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THE IMPACT OF SEMI-RIGID JOINTS ON THE STIFFNESS OF LIGHT WOOD-FRAME STRUCTURES

This paper contains an overview and analysis of semi-rigid joints in various types of construction. The behaviour of these joints is compared with the behaviour of joints in light wood-frame structures. The results of experimental tests of joints in light wood-frame structures are presented. In the experiments, displacements were recorded to calculate the bearing of elements in the joints as well as the rotation of the loaded element relative to the supporting element in the joint. A simple numerical model describing the semi-rigid behaviour of the joint is presented.

Keywords: light wood-frame structures, semi-rigid joints, experimental tests, joint stiffness

Introduction

Although timber structures are not as popular in Poland as in North America, Scandinavia or Germany, many buildings of this type are erected there. Polish factories also manufacture a large number of precast wall, floor or roof elements for export. The diversity of structural systems, from traditional kinds to various types of light wood-frame systems, necessitates the investigation of structure behaviour.

Buildings built using lightweight wood-frame technology possess high material load capacity, but the problem is to ensure the overall stiffness of the entire building. Due to the high flexibility of wood to external loads and relaxation processes, the assurance of high spatial stiffness of the building becomes problematic. Joints are the most vulnerable points of the structure to loss of capacity and stiffness, and have thus been the subject of analyses and experimental tests [Foschi 1977; McCutecheon 1985; White and Dolan 1995]. The impact of joint stiffness on the functioning of the entire structure is seen best on the basis of deformation of elements in the joint [Salenikovich 2000; Alam and Ansell 2012].

Light wood-frame structures work in a different way than other types of structures. Joints in a light wood-frame building are less rigid than the joints in

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other structures. Timber elements are more susceptible to external static load and other forms of load (such as temperature), changes in humidity, biodegradation, and loss of load capacity over time. Incorrect formation of the joint or incorrect use of the building may result in damage of the individual structural elements or even collapse of the structure [Vanya 2012; Krentowski 2015]. The stiffness of the structure may also be affected by reinforcement of the joints with the use of steel plates, additional wood elements or adhesives [D'Amico et al. 2012; Arciszewska-Kądzior et al. 2015; Nowak et al. 2016; Rapp 2016;]. Investigation of joint behaviour makes it possible to model the functioning of the entire structure. Joints of lightweight timber structures behave differently from joints in the well-researched steel, pre-cast RC or masonry structures.

Materials and methods

Behaviour of joints in precast RC structures

In the case of precast RC structures the stiffness of the junction is a function of the stiffness of the individual connected elements. It is provided by tensile steel reinforcement bars and the working of the concrete under a range of compressive stresses.

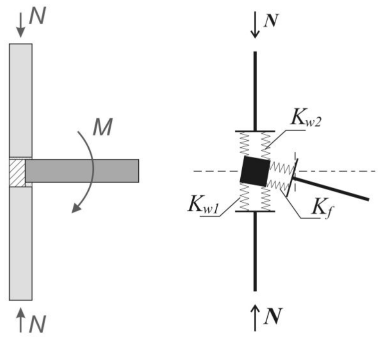


Fig. 1. Wall-to-floor connection and static diagram of precast reinforced concrete elements [Lewicki et al. 1979]

The stiffness of the junction in precast RC structures is usually lower than the stiffness of its vertical and horizontal elements. The stiffness of the junction is calculated on the basis of experimental tests, and may be described by a formula known from structural mechanics:

$$K_{test} = \frac{M}{\varphi} \tag{1}$$

where:
M is the bending moment acting on the horizontal element in the junction,
φ is the angle of rotation of the horizontal element under the moment M.

Depending on the stiffness of the junction, the M-φ relationship is described by a more or less non-linear function. In the case of rigid joints the rotation angle increases insignificantly even at high values of bending moment, while for semi-rigid joints even a small increase in the external loads causes a significant increase of rotation angle, as shown in figure 2.

Assuming that two wall elements and one floor element are connected in the junction (fig. 1), the stiffness of the individual components and the experimentally determined connection stiffness satisfy the equation [Lewicki et al. 1979]:

$$\frac{1}{K_{test}}=\frac{1}{K_{w1}+K_{w2}}+\frac{1}{K_f} \qquad (2)$$

and the stiffness of the wall element is calculated as [Lewicki et al. 1979]:

$$K_{w1}(K_{w2})=\frac{E_{p,\max} I_0}{0.5\, h_z}\zeta_{pl} \qquad (3)$$

where:

K_{test} is the experimentally determined stiffness of the joint,

K_f is the stiffness of the floor element,

K_{w1}, K_{w2} are the stiffness of the bottom and top-storey wall respectively,

$E_{p,\max}$ is the average modulus of elasticity of the band of the junction with maximum stiffness,

I_0 is the second moment of inertia of the examined band of the junction,

h_z is the thickness of the junction,

ζ_{pl} is a factor expressing the plasticity of the joint material under a load close to destructive load.

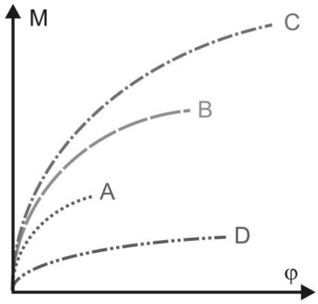


Fig. 2 The M-*φ* relationship for rigid joints (A, B, C) and semi-rigid joints (D) [Lewicki et al. 1979]

Behaviour of joints in steel structures

In steel structures, as for precast RC structures, the stiffness of the joint is a function of the *M-φ* relationship (fig. 3). The shape of the *M-φ* curve is conditioned by the way in which the horizontal elements (beams) and vertical elements (columns) are connected in the joint.

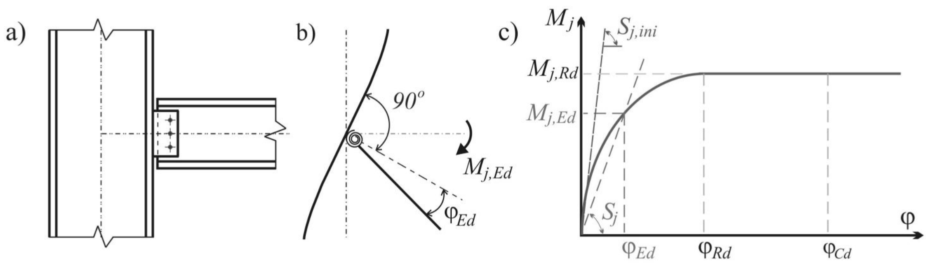


Fig. 3. Semi-rigid joint in a steel structure according to Eurocode 3: a) construction of the joint, b) deformation of the joint, c) moment–distribution (*M-φ*) relationship

The type of joint construction (butt connection, overlapping, overlaying, ribs), the means of connection (fasteners, welded) and the number of individual components in the analysed joint influence the behaviour and deformation of the joint [PN-EN 1993-1-8:2006/NA:2011; Bródka and Broniewicz 2013]. Depending on the stiffness of the element, the connection with the remainder of the joint may be treated as nominally pinned, rigid or semi-rigid [PN-EN 1993--1-8:2006/NA:2011].

The connection is treated as rigid when the initial stiffness satisfies:

$$S_{j,ini} \geqslant \frac{k_b E I_b}{L_b} \tag{4}$$

and as pinned when:

$$S_{k,ini} \leqslant \frac{0.5 E I_b}{L_b} \tag{5}$$

where:

E is the modulus of elasticity of the steel,

I_b is the second moment of inertia of the beam element,

L_b is the span of the beam (between the axes of the studs),

k_b is a coefficient equal to 8 or 25 according to section 5.2.2.5 of the standard [PN-EN 1993-1-8:2006/NA:2011].

If neither of the conditions is met, the connection is treated as semi-rigid.

Behaviour of joints in masonry structures

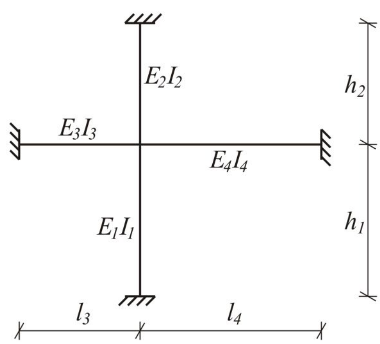


Fig. 4. Simplified frame diagram to calculate joint moment according to Eurocode 6

Masonry structures are another type of structure in which connections may be treated as semi-rigid. A former, now deprecated standard [PN-B-03002:2007] provided two models of calculation of the connection, one of which was the continuous model. In this model, the bending moment needed to compute the eccentricity of the load was reduced due to the semi-rigid behaviour of the joint. The reduction factor was taken as 0.85.

In the current Eurocode 6 standard [PN EN 1996-1-1+A1:2013-05/NA:2014-10] the connection is again assumed to exhibit semi--rigid behaviour. This means that the moments in the joint are smaller than those calculated for a scheme with rigid nodes. The value of the moment in the joint (fig. 4) is multiplied by the factor

$$\eta = 1 - \frac{k_m}{4} \tag{6}$$

where:

$$k_m = \frac{n_3 \dfrac{E_3 I_3}{l_3} + n_4 \dfrac{E_4 I_4}{l_4}}{n_1 \dfrac{E_1 I_1}{h_1} + n_2 \dfrac{E_2 I_2}{h_2}} < 2.0 \quad (7)$$

where:

n_i is the stiffness factor of the member, taken as 4 when it is fixed at both ends, otherwise 3,

E_i is the modulus of elasticity of the member,

I_i is the second moment of inertia of the member,

h_i is the clear height of the vertical member,

l_i is the clear span of the horizontal member.

Behaviour of joints in timber structures

Timber structures behave differently from the previously described structures. Alongside the *M-φ* relationship describing the behaviour of the joint due to rotation of a component relative to the rest of the joint, the *P-Δ* relationship is also taken into account. This dependence represents the deformation along or perpendicular to the axis of the component resulting from the deformability of the wood. Deformation of the components is caused by pushing one element in another.

Deflection of a building under an external load is shown in figure 5.

Deformation of the building under a load causes a change in the position of a component relative to other components in the joint. The joints of light wood-frame structures most vulnerable to deformation are shown in figure 6.

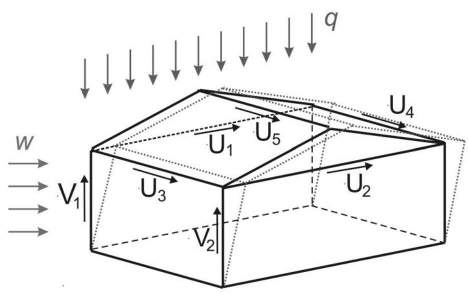


Fig. 5. Simplified deflection plan of entire structure under external loads

The entire building may be subject to a variety of static schemes due to the incorporation of objects into the structure. In the case of precast wall, floor and roof elements assembled on site, the rigidity and accuracy of the joints are decisive factors for spatial rigidity and SLS conditions.

Buildings based on light wood-frame technology are relatively flexible to external loads, particularly lateral forces caused by wind or seismic impacts [EN 1995-1-1:2010].

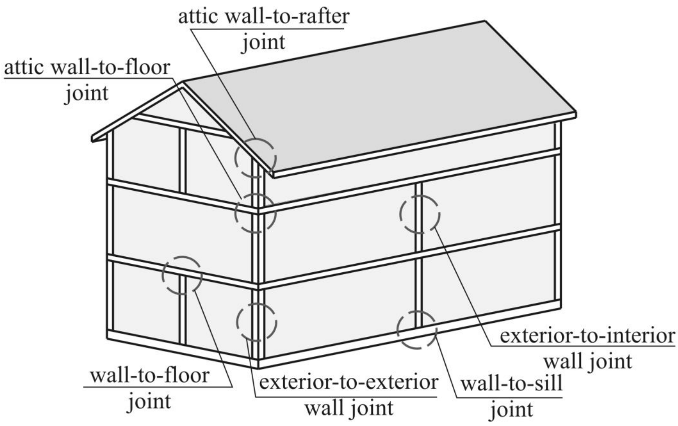


Fig. 6. The joints most vulnerable to deformation in light wood-frame buildings

Standards and rules for the design of light wood-frame structures require the division of horizontal loads between individual walls parallel to the direction of the load (fig. 7). Consideration of horizontal loads is very important in the case of light wood-frame buildings, due to their light weight.

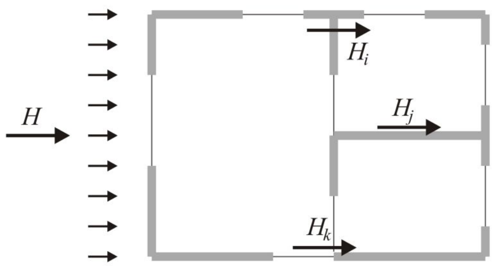


Fig. 7. Distribution of horizontal loads between walls

The load is distributed proportionally to the stiffness of individual walls, using the formula:

$$H_i = H \cdot \frac{K_i}{\sum K_i} \tag{8}$$

where:

H_i is the horizontal load acting on an individual wall element,
H is the magnitude of the resultant horizontal load,
K_i is the stiffness of the individual wall element.

It is important to compute correctly the stiffness of the wall element, taking account of the structure of the element and the composition of joints and connections.

The behaviour of the joint is influenced by its construction, the direction of external loads, changes in humidity, and the passage of time (rheological changes).

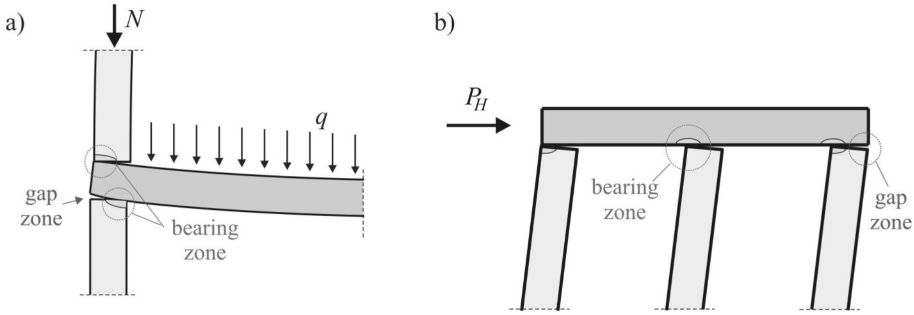


Fig. 8. Deformation of floor-to-wall joint under external load: a) vertical, b) horizontal

The cooperation of elements intersecting at the joint depends on the direction and type of external loads. In analysis of floor-to-wall joints, different schemes may apply for vertical loads (fig. 8a) and for horizontal loads (fig. 8b).

The above diagrams show (in both cases) the deformation of the joint as a sum of the deformations described by the *M-φ* and *P-Δ* relationships. The bearing zone on tangent edges of connected elements does not cover the whole surface of the designed connection. According to the rotation of the horizontal element relative to the vertical one, bearing zones appear on part of the top surface of the stud. Depending on the direction of the external load, gap zones appear between the components on the left or right side of the joint.

Experimental tests of joints in timber structures

Experimental studies have been carried out for whole buildings [Filiatrault et al. 2010], for components such as walls [Salennikovich 2000; Baszeń and Miedziałowski 2004] and floors [Kamiya 1990; Baszeń and Miedziałowski 2004], and also for single connections and joints [McCutcheon 1985; Arciszewska-Kędzior et al. 2015; Hataj et al. 2015].

Joints connecting horizontal elements (wall plates) with vertical elements (studs) were tested in a previously conducted study [Baszeń and Miedziałowski 2015]. Two types of joints connecting the bottom wall plate to the stud were made. The first was a joint with restricted vertical displacement in the axis of the applied load, representing a wall based on the foundation, while the second was a joint with permissible vertical displacement, representing a lintel. The specimen (fig. 9) was made of elements with cross-section 45 mm × 90 mm. The load was applied to the vertical element along its axis and transferred onto the horizontal beam.

The aim of the experiment was to determine the *P-Δ* relationship and calculate the translational stiffness of a joint, calculated as the tangent stiffness at a given point of the *P-Δ* curve.

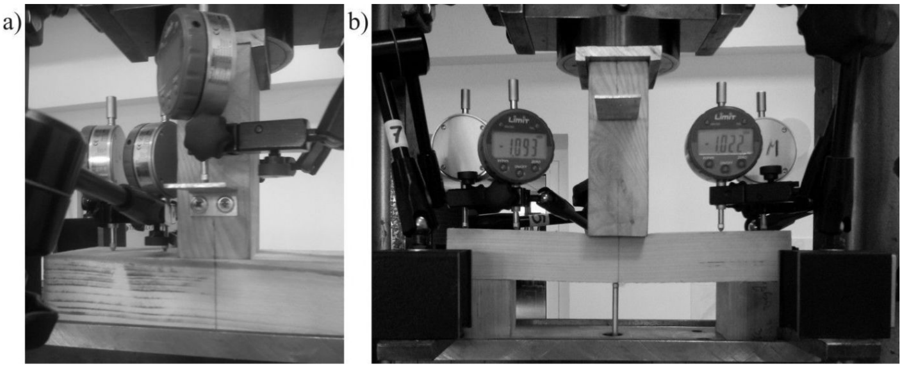


Fig. 9. Stud-to-horizontal beam joint: a) with restricted vertical displacement, b) with permissible vertical displacement

The values of the displacement *Δ* in joints with restricted displacement were determined as the pushing of the stud into the surface of the horizontal beam – the relative displacement of the tangent edges of those elements (fig. 9a). In joints with permissible vertical translation the displacement *Δ* was determined as the vertical displacement of the bottom edge of the horizontal beam in the axis of the applied load (fig. 9b).

The experiments were conducted until loss of the load capacity of the horizontal beam. In the case of a joint with restricted displacement, the loss of load capacity occurred at the moment of material plastification in the bearing surface of the horizontal beam. In the case of joints with permissible displacement, the loss of capacity resulted from cracking of the horizontal beam [Baszeń and Miedziałowski 2015].

In a subsequent series of experiments the *M-φ* relationship was determined [Baszeń 2017]. The tests were conducted on a micro-scale specimen. A beam with 22 mm × 46 mm cross-section supported by two elements with a clear span of 170 mm was used in the experiment. The horizontal beam represented the top plate of the wall, and the supporting elements represented the wall studs.

The load was applied by a timber element set in the mid-span of the beam (fig. 10).

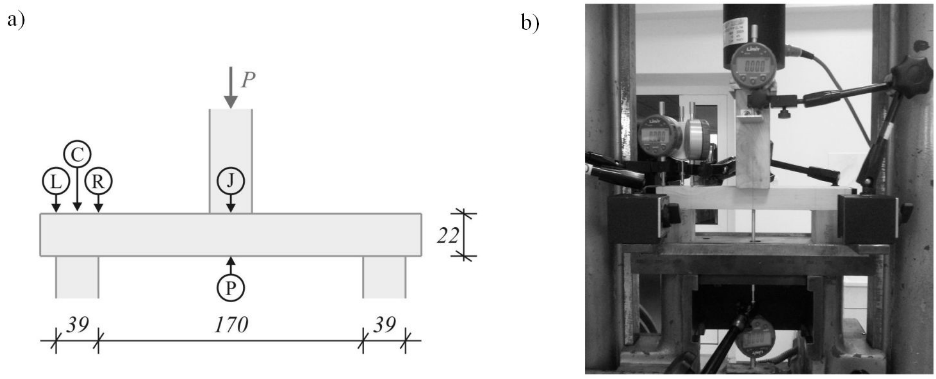


Fig. 10 Calculation of rotation of a horizontal beam over a vertical support: a) dimensions and arrangement of measurement points, b) specimen on the test stand

Five measurement points were used to record vertical displacement: three over the support and two in the mid-span of the specimen. The recorded values of displacement over the support in the surface of the stud were used to calculate the rotational angle of the joint.

In the experiments a simplified model of the joint was adopted. Only the timber framing elements were included, neglecting the presence of sheathing. Sheathing participates in the distribution of external loads onto the individual components of a structural element. Depending on the type of element, the participation of the timber framing is up to 95% in the case of wall elements and up to 85% in the case of floor elements. The greater participation of sheathing in carrying external loads in the case of floors is due to the fact that the number of sheathing-to-framing fasteners is greater than in the case of walls. More connectors provide better cooperation between individual elements of the floor.

Because the goal of the experiment was to investigate the flexibility of joints resulting from the material properties of the wood, the study neglected the influence of the fasteners on the stiffness and deformability of the joint. Only the deformation of wooden elements was observed.

Numerical analysis of joint behaviour

The results obtained from the experiments could be used as input data for the construction of a model describing the behaviour of joints. It is possible to create a calculation model enabling the accurate mapping of joint working in timber structures.

Modern computers with high computing power make it possible to solve problems even with a multitude of unknowns. Finite Element Method analysis is a good solution for the calculation problem.

The semi-rigid behaviour of the joint in the proposed numerical model (fig. 11) is reflected by modification of the global stiffness matrix of the analysed structure. The semi-rigid behaviour of the joint is represented by contact elements placed on the tangent edges of components in the joint. These contact elements will work only in the case of compression; for tension these elements will be inactive, allowing a gap zone to appear.

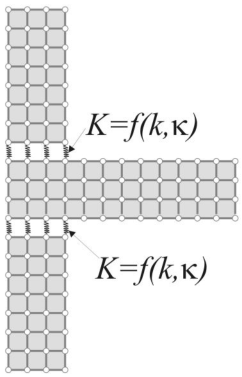


Fig. 11. Proposed numerical model (FEM)

The set of equations for the whole system is described by the known relation:

$$\boldsymbol{K} \cdot \boldsymbol{d} = \boldsymbol{P} \tag{9}$$

where the stiffness matrix is described as:

$$\boldsymbol{K} = \sum_t \sum_e \left(\boldsymbol{K}_e + \boldsymbol{K}_j\right) \tag{10}$$

and the load vector as:

$$\boldsymbol{P} = \sum_t \sum_e \left(\boldsymbol{P}_p + \boldsymbol{P}_\rho\right) \tag{11}$$

where:

$\boldsymbol{K_e}$ is the stiffness matrix of the individual flat shell element, describing the individual timber members,

$\boldsymbol{K_j}$ is the stiffness matrix of contact elements, describing the connection between individual timber members,

$\boldsymbol{P_p}$ is the static load,

$\boldsymbol{P_\rho}$ is the rheological load,

d is the vector of displacements.

Flat shell finite elements are derived by the superposition of plate finite elements with plane stress finite elements. Each node of the flat shell element has five degrees of freedom. Stiffness is absent in the direction of the rotation axis perpendicular to the shell element.

The stiffness of the individual flat shell element is calculated as:

$$\boldsymbol{K}_e = \boldsymbol{K}_e^p + \boldsymbol{K}_e^{ps} + \boldsymbol{K}_e^s \tag{12}$$

where:

$\boldsymbol{K}_e^p$ is the stiffness matrix of the plate element,

$\boldsymbol{K}_e^{ps}$ is the stiffness matrix of the plane stress element,

$\boldsymbol{K}_e^s$ is an additional stiffness serving to make the stiffness matrix invertible.

Results and discussion

Translational stiffness of joint

The experiments demonstrated the non-linear, semi-rigid behaviour of joints in light wood-framed structures.

Experimental investigations of the *P-Δ* relationship show that, irrespective of the boundary conditions, the deformations of wall plate-to-stud joints are non-linear (fig. 12).

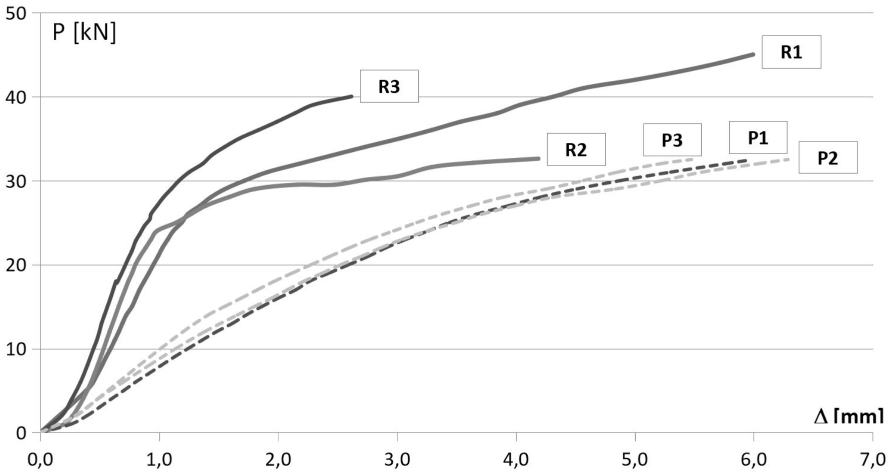


Fig. 12. The *P-Δ* relationship for joints with restricted (*R*) and permissible (*P*) vertical displacement

The results obtained from the experiments show that the translational stiffness of a joint is at a similar level for joints with restricted and permissible displacement, under higher values of load. In the case of lower values of external load the initial translational stiffness for joints with restricted displacement is significantly higher than for joints with permissible displacement.

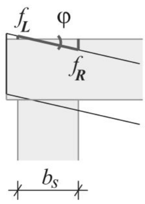
The experimentally determined maximum stiffness of a joint with restricted vertical displacement was 38 kN/mm for a compressive stress of 3 MPa, and decreased to 12.5 kN/mm for a compressive stress of approximately 7 MPa. When the compressive stress was further increased to 8-9 MPa, the translational stiffness decreased to a value in the range 3.6-6 MPa, depending on the specimen.

In joints with permissible vertical displacement the maximum translational stiffness was obtained for a compressive stress of 1.5 MPa, and was approximately 10 kN/mm. For a compressive stress of 7 MPa it decreased to approximately 3.6 kN/mm.

Rotational stiffness of joint

The rotational stiffness of the joint is described by the *M-φ* relationship. To enable use of this relationship the joint must be semi-rigid. The experiment was carried out on beams that were not fixed, but freely supported on the vertical elements. The external load causes stresses in the supporting. The phenomena of bearing and friction in the support zone cause the node to function as partially fixed. Because the moment in this node is not easy to establish, the *M*-φ relationship is replaced by M_{span}-φ, where the rotational angle is related to the maximum span moment. Another possibility is to relate the angle φ to the value of the external load and determine the *P*-φ relationship.

The method of calculation of the rotational angle is presented in figure 13, and the relationship between the maximum span moment and the rotational angle of the supported joint is shown in figure 14.



$$\varphi = \frac{f_R - f_L}{b_s}$$

Fig. 13. Calculation of rotation angle of beam over support element

The results obtained from the experiments show the visible non-linearity of the *M*-φ relationship curves, especially in case of higher values of external load. The average initial rotational stiffness of the joint was 17.6 kNm for 2.9 MPa bending stress calculated at the mid-span of the horizontal beam. The rotational stiffness decreased to 4.4 kNm for 9.5 MPa bending stress.

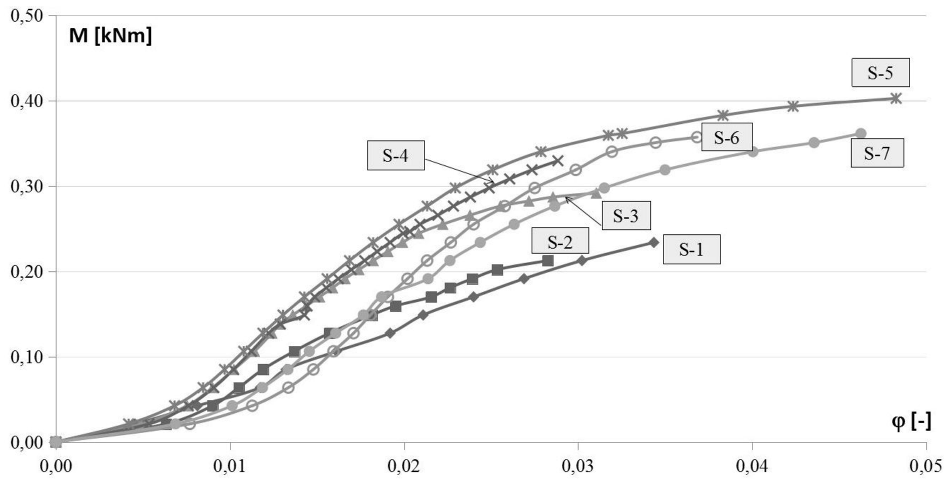


Fig. 14. The M-φ relationship for the analysed joint

Numerical analyses

Numerical analyses were performed to observe the behaviour of the floor-to--wall joint. In the analyses a model of the joint with real dimensions was implemented. The problem was described by finite elements in a 2D layout. As in the experimental tests, only the wooden framing of the walls and floor was included, neglecting the presence of sheathing and fasteners. The floor joist and wall stud were described by 4-node shell elements, and the contact zones between the joist and studs by contact elements (fig. 15). The displacement in the plane was restricted for nodes of the bottom edge of the stud of the bottom-storey wall.

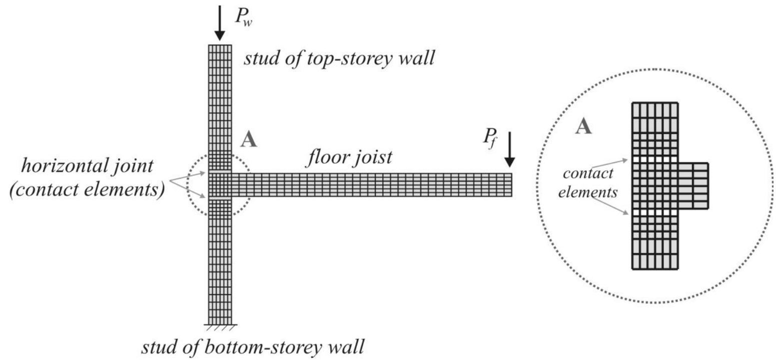


Fig. 15. Discretization of floor-to-wall joint

External loads were applied as two forces at the end of the cantilever beam representing the floor joist and in the axis of the top vertical element representing the stud of the top-storey wall. Loads were applied stepwise for the force at the end of the cantilever (P_f= 1 kN, 2 kN, 3 kN, 4 kN, 6 kN, 8 kN) with a constant value for the force over the stud (P_w = 12 kN).

Numerical computation for different loads allows one to obtain the rotational angle of the joist relative to the vertical elements. Joist stiffness was defined as the ratio of bending moment to the calculated rotational angle. The relationship between external load and the stiffness of the horizontal element is shown in figure 16.

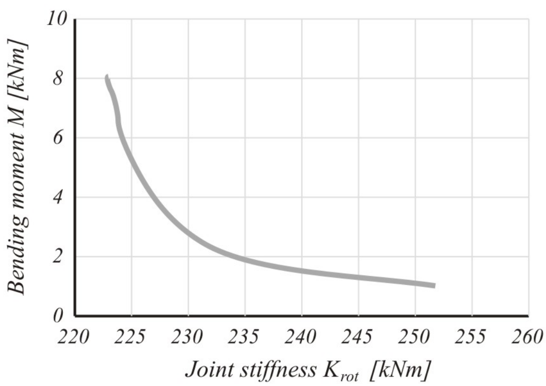


Fig. 16 Relationship between stiffness and bending moment

Conclusions

The semi-rigid behaviour of joints in a light wood-frame structure clearly affects the behaviour of the entire structure. The overall spatial stiffness of the building is determined by the stiffness of individual joints. An increase in the bending or compressive stresses in structural components of light wood-frame structures reduces the joint stiffness, and consequently the stiffness of the entire structure also decreases.

Experiments have shown that:

- joints of light wood-frame structures work differently from those of other kinds of structures;
- the behaviour of joints of a timber structure is more complex than in the case of steel, precast RC or masonry structures;
- the stiffness of a joint in a timber structure is a function of translational and rotational stiffness;
- translational and rotational stiffness decrease with an increase in the stresses in the structural components of the structure;
- non-linear behaviour is more visible in the case of joints with restricted displacement in the load direction;
- knowledge of joint stiffness enables correct distribution of the external horizontal load to the individual wall elements.

Numerical analysis may be used to determine how the joint will behave and deform, by constructing a numerical model using the experimentally determined stiffness of the joint.

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