-based fasteners in their connections. These are called mechanical connections and usually have lateral loaded dowels to transfer forces between members. The bolted connection is the most common fastener type used nowadays in timber structures because it is quick, easy to install, and allows field assembly without structural elements or members surface preparation. In structural and numerical analysis, timber

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connection can be considered as a hinge in truss system, or rigid as in an idealized frame. However, to obtain a more realistic and reliable behavior, the semi-rigidity of the connection should be considered as economical results [6]. A numerical study using the finite element method showed that the degree of semi-rigid connections is important in the design phase [7, 8]. In two-hinged gable frame structures, for instance, moment redistribution could substantially increase the deformation of structures that rely on the stiffness of multi-nailed moment connections [9].

A-three-hinged gable frame structure is chosen for this study, as it is easily assembled and is typical for factory buildings or warehouses. This construction provides benefits when non-prismatic member are taken into consideration, and, therefore, could be used for longer spans more effectively than an ordinary portal with prismatic members. In this study, a full-scale connection composed of LVL timber members, steel gusset plates and bolts is produced and tested experimentally for a monotonic load to validate the moment-rotation response generated by the finite element analysis with ABAQUS. In addition, the evaluation of the degree of the semi-rigid connections using these predicted moment-rotation curves is performed thoroughly to comprehend its influence on the load bearing capacity of the selected three-hinged gable frame. The structural model of the gable frame constitutes two identical springs to facilitate the effect of the semi-rigid connection.

2. Experimental Program

A full-scale connection shown in Fig. 1 was produce and tested experimentally in this study. This connection model is actually a beam-column joint of a three-hinged gable frame structure having a span of 8 m and with a height of 5 m. The connection comprises eight bolts of 9.45 mm diameter, 3.8 mm thick gusset plates and non-prismatic LVL timber members (species *Paraserianthes falcataria*; the moisture content is 12.8%; air-dry specific gravity of 0.38). Fig. 1 shows the connection having a 15 mm gap to facilitate rotation without direct contact between the connection members and to allow resistance developed by possible friction between the timber members and the gusset plates. It is also intended to accomodate the lateral force acting on the bolts as the timber members rotate during ramp loading. After closing this gap on one side, partial contact between the two timber members will significantly affect the rotational resistance of the joint. However, it is beyond the scope of this study.

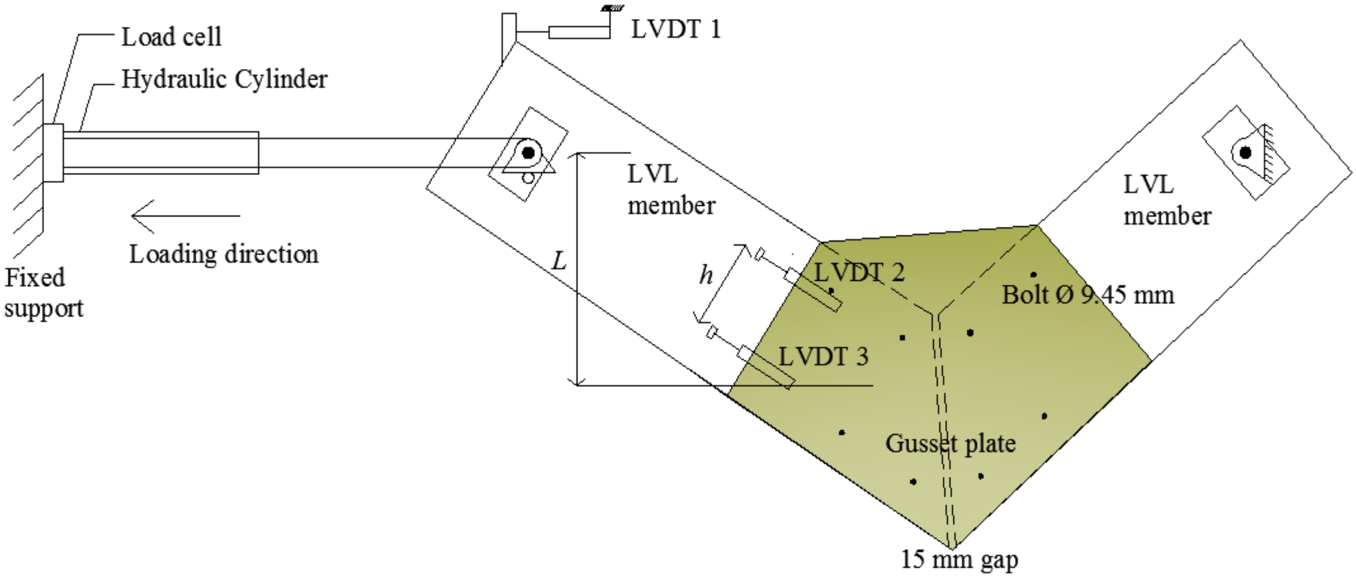


Fig. 1 Test set up of the joint model

Fig. 1. The test specimen is connected to a pinned support at the right, while on the left it is connected to a hydraulic jack equipped with load cell. The movements are restrained, except in the horizontal direction. The distance between theload application and the connection is 1590 mm, and its deviation during load application is measured using one linear variable displacement transducer (LVDT). The joint rotation, which is basically the results of both horizontal and vertical slip between

the timber member and gusset plate ignoring bending of the timer members, is measured using two LVDTs similar to previous studies, conducted by the author [10] which is a simplification of the method used by Leijten [11, 12, 13]. The joint was monotonically or ramp loaded by pulling the left arm to the left by the hydraulic jack. This results in a gap closing mode at the inside corner. Measurement of displacement and magnitude of the applied load were continuously recorded and saved. A visual observation was also performed during the test, especially concentration on the part of the timber member in the vicinity of the gap.

3. Finite Element Model

Fig. 3 shows the finite element model of the connection described in the experimental program developed using ABAQUS v.11 [14]. Both gusset plates and LVL timber members had a pre-drilled hole of 11 mm in diameter, and were discretized using eight-node solid elements. A surface-to-surface (penalty) contact model was implemented at contacts between components of the connection: timber members, gusset plates and bolts via contact pair of master and slave surface elements provided by ABAQUS. The normal and sliding contact stiffness was automatically calculated based on the material properties and frictional coefficient between the gusset plate and LVL timber member was assumed 0.1 [15].

In this finite element analysis, the end of the right arm of the connection model is fixed, while the end of the left arm is loaded uniformly . As the arms of the connection model are relatively short and stiff, the resulting bending deformation of this arm due to applied load is negligible. The rotation of the connection model is mainly controlled by the slip between gusset plates and timber members at the connection area. Here, the rotation of the connection model is defined as the summation of the displacement of nodes *a* and *b* divided by the distance between these two nodes (l). The connection moment is a result of the multiplication between the applied load and distance between end of the left arm to the connection centroid (L) as shown in Fig. 1.

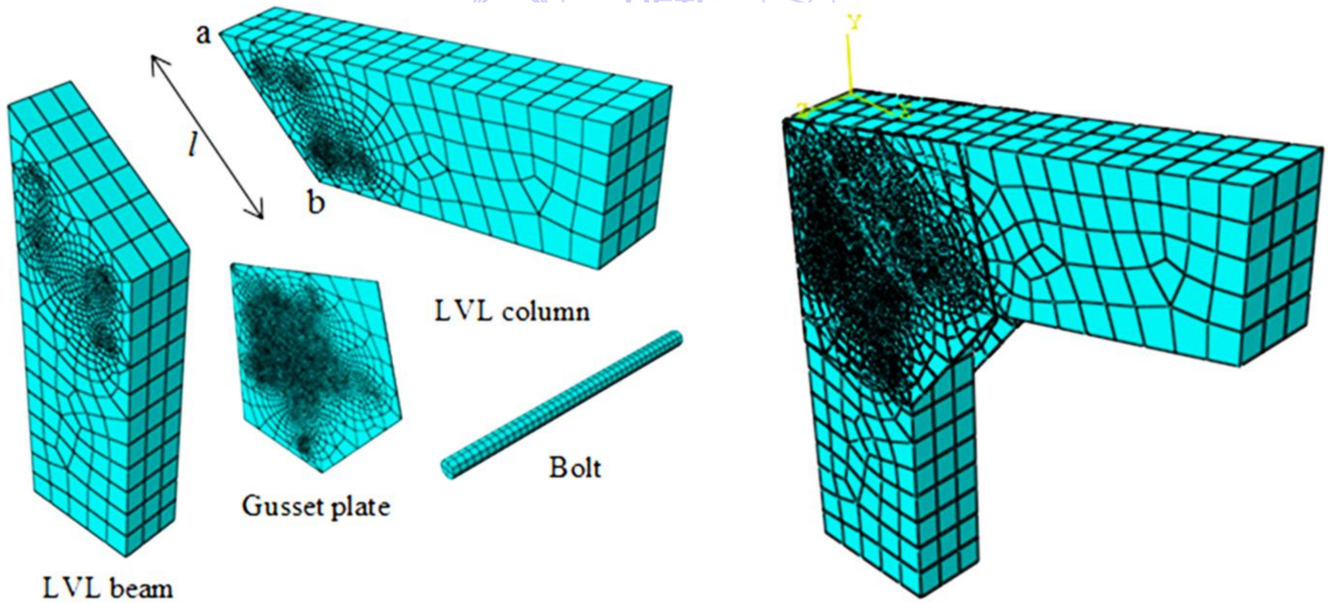


Fig. 2 A-3D finite element model of the connection system

The LVL timber member is modeled as elastic orthotropic material, engineering properties of which were summarized from previous study [16,17]. The bolt and steel gusset plates were assumed as elastoplastic material [16,18]. Therefore, 9 constants are required for the elastic orthotropic material; these constants are summarized in Table 1. E_R, E_T, E_L are the moduli of elasticity in the three material axes; G_{RT}, G_{LT}, G_{LR} are the shear moduli; v_{RT}, v_{LT}, v_{LR} are the Poisson's ratio. As an elastopastic material, the bolts and gusset plates were assumed to have the following engineering properties: a modulus of elasticity of 200 GPa, a yield strength of 400 MPa with a tensile strength of 500 MPa, and a Poisson's ratio 0.3 [15].

Table 1 Material constants of LVL *Paraserianthes falcataria* timber member

E_R	70	MPa
E_T	75	MPa
E_L	1500	MPa
G_{RT}	22	MPa
G_{LT}	49	MPa
G_{LR}	50	MPa
v_{RT}	0.05	
v_{LT}	0.30	
v_{LR}	0.38	

4. Results and Discussions

The relation between moment and rotation of the connection obtained experimentally as well as with the finite element analysis are presented both in Fig. 3. The Indonesian national design code is still based on permissable stresses and, therefore, design using the linear-elastic reponse is evaluated. In the linear-elastic region, the curve given by finite element analysis demonstrated a slightly stiffer response when compared to that of the experiment. This is considered as a result of the imperfection during joint speciment production. The moment resistance of the experiment and the finite element analysis is approximately 4.18 kNm at 0.015 radians. Fig. 3 shows that above this value, an in-elastic response is clearly noticed due to combination of plastic deformation of wood fibers beneath the bolts and in-elastic bending deformation of the bolts. The connection showed a very ductile failure as slender bolts were used (ratio of timber member divided by bolt diameter was 21). Fig. 4 shows plastic deformation of wood fibers beneath the bolts and the direction of bolts displacement which indicates joint rotation after termination of the test.

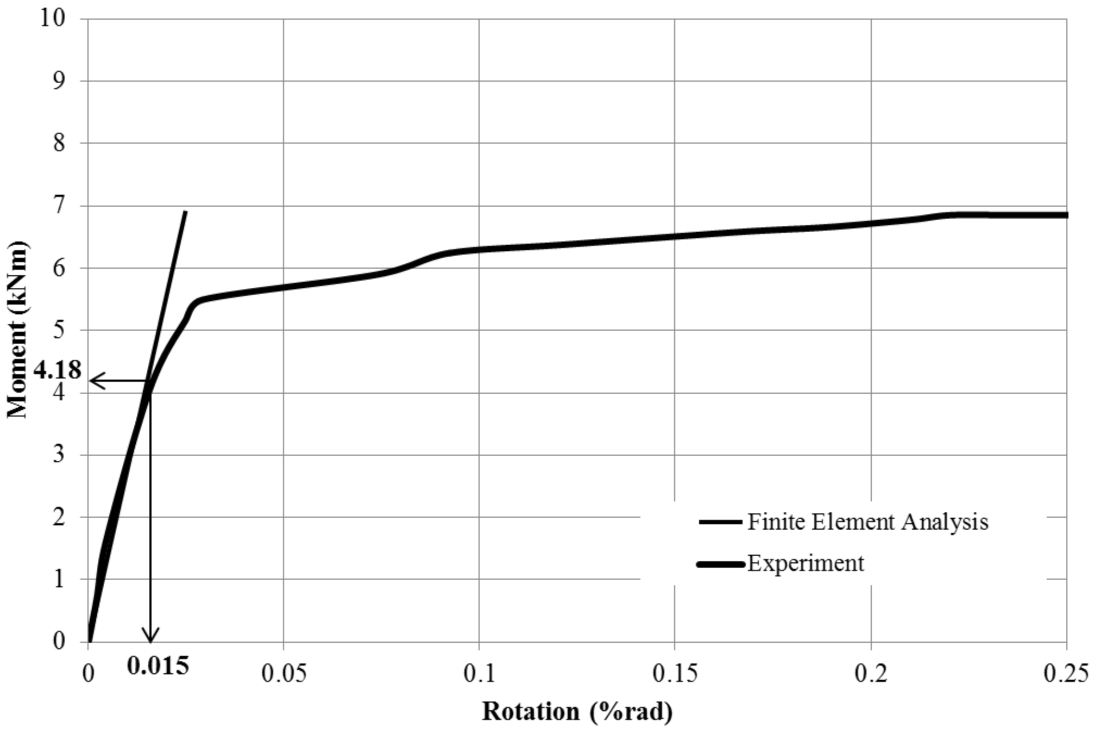


Fig. 3 Experiment and predicted moment – rotation curves (joint with 4 bolts of 9.45 mm in diameter)

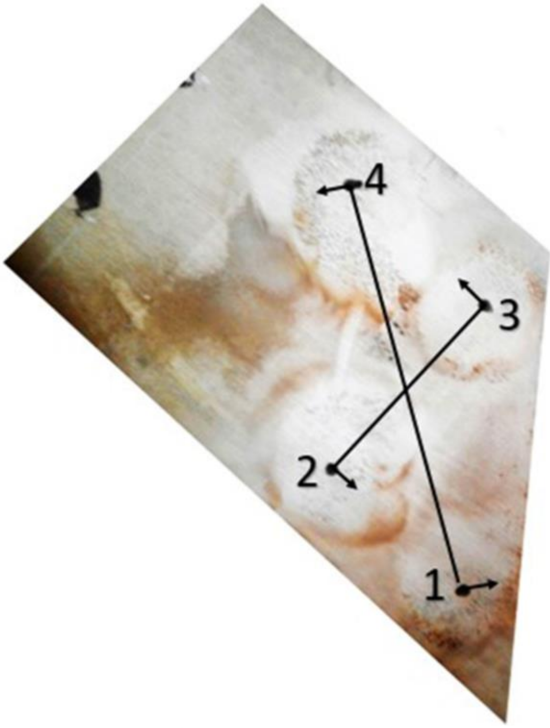
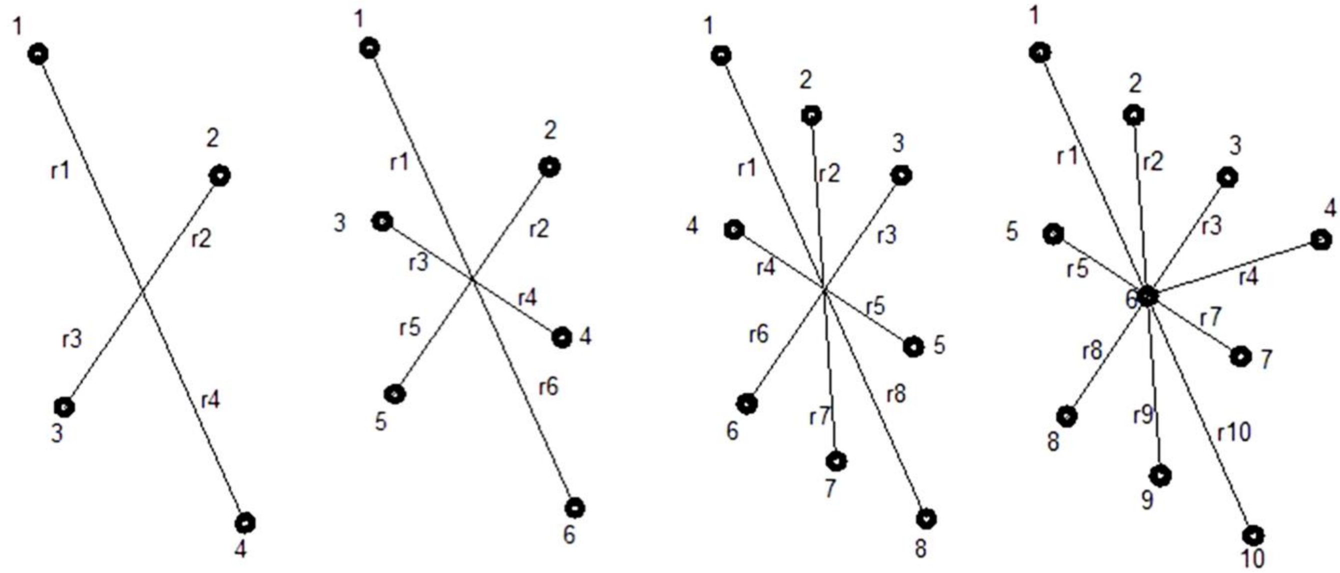


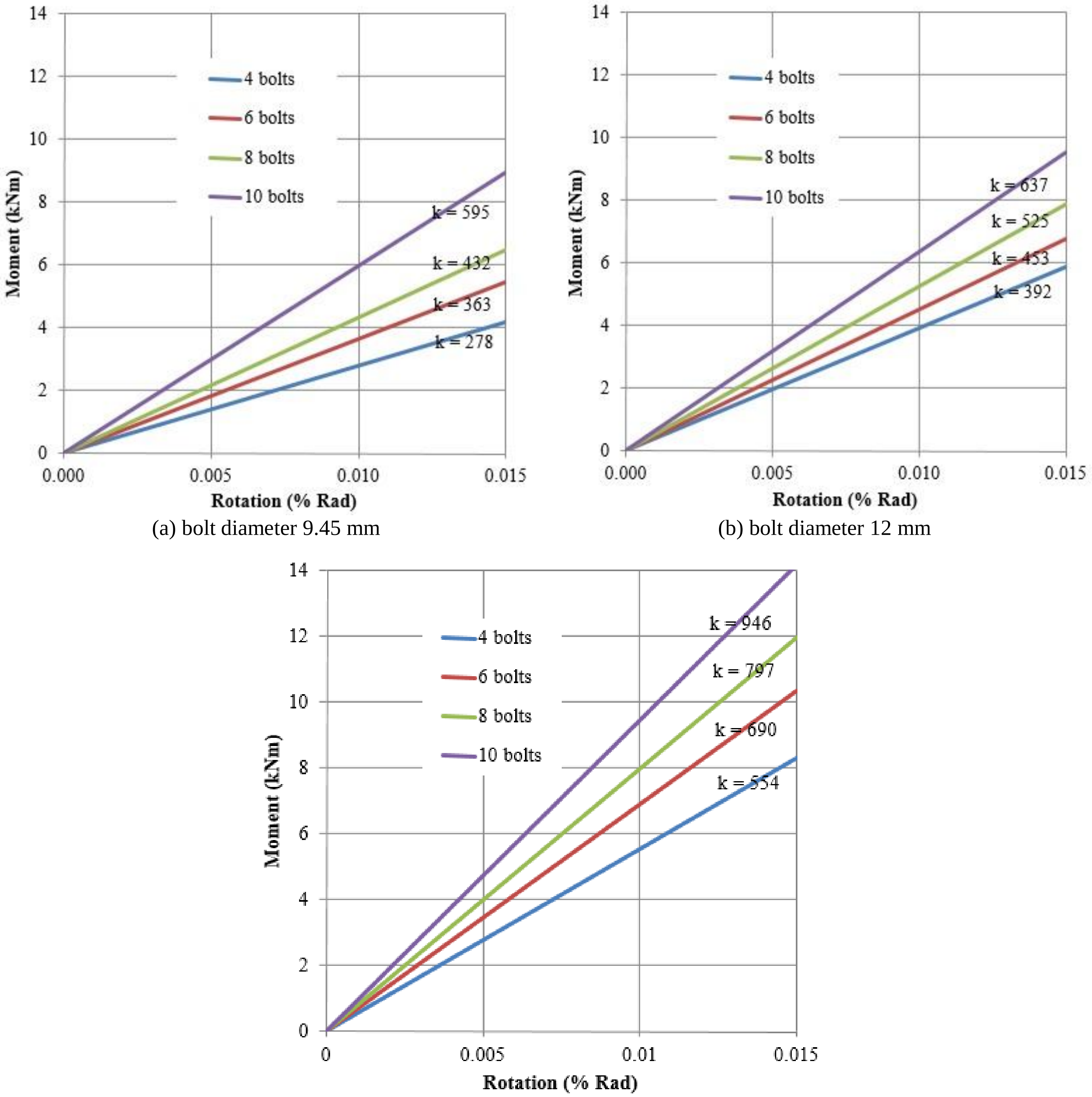
Fig. 4 Embedment deformation of LVL timber member of the bolts after termination of the test(front view of left part)

The engineering properties of the LVL timber member are presented in Table 1, which result were derived after several trials considering the general rule and relation among these properties based on previous study [16, 17, 19]. Fig. 3 shows that the linear elastic part of the moment-rotation curve of the FEM was in good agreement with the experimental curve. Rotational stiffness (k) or slope of the linear elastic part of the moment-roration curve given by the FEM is 278 kNm/%rad, while the experiment is 267 kNm/%rad; prediction overestimates the experimetal stiffness by 4%. This finite element model of the connection along with its prescribed engineering properties of the LVL timber members, gusset plates, and bolts are believed to be valid for further numerical studies concerning slight changes in bolt diameter and configuration. Similar finite element models of moment-rotation joint were developed for different kinds of bolt diameters (9.45 mm, 12 mm, and 16 mm) and for different bolt patterns as shown in Fig. 5. The predicted moment-rotation curves of these connection models are presented in Fig. 6 where a linear elastic response up to a rotation of 0.015 radians was considered. The rigidity level as indicated by the slope of the curve increases, as the number of bolts increases.



(a) 4 bolts (b) 6 bolts (c) 8 bolts (d) 10 bolts

Fig. 5 The variation in bolt configurations considered by the finite element model



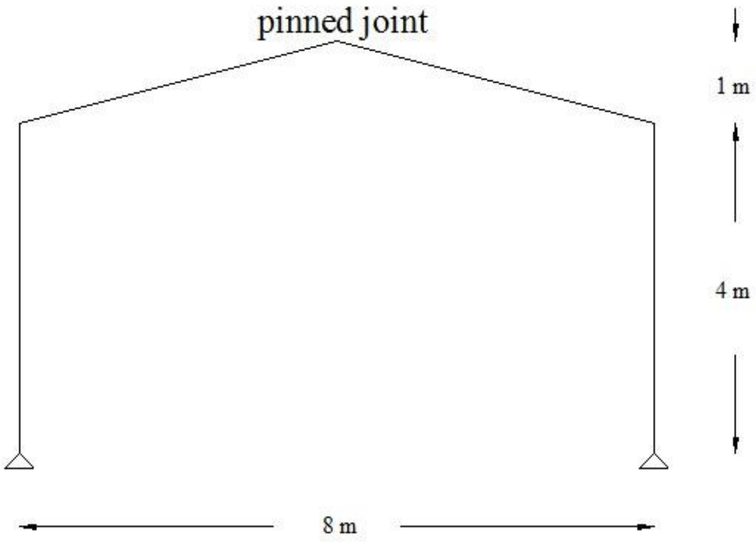
(a) bolt diameter 9.45 mm

(b) bolt diameter 12 mm

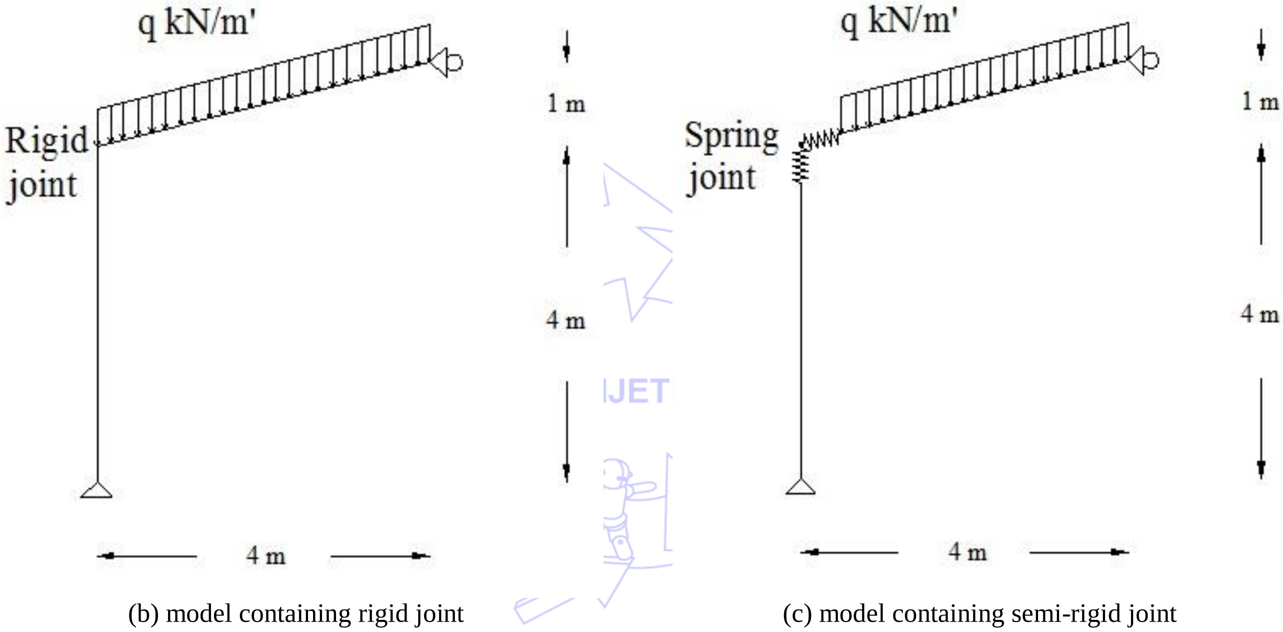
(c) bolt diameter 16 mm where k is the rotational-stiffness in [kNm/%rad]

Fig. 6 Predicted moment-rotation curves based on finite element model prediction

The remaining of this section is devoted to demonstrate the utilization of the above FEA findings for a practical case involving LVL timber members as main structural component. A three-hinged gable frame structure having a span length of 8 m and height of 5 m was considered in this analysis and idealized as shown in Fig. 7(a). The cross-section of each LVL timber member at one end is 200 mm by 200 mm, and 200 mm by 400 mm at the other end. This gable structure was simplified into two models for structural analysis purpose, as can be seen in Fig. 8 where only half of the geometry is considered due to symmetrical configuration. Fig. 7(b) is a structural analysis model with rigid beam-column connection as commonly found in most structural analysis textbooks, while Fig. 7(c) is a structural analysis model which introduced flexible beam-column joint [20] through two identical rotational springs. Knowing beam-column joint configuration, the constant of rotational springs is provided in Fig. 6. This rotational spring constant is 278.; 363.; 432.; and 595. kNm/%rad, respectively for the beam-column connections having 4, 6, 8, and 10 bolts of 9.45 mm in diameter.



(a) idealized model

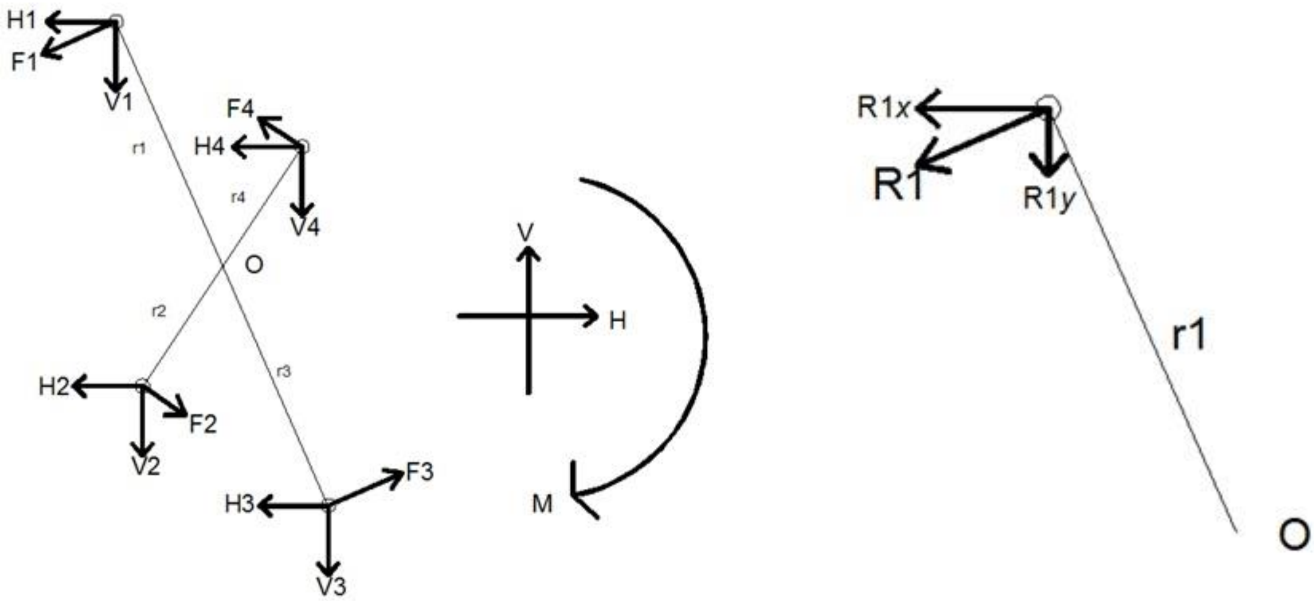


(b) model containing rigid joint (c) model containing semi-rigid joint

Fig. 7 Three-hinged gable frame structure of this study

A structural analysis was carried out for both models, i.e., the model having a rigid joint and a semi-rigid joint [21], to evaluate the maximum external load q when the highest lateral force of the bolt (R) in the beam-column joint reaches its yield load carrying capacity Z [22] is presented in Appendix 1, which is 9.57 kN in direction parallel-to-grain and 7.12 kN in direction perpendicular-to-grain. If the lateral force of bolt is neither parallel- nor perpendicular-to-grain, which is generally found in moment-resisting joints, the well known Hankinson formula [23, 24, 25] can be applied.

A calculation example of load bearing capacity of the gable frame with a rigid beam-column joint having 4 bolts 9.45 mm in diameter and external load q along the rafter (see Fig. 7. (b)) developing internal forces H, V and moment M at the vicinity of beam-column joint is presented as follows. Evaluation of internal forces acting on each bolt was performed using rigid plate assumption, where the horizontal force H and vertical force V are equally shared by all bolts. The moment M is resisted by the moment-couple proportional to the distance of each bolt to centroid of the bolt configuration, as indicated in Fig. 8 (a). Finally, the lateral force acting on bolt number i can be further simplified according to Fig. 8 (b) where the lateral force equals to R_i as the resultant force of summation of forces in horizontal axis (F_{ix} and H_i) and summation of forces in vertical axis (F_{iy} and V_i).



(a) caused by internal forces *H*, *V* and *M* (b) force resultant on bolt number 1

Fig. 8 Lateral forces acting on the bolts

The maximum load bearing capacities (q) of the gable frame structure depicted in Fig. 8 for different configurations of beam-column joints are summarized in Table 2 and Table 3, respectively for rigid connection and a semi-rigid connection model. Tables 2 and 3 clearly show that the gable frame structure with a rigid beam-column joint had a higher load bearing capacity than the gable frame with the semi-rigid beam-column connection. This is about 1.20 times (min 1.17; max 1.25) for the four differences bolt configuration described in Fig. 5. It is also well observed that in the gable frame structure with the semi-rigid beam-column joint, the maximum load bearing capacity increases as the rigidity level of the semi-rigid joint increases (for this case by increasing number of bolts). Therefore, a rigid beam-column joint assumption in structural analysis of timber structures such as the gable frame considered in this study, will overestimate the actual load bearing capacity.

Table 2 Maximum load bearing capacity of gable frame structure with rigid beam-column joint

The number of bolts	q (kN/m’)	M (kNm)	V (kN)	H (kN)	F_i (kN)	F_{ix} (kN)	R_{iy} (kN)	ΣR_x (kN)	ΣR_y (kN)	R (kN)	α(°)	Z_α (kN)
4	0.57	4.85	1.13	1.13	7.01	3.82	5.88	4.95	7.01	8.58	54.77	8.58
6	0.69	5.86	1.37	1.37	6.66	3.62	5.58	4.99	6.95	8.56	54.30	8.56
8	0.87	7.31	1.71	1.71	6.15	3.35	5.16	5.06	6.87	8.53	53.62	8.53
10	1.01	8.57	2.00	2.00	5.71	3.11	4.79	5.11	6.79	8.50	53.03	8.50

Table 3 Maximum load bearing capacity of gable frame structure with semi-rigid beam-column joint

The number of bolts	q (kN/m’)	M (kNm)	V (kN)	H (kN)	F_i (kN)	F_{ix} (kN)	R_{iy} (kN)	ΣR_x (kN)	ΣR_y (kN)	R (kN)	α(°)	Z_α (kN)
4	0.49	4.41	1.56	1.56	6.38	3.47	5.35	5.03	6.91	8.54	53.93	8.54
6	0.59	4.99	2.04	2.04	5.66	3.08	4.75	5.12	6.79	8.50	52.96	8.50
8	0.72	5.69	2.62	2.62	4.79	2.61	4.01	5.23	6.64	8.45	51.76	8.45
10	0.81	6.21	3.05	3.05	4.14	2.25	3.47	5.31	6.52	8.41	50.87	8.41

5. Conclusions

In this study, a 3-D finite element model was developed using ABAQUS to predict the linear-elastic moment-rotation behaviour of timber joints where a flexible surface-to-surface contact model was included into the model to consider contact condition among the joint components, i.e., the gusset plates, bolts, and LVL timber members. An experimental program was carried-out to validate the results of this numerical study. The evaluation showed that the numerical analysis and the experiment agreed very well in the linear-elastic region. The joint rotation is smaller 0.015 radians. The predicted curves demonstrated that joint models with a greater number or bolts diameters show higher rotational stiffness. The results of this numerical study were further utilized to evaluate the consequences of two different joint modeling techniques: rigid and semi-rigid, on the load bearing capacity of a three-hinged gable frame timber structure. It is found that the gable frame structure with the rigid beam-column joint overestimates the load bearing capacity by around 20% compared to the gable frame structure with semi-rigid beam-column joint.

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Appendix 1. European Yield Theory for Bolted Connection

Lateral resistance of steel-wood-steel bolted joint (Z) is the lowest among these two following equations:

$$Z = 0.4F_{em}Dt_m \tag{A-1}$$

$$Z = \sqrt{2M_{yb}F_{em}D} \tag{A-2}$$

where F_{em} is bearing strength of timber member (21.93 MPa for loading parallel-to-grain; 12.10 MPa for loading perpendicular-to-grain) , t_m is thickness of wood member (200 mm), D is bolt diameter (9.45 mm), M_{yb} is bending yield moment of the bolt and is equal to $F_{yb}D^3/6$, and F_{yb} is the average between yield and tensile strength (450 MPa)