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ASSESSMENT OF NAILED CONNECTIONS IN EXISTING STRUCTURES

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ABSTRACT: This paper presents the development of an assessment scheme for a visual qualitative evaluation of nailed connections in existing structures, such as board trusses. In terms of further use and preservation, a quick visual inspection will help to evaluate the quality of a structure regarding its load-bearing capacity and deformation behaviour. Tests of old and new nailed joints in combination with a rating scheme point out the correlation between the load-bearing capacity and condition of a joint. Old joints of comparatively good condition tend to exhibit better results than those of poor condition. Moreover, aged joints are generally more load-bearing than newly assembled ones.

KEYWORDS: nailed trusses, nailed constructions, Corrosion, Preservation, Re-use, Structural Post-strengthening

1 INTRODUCTION

Carpentry-based timber construction in the 20th century underwent a strong development towards engineered timber construction, which was driven by the development of new materials (glulam, laminated veneer lumber, wood-based boards), on the one hand, and new connections, on the other. Moreover, steel structures were replaced by timber structures in times of material and resource scarcity, and in this context, nails played a decisive role.
Utilising nail construction, smaller board cross-sections could be economically connected to form efficient trusses with large spans, without having to pay attention to patents, as was the case with the various types of shear connectors, and could also be erected by untrained workers. A nailed truss construction from this time is depicted in Figure 1.

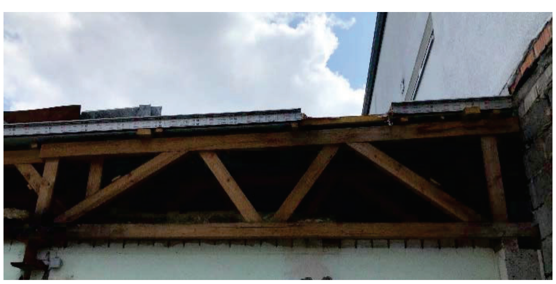

Figure 1: *Bailed truss Type-II*

However, acceptance of the structural use of the nail had to be created first. While dowels and bolts were widely used in many applications at the beginning of the last century, the nail in a load-bearing function was still prohibited. The nail only become part of the standardised codes after extensive basic research in the 1930s [1-3] and, thus, establish itself in the construction industry. Until the introduction of nail plates, countless roof trusses were built with nailed joints. Nailed trusses were still used in countries with a planned economy until about 1990.

The question of continued use and preservation is an important issue for many of these structures, leading to the need for engineering assessment. In this context, strongly varying conditions might be found. The nodes, which are, in many cases, decisive for the load bearing capacity, might especially exhibit deficiencies. A scheme is developed here for a simple, rapid evaluation of the condition of nailed connections in terms of load-bearing capacity and serviceability to generally support a decision about their further use.

With the help of this scheme, the condition of the nail connections is to be determined regarding nailing patters, edge distances and spacing, the presence of small or large cracks or fractures and moisture damage. The effort should be limited to the extent that non-destructive examinations are used almost exclusively. This means that the assessment is carried out mainly by visual inspection. In order to obtain information on the load-bearing behavior of nailed connections, the objectives of this paper are to determine the load-bearing capacities based on experimental tests on aged connections of nailed trusses, and combine these with visual classification based on the scheme presented.

The further use of such connections or load-bearing structures is to be supported by the combination of visual and experimental examinations. In addition to the examination utilising the scheme, the corrosion of the nails is also considered.

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2 ASSESSMENT OF CONNECTIONS

2.1 CLASSIFICATION SCHEME

A comprehensible rating scheme had to be developed to support a visual qualitative evaluation. In this context, Imamura and Kiguchi [4] presented a scheme based solely on the degree of corrosion of the nails. In this case, the condition of the nail was assigned from completely intact to failure. This approach was extended to the evaluation scheme for nailed joints of roof trusses, which also considers the integrity of the wood. This scheme is shown in Figure 2.

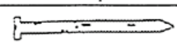
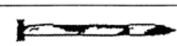
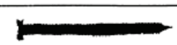
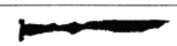
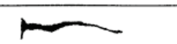
Rating	Standard	Example
1	Scarcely rusted	
2	Partially rusted, no visible defect	
3	Totally rusted, no defect inside	
4	Partially defect, with original length	
5	Failure	

Figure 2: *Rating scheme of corrosion of the nails from* [4]

The classification into different categories is based only on the external appearance. Consequently, various conditions were imposed to enable the categorization of the individual nodes. These conditions are depicted in Figure 3. Each category is divided into a side and bottom view.

The reference to the load, depending on the grain direction, is established for this purpose. The assessment is carried out for the diagonal components loaded in the direction of the grain.

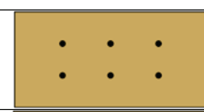
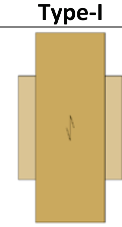
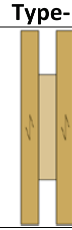
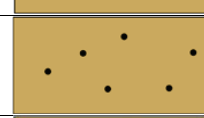
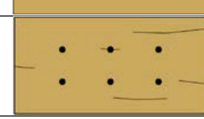
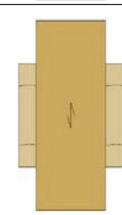
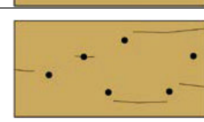
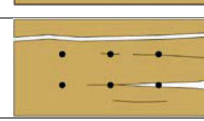
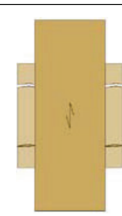
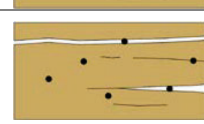
Category 1 is reached if the penetration length, the edge distance and the spacing are in accordance with the regulations applicable. In addition, at least 90 % of the nails must be hammered flush with the connecting timber, otherwise the joint falls directly into category 2. Category 1 should not generally show any visual damage.

A joint is classified as category 2 if either the penetration length, the edge distance or the spacing are not in accordance with the regulations applicable. No visible damage should be apparent in category 2 either.

Category 3 is divided into subcategories: 3a and 3b. In 3a, the joint complies with the penetration length, the edge distance and the spacing according to the regulations applicable. The joint is classified as 3b if any condition from 3a is not fulfilled. Basically, a joint is classified as category 3 if it has cracks smaller than 1 mm, less than 90 % of the nails are nailed flush with the connecting timber or there are first signs of deterioration of the wood.

In category 4, a distinction is again made between subcategories: 4a and 4b. The subcategories are the same as for condition 3. The criteria for classification as category 4 are any visible defects, including cracks larger than 1 mm wide, splitting, local moisture damage, wood decay or insect infestation.

Figure 3: *Classification scheme for aged nail joints*

Category		Description		Illustration: side view	Illustration: bottom view Type-I	Illustration: bottom view Type-II
1		➢ Specifications for edge distance, penetration length and spacing are in accordance with the code regulation. ➢ > 90 % of the nails flush with connecting timber ➢ Timber without visible damage / cracks				
2		➢ Specifications for edge distance, penetration length or spacing are in accordance with the code regulation. ➢ Timber without visible damage / cracks				
3	a	➢ small cracks, width < 1 mm ➢ < 90 % of the nails flush with connecting timber ➢ Indications of moisture impact	Specifications for edge distance, penetration length and spacing are in accordance with the code regulation.			
	b		Specifications for edge distance, penetration length or spacing are not in accordance with the code regulation.			
4	a	➢ crack width > 1 mm ➢ splitting and disconnection ➢ moisture impact ➢ timber decay ➢ insect infestations	Specifications for edge distance, penetration length and spacing are in accordance with the code regulation.			
	b		Specifications for edge distance, penetration length or spacing are not in accordance with the code regulation.			

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2.2 APPLICATION OF THE CATEGORIZATION

The scheme was applied to two test series with the same roof shape, which was the mono-pitched roof. Type I consisted of a chord as the middle member and struts and posts as side members, respectively. The construction is shown in Figure 4 and 5. A total of 62 connections were tested. Furthermore, 11 connections of Type II were tested. In the latter, struts and posts are between two bottom chords, as shown in Figure 6 and 7.

The scheme to evaluate the condition was applied to all joints. Only timber members which are loaded in the direction of the grain were analysed. The chord, which is loaded orthogonal to the grain, can be neglected for the categorization.

The designations of the joints AH – “Adorf Hessen” and GH – “Gießen Hessen” refer to the region in Germany, while PD refers to the roof shape. In both cases the roof shape was a pitched roof.

Joint Type I

The connections were made with a different number of nails and two different nail diameters. The posts and struts are on the outside in this type. The chord between the posts and struts is a single beam with the dimensions 8 x 16 cm. The chord has not been categorized due to its full integrity and the loading, which is not parallel to the grain. The assessment of the diagonal side members with a geometry of 2.3 x 10 cm is illustrated by the examples in Figure 4 and 5.

(a)
side view A:
post cat. 2,
strut cat. 2

(b)
side view B:
post cat.: 3b,
strut cat.: 3b

Figure 4: Evaluation of Joint Type I B09

The post for joint B09 in Figure 4a was assigned to Category 2 based on a nonstandard nailing. No cracks could be found. The strut on this side was assigned to category 2 because no cracks were discernible here either and standard nailing was missing. On the other side of the connection, the post was assigned to category 3b due to nonstandard nailing and small cracks up to 1 mm wide.

The strut on side B was classified in category 3b as it also lacked standard nailing and showed small cracks up to 1 mm in the lower part of the board.

(a)
side view A:
post cat.: 4b,
strut cat.: 4b

(b)
side view B:
post cat.: 4b,
strut cat.: 4b

Figure 5: Evaluation of Joint Type I B12

The posts for joint B12 in Figures 5a and 5b were classified as category 4b due to nonstandard nailing and cracks larger than 1 mm. The struts on both sides were classified as category 4b because there were also cracks larger than 1 mm and a standard nailing was missing.

Joint Type II

The connections of Type II were made with a different number of nails. In this type, the posts and struts formed the middle parts. The chords were on the outside, each with dimensions of 2.3 x 10 cm. Since the direction of the force is not in the direction of the grain of the chord, it was assumed to have no influence on the evaluation. The middle members with a geometry of 2.3 x 10 cm have been evaluated using two examples in Figure 6.

bottom view:
cracks
→ post cat.: 4

Figure 6: Evaluation of Joint Type II A01

The post for Joint A01 is decisive for the evaluation of the entire joint. It was classified as category 4b due to the bottom view because there was a crack of more than 1 mm (see Fig. 6).

bottom view:
cracks →
post cat.: 4,
strut cat.: 4

Figure 7: Evaluation of Joint Type II A02

The post and the strut were again taken as significant for the evaluation of joint A02. These were classified as category 4 due to the bottom view, since cracks larger than 1 mm were present there (see Figure 7b).

3 ANALYTICAL CALCULATION

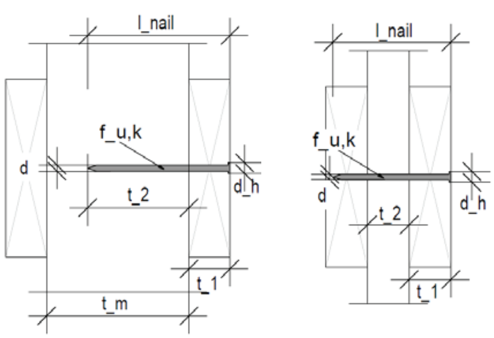
The characteristic lateral resistance for the shear loaded connections was calculated according to EN 1995-1-1:2010 for purposes of comparison [7]. The geometrical and the material parameters used in this calculation are documented in Table 1. The construction of each type with the geometrical parameters is shown in Figure 8.

Table 1: Parameters for the calculation of the two test series

$f_{u,k}$ [N/mm²]	ρ_m [kg/m³]	ρ_k [kg/m³]	f_{ax} [N/mm²]
600	420	350	2.45

Calculation of the load-bearing capacity

The following steps show the calculation of the shear capacity for the two test series.



(a) (b)

Figure 8: Geometry of (a) Type I and (b) Type II

It could be shown using pre-calculations that for all diameters of the nails, mechanism d was decisive for Type I, in which a yield hinge was formed in the middle timber, and mechanism c for Type II, with embedment failure in all timbers.

The yield moment $M_{y,Rk}$ is calculated as

$$M_{y,Rk} = 0.30 \cdot f_{u,k} \cdot d^{2.6} \tag{1}$$

where $f_{u,k}$ is the tensile strength and d the diameter of the nails. The embedment strength $f_{h,k}$ is calculated as

$$f_{h,k} = 0.082 \cdot \rho_k \cdot d^{-0.3} \tag{2}$$

where ρ_k is the characteristic density. The withdrawal strength $f_{ax,k}$ is calculated as

$$f_{ax,k} = 20 \cdot 10^{-6} \cdot \rho_k^2 \tag{3}$$

The head pull-through resistance is calculated as

$$f_{head,k} = 70 \cdot 10^{-6} \cdot \rho_k^2 \tag{4}$$

The ratio of embedment strength is calculated as

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} = 1 \tag{5}$$

with $f_{h,2,k} = f_{h,1,k}$. The load-bearing capacity for the joints of Type I is calculated as

$$F_{v,Rk} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta}\left[\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - \beta\right] \tag{6}$$

with the parameters determined previously. The load bearing capacity for Type II is calculated as

$$F_{v,Rk} = \frac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta} \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \cdot \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right] + \beta \cdot \left(\frac{t_2}{t_1}\right)^2 - \beta \cdot \left(1 + \frac{t_2}{t_1}\right)}\right] \tag{7}$$

To include the rope effect $\Delta F_{ax,Rk}$, the significant withdrawal force is calculated as

$$F_{ax,Rk} = min\begin{cases} f_{ax,k} \cdot d \cdot t_2 \\ f_{ax,k} \cdot d \cdot t_1 + f_{head,k} \cdot d_h^2 \end{cases} \tag{8}$$

where d_h is the nail head diameter. The additional force is calculated as

$$\Delta F_{v,Rk} = min\begin{cases} 0.25 \cdot F_{ax,Rk} \\ 0.15 \cdot F_{v,Rk} \end{cases} \tag{9}$$

The total load capacity of a nail is calculated as

$$F_{v,Rk,total} = F_{v,Rk} + \Delta F_{v,Rk} \tag{10}$$

The dimensions of the nails correspond to those found in the components. The standard dimensions of nails nowadays are different. Nails with diameters of 2.8, 3.1 and 3.4 mm are used in these specimens.

The analytical calculated capacity according to EC5 is documented in Table 2 for each diameter.

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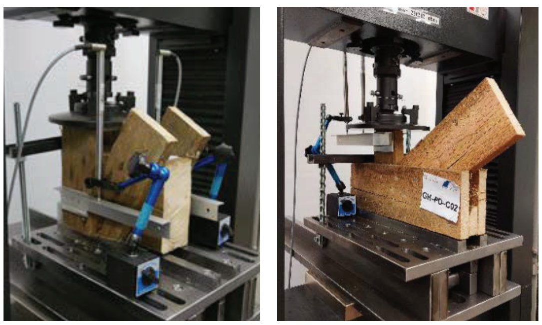
Table 2: Load capacity per nail diameter

d [mm]	d_h [mm]	t_m [mm]	t_1 [mm]	t_2 [mm]	$F_{v,k}$ [N]	$F_{ax,k}$ [N]	$F_{v,k,total}$ [N]
2.8	6	80	23	42	588	288	660
3.1	6	23	23	23	604	175	647
3.4	7	80	23	57	727	475	836

4 EXPERIMENTAL TESTING OF JOINTS

The joints evaluated with the scheme developed were finally tested to determine the capacity of the connections. The whole test programme included shear tests on specimens taken from trusses of existing structures and additional tests to determine material parameters. The evaluations of all tests are presented by Schwendner et al. [6].



(a) (b)

Figure 9: Test set-up for joints (a) Type I; (b) Type II

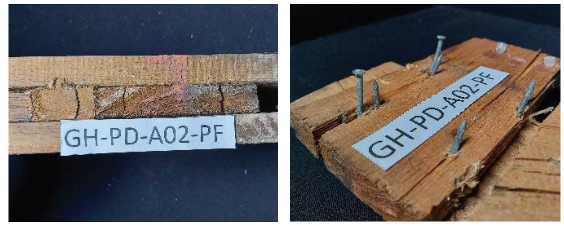
Only the experimental tests on joints will be discussed in this paper. The failure of the joints was defined corresponding to a displacement of 15 mm. Figure 9a shows the test set-up for Type I and Figure 9b for Type II. Both configurations can be tested with the set-up developed.

5 TEST RESULTS AND VALIDATION

In this section, the experimental test results are compared with the results of the analytical calculation according to EC5. In a second step, the scheme developed is validated. Therefore, the categories are compared with the ratio between the experimental test results and the strength calculated according to EC5.

5.1 TYPICAL FAILURE MODES

Application examples of the rating scheme show a correlation of the joint and the failure mode. The joints which are depicted in Figure 11a and b, for example, were evaluated in advance with condition 4 and show that not all nails have formed plastic hinges after the execution of the test; the wood was unable to contribute to the embedment strength due to the cracks and, thus, a lower load-bearing capacity of the joint results.



(a) (b)

Figure 10: Type II: joint GH-PD-A02 before and after testing

As a further example, Figure 11a and b show a test specimen of Type II before and after the shear test. Here, the rating 4 also indicates a lower load-bearing behaviour. After testing, it can be seen that the specimen could not reach its full load-bearing capacity in the area of the crack, due to the missing yield hinge. Consequently, not all nails reached their capacity.

(a) (b)

Figure 11: Type I: joint AH-PD-B06 before and after testing

5.2 COMPARISON BETWEEN EXPERIMENTAL TEST RESULTS AND ANALYTICAL CALCULATION

The geometrical components and the assessment category for each joint taken from the aged roof trusses are documented in Table 3 to 6. Additionally, the number of nails, the analytically calculated characteristic strength $F_{v,Rk}$ according to EC5 and the maximum strength F_{max} determined by the experimental tests are given. As a final result, the ratio between F_{max} and $F_{v,Rk}$ is calculated.

Nails with a diameter of $d = 3.4$ mm and $d = 2.8$ mm were used for Type I. The calculated load capacity in relation to the number and diameter of existing nails per joint are shown in Table 3 and 4.

Nails with a diameter of $d = 3.1$ mm were used for Type II. The calculated load capacity in relation to the number of nails per joint are shown in Table 5 and 6.

Table 3: Type I, Serie AH-PD-PF (ST = strut)

Joint	α	Category	$n_{2.8}$	$n_{3.4}$	$F_{v,Rk}$	F_{max}	$F_{max}/F_{v,Rk}$
	[°]	[-]	[-]	[-]	[kN]	[kN]	[-]
A02-ST	90	3	7	8	11.31	24.62	1.4
A04-ST	90	2	9	6	10.96	26.75	1.9
A05-ST	83	2	9	6	10.96	30.09	1.9
A06-ST	90	2	7	6	9.64	27.72	2.2
A08-ST	90	3.5	0	14	11.70	26.33	2.5
A09-ST	83	3	7	6	9.64	20.20	2.0
A10-ST	90	2.5	11	6	12.28	23.90	1.8
A11-ST	83	4	7	7	10.47	22.71	2.7
mean-A-ST		**2.8**			**10.87**	**25.29**	**2.0**
B04-ST	90	3	0	13	10.87	30.70	2.6
B06-ST	90	3	0	13	10.87	26.93	2.6
B07-ST	83	2.5	0	12	10.03	21.79	1.9
B08-ST	90	2.5	0	13	10.87	29.10	1.9
B09-ST	83	2.5	0	12	10.03	27.21	1.7
B10-ST	90	4	0	14	11.70	20.09	2.5
B11-ST	83	3	0	12	10.03	22.94	1.8
B12-ST	90	4	0	12	10.03	15.70	1.5
mean-B-ST		**3.1**			**10.55**	**24.31**	**2.1**

Table 4: Type I, Serie AH-PD-PF (PF = post)

Joint	α	Category	$n_{2.8}$	$n_{3.4}$	$F_{v,Rk}$	F_{max}	$F_{max}/F_{v,Rk}$
	[°]	[-]	[-]	[-]	[kN]	[kN]	[-]
A02-PF	90	4	9	6	10.96	15.64	2.2
A04-PF	90	2.5	9	6	10.96	21.09	2.4
A05-PF	83	3	9	6	10.96	20.94	2.7
A06-PF	90	2	4	11	11.84	25.86	2.9
A08-PF	90	3.5	0	12	10.03	25.56	2.2
A09-PF	83	3	9	6	10.96	21.54	2.1
A10-PF	90	3	11	6	12.28	21.70	1.9
A11-PF	83	3	8	6	10.30	27.48	2.2
mean-A-PF		**3.0**			**11.03**	**22.48**	**2.3**
B04-PF	90	2.5	0	12	10.03	26.22	2.8
B06-PF	90	2.5	0	12	10.03	25.77	2.5
B07-PF	83	2.5	0	12	10.03	18.59	2.2
B08-PF	90	2.5	0	12	10.03	19.10	2.7
B09-PF	83	2.5	0	12	10.03	16.77	2.7
B10-PF	90	2.5	0	11	9.20	23.34	1.7
B11-PF	83	3	0	12	10.03	18.43	2.3
B12-PF	90	4	0	11	9.20	13.92	1.6
mean-B-PF		**2.8**			**9.82**	**20.27**	**2.3**

Table 5: Type II, Serie GH-PD-ST (ST = strut)

Joint	α	Category	$n_{3.1}$	$F_{v,Rk}$	F_{max}	$F_{max}/F_{v,Rk}$
	[°]	[-]	[-]	[N]	[N]	[-]
A02	35	4	13	16.82	22.55	1.3
B02	35	3	7	9.06	13.06	1.4
A03	30	3	14	18.12	29.33	1.6
B03	30	3	16	20.70	34.05	1.6
C02	35	3	14	18.12	30.95	1.7
C03	30	2	14	18.12	49.99	2.8
mean-ST		**3.0**	**13**	**16.82**	**29.99**	**1.8**

Table 6: Type II, Serie GH-PD-PF (PF = post)

Joint	α	Category	$n_{3.1}$	$F_{v,Rk}$	F_{max}	$F_{max}/F_{v,Rk}$
	[°]	[-]	[-]	[kN]	[kN]	[-]
A01	85	4	8	10.35	20.17	1.9
B01	85	4	8	10.35	5.70	0.6
C01	85	4	8	10.35	16.58	1.6
A02	90	4	8	10.35	15.03	1.5
B02	90	4	8	10.35	15.18	1.5
mean-PF		**4.0**	8	**10.35**	**14.53**	**1.4**

The experimental test results achieved a higher load than the calculation according the EC5 for all joints, with the exception of joint B01.

5.3 VALIDATION OF THE ASSESSMENT SCHEME

The results from experimental testing are used for the validation of the classification system. Thus, the characteristic capacity per nail was assigned to the respective category for each test.

Figure 12a shows the graphs for Type I. Based on the shear tests, it was possible to establish a trend line showing a clear tendency for a decreasing quality to correspond to a lower load-bearing capacity (see Figure 12). On average, a ratio of 2.6 between experimental and analytical results can be obtained for Category 2, 2.2 for Category 3 and 1.7 for Category 4.

Figure 12b shows the graph for Type II. By assigning the load-bearing capacities to the pre-rated categories, a trend line could again be generated here. This supports the hypothesis of the correlation of the assessed category and capacity. The average ratio of category 2 is 2.8, of category 3, 1.6 and of category 4, 1.4.

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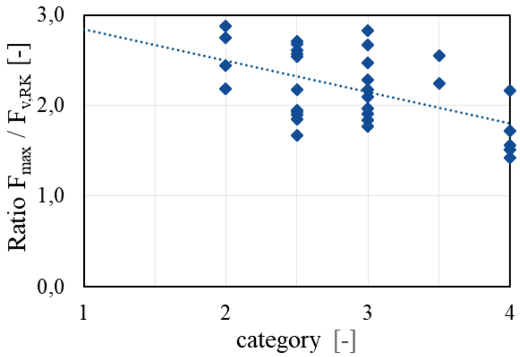
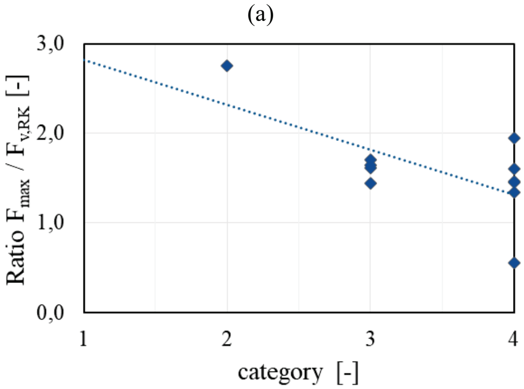


(a)



(b)

Figure 12: Load-bearing capacity per nail depending on the category, (a) Type I, (b) Type II

6 SUMMARY AND CONCLUSION

A classification scheme which is based on the visual inspection of individual joints was successfully applied and confirmed. It could be shown by investigating joints of aged nail trusses that a lower bearing capacity was reached on average with a decreasing rating.

In addition, the rating can also predict the possible form of failure, such as the preliminary splitting of the wood so that the nails cannot form a yield hinge. Cracks indicate that the nails cannot develop their full load-bearing capacity. The category and assignment correspond to the quality of the joints in terms of their load-bearing capacity.

Regarding further research on aged nail trusses, experiments should be carried out on complete trusses. An assessment of the load-bearing behaviour of the intact trusses is missing because only joints and their parameters have been investigated so far. This could be used to describe further factors, such as bending or buckling.

In addition to other component tests in the future, an exact evaluation of the load-bearing capacities measured so far per condition should be determined. This means that a percentage load-bearing capacity can be assigned per category of the connection according to Eurocode 5. The aim here is that the load-bearing capacity of worse joints may have to be corrected. For this purpose, further investigations are needed, for example, to check repair measures on aged structural members. The aim is to gain more knowledge about the compliance of the connections. Is it sufficient to hammer additional nails into a component to reactivate defective parts? How does a replaced board affect the whole system? Which joints need to be repaired?

Large-scale tests are necessary to answer these questions. In addition to determining the load-bearing capacity, other factors should also be considered. Due to the slenderness of the components, the fire protection class is often significantly lower than F30. A possible fire can have a direct influence on the nails and lead to an abrupt collapse of the construction. Measures should be considered to increase the stability in case of fire.

Another factor is the influence of moisture. Although the thinness of the component results in less swelling, the slenderness of the boards means that the full cross-section is probably saturated and could lead to a faster failure of the component in the long term.

In addition to these open points, an application of the evaluation scheme in existing buildings would be desirable. The target is to evaluate the load-bearing capacity of the roof structure without removing components in order to be able to plan possible replacement measures in an economical and sustainable way.

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