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Research paper

Capacity of connections on punched metal plate fasteners made of spruce wood reinforced with perforated plates and screws

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Abstract: In Poland, until 2010, a number of buildings were built with the support structure of roofs in the form of timber trusses connections with punched metal plate fasteners. Standardisation changes, i.e. the introduction of Eurocodes in 2010, resulted in, among other things, increasing the values of recommended climatic loads. Due to the above, in structures built several years ago, the load capacity limit for both timber and fasteners is exceeded. Modernisation of structures with these trusses usually leads to the need to strengthen existing nodes. The issue of strengthening connections with punched metal plate fasteners is poorly recognised in experimental (failure) tests, and thus there are no detailed standard guidelines. The purpose of the experimental tests was to identify the behaviour of tension connections on metal plate fasteners and to evaluate the impact of the additional reinforcement in the form of perforated plates and screws. The analysis of the results obtained from the experimental tests allows us to conclude that the reinforced nodes achieved a much higher ultimate load than the one resulting from the design calculations.

Keywords: failure testing, timber structures, design of connections, roof trusses, reinforcement of connections

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1. Introduction

In Poland, until 2010, a number of buildings were built with roof support structures of roofs in the form of timber trusses connections with punched metal plate fasteners. Timber roof trusses connected with punched metal plate fasteners constitute a wide range of construction solutions for roofing larger buildings with more complex structures. [1,2]. The truss production process allows a very economical use of material resources while reducing implementation time, replacing heavy concrete ceilings with light, ready-made wooden structures (Fig. 1), as well as better organisation of usable space [3,4].

Fig. 1. Prefabricated timber trusses with parallel chords prepared for transport

The reliability of timber structures connected with punched metal plate fasteners and their correct use are influenced by many factors at various stages of the construction of the facility. When designing an object, the degree of complexity of the structure and the properties (anisotropic structure) of timber should be taken into account [5,6]. The standardisation changes that formed the basis of the calculations and design resulted in more stringent requirements for the timber structures connected to the metal plate fasteners [7,8]. Structures were often designed with small safety margins, where stress in the rods and nodes reached approximately 95%, which currently constitutes the main reason for the reinforcement of such structures. Other reasons for strengthening these types of construction include increased operational loads resulting from changes in the way the structure is used, errors made in the construction process, misuse inconsistent with the intended purpose and design assumptions, as well as irresponsible human actions. Modernisation of facilities with wooden beams connected with punched metal plate fasteners usually leads to the need to strengthen the nodes [9]. To reinforce these types of connection, various methods can be used to obtain a connection with better strength parameters. The basic way to strengthen existing nodes is to attach additional elements, most often with pin connectors. Regardless of the method of connecting the reinforced element with the reinforced element, it is necessary to recognise the distribution and distribution of stresses to estimate the load-bearing limit state deformability of the connection itself [10, 11].

A popular material for reinforcing the connections of timber structures is steel [12]. Therefore, a rational way to reinforce connections with punched metal plate fasteners is to use perforated plates attached with specialised screws used for metal-timber connections. The perforated plate is made of galvanised sheet metal with a thickness of 1.5 or 2 mm. They are easy to install. The optimal arrangement of holes limits the delamination of timber.

Perforated plates are resistant to tensile forces. To avoid eccentricities, it is recommended to use plates in pairs, connected on both sides to timber elements [7]. The issue of strengthening connections with punched metal plates is poorly understood in experimental (destructive) tests. There is no specific information on the effectiveness of the reinforcement itself or more detailed recommendations regarding the design or load-bearing capacity assessment of the reinforced connection. Only in the national annex of the PN-EN 1995-1-1 standard at point NA.2.8.4.2 [13] provides a method for determining the load-bearing capacity of connections made using connectors of similar susceptibility. According to the guidelines, when using different types of connectors, it is recommended that the load capacity of the connectors that transfers a smaller part of the force that occurs in the connection/element is taken into account with a factor of 0.65.

This article presents the results of experimental (destructive) tests that evaluated the load-bearing capacity and effectiveness of a reinforced connection for punched metal plate fasteners using perforated plates and screws. These connections were subjected to axial tension by force acting along the fibres of the connected timber elements.

2. Research objective and scope

The tests were aimed at determining the load-bearing capacity and strengthening effectiveness of first unreinforced and then reinforced punched metal plate fasteners. An additional objective of the research was to investigate the damage to the timber elements and connectors and to observe the behaviour of the above-mentioned connections. The research constitutes the basis for further experimental research related to strengthening the connections of timber elements with punched metal plate fasteners.

The test program included three groups of joints of timber elements, made of C24 class spruce wood, with a cross-section of 45×195 mm and a length of 600 mm each. The length of the test element was 1202 mm. In each group of connections, 7 trial elements were tested. A summary of all tested elements is presented in Table 1, and the diagrams and dimensions of the tested trial elements are shown in Fig. 2.

The first group (GB1) consisted of unreinforced joints, where timber elements were connected using a pair of MiTek T150 punched metal plate fasteners, each with a thickness of 1.5 mm and dimensions of 176×185 mm (Fig. 2a). In the second group (GB2), connections were made using a pair of perforated plates NP20/160/400 joined with 2×12 CSA screws, 5×35 mm each, on both sides of the test specimen (Fig. 2b). The joints of the third group (GB3) included connections from the first group reinforced with perforated plates and screws (Fig. 2c).

The test specimens of groups GB1 and GB3 were previously manufactured and supplied by a professional plant producing prefabricated timber roof trusses with punched metal plate fasteners. The connections in group GB2 and the reinforcement of group GB3 were made in a research laboratory, where the perforated plates were fastened by screwing without pre-drilling holes. The screws were placed according to the principles of mechanical connector spacing specified in Chapter 8.7 of the PN-EN 1995-1-1 standard [14]. The perforated plates and screws were supplied by a specialised plant manufacturing structural connectors for timber constructions.

Table 1. Summary of research elements

Group	Research element	Cross-section $b \times h$ [mm]	Length L [mm]	Material	Connector type	The number of samples
GB1	PK1÷PK7	45 × 195	1202	Spruce wood C24	Punched metal plate fasteners T150	7
GB2	PPW1÷PPW7	45 × 195	1202	Spruce wood C24	Perforated plates NP20/160/400	7
					Screws CSA 5x35	
GB3	PKPW1÷ PKPW7	45 × 195	1202	Spruce wood C24	Punched metal plate fasteners T150	7
					Perforated plates NP20/160/400	
					Screws CSA 5x35	

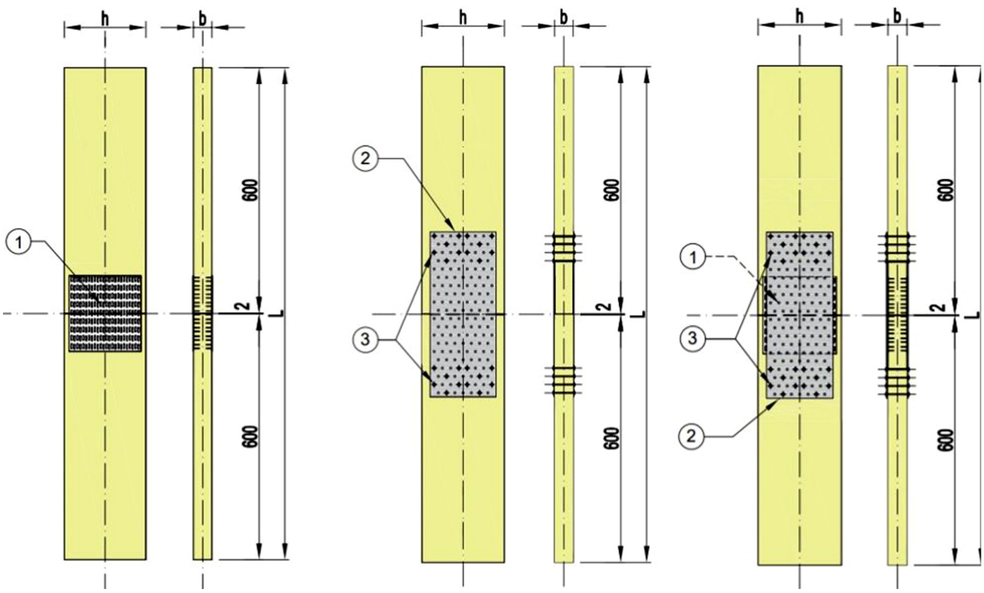


Fig. 2. Dimensions and shapes of the test specimens: (a) PK, (b) PPW, (c) PKPW; (1 – punched metal plate fasteners, 2 – perforated plate, 3 – screws: 12 pieces)

Special anchoring fixtures (Fig. 3) were custom-made to secure the test specimens in the strength-testing machine.

The anchoring fixtures were designed to accommodate test specimens of two different thicknesses, namely 45 mm and 60 mm. They were fabricated using sheets of S235 JR carbon

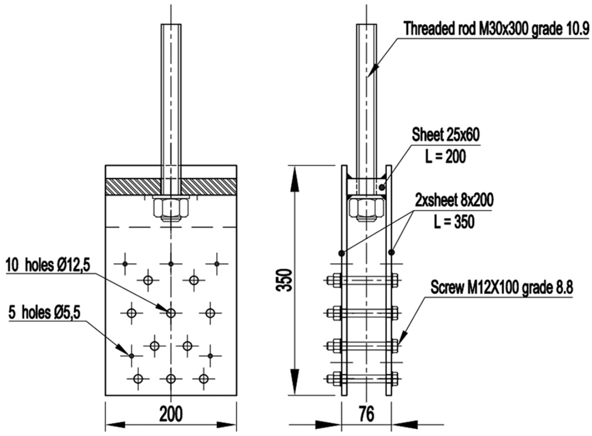


Fig. 3. Anchoring fixtures

steel. In the case of securing elements from groups GB1, GB2, and GB3 with a thickness of 45 mm, two additional plates measuring $8 \times 200 \times 230$ mm were attached. The attachment of the samples to the fixtures took place in the workshop within the structural testing laboratory.

Since screws were used to secure the specimens, holes were drilled in the wooden elements with a diameter that matched the diameter of the screw shank. No preloading of these screws was applied.

3. Experimental research

3.1. Test of wood material

The material tests aimed at determining some of the physical and mechanical properties of the wood of the tested elements. As part of these studies, the following parameters were determined:

- moisture content,
- surface temperature of the element,
- tensile strength parallel to the grain.

The moisture content of the test specimens was measured using the resistance method with a moisture meter (Laserliner MultiWet – Master Compact Plus) – Fig. 4a. Measurements were taken at multiple points for each test element. The average moisture content of the test specimens in group GB1 was $\omega_{\text{mean}} = 11\%$, in group GB2 it was $\omega_{\text{mean}} = 11.4\%$, and for the elements in group GB3, it was $\omega_{\text{mean}} = 11.7\%$.

During the tests, surface temperature of the test specimen was also recorded using a thermal detector (Bosch PTD 1) – Fig. 4b. The average surface temperature of the test specimens for group GB1 was $T_{\text{mean}} = 21.2°\text{C}$, for group GB2 it was $T_{\text{mean}} = 21.7°\text{C}$, and for the elements of group GB3, it was $T_{\text{mean}} = 22.3°\text{C}$.

(a) (b)

Fig. 4. Measurement of moisture content and surface temperature of the test specimen

The tensile strength parallel to the grain was determined using a Cometech strength test machine in the Building Materials Laboratory at Rzeszow University of Technology. The tensile strength of the wood parallel to the grain was assessed based on tests conducted on small samples in accordance with standards [15, 16]. A total of 20 samples were used for these tests. The samples were prepared and extracted following the guidelines outlined in the standard [17]. The sample was loaded uniformly at a constant rate. Failure occurred after approximately 2 minutes – Fig. 5. The static tensile test revealed that the average tensile strength of wood parallel to the grain is $f_{t,0,\mathrm{mean}} = 85.79$ MPa. The standard deviation was $s = 3.70$ MPa, and the coefficient of variation was $v = 2.25\%$.

(a) (b)

Fig. 5. Testing the tensile strength of wood parallel to the grain

The research did not include the determination of wood density. The value of wood density was adopted based on the studies according to [18], and for spruce wood, it is $\rho_k = 470$ kg/m³.

3.2. Characteristics of material connectors

The functional properties of the punched metal plate fasteners were determined based on the manufacturer's declaration of functional properties of the plates. MiTek T150 plates were made from S250GD steel, and the characteristic anchoring strength of the plate was $f_{a,0,0} = 2.61$ MPa, with a characteristic tensile strength $f_{t,0} = 251$ MPa. The plates' coating was protected against corrosion through hot-dip galvanisation.

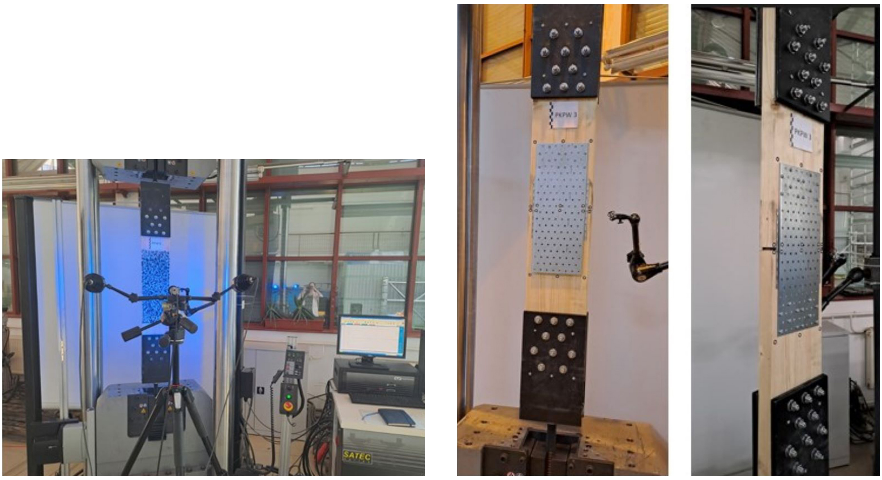
CSA 5x35 screws were made of carbon steel and protected against corrosion by zinc electroplating. On the basis of the manufacturer's technical data sheet, the characteristic capacity values for a single screw were determined. The characteristic axial withdrawal capacity is $F_{\mathrm{ax,Rk}} = 2.11$ kN, and for shear, it is $F_{\mathrm{v,Rk}} = 1.99$ kN. These values are specified for wood of grade C24.

3.3. Appropriate research

The tensile strength testing of the elements in groups GB1, GB2 and GB3 was conducted at the Faculty of Structural Research Laboratory at Rzeszow University of Technology, using the INSTRON 1200kN-J1D strength testing machine. Experimental research was conducted in accordance with the applicable standards [19, 20]. Before testing, a geometric inventory of the test specimens was performed (Fig. 6). This inventory included measurements of both cross-sectional dimensions, the positioning of fasteners, and deviations from their straightness according to the norms [21, 22].

Fig. 6. Inventory of samples

The testing involved placing the sample in anchoring fixtures on the strength-testing machine, followed by the installation of measuring instruments. Subsequently, the samples were subjected to axial tension parallel to the grain of the timber elements. A view of the testing setup and the connections of the elements before and after reinforcement is depicted in Fig. 7.



(a) (b)

Fig. 7. Research setup: (a) general view, (b) view of the connection from group GB3 prepared for testing before reinforcement and after reinforcement

The tests were carried out until failure occurred. The maximum load, F_{est}, was estimated on analytical calculations. For GB1 (unreinforced) connections, the maximum load was taken as $F_{est} = 85$ kN. Reinforcement was designed to be approximately 50% of the maximum load capacity. The reinforcement in the form of perforated plates and screws was installed after the initial loading of the GB1 connection. The initial load was set at 30% of the maximum load capacity, F_{est}. The loading procedure for the test specimens was carried out in accordance with the guidelines outlined in Section 8 of standard [20].

Initially, the connection was loaded to 0.4 of the maximum load, F_{est}, and held at this load for 30 seconds. Subsequently, the load was reduced to $0.1F_{est}$ and remained for another 30 seconds. After this period, the load was increased until the connection failed or a displacement of 15 mm was reached. The load was performed at a constant loading rate equivalent to $0.2F_{est}$ for a minute. Beyond $0.7F_{est}$, a constant displacement rate was applied, with the aim of failure within 3 to 5 minutes or when a displacement of 15 mm was achieved.

During the tests, the following parameters were recorded:

- vertical displacements along the axis of the tested connection,
- deformations,
- the value of the destructive force,
- the mode of connection failure.

Displacement and deformation measurements were carried out using a noncontact method through the ARAMIS Adjustable 12 M digital image correlation system. The optical axis of the system's camera was set perpendicular to the axis of the tested connection. To record displacements in each test element, measurement points in the form of attached markers were placed. For the GB1 (unreinforced) connections, 12 measurement points were used, 10 measurement points for the GB2 connections, and 14 measurement points for the GB3 (reinforced) connections. Additionally, next to the test specimen, a stationary element with 5 measurement points attached was placed. Deformations were recorded using a black-and-white pattern. The measurement of the relative displacement (slip) between the connected elements was performed every 5 seconds. The measurement of the force applied to the test specimen was conducted using the measuring head of the INSTRON 1200 kN J1D strength test machine, which also measured the shortening of the test element. The arrangement of measuring instruments is depicted in Fig. 8.

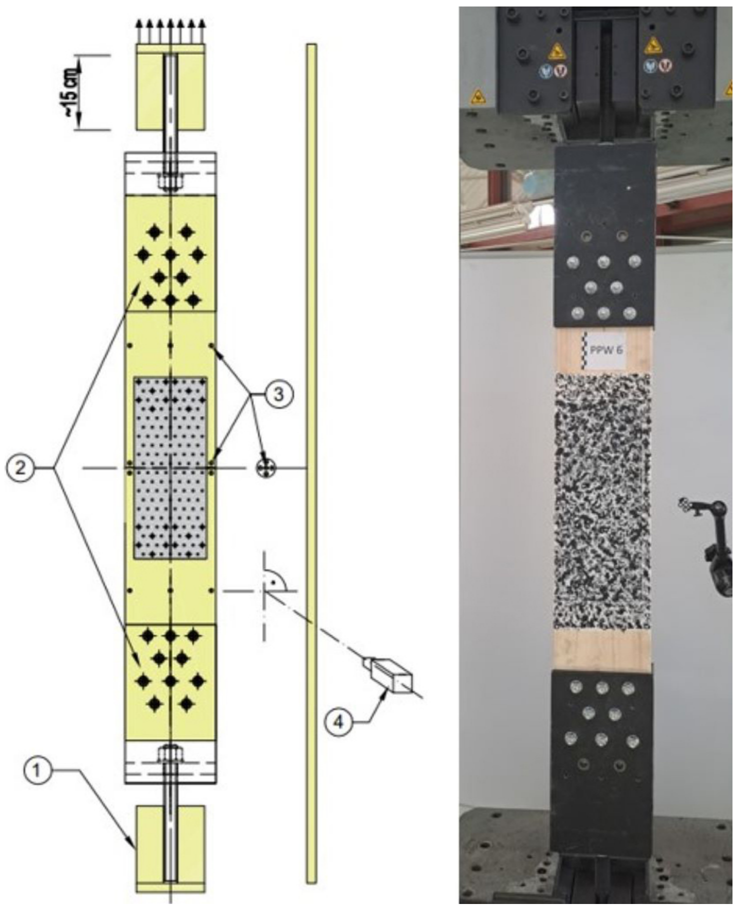


Fig. 8. View of the arrangement of measuring instruments for a GB2 test element: (a) Schematic diagram; 1 – strength-testing machine jaws, 2 – anchoring fixture, 3 – measurement points for digital image correlation, 4 – ARAMIS system camera, (b) actual view

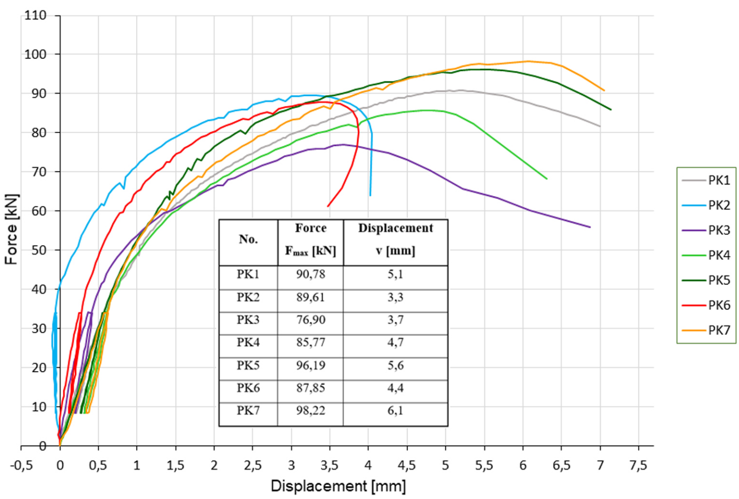
4. Research results, analysis and assessment

The primary results of the tests on the GB1, GB2 and GB3 connections were the force-displacement (F–v) curves and the mode of connection failure. The largest deformations, determined using GOM Correlate software, were found to be minimal, with values ranging from 1% to 3%. Consequently, they were omitted from further analysis of the results.

4.1. Force-displacement curves

The primary characteristic of the behaviour of the tested connections is the relationship between force and vertical displacement. Vertical displacement, denoted as v, was considered as the difference in displacements of measurement points marked on the wood elements. The points located farthest from the centre of the connection, as the most representative, were taken into account. The displacements measured by the ARAMIS system were obtained as the average of three distances between the aforementioned measurement points using the GOM Correlate software. The paths of static equilibrium obtained in this way were used to determine the ultimate force. The graphs of the obtained paths of static equilibrium and the values for each group of connections are shown in Figures 9a–c. The maximum recorded force during the test was considered the limit load for each connection or when the displacement reached 15 mm, according to Section 8 of standard [20].

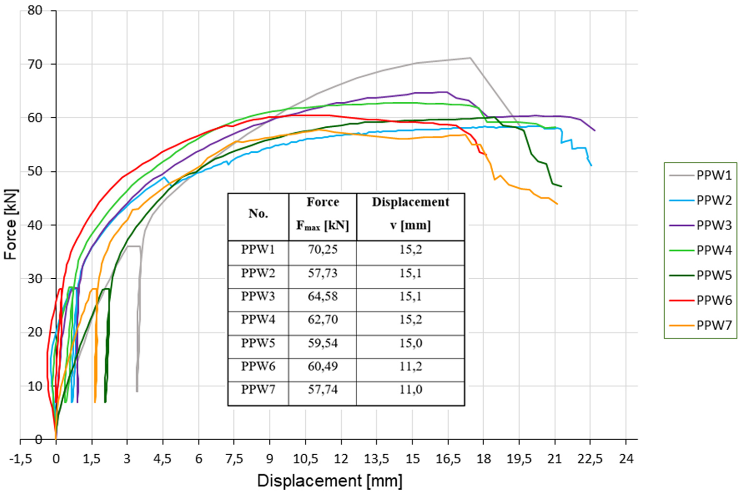
Upon analysis of the above charts, two distinct stages of connection operation are clearly evident. The boundary of the first stage is the attainment of a value of approximately 50% of the destructive force. Displacement values are very small during this stage, indicating a minor



No.	Force F_{max} [kN]	Displacement v [mm]
PK1	90,78	5,1
PK2	89,61	3,3
PK3	76,90	3,7
PK4	85,77	4,7
PK5	96,19	5,6
PK6	87,85	4,4
PK7	98,22	6,1

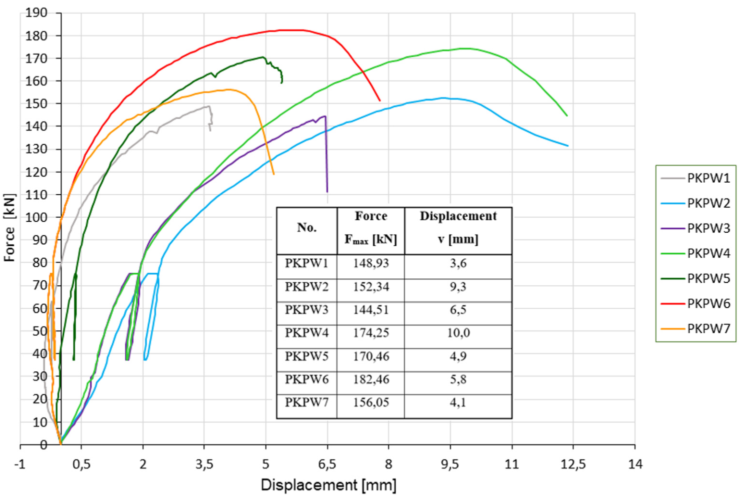
(a)

Fig. 9. The paths of static equilibrium and the values of the obtained results for the research group: (a) GB1, (b) GB2, (c) GB3



No.	Force F_{max} [kN]	Displacement v [mm]
PPW1	70,25	15,2
PPW2	57,73	15,1
PPW3	64,58	15,1
PPW4	62,70	15,2
PPW5	59,54	15,0
PPW6	60,49	11,2
PPW7	57,74	11,0

(b)



No.	Force F_{max} [kN]	Displacement v [mm]
PKPW1	148,93	3,6
PKPW2	152,34	9,3
PKPW3	144,51	6,5
PKPW4	174,25	10,0
PKPW5	170,46	4,9
PKPW6	182,46	5,8
PKPW7	156,05	4,1

(c)

Fig. 9. [cont.]

pulling of connectors from the wood. As the force continues to increase, the joint starts to operate in the second stage. This stage is characterised by the extraction of wooden dowels in the case of connection in group GB1, as well as the shearing and pulling out of dowels and screws in connection of groups GB2 and GB3. The second stage is also marked by a significant increase in displacements. The perforated plate does not deform and is not engaged in joint operation. Negative displacement values indicate the initial fit of the connectors to the wood, both in the supports and within the joint. The displacements recorded for the destructive force in the GB1 and GB3 were small, not exceeding 10 mm. Only in group GB2, which comprises connections with perforated plates and screws, were vertical displacements relatively large, exceeding 10 mm. The average values of the destructive force for the connections and the characteristic force calculated according to Appendix D point D7.2 of the PN-EN 1990:2004/Ap1 standard [23] are as follows:

- group GB1 $\bar{F}_{\max} = 89{,}33$ kN, $F_k = 74{,}61$ kN
- group GB2 $\bar{F}_{\max} = 61{,}86$ kN, $F_k = 52{,}52$ kN
- group GB3 $\bar{F}_{\max} = 161{,}29$ kN, $F_k = 131{,}21$ kN

The differences in the results obtained between individual test samples within the same group fall within the acceptable tolerances specified in the standard [20]. The sum of the characteristic load capacity obtained from the tests of connections in groups GB1 and GB2 is approximately 3% less than the characteristic load capacity obtained from the tests of connections in group GB3. This discrepancy is not in line with the provision found in the national annex [13], which requires a load reduction factor of 0.65 for connections in group GB2.

4.2. Connections failure mode

When comparing the behaviour of reinforced connections with those that are not reinforced, a similar failure mode of the connections can be observed. In most cases, the failure mode included pulling and shearing of connectors.

In the connections of group GB1 (unreinforced), the observed failure was the gradual pulling out of the spikes of the punched metal plate fasteners from the wood (Fig. 10a). The extraction of plate spikes occurred progressively, depending on the increasing load values applied to the connection. As a final result, for most of the examined joints, a symmetrical form of plate damage was recorded (Fig. 10b). In one tested connection, a block tearing failure of the joint was observed (Fig. 10c). At the point where the spikes detached, there was a bending of the metal plate fasteners. The failure mode in the joints of group GB2 was the pull-out and shearing of screws (Fig. 11). The connection failure occurred progressively, depending on the tensile force

Fig. 10. Failure mode in group GB1: (a) pulling out of the spikes of metal plate fasteners, (b) symmetric form of failure, (c) block tearing

values. Similarly to the research described in the literature [24]. As a result, the joint failure was symmetrical. No significant damage to the perforated plate was noted; it was simply bent at the point of detachment and screw shear. Plastic deformation of the plate was not observed.

Fig. 11. Connection failure mode in group GB2

In the reinforced connections of group GB3 (Fig. 12), slight pulling of the spikes of the punched metal plate fasteners and screws was observed. However, no shearing of connectors was recorded. There was no damage to the perforated plate and it did not exhibit plastic deformation. The effectiveness of reinforcement using perforated plates and screws can be observed.

Fig. 12. Failure mode in Group GB3

5. Summary and final conclusions

The research had an exploratory character. Analysis of the results obtained from experimental testing justifies the conclusion that the characteristic load capacity of the tested connections using metal plate fasteners and perforated plates (group GB3) is approximately equal to the sum of the characteristic load capacity values obtained from the tests of connections in groups GB1 and GB2. The method applied to reinforcing tension connections with metal plate fasteners using perforated plates and screws shows potential for practical application. On the basis of the results of the research, it is recommended to use this method for reinforcing connections realised on punched metal plate fasteners. Design procedures for strengthening various types of connections in timber structures are poorly documented and require appropriate experimental research and analysis. The results of these efforts will allow for the development of detailed and unambiguous design procedures and guidelines for the execution and acceptance of reinforced connections on punched metal plate fasteners.

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Nośność złączy na płytki kolczaste wykonanych z drewna świerkowego wzmocnionych płytkami perforowanymi i wkrętami

Słowa kluczowe: badania niszczące, konstrukcje drewniane, projektowanie złączy, wiązary dachowe, wzmacnianie połączeń

Streszczenie:

W Polsce do 2010 roku wybudowano szereg obiektów z konstrukcją nośną dachów w postaci drewnianych wiązarów z węzłami na płytki kolczaste. Zmiany normalizacyjne, tj. wprowadzenie w 2010 r. eurokodów dotyczących obciążeń, spowodowało między innymi zwiększenie wartości zalecanych obciążeń klimatycznych. W związku z powyższym w konstrukcjach wybudowanych kilka lat temu dochodzi do przekroczenia limitu nośności zarówno dla drewna, jak i łączników. Modernizacja obiektów z wiązarami drewnianymi, z reguły prowadzi do konieczności wzmacniania istniejących węzłów. Zagadnienie wzmacniania złączy na płytki kolczaste jest słabo rozpoznane w badaniach doświadczalnych (niszczących), a tym samym brak jest szczegółowych wytycznych normowych. Celem przeprowadzonych badań doświadczalnych było rozpoznanie zachowania się rozciąganych złączy na płytki kolczaste oraz oceny wpływu zastosowanego dodatkowego wzmocnienia w postaci płytek perforowanych i wkrętów. Analiza otrzymanych wyników badań doświadczalnych upoważnia do stwierdzenia, że wzmocnione węzły osiągnęły znacznie większe obciążenie granicznie niż to, które wynikało z obliczeń projektowych.

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