Connection of timber members

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Abstract. Expanding applicability of timber structures, especially from engineered wood is a task of utmost importance due to wood’s replenishment properties as a material. This paper reviews new modification of mentioned above connections in lattice timber structures. This connection is comprised of timber members with longitudinal slot in combination with lateral round holes and steel plate with laterally welded-in rods which is inserted into the slot from the side of the timber member in direction of the holes axis. The distance between holes is equal to the distance between welded-in rods. Welded-in rods essentially work as dowels providing mandatory installation of tightening bolts laterally to the plane in which steel plate is located. The bolt holes in the plate can be larger or equal to the bolt diameter. This research focused primarily on the development of calculation methods for the described connection validated by finite element models analysis and experimental data.

Keywords: connections of timber structures, wood, dowels, finite element model analysis, test of timber connections, slotted steel plate.

1 Introduction

The role of timber structures in the reduction of building industry emission footprint is shown in several studies [1], [2], [3]. Structural steel as a recycled material also belongs to the category of sustainable materials. Timber as a structural material, especially in its revived form as engineering wood, gains more traction in the field of sustainable building structures. Each building material has its own advantages and disadvantages in terms of its applicability and set of restrictions in structural shapes it can be utilized for. One of the restrictions regarding to the timber structures is a required special design approach to connections of structural members made of it [4]. Load transfer, in most cases of joints between structural members for the structures assembled on the construction site, is provided by bolt or dowel connections [5], [9]. Widening field of applicability of timber structures and increased cross-section areas of structural members made of engineering wood require larger amount of load to be transferred through joints thus new paths and design solutions of the joints between structural members are being developed [6], [8]. The design and development stages of high load timber joints are very cost intensive tasks with the results having very narrow scope of possible applicability beyond certain structural shape or types of structures. Finite element models analysis (FEMA) validated by small scale experiments can contribute to lowering the cost of design and development stages. Therefore FEMA coupled with experiment were utilized as main tools in conducting this study as others did in studies [7], [19].

2 Methods

2.1 Object of research

The subject of this study is a steel-timber joint of composite type where load transfer is provided by rods welded in steel plate, which essentially, utilized as dowels Figure 1. Composite structure of the connection is explained by the fact that some parts of a cross-section in the connection are subjected to shear, whereas others are transferring compressing force. The functionality of this joint also depends on tightening bolts which also can optionally participate in the load transfer. This design solution of the joint can be used primarily in lattice timber structures such us trusses, spatial roof systems, and
brace members of the frames of multi-storey buildings [16]. Therefore main load was presented as tension force applied to the steel slotted plate in longitudinal direction to the grain.

In the Figure 1 a) an ending of structural member prepared with centrally located slot for accommodation of steel plate with welded-in rods is shown. Tightening bolts go through structural member in direction normal to the steel plate whereas welded-in rods are located in its plane (Figure 1 b).

**Figure 1.** Timber-steel slotted joint.
The difference between the diameter of tightening bolts and holes diameter can be seen on Figure 2 b). This was done on purpose to examine effects of welded-in rods working as dowels. Ends of timber structural members need to be prepared as shown in Figure 2 a). Novelty of this connection, in distinction from usual dowel type connections [9-11] is determined by slotted steel plate with welded-in rods splitting holes in timber member, thus known calculation methods [12], [14] cannot be directly used for assessment of its load bearing capacity. Most alike types of connections are described in [21].

2.2 Numerical studies
FEMA is a known and widely acclaimed tool for designing timber connections [13], [15], [20]. This study was conducted with the help of Ansys Mechanical. In order to assess effectively all relevant factors influencing load bearing capacity of timber connection this study focused on following components:

- Stress-strain state of timber part of connection;
- Diameter of welded-in rods;
- Amount of force transferred through welded-in rods and tightening bolts working as shear dowels respectively in combined version of connection;
- Diameter of tightening bolts working as shear dowels;
- Pretension in tightening bolts working as tightening components only;
In overall, 10 finite element models (FEM) of timber connection were explored. All analyses were executed in non-linear settings. Nonlinear analysis consisted of orthotropic and nonlinear behavior of material and nonlinear contacts. Yield strength of a timber was set at 25 MPa and yield strength of a steel plate at 350 MPa.

| Model number | Description of factors involved in each model |
|--------------|-----------------------------------------------|
| 1.           | Model includes slotted 3mm thick steel plate with welded-in rods working as dowels with diameter 10 mm. Diameter of all 4 tightening bolts is 10 mm, diameter of all 4 holes in the steel plate is 16 mm. |
| 2.           | Model includes slotted 3mm thick steel plate with welded-in rods working as dowels with diameter 10 mm. Diameter of 2 tightening bolts is 10 mm and 2 others - 16 mm, diameter of all 4 holes in the steel plate is 16 mm. |
| 3.           | Model includes slotted 3mm thick steel plate with welded-in rods working as dowels with diameter 12 mm. Diameter of all 4 tightening bolts is 10 mm, diameter of all 4 holes in the steel plate is 16 mm. |
| 4.           | Model includes slotted 3mm thick steel plate with welded-in rods working as dowels with diameter 10 mm. Diameter of all 4 tightening bolts is 10 mm; diameter of all 4 holes in the steel plate is 16 mm. 5 kN pretension is applied to each tightening bolt. |
| 5.           | Model includes 3mm thick slotted steel plate with welded-in rods working as dowels with diameter 10 mm. Diameter of all 4 tightening bolts is 12 mm, diameter of all 4 holes in the steel plate is 12 mm. |
| 6.           | Model includes slotted 3mm thick steel plate with weld-in rods working as dowels with diameter 10 mm. Diameter of 2 tightening bolts is 10 mm and 2 others 12 mm, diameter of 2 holes in the steel plate is 12 mm and 2 others – 16 mm. |
| 7.           | Model includes slotted steel plate 3 mm thick and 4 bolts with diameter 12 mm installed in holes with diameter 12 mm. |
| 8.           | Model includes slotted 3mm thick steel plate with weld-in rods working as dowels with diameter 16 mm. Diameter of all 4 tightening bolts is 10 mm, diameter of all 4 holes in the steel plate is 16 mm. |
| 9.           | Model includes slotted 3mm thick steel plate with weld-in rods working as dowels with diameter 16 mm. Diameter of 2 tightening bolts is 10 mm and 2 others –16 mm, diameter of all 4 holes in the steel plate is 16 mm. |
| 10.          | Model includes slotted 3mm thick steel plate with weld-in rods working as dowels with diameter 10 mm. Diameter of all 4 tightening bolts is 10 mm, diameter of all 4 holes in the steel plate is 16 mm. 8 kN pretension is applied to each tightening bolt. |

Due to multiple different variables influencing load bearing capacity of the connection some priorities were assumed for them. Aside from load bearing capacity of the steel slotted plate weakened by holes main focus was directed at overall compressed bearing area of holes for tightening bolts and overall compressed bearing area of notches for welded-in rods described as $A_{rc}$ and $A_{b}$.  

2.3 Experiments

Tests were developed to validate theoretically obtained values of load bearing capacity of the studied connection. The test specimen was consisted of 2 slotted steel plates with welded-in rods and timber member with prepared ends. The length of timber piece was approximately 600 mm.
In overall 2 variations of connection were tested related to the model 8 and the model 9 described in Table 1. All timber specimens were selected to exclude imperfections as knots and fissures. Connections before assembly are shown in Figure 3.

![Figure 3](image)

**Figure 3.** Testing specimen described as Model 8 before assembly.

Process of testing of 2 variations of connection is shown in Figure 4 a) and Figure 4 b).

![Figure 4](image)

**Figure 4.** Process of testing 2 variations of studied timber connection.

3 Results and Discussion

3.1 Numerical studies

After conducting numerical simulations in Ansys Mechanical following results were obtained. The range of load bearing capacity of the connection was from 55 kN to 115 kN. Stiffness of modeled
timber joint expressed as a ratio of an applied force to displacement of steel plate, assuming deformation of steel plate negligible, was in range of 40 to 110 kN/mm. Models where bolts were utilized only as tightening elements demonstrated least stiffness and ultimate strength whereas connections with tightening bolts transferring shear forces showed the most stiffness and ultimate strength. Models with additional pre-stressing in tightening bolts have stiffness laying in between upper and lower bounds of a stiffness range.

Bearing areas $A_{rc}$ for welded-in rods and $A_{cb}$ for bolts in shear were obtained from models. The results of numerical simulations and variable parameters are shown in the Table 2.

Table 2. Results and variable parameters of FEMA in ANSYS Mechanical.

| Model number | Bearing area of weld-in rods $A_{rc}$ (cm²) | Bearing area of bolts in shear $A_{cb}$ (cm²) | Pre tension (kN) | Stiffness K (kN/m m) | Yield strength $N_y$ (kN) | Net Area of weaknesses $A_0$ (cm²) | Theoretic al yield strength (k N) |
|--------------|------------------------------------------|--------------------------------------------|----------------|----------------------|--------------------------|-------------------------------|--------------------------------|
| 1            | 34.72                                    | -                                          | -              | 42                   | 53.6                     | 10                            | 51.6                          |
| 2            | 34.72                                    | 24.1                                       | -              | 93                   | 95                       | 10                            | 88.3                          |
| 3            | 43.9                                     | -                                          | -              | 41                   | 70.35                    | 12                            | 64.7                          |
| 4            | 34.72                                    | -                                          | 20             | 45                   | 53.6                     | 10                            | 51.6                          |
| 5            | 34.72                                    | 36.15                                      | -              | 110                  | 95                       | 10.8                          | 112.0                         |
| 6            | 34.72                                    | 18.07                                      | -              | 87                   | 85.72                    | 10.8                          | 77.3                          |
| 7            | -                                        | 36.15                                      | -              | 82                   | 72.9                     | 10.8                          | 86.8                          |
| 8            | 61.4                                     | -                                          | 40             | 72.9                 | 16                       | 72.7                          |                               |
| 9            | 61.4                                     | 27.1                                       | -              | 95                   | 95                       | 16                            | 115.5                         |
| 10           | 34.72                                    | -                                          | 32             | 61.2                 | 65.2                     | 10                            | 51.6                          |

The results of FEMA are presented in the compounded graph in Figure 5. Stress-strain state of one of the models is presented on Figure 6.

Strength of a connection transferring tension force in general can be determined by several limit states:

a) rupture of wood in weakened cross-section of timber member;
b) failure of steel plate under the load;
c) failure of bearing compressed areas under the welded-in rods;
d) shear failure of wood in area between 2 welded-in rods;
e) failure of bolts in shear (for connections of combined type) due to bearing failure of wood;
f) failure of bolts in shear;
g) displacements or slip of steel plate exceeding serviceability limit state.

Strength of a timber connection is defined by minimal value of actual strength calculated for each limit state. Actual strength can be described as a function of several variable parameters:

$$[N] \leq N(A_{rc}, A_{cb}, f_{yc}, A_n, A_{np}, f_{ys})/\gamma_c$$

where $f_{yc}$ – yield strength of wood in tension for defined grade and species, $A_n$ – net cross section of a timber member, $A_{np}$ – net cross section of a steel plate, $f_{ys}$ – yield strength of steel, $\gamma_c$ – safety factor from 1.1 to 1.4.

For instance actual strength of a connection defined by failure mode c) should be determined as follows:

$$N = n*l*(d-t)*f_{ys}$$

where $n$ – number of weld-in rods per one plate, $l$ - width of a timber member, $t$-thickness of a steel plate.
Actual strength of a connection defined by failure mode d) should be determined as follows:

\[ N = 2h*b*f_{sht} \]  

\( f_{sht} \) – yield strength of wood in shear for defined grade and species.

**Figure 5.** Results of FEMA in Ansys Mechanical.

**Figure 6.** Stress-strain state of connection.

Several dependences were observed after FEMA in Ansys Mechanical. Load- displacement curves for reviewed models tended to be grouped as following:

a) models with tightening bolts transferring load as dowels complementary to welded-in rods in the steel plate (steeply inclined curves 7, 2, 6, 9, 5 Figure5);

b) models with tightening bolts working only as bounding elements of connection (slightly inclined curves 1, 3, 8 Figure5)

c) same as b) only with additional tightening pretension force from 20kN to 32 kN

For group b) increase in overall bearing area of welded-in rods \( (A_c) \) leads to higher yield strength of connection but not linearly. Stiffness of connection described as load/displacement ratio \( (kN/mm) \) doesn’t change with increased \( A_c \). For group a) increase in overall bearing area of tightening bolts transferring load as dowels \( (A_{cb}) \) has more effect on strength and stiffness than that of an \( A_c \). Also non linear load-slip dependence, like in [18] was observed.
As a result of FEMA following equations were empirically deduced to optimize accuracy of calculations for c) and e) failure modes (without pretension configuration):

\[ N \leq \frac{37}{1+28.02e^{-0.12A_{cr}}} \left( 1 - \frac{A_{ch}}{A_{cr}+A_{cb}} \right) + A_{ch}f_{ytc} \]  
\( (4) \)

Where \( f_{ytc} \) - yield strength of wood in compression for defined grade and species, 
\( A_{cb} \) – overall bearing area of tightening bolts transferring load as dowels (cm\(^2\)).
\( A_{cr} \) – overall bearing area of weld-in rods (cm\(^2\)).

3.2 Experiments

After testing of specimens with parameters as in models 8 and 9 following failure modes were observed:

- a) Bearing failure of timber in areas of contact with welded-in rods with subsequent sliding of a steel plate to the point where tightening bolts start participating in force transfer i.e. all bolt-hole gaps in steel plate were closed (Model 8);
- b) Bearing failure of timber in areas of contact with welded-in rods following by rapture failure of timber in weakened area of cross-section (Model 9) like in [17].

Observed failure modes presented in Figure 7.

![Failure mode of model 8 test specimen.](image1)

![Failure mode of model 9 specimen.](image2)

**Figure 7.** Failure modes of tested specimens.

Load-displacement curves for described specimens obtained in testing machine are shown in Figure 8. It is worth mentioning that displacements in Figure 8 were measured for movable cross-head, therefore, contained slips of a steel plate inside the grips. Also data in the graph accounts for double sided connection i.e. 2 steel plates. In order to exclude slips from measurements extensometer with one of its leg placed on steel plate and other on timber member was utilized. Load displacement curve with displacement of a steel plate is shown in Figure 9. Data for model 8 shown in Figure 9 were erroneous due to failure of connection from other side of double sided specimen.
Figure 8. Test results as load /displacement curve for 8 and 9 models.

Figure 9. Test results as load / steel plate displacement curve for 8 and 9 models.

By comparing results of FEMA with experiment data following points can be summarized:

- There is substantial difference between stiffness obtained during experiment and FEMA in Ansys Mechanical, which possibly can be explained by imperfections in the experiment specimen comparatively to a finite element model, such as uneven surface of a weld throat and gaps.
- Yield strength of connection is more in line with experiment data which confirms underlying hypothesis of choosing $A_{cc}$ and $A_{bc}$ as key parameters.

4 Conclusions

After conducting FEMA in Ansys Mechanical and experiment with timber connection following results were achieved:

- Calculation method was deduced for assessing approximate yield strength of studied type of timber connection;
- Testing methods were developed for this types of connections;
- Finite elements model with solving process in nonlinear settings were optimized for reasonable calculation time.
At the end of this study in general following conclusions can be made:

- Design solution of studied timber connection was rendered viable and can, with proper further testing, be utilized in real design projects with timber members in tension;
- Most high performing variants of timber connection are of a “combined” type, where tightening bolts transfer force as dowels complimentary to the welded-in rods;
- “combined” type of connection has yield strength 30 % higher than of a initial type, where tightening bolts only bound timber and steel plate together without transferring any force;
- Quality of manufacturing affects strength of a connection substantially, therefore relevant techniques should be developed;
- Further studies can address question of how this connection transferring compressing force.

References
[1] Padilla-Rivera Alejandro and Blanchet Pierre 2017 Carbon footprint of pre-fabricated wood buildings International symposium on sustainable design pp 88-95 DOI: 10.5151/sbds-issd-2017-015
[2] Julie LyslosKullestad, Rolf AndréBohne and Jardar Lohne High-rise Timber Buildings as a Climate Change Mitigation Measure A Comparative LCA of Structural System Alternatives Energy Procedia 96 pp 112-123
[3] Gold S, and Rubik F 2009 Consumer attitudes towards timber as a construction material and towards timber frame houses - selected findings of a representative survey among the German population Journal of Cleaner Production 17(2) pp 303-309 DOI: 10.1016/j.jclepro.2008.07.001
[4] Franke S and Magnière N 2014 Discussion of testing and evaluation methods for the embedment behavior of connections INTER Meeting 47 pp 93-102
[5] Gattesco N 1998 Strength and local deformability of wood beneath bolted connectors Journal of Structural Engineering 124(2) pp 195-202 DOI: 10.1061/(ASCE)0733-9445(1998)124:2(195)
[6] Biger J P, Bocquet J F and Racher P 2000 Testing and designing the joints for the pavilion of Utopia World Conference on Timber Engineering 4.3.3
[7] Hong J P 2007 Three-dimensional nonlinear finite element model for single and multiple dowel-type wood connections PhD thesis (University of British Columbia)
[8] Lisitskii I I and Zhadanov V I 2019 Joint connections on the basis of the pasted flat cores in truss structures Vestnik of Volga State University of Technology. Series: Materials. Structures. Technology pp 59-68
[9] Franke S and Quenneville P 2011 Bolted and dowelled connections in Radiata pine and laminated veneer lumber using the European Yield Model Australian Journal of Structural Engineering 12(1) pp13-27
[10] Jorissen A 1998 Double shear timber connections with dowel type fasteners PhD thesis (Delft University of Technology)
[11] Dorn M 2012 Investigations on the serviceability limit state of dowel-type timber connections PhD thesis (Vienna University of Technology)
[12] Sandhaas C 2012 Mechanical behaviour of timber joints with slotted-in steel plates PhD thesis (Delft University of Technology)
[13] Hochreiner G, Bader T K, Schweigler M and Eberhardsteiner J 2017 Structural behaviour and design of dowel groups Experimental and numerical identification of stress states and failure mechanisms of the surrounding timber matrix Engineering Structures 131 pp 421-437 DOI: 10.1016/j.engstruct.2016.10.043
[14] Sawata K, Kawamura H, Takanashi R, Ohashi Y and Sasaki Y 2016 Effects of arrangement of steel plates on dowel type cross laminated timber joints with two slotted-in steel plates subjected to lateral force World Conference on Timber Engineering (Austria: Vienna) pp 1556-63
[15] Gecys T, Daniunas A, Bader T K, Wagner L and Eberhardsteiner J 2015 3D finite element analysis and experimental investigations of a new type of timber beam-to-beam connection
Engineering Structures 86 pp134-145 DOI: 10.1016/j.engstruct.2014.12.037

[16] Iraola B, Cabrero J M, Gil B 2016 Pressure-overclosure law for the simulation of contact in spruce joints World Conference on Timber Engineering, At Vienna pp 2019 -27

[17] Cabrero J M and Yurrita M 2018 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 pp 895-910

[18] Hirai T 1983 Non-linear load-slip relationship of bolted wood-joints with steel side members – II – Application of the generalized theory of beam on an elastic foundation Makusu Gakkaishi 29(12) pp 839-844

[19] Resch E and Kaliske M 2012 Numerical analysis and design of double-shear dowel-type connections of wood Engineering Structures 41 pp 234–241 DOI: 10.1016/j.engstruct.2012.03.047

[20] Diaz J J, Nieto P J, Luengas A L, Dominguez F J S and Hernandez J D 2013 Non-linear numerical analysis of plywood board timber connections by DOE-FEM and full-scale experimental validation Engineering Structures 49 pp 76–90 DOI: 10.1016/j.engstruct.2012.11.003

[21] O’Loinsigh C, Oudjene M, Shotton E, Pizzi A and Fanning P 2012 Mechanical behaviour and 3D stress analysis of multi-layered wooden beams made with welded-through wood dowels. Composite Structures 94 pp 313–21 DOI: 10.1016/j.compstruct.2011.08.029