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# Numerical and Experimental Analysis of Timber Portal Frame with Semi-Rigid Knee Joints

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**Abstract.** The problems related to development of rotational deformations of a knee joints and crack propagation in rafter elements of timber portal frames were discussed in this article. The rotational displacement, which develops between the rafter and column members due to the bending deformations of the bolts in the knee joint with a simultaneous embedment into the wood, increases the global deformations of the portal frame. Additionally, to axial force and bending moment the rafter elements are heavily loaded with shear force at the sections near knee joint especially. All effects together create very complicated complex of affecting factors. In the current study the design methodology of timber portal frames has been revised with the purpose to develop a more comprehensive set of design conditions for timber portal frames with dowel type fasteners in the knee joints. It is suggested to use the Hoffman failure criterion taking into account the difference of strength in tension and compression to manage the effects of plastic yielding combined with the crack development in wood assumed it as an orthotropic material. It has been proved by case studies of timber portal frames under service loads, as well as by tests and theoretical considerations. The set of design conditions must be supplement by additional clauses comprising Hoffman failure criterion regarding timber sections heavily loaded in shear and cross grain tension. Also, it is recommended that the design capacity of dowel type fastener should be decreased when shear force transferred by the bolt acts perpendicular to the grain direction inducing tension cross grain direction. Other measure may be application of some surface strengthening method.

## 1. Introduction

Timber portal frames with mechanical fasteners in knee joints are a more appropriate solution regarding transportation and assembling conditions compared to the prefabricated glued units with finger joints at knee joint. At the same time, by replacing the rigid glued finger-joint connection with the semi-rigid mechanically fastened junction, the multiaxial stress state in sections of anisotropic material weakened by bolt holes and loaded by concentrated shear transferring forces that are induced by bending moment at the knee changes considerably. During recent years, timber portal frames with semi-rigid connections made by using mechanical fasteners (bolts) have gained more attention due to some obviously important problems arising after some years or decades under service loads. There are some characteristic zones of rafter and column elements appearing subjected to the progressive development of cracks. Cracks often develop in rafter elements at the sections heavily loaded in shear and/or tension perpendicular to the direction of the grain near the knee joint, see Figure 1. The rotational displacement between rafter and column elements connected in the knee joints develops due to the bending deformations of the bolts

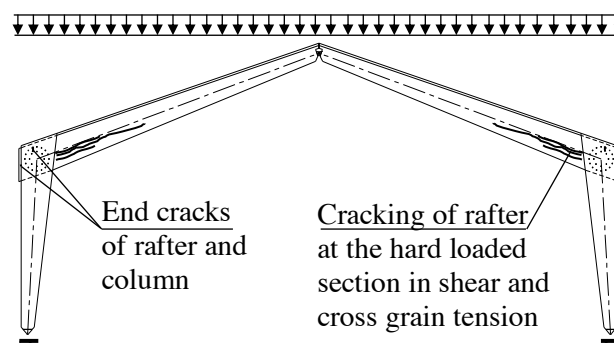


and a simultaneous embedment into the wood, as well as 1 mm of specified clearance in a hole- all together these circumstances increase the global deformations of the portal frame system. Therefore, a setup of bolted knee joints demonstrates the complicated system formed by bolts and members of anisotropic wood material into which they are embedding under variable and permanent loads during the service life. It is designed so that the resisting shear forces transferred by bolts balance the applied moment in the knee. It is considered that serious consequences may be expected regarding the semi-rigidity of the knee connection due to the non-linear behavior of dowels in bending and plastic deformations when the bolt is embedded into the wood surface. Crack development observed during the inspection of the portal frames, which have been in service for some decades, suggests that in some aspects the design conditions provided by Eurocode 5 appear to be inaccurate regarding the predictive abilities of strength capacity of anisotropic timber material and stiffness parameters for semi-rigid dowelled connections. In order to avoid the risks of splitting failure of wood, a new framework needs to be implemented for more comprehensive design conditions using developed wood fracture theories. To ensure the sustainable maintenance of the timber portal frame with the semi-rigid dowelled knee connection, the more comprehensive design concepts need to be developed. With a proper modelling of mechanical behavior of moment resisting L-type connections, further improvements on the design efficiency of timber portal frames can be obtained.

In this current study three aspects of the portal frame problem are discussed: 1) splitting capacity of wood; 2) cross grain strength of wood at the end areas beyond bolts; 3) reducing rigidity of bolted connections.

## 2. State of art

The theoretical background for portal frames with semi-rigid knee joints was developed by researchers step by step [1]. In general, structural timber design codes [2] facilitate the development of the solutions which are capable to balance the expected adverse consequences (failure or non-functionality) using acceptable simplifications for the models analysed. Such an approach is sufficiently rational for traditional structures (beams, columns, trusses) when anisotropic wood material has been loaded mainly in the direction of the grain with limited shear zones, moreover tension perpendicular to the grain is almost avoided. Structural components, such as rafter and column members of portal frames with moment-resisting semi-rigid connections in knee joints, demonstrate some tendency to crack propagation under multiaxial stress combination in an anisotropic material (Figure 1), what is not subjected to any examination by design conditions for eccentrically loaded structural members.



**Figure 1.** Design model of portal frame (worked example)

The next step is a detailed analysis of a two-dimensional finite element model considering timber as an elastic orthotropic material. For the first time a more comprehensive concept on examination of moment resisting dowel-type connections in timber structures was presented by Racher [1;4] when

preparing a background for Eurocode 5 [2]. Considering the orthotropic behavior of timber, the stiffness and bearing capacity of the joint was evaluated in relationship with the fastener slip modulus, depending on the angle between the force and direction of the grain, as well as taking into account the geometric layout of a joint. For a simplified elastic analysis, it was assumed that connected members (columns and rafters) are stiffer and stronger than the connection. Therefore, the rotational displacement of the joint results by the displacements of individual fasteners, and moment capacity may be expressed by the sum of fastener reactive moments about the rotation centre.

During the last few decades, some serious research has been performed with the purpose to find a more appropriate model for simulation of behavior of semi-rigid knee joints in timber portal frames. Leichti examined the behavior of the semi-rigid connection in a two storey orthogonal frame and expressed moments at the connections as the function of stiffness properties [3]. Bouchair with colleagues [4] developed a two dimensional finite element model of moment-resisting dowel joint, considering the non-linearity induced by embedding of a bolt into wood and modelling the fastener plastic deformations in bending by two nonlinear springs. Good agreement of numerical and experimental results was presented. It was found that the interaction between the shear and the tension stresses perpendicular to the grain is an important factor which needs to be evaluated by some criterion for the design purposes. A more comprehensive understanding of stress value variation within the multi-dowel timber connection was achieved using finite element simulations which help to reveal unexpectedly high influence of seemingly negligible moisture changes in wood to increasing of tensile stresses around weak resistance zones of anisotropic material [5].

Ormarsson and Blond presented a refined formulation for the determination of force to be transferred by an individual dowel located around a circle in relationship of a “location angle” [5;6]. A highly developed three-dimensional nonlinear finite element model for the simulation of behavior moment-resisting steel-to-timber dowel-type connection incorporating the Hill criterion of plastic yielding of timber and the Hoffmann failure criterion for the control of damage evolution under shear and tension perpendicular to grain representing good accuracy with the experimental results was recently elaborated and published by well known researchers Xu, Bouchair and Racher [7]. Using modern structural analysis software Hochreiner et al [8] studied the strain field intensity in the surrounding timber matrix disclosing the history of crack formation in wood. It is emphasized in conclusions that the extended knowledge about tensile stresses perpendicular to the direction of the grain in combination with the shear stresses may be the main factor for the evaluation of moment resisting capability of timber connection. Bader et al [9] in their study proved that the force distribution among dowels located in a circle is non-uniform and is dependent on the angle to the grain. Schweigler et al [10] proposed a new engineering model for non-linear analysis of semi-rigid joints.

Schoenmakers and Jorissen [11] presented an extensive overview of failure mechanisms and analytical analysis of parameters involved, also providing the expressions for the determination of the failure mechanism being governing. The models for both main failure mechanisms regarding dowel-type fasteners embedment failure and primary splitting of the timber depending on the loaded edge distance were constituted. A comprehensive review of fracture mechanics concepts is presented by Moura and Dourado [12]. Fracture characterisation for Mode I of an opening crack propagation and shear Modes II and III is contributed basing on theory and experiments. Only the great variety of experimental models and large test data samples for each model needed cause difficulties for development and implementation of fracture mechanics based design methods in the design codes.

### 3. Materials and methods

In this study analytical, experimental and numerical methods were combined with the purpose to disclose stress and strain components leading to unfavourable consequences during the service life of a

portal frame structure, such as crack propagation and the possible loss of system functionality due to the increase of deformations.

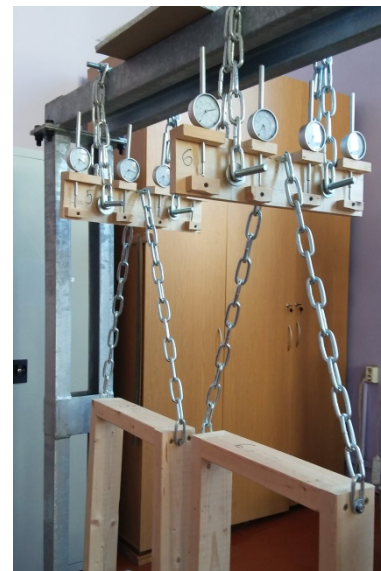
In order to move closer to theoretical design values according Eurocode 5 the portal frame structure has been developed, see Figure 1. It is a three hinged timber portal frame under a linear vertical load of 6.5 kN/m. The span of the frame is 30 m, the height at the apex point- 14.5 m and the slope of rafters- 19°. The material is glue laminated timber of strength class Gl24h specified by EN 14080:2013. The columns are designed of paired elements with maximal cross section sizes at support hinges 2x150x600 mm, tapered up to the knee joint at maximal sizes of 2x150x1600 mm. The rafter element section sizes are 200x1600 mm at the knee. The moment resisting knee joint is designed by using 34 bolts (grade 8.8) of a diameter of 24 mm. A semi-rigid reaction of the dowelled moment-resisting connection is modelled by a linear rotational spring specifying the proposed rotational stiffness in terms of moment per radian of rotation induced (kNm/rad) using Axis VM software.

The development of rotational deformations under short-term and long-term static loading was studied experimentally testing L-shape models under short-term and long-term static loads in a controlled laboratory microclimate. One day after a full load was attached, ranging from 90% of the design value, the initial elastic displacements were measured. Dial gauges of an accuracy of measurement 0.01 mm mounted on a rigid frame were used (Figure 2(a)). The measurements of vertical displacements of a free end-point, as well as the moisture content of wood were taken at three day intervals. The experimental models were made of softwood lumber (Picea Abies) classified as softwood lumber of the strength class C30, cross-section sizes 21x95mm. The moisture content of wood varied in the range of 5% to 11%, was measured using the Wood Moisture Meter MD-2G. The average mass density of the wood varied from 330 to 400 kg/m<sup>3</sup> in dry condition. The model represented connection like the one used in real portal frame. It was designed by twelve bolts M4 (metric threaded rods,  $d=4$  mm grade 4.8, DIN 976), which were placed around two circles, four bolts locating around the internal circle and eight bolts placing around the external one. Friction between timber elements was not avoided.

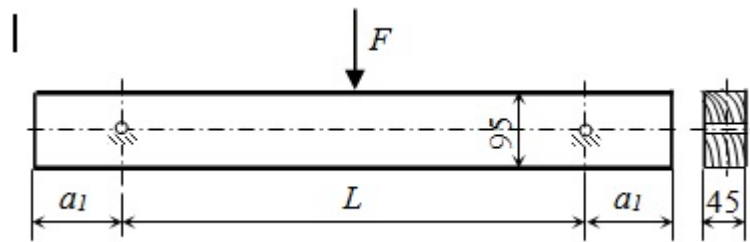
(a)



(b)



**Figure 2.** Longterm tests: (a) L-type models; (b) cross grain tension capacity test

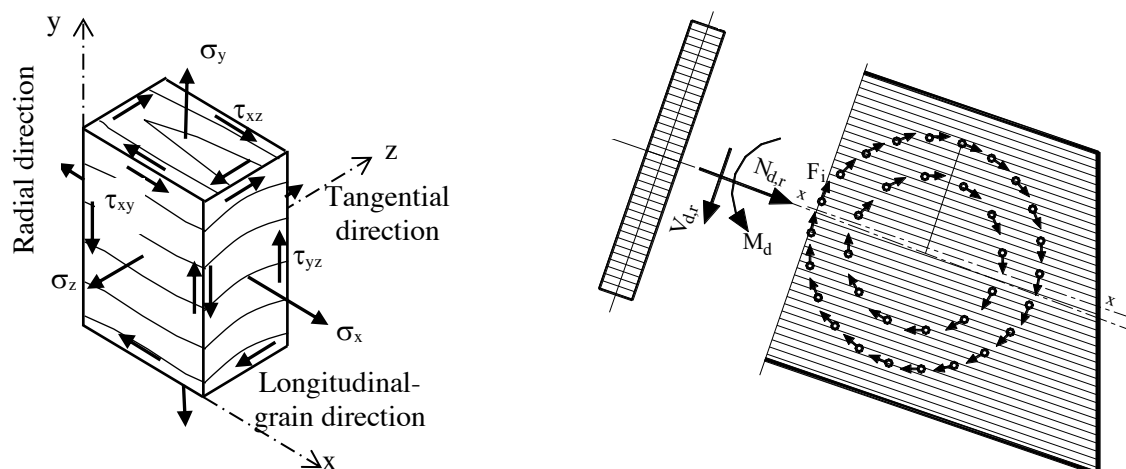


**Figure 3.** Design model of cross grain tension capacity test

Beam type test models were made for the experimental testing of the capacity of wood in cross grain tension under static load (Figure 2(b)). The main purpose of the tests performed was to examine the relationship between the force capacity in cross grain tension and the distance ( $a_1$ ) of the free end. Three samples, each consisting of ten softwood (*Picea Abies*) beam models (cross section of 95x45 mm) were tested under short-term load by a universal testing machine INSTRON up to the failure maintaining test time for approximately 100 s. The samples differed by the end distances  $a_1$  over supports ( $a_1 = 7d$ ;  $8d$ ;  $10d$ , where  $d$  is the diameter of the support bolt), as shown in the design model, see Figure 3.

#### 4. Consideration on multiaxial behaviour of timber

It has been found that a multiaxial stress-strain state in rafter sections near the knee joint normally can lead to exceeding of failure capacity followed by crack propagation during long-term service. The connection is subjected to a moment, and stress resultants, i.e. shear forces between members are transferred by an individual fasteners (bolts) located around a circle, and normally force direction complies with a tangential one. Multiaxial loading of rafter element, especially in sections near the bolt holes, see Figure 4, induces a complex stress state constituted by normal stress  $\sigma_x$  in the strong axis direction induced by a bending moment ( $M_d$ ) and an axial force ( $N_{dr}$ ), shear stresses induced by shear force ( $V_{dr}$ ), as well as normal stress  $\sigma_y$  in a weak direction due to the reaction of bolts transferring rotational shear forces  $F_i$  between rafter and column members, see Figure 4. It has been found by investigations in fracture mechanics that for fibrous materials, as wood, cross grain tension stress interaction with shear is a more unfavourable factor combination in failure.



**Figure 4.** Illustration for multiaxial loading of a rafter section near knee joint

In this study the Hoffman failure criterion [7] is applied for examination of material strength in multiaxial loading. The Hoffman failure criterion is an extension of the Tsai-Hill failure criterion, which incorporates different tensile and compressive strengths of wood, as well as takes into account the plastic yielding combined with the damage evolution making this criterion more consistent with real behavior of wood. The Hoffman failure criterion is expressed by the following equation:

$$C_1(\sigma_y - \sigma_z)^2 + C_2(\sigma_z - \sigma_x)^2 + C_3(\sigma_x - \sigma_y)^2 + C_4\sigma_x + C_5\sigma_y + C_6\sigma_z + C_7\tau_{yz}^2 + C_8\tau_{xz}^2 + C_9\tau_{xy}^2 = 1 \quad (1)$$

$$C_1 = \frac{1}{f_{t,90}f_{c,90}} - \frac{1}{2f_{t,0}f_{c,0}}; C_2 = C_3 = \frac{1}{2f_{t,0}f_{c,0}}; C_4 = \frac{1}{f_{t,0}} - \frac{1}{f_{c,0}}; C_5 = C_6 = \frac{1}{f_{t,90}} - \frac{1}{f_{c,90}}; C_7 = C_8 = C_9 = \frac{1}{f_v^2} \quad (2)$$

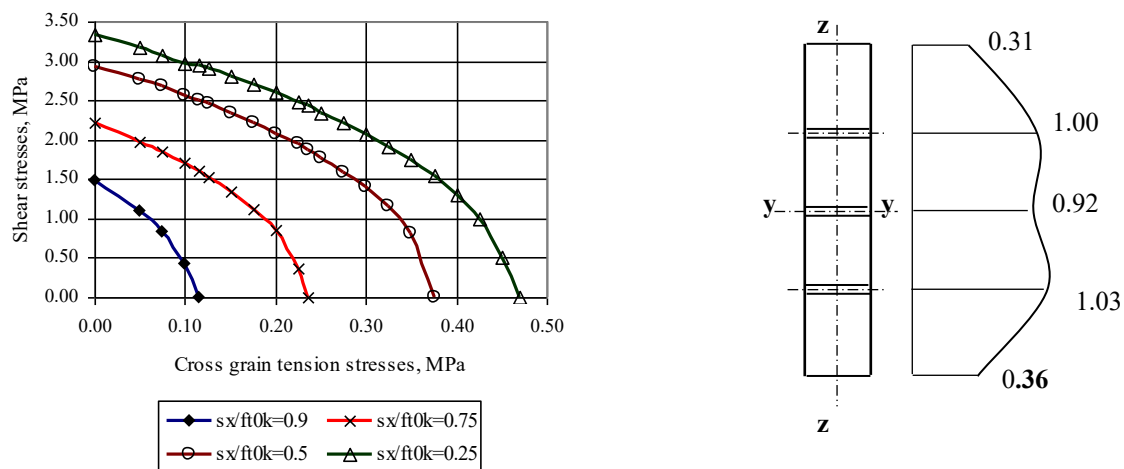
where  $f_{t,0}$  ( $f_{c,0}$ ) is tension (compressive) strength of wood in the direction of the grain,  $f_{t,90}$  ( $f_{c,90}$ ) is tension (compressive) strength of wood in the direction perpendicular to the grain,  $f_v$  is shear strength of the wood. The glulam used corresponds to the strength class Gl24h according to EN 14080, and the characteristic strength values are:  $f_{t,0} = 19.2$  MPa,  $f_{t,90} = 0.5$  MPa,  $f_{c,0} = 24$  MPa,  $f_{c,90} = 2.5$  MPa,  $f_v = 3.5$  MPa.

Using the Hoffman strength criterion the allowable cross grain tension and shear stress values combinations were found, which satisfy the equation (1) for different loading levels in the axial direction representing varying stress intensity in rafter section depending on distance between the neutral axis and the bolt hole center. The results of the numerical study show that in a combined stress state with actual shear stresses and tension perpendicular to the grain the allowable portion of stress values is quite limited. For example, if normal stresses  $\sigma_x$  in the direction of the grain utilize 75% of resistance, only 40% of the cross grain tension capacity and approximately 24% of shear capacity may be exploited simultaneously, see strength surface curves in Figure 5. It is pointed out that the examination of strength failure criteria is a significant step in the design of timber structures such as portal frames. And so, applying to the examination of rafter member section near the knee joint the unfavourable combination of stress values leading to the exceeding of failure capacity and crack initiation may be found in the quarters of the section where the full complex of stresses is relevant:

- normal stresses along the direction of the grain induced by bending moment about strong axis of a rafter section, (Figure 4):  $\sigma_{x,l} = M_{d,y} \cdot y / I_y$ ,
- normal stresses induced by bending moment about weak axis of rafter section:  $\sigma_{x,z} = M_{d,z} \cdot y / I_z$ ,
- tangential stresses induced by shear forces:  $\tau_{xz} = V_{d,z} \cdot S_y / b / I_y$ ,  $\tau_{xy} = V_{d,y} \cdot S_z / b / I_z$ ,
- tension stresses perpendicular to the direction of the grain induced by bolt reaction:  $\sigma_y = F_i / b / a$ ,
- compressive stress induced by axial force in rafter:  $\sigma_{xc} = N_{dr} / A$ ,

where  $M_d$ = bending moment in the section examined,  $V_d$ = internal shear force,  $F_i$ = reaction of bolt to the rotational movement induced by the bending moment, here assumed equal to the dowel resistance in double shear,  $y$ = distance from the neutral axis,  $I_y$ = the 2nd moment of the weakened rafter section about the strong axis (y-y),  $I_z$ = the 2nd moment of section about weak axis (z-z),  $S_y$ = the first moment of area about axis y-y,  $S_z$ = the first moment of area about axis z-z,  $b$ = width of section,  $A$ = area of section,  $F_i$ = reaction of the bolt representing the splitting force,  $a$ = size of the zone supposedly assumed resisting evenly to the unilaterally attached splitting force (applied  $a = 6d$ ). Note, that there is adopted  $a = 6d$  instead of  $7d$  predicting some wider safety margin for the end sections of timber elements tended to the formation of fissures.

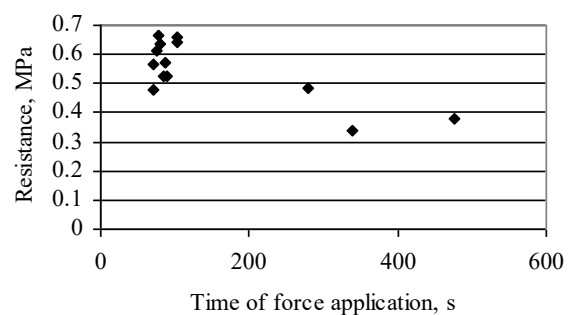
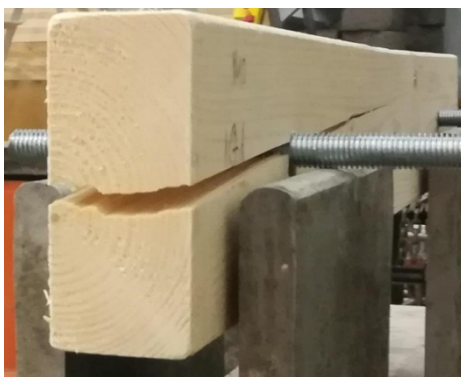




**Figure 5.** Illustration of splitting capacity space: a) Strength boundary diagrams for different loading levels in strong axis direction ( $s_x = \sigma_x$ ); b) prognosis of splitting capacity employment according to the Hoffman criteria along the depth of the rafter section

### 5. Cross grain tension capacity

Cracks forming at the ends of column and rafter elements are induced by concentrated shear forces transferred by bolts representing an opening mode (Mode I) of fracture. The purpose of the tests in cross grain tension was to examine whether some relationship between the free end length beyond the bolt and splitting capacity exists. All specimens demonstrated a brittle failure mode, see Figure 6. The density of wood and moisture content was measured for each specimen. The moisture content of wood varied from 4 to 12%. Corresponding corrections were involved in processing the test results assuming that the splitting capacity increases by 2.5% per one percent in the reduction of moisture content. The resistance of the end area in tension perpendicular to the grain was determined as a ratio of splitting force (= half of beam reaction) on the end area. The average resistance values of three samples tested are presented in Table 1. The results of the tests show that enlargement of the end distance parameter  $a_1$  does not provide any increase of force capacity in the tension perpendicular to the grain, it is exactly the opposite. The test time for some specimens was longer- up to 480 s. It was noticeable, that the tensile strength perpendicular to the grain is very sensitive to the time of the application of load. It is clear from Figure 6, the longer the test time the lower the resistance.



**Figure 6.** Cross grain tension tests: a) characteristic failure mode in cross grain tension; b) resistance in cross grain tension depending on time in short-term tests



**Table 1.** Results of short-term tests in cross grain tension.

Type of model regarding end distance $a_1$	Average resistance of area ( $N/mm^2$ )	Full capacity of end area in cross grain tension ( $N$ )	Coefficient of variation (%)	Density of dry wood ( $kg/m^3$ )
$7d$	0.75	3142	13.5	337
$8d$	0.71	3344	12.1	320
$10d$	0.59	3521	11.0	359

This affecting relationship observed activated a task to also perform long-term tests of models under a permanent design load, see the scheme in Figure 6(b). Geometric parameters of the test beams were selected so that the weakest sections are in the cross grain tension, and test load comes from a decisive strength condition perpendicular to the direction of the grain taken with the simplification of a unique stress distribution along the distance  $a_1$ :  $\sigma_{t,90} = V_d / (2b \cdot a_1) \leq f_{t,90,d}$ , where  $a_1$  = varying parameter  $7d, 8d, 9d, 10d$ . During a three month period no crack formation was observed in the wood.

## 6. Rotational stiffness of a knee connection

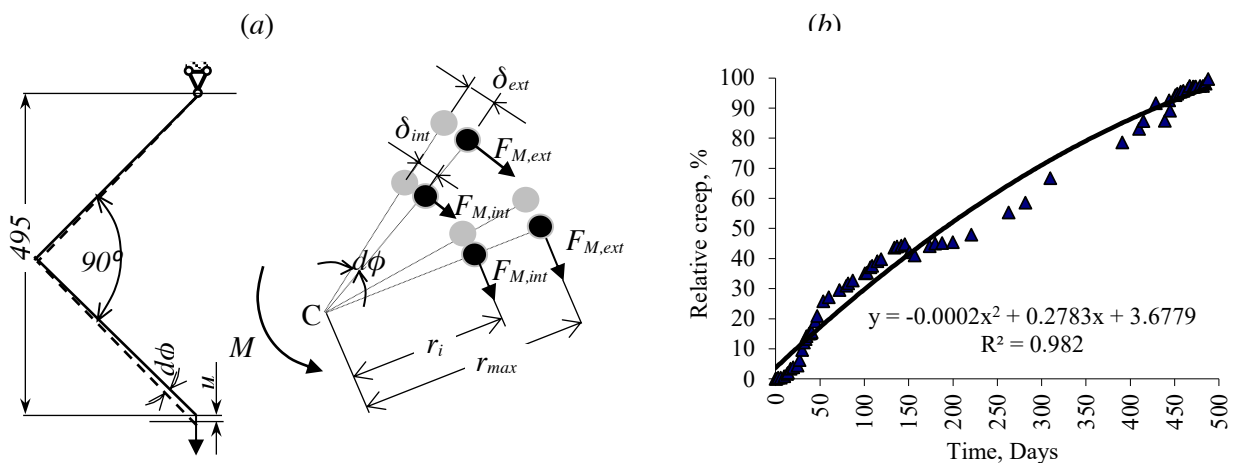
Due to both the embedment of the fasteners in wood and their deformations in bending some rotational movement develops between the members connected in a knee joint with mechanical fasteners. Rotational movement in knee joints produces a sharp increase of corner displacements in the horizontal direction, as well as a remarkable increase of vertical displacement of the apex point of the portal frame. The stiffness properties of wood are distinct from that's of steel bolts, moreover, wood is an anisotropic material. The cross-grain tension stresses induced by shear forces may cause crack development and/or wood failure. The displacement between connected members, see Figure 7, arises from a combination of movement due to the tolerance on the hole diameter and the embedment of the fastener into wood material, as well as by the bending deformation of the fastener (bolt).

To specify moment release for the semi-rigid knee connections the initial value of rotational stiffness assuming the linear elastic spring model may be judged from logical considerations. When the design load is applied and friction forces between the rafter and column surfaces are disregarded it is assumed the limit of possible mutual displacement of connected members, in the amount of 2 mm ( $\delta_{ext} = 2$  mm (Figure 7(a)) considering the value of 1 mm passing allowable clearance in the hole and one more millimeter is added as the normal increase of deformation due to bending of the bolt and embedding into the wood surface. Thus, the possible rotation of the rafter axis may be defined as the ratio  $2/r_{max}$ , where  $r_{max}$  is radius of external circle in mm ( $r_{max} = 641$ ). The rotational stiffness of spring is equal to the ratio of design moment in a knee and angle  $d\phi$  (kNm/rad).

So, in the worked example of the portal frame (see parameters in Chapter 3), the bending moment in a knee section assuming rigid connection is 480 kNm. Corresponding initial stiffness property introduced into the model data table modelling the linear elastic spring in a knee joint using Axis VM software in this case is adopted  $K_y = 480 / 0.00312 = 153846$  kNm/rad. It has been found by linear static analysis that the rotational displacement in the semi-rigid knee joint promotes an increase of global deformations of a portal frame about 1.6 times and moment increases 1.15 times compared to the traditional design model with a rigid connection. When geometric imperfections of shape are also introduced additionally, the displacement of the apex point may increase by 2–7% depending on the stiffness of the connections.

It is expected that the rotational displacement of the knee joint continues to increase during the service life since wood is known as a material subjected to creep development. Five L-shape models have been loaded by force 1.16 kN presenting a full permanent design load. The average value of angle

increment after one day when full load was applied is  $d\phi = 0.8^\circ$ , which corresponds approximately to the slip value  $\delta_{ext} = 0.6$  mm. The models were held under a load for 490 days. During this time an angle increment reached  $2.4^\circ$ , and the corresponding slip value  $\delta_{ext} = 1.5$  mm. As the time-dependent parameter the relative creep (known also as the creep coefficient) was quantified. The relative creep is expressed in terms of the initial elastic displacement ( $u_o$ ):  $c_r = 100 \times (u_t - u_o) / u_o$ , where  $u_t$  is the displacement measured at time  $t$ . Long-term test results of models prove the remarkable increase of deformations, see the graphic in Figure 7(a). The 2nd order polynomial equation demonstrates a good fitting to the test data, see Figure 7(b). Analysing mathematical properties of this function it was found that the maximal value of the displacement may be expected approximately after two years.



**Figure 7.** Illustration for rotational displacement in dowelled timber-to-timber connection: (a) geometry; (b) development of creep in time

## 7. Conclusions

It has been proved by the results of this study that the set of design conditions of heavily loaded structural timber members such as portal frame elements must be supplemented by additional clauses especially when multiaxial stress state occurs incorporating action inducing tension stresses perpendicular to the direction of the grain.

It is recommended to reduce the degree of design capacity utilization for dowel type fastener when shear force transferred by the bolt induces cross grain tension.

It is suggested the Hoffman failure criterion to use for the examination of failure capacity of heavily loaded timber elements since this criterion incorporates all important issues for wood: different resistance in tension and compression regarding to the direction of the grain, effects of plastic yielding combined with the crack development in a glulam element.

Some specifications should be developed for code applications regarding additional global deformations of the portal frame with semi-rigid moment connections in knee joints. The rotational stiffness modulus determined by long-term tests makes it possible to cope with the global deformation of a frame depending on the rotational movements in a knee. The results of L-shape model long-term tests obtained in this study prove an assumption to define the initial rotational stiffness (or spring constant)  $K_y$  as a ratio of design moment capacity to the increment of the angle in radians determined from the assumption on two millimeter slip development along the external circle of the bolt location.

It is suggested that force distributors should be used in the moment-resisting connections, such as toothed shear connectors to avoid force concentration and crack propagation.

In order to ensure the stiffness of the moment-resisting connections made with dowel type fasteners, further research is needed for the assessment of the allowable level of utilization of shear capacity.

### Acknowledgment

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