World Conference on
Timber Engineering
Oslo 2023

TESTING OF CONNECTIONS TAKEN FROM OLD NAILED ROOF TRUSSES

Sascha Schwendner[1], Lars Völlmecke[2], Phien Ho[3], Jens Fischer[4], Werner Seim[5]

ABSTRACT: Experimental testing of nailed connections taken from old roof trusses is presented in this paper. To enable the further use and preservation of nailed roof trusses, it is important to understand how the nail corrosion and aging processes of steel and wood affect the load-bearing capacity and deformation behaviour of such structures. The hypothesis was investigated whether corroded nails allow an increase in load-bearing capacity. Several old and new joints were tested in a first test series, and the results were very promising regarding the initial assumption. However, more tests must be carried out to verify the results.

KEYWORDS: Nailed Connection, Corrosion, Degradation, Preservation, Withdrawal, Shear Loaded

1 INTRODUCTION

Connections with nails experienced a real boom in the middle of the 19th century and often turned out to be the construction method of choice, especially in times of material and resource scarcity. In addition, labour costs were significantly lower than today, which meant that the large amount of time required to create the trusses with many fasteners was still economical. Nevertheless, nails were not allowed for load-bearing purposes at the beginning of the 20th century in Germany until they were established as widely accepted fasteners through extensive basic research in the 1930s [1,2].

The combination of comparatively small timber sections and nails led to the development of trusses, which achieved large spans with a minimal use of materials. Moreover, nails were not subject to patent protection and could also be installed by untrained laborers, two important advantages compared, for example, to connections with dowels.

A large variety of different types of roof trusses were developed over the years. The load-bearing capacity of nailed trusses might be questioned today when it comes to discussing further use and preservation. Takanashi et al. [3] have already suggested an increase of the load-bearing capacity of nailed connections from existing structures based on their pull-out tests on artificially aged nail joints.

The aim of this paper is to investigate the load-bearing capacity of aged timber joints and the influence that corrosion of nails can have on this. Against this background, an investigation into how aging, especially nail corrosion, affects the load-bearing capacity and the deformation behaviour of the nailed joints was undertaken. The basic assumption of a potential increase in the load-bearing capacity due to partially corroded nails was investigated by tests on aged joints from nailed roof trusses taken from existing buildings. The truss nodes were examined with a newly developed testing device. Additional tests were carried out on single components to analyse their specific properties and determine parameters. This can provide conclusions about differences between the aged connections and new construction. Therefore, geometrically identical node connections were produced with new materials and tested (see Figure 1). All test results will be compared and discussed in the following.

(a) (b)

Figure 1: *(a) Type-I_old: joint B04 and (b) Type-I_new: joint SV-1947-01*

2 EXPERIMENTAL TESTING

In order to be able to investigate the behaviour of aged nodes, trusses were chosen from a structure which had been used primarily for roofing a car wash and later a beverage store. After demolition, the trusses were stored

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[1] Timber Structures and Building Rehabilitation, University of Kassel, Germany, s.schwendner@uni-kassel.de
[2] Timber Structures and Building Rehabilitation, University of Kassel, Germany, l.voellmecke@uni-kassel.de
[3] Timber Structures and Building Rehabilitation, University of Kassel, Germany, phienho@uni-kassel.de
[4] Timber Structures and Building Rehabilitation, University of Kassel, Germany, fischer.jj@t-online.de
[5] Timber Structures and Building Rehabilitation, University of Kassel, Germany, wseim@uni-kassel.de

https://doi.org/10.52202/069179-0529

outside under a tarp sheathing for several years (see Figure 2). This structure has been exposed over time to different climatic conditions, which have caused significant corrosion at the joints and local cracks to appear in the timber. On the other hand, some nodes are completely undamaged, so, there was a wide range between intact and considerably damaged nodes.

2.1 TESTING PROGRAMME

The test programme consists basically of two groups. The elements selected for the first group (Type I-old) are taken from the original trusses. These specimens showed cracks in the timber and corrosion of the nails, together with irregularities of the nailing pattern.
The elements selected for the second group (Type I-new) were made with new materials and served as a reference to evaluate the load-deformation behaviour of aged elements (see Figure 1). The specimens in the second group are identical to those taken from the trusses to be comparable to each other.

Figure 2: *Roof trusses*

In a first step, parameter tests were carried out to get an initial impression of the quality of the materials which were used for the joints. The parameters determined were

- the density of the timber,
- the embedment strength of the timber,
- the tensile strength of the steel,
- the yield moment of the nails and
- the withdrawal strength of the nails.

In terms of timber materials, a distinction was made between strut and chord. The position in the truss beam does not play a role in the determination of the parameters for the nails.

An overview of the whole test programme is given in Table 1. The tests to indicate the specific properties and the tests on connections are listed here.

Table 1: *Overview of the test programme*

Number of tests	Old	New
Density ρ_k	27	18
Embedment strength $f_{h,k}$	16	16
Tensile strength $f_{u,k}$	20	8
Yield moment $M_{y,k}$	10	10
Withdrawal strength $f_{ax,k}$	18	10
Connection I-old	8	15
Connection I-new	8	15

2.2 DENSITY AND EMBEDMENT STRENGTH

The density of the timber in both groups was tested according to DIN 52182 [4]. The test specimens were dried until the weight dropped to under 0.1 %.

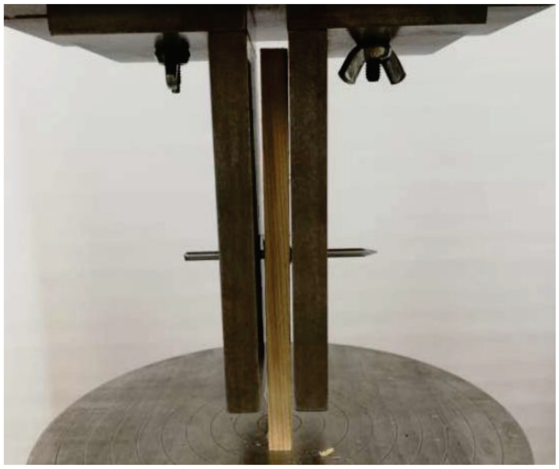


Figure 3: *Test set-up to determine the embedment strength of the timber*

The embedment strength of the timber was tested according to EN 383:2007 [5]. Figure 3 shows the test set-up to determine the embedment strength of the timer elements. The embedment strength is determined at a deformation of the specimen of 5 mm with the maximum force *F*, the thickness of the timber specimen *t* and the diameter of the fastener *d*, according to equation (1). Based on the code for all specimens, the loading was parallel to the grain.

$$f_{h,k} = \frac{F_{\max}}{t \cdot d} \tag{1}$$

2.3 TENSILE STRENGTH AND YIELD MOMENT

The tensile strength of the fasteners was determined according to DIN 2002-001 [6] after the nail heads had been cut, whereby the clamping of the nails had to be reduced due to their length. This had no influence on the fracture pattern of the nails. Testing at a constant speed, the maximum strength is calculated with the maximum force and the cross-section for the original nominal diameter of 3.40 mm of the nail. Figure 4 shows the aged and new nails and the test set-up to determine the tensile strength of single nails.

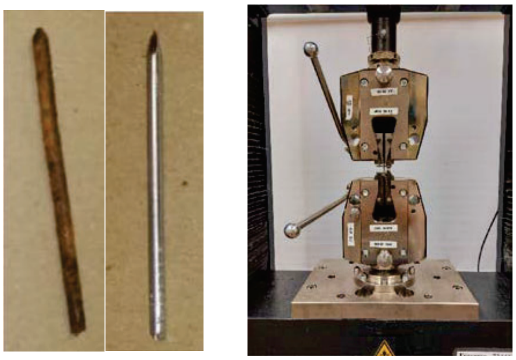

(a) (b)

Figure 4: *(a) Aged and new nail and (b) test set-up to determine the tensile strength of a single nail*

The yield moment of the nail was determined with a test set-up proposed by Werner and Siebert [7]. The construction is depicted in Figure 5. The nail in the frame can be deformed with a long cantilever with the length l_v. A load cell is located at the end of the lever arm. The yield moment of a single nail can be determined with the force F_{max} detected in the load cell and the length of the lever arm, see equation (2).

$$M_{y,Rk} = F_{\max} \cdot l_v \tag{2}$$

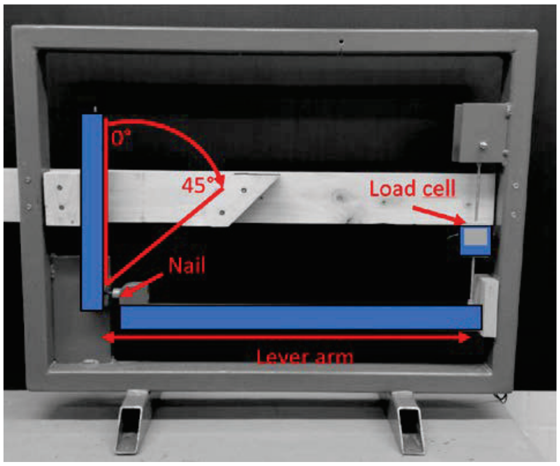
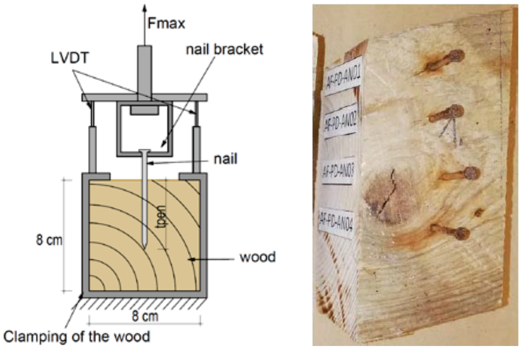


Figure 5: *Test set-up to determine the yield moment of a single nail*

2.4 WITHDRAWAL STRENGTH

The withdrawal strength of the nails was determined according to EN 1382:2016-07 [8]. The test set-up is shown in Figure 6. To determine the withdrawal strength of the aged nails, parts of the truss beams were cut out to obtain realistic results. The test specimens were conditioned under a humidity of 65 ± 2 % at 20 ± 2 °C prior to testing.



(a) (b)

Figure 6: *(a) Test set-up and (b) specimen for the determination of the withdrawal strength*

2.5 TESTS ON CONNECTIONS

In a second step, tests on joints cut out from the trusses were carried out. A special test set-up was developed for this so that the connection of posts and struts that act in a node can be tested separately, one after the other (see Figure 7). The set-up was designed in such a way that different inclinations of the struts could be tested. The chord of the truss beam ran continuously as one section between the two supports, whereas the posts and the struts were separated into two parts, each connected sideways to the chord. Each pair of posts or struts in the test set-up were loaded and could deform up to 50 mm, which ensured that the maximum load was achieved in the connections when the tests were carried out. The test set-up is depicted in Figure 7 (b). Due to the test set-up developed, it was necessary to test the posts first and then the struts.

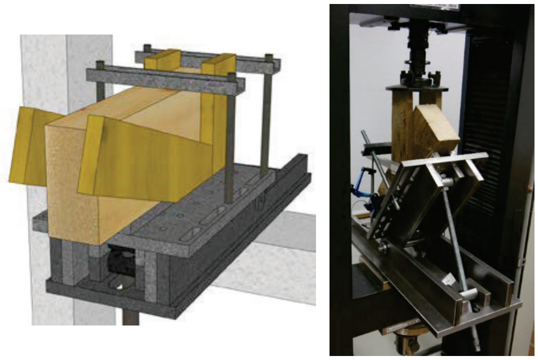

(a) (b)

Figure 7: *Test set-up for joints (a), isometric view and (b) photo*

All tests were performed and evaluated under the monotonic loading protocol according to EN 26891 [9]. Displacement transducers were installed to measure the relative displacement between the chord and the posts and struts, respectively. The load-bearing capacity of the connections was, firstly, estimated at the mean value level to determine the loading rate. This resulted in a loading rate of 41.6 and 62.3 N/sec, depending on how many fasteners were arranged in the node to be tested. This ensures that failure was achieved in a time span of 120 s ± 30 sec. The test specimens were conditioned under a humidity of 65 ± 2 % at 20 ± 2 °C prior to testing.

3 RESULTS AND DISCUSSION

The results of the experimental investigations are presented and evaluated in this section. Firstly, the experiments for determining the parameters of the single elements of the truss are discussed, followed by a consideration of the nodes. Finally, the 5 % fractile values are derived from experimental results which are then compared with the characteristic values from EC 5 [10]. This makes it possible to compare whether the aged trusses have similar properties to the nodes created with new materials. The values according to EC 5 are calculated as follows:

$$\begin{aligned} f_{h,k} &= 0.082 \cdot \rho_k \cdot d^{-0.3} \\ &= 0.082 \cdot 350 \cdot 3.4^{-0.3} = 19.9 N / mm^2 \end{aligned} \tag{3}$$

$$\begin{aligned} M_{y,k} &= 0.3 \cdot f_{u,k} \cdot d^{2.6} \\ &= 0.3 \cdot 600 \cdot 3.4^{2.6} = 4336 Nmm \end{aligned} \tag{4}$$

$$\begin{aligned} f_{ax,k} &= 20 \cdot 10^{-6} \cdot \rho_k^2 \\ &= 20 \cdot 10^{-6} \cdot 350^2 = 2.45 N / mm^2 \end{aligned} \tag{5}$$

3.1 MATERIAL PARAMETERS

Table 2 summarises the parameters determined for the elements from the trusses and compares them to the specifications according to EC 5. The values are always given as characteristic values. The exact number of samples and the coefficient of variation (COV) are also given.

Despite considerable signs of aging of both nails and wood, the test results meet the specifications of EC 5 regarding density, embedment strength and tensile strength.

(a) (b)

Figure 8: *(a) Tensile failure of old and (b) new nails*

In terms of tensile strength, it is noticeable that the new nails have a significantly higher characteristic value. This is probably due to the standardised production and better steel quality compared to the nails from the old truss. Only the yield moment and withdrawal strength exhibit shortcomings for the specimen taken from the aged truss.

Table 2: *Truss Type I-old: results of parameter testing – characteristic values*

Parameter			Value	n [-]	COV [%]
Density ρ_k [kg/m³]		chord	401	27	1.6
		strut	392	28	11.0
		EC5	350		
Embedment strength $f_{h,k}$ [N/mm²]		chord	22.1	16	4.0
		strut	21.0	16	20.6
		EC5	19.9		
Tensile strength $f_{u,k}$ [N/mm²]	nails ø = 3.4 mm	old	781	20	10.3
		new	1045	8	0.4
		EC5	600		
Yield moment $M_{y,k}$ [Nmm]		old	4102	10	5.4
		new	6212	10	1.1
		EC5	4336		
Withdrawal strength $f_{ax,k}$ [N/mm²]		old	2.22*	18	52.6
		new	2.94	10	13.0
		EC5	2.45		

** minimum value*

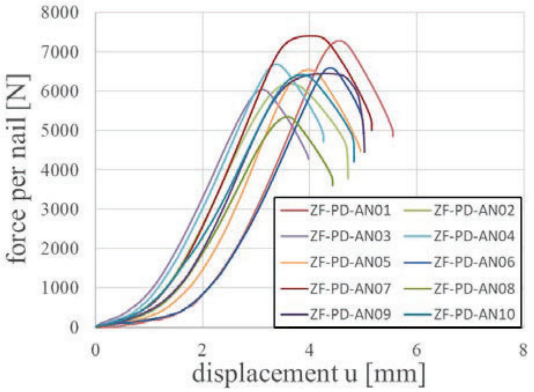


Figure 9: *Tensile strength test results with corroded nails*

Tensile strength

Three different test series were carried out to determine the tensile strength of the nails. Nails with corrosion were taken from the aged roof trusses for the first test series. The second series of tests was also on nails from aged roof trusses but with the difference that the nails were galvanized. This layer protected the nails from corrosion. The third test series was on new nails. When testing the tensile strength of the nails from the first test series, it was interesting to see failure always occurring in a zone of the nails which was originally situated close to the contact surface between the middle and side parts. This can be an indication of slightly higher corrosion in the small gap between the two timber parts. The specimens tested are depicted in Figure 8. The test results are shown in Figure 9 to 11.

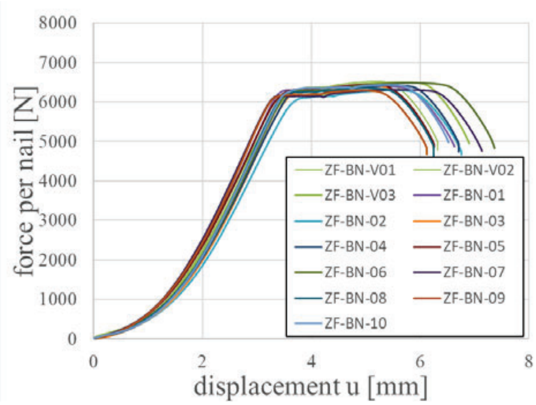


Figure 10: *Tensile strength test results with galvanised, aged nails*

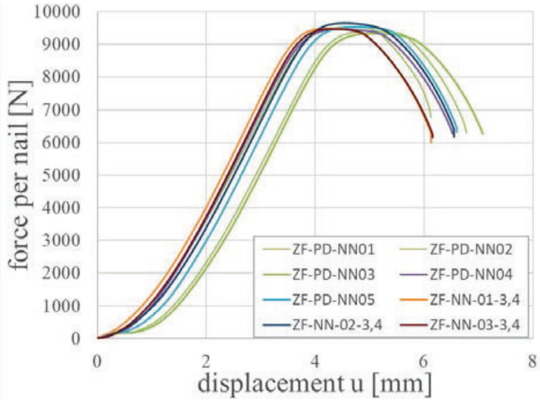


Figure 11: *Tensile strength test results with new nails*

Table 3: *Yield moment $M_{y,k}$ for old and new nails*

d = 3.40 *mm*, l_v = 655 mm			
old nails		new nails	
	$M_{y,k}$ *[Nmm]*		$M_{y,k}$ *[Nmm]*
FM-PD-AN01	3596	FM-PD-NN01	5964
FM-PD-AN02	3701	FM-PD-NN02	6213
FM-PD-AN03	2672	FM-PD-NN03	6220
FM-PD-AN04	2324	FM-PD-NN04	6391
FM-PD-AN05	3948	FM-PD-NN05	6539
FM-PD-AN06	3558	FM-PD-NN06	5894
FM-PD-AN07	2818	FM-PD-NN07	6399
FM-PD-AN08	2838	FM-PD-NN08	6364
FM-PD-AN09	3028	FM-PD-NN09	5945
FM-PD-AN10	3234	FM-PD-NN10	6192

Yield moment

A very homogeneous result with a low COV is observed for the yield moment, both for the old and new nails. It can be seen that the old nails only achieve about 2/3 of the yield moment of the new nails. The value achieved by the old nails corresponds to that calculated according to EC5. New nails, thus, achieve significantly higher yield moments. The result of each test is given in Table 3.

Table 4: *Withdrawal strength test results for old nails*

old nails					
	b [mm]	*t* [mm]	*d* [mm]	t_{pen} [mm]	f_{ax} [N/mm]
AF-AN-01	80.0	45.5	3.36	41.5	4.78
AF-AN-02	80.0	80.0	3.55	56.0	2.39
AF-AN-03	80.0	80.0	3.44	56.0	3.02
AF-AN-04	80.0	80.0	3.48	56.0	9.51
AF-AN-05	80.0	80.0	3.55	56.0	8.86
AF-AN-06	80.0	80.0	3.45	56.0	10.8
AF-AN-07	80.0	80.0	3.55	56.0	8.97
AF-AN-09	80.0	80.0	3.55	56.0	4.24
AF-BN-01	75.8	48.0	3.42	40.7	7.65
AF-BN-02	75.8	48.0	3.44	41.4	3.38
AF-BN-03	76.7	46.0	3.45	39.2	2.22
AF-BN-04	76.7	46.0	3.42	40.1	3.09
AF-BN-05	76.5	45.0	3.33	42.2	6.11
AF-BN-06	76.5	45.0	3.46	41.5	4.77
AF-BN-07	75.7	46.0	3.38	42.6	2.51
AF-BN-08	75.7	46.0	3.35	42.0	2.82
AF-BN-09	76.3	46.4	3.33	41.0	4.61
AF-BN-10	76.3	46.4	3.36	42.1	4.40

Withdrawal strength

The withdrawal strength is generally subject to a high degree of scattering, which is why the results are not very robust and, together with the characteristic value, the minimum value determined in the tests is given in Table 2. Consequently, further tests are required. Old nails were assigned their original diameter for the calculation of the withdrawal strength due to the small amount of corroded material. The result of each test for the old nails is given in Table 4. The test results for corroded nails taken from the first test series with the tests AF-AN-01 to -09 are shown in Figure 12. The test results for galvanized nails taken from the second test series with the tests AF-BN-01 to -10 are shown in Figure 13.

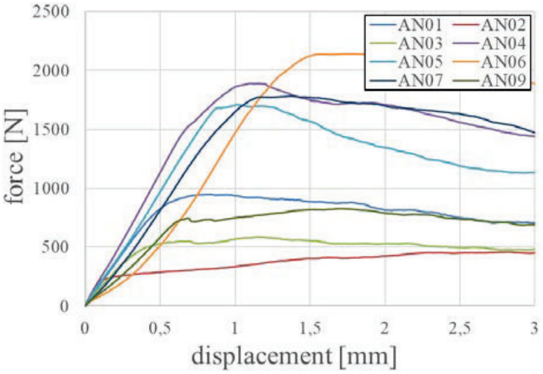


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Figure 12: Withdrawal strength test results with corroded nails

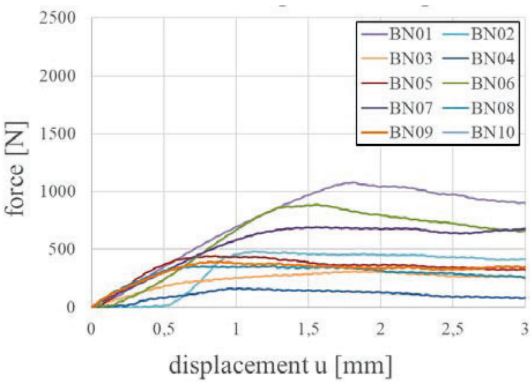

***Figure 13:** Withdrawal strength test results with galvanised, aged nails*

The test results made with new materials are given in Table 5 and shown in Figure 14.

***Table 5:** Withdrawal strength test result for old nails*

new nails			
b x t = 80 x 80 mm, *d = 3.40 mm*, t_{pen} = 56 mm			
Test	f_{ax} [N/mm^2]	Test	f_{ax} [N/mm^2]
AF-NN-01	5.47	AF-NN-05	4.12
AF-NN-02	12.0	AF-NN-06	3.51
AF-NN-03	6.55	AF-NN-07	5.75

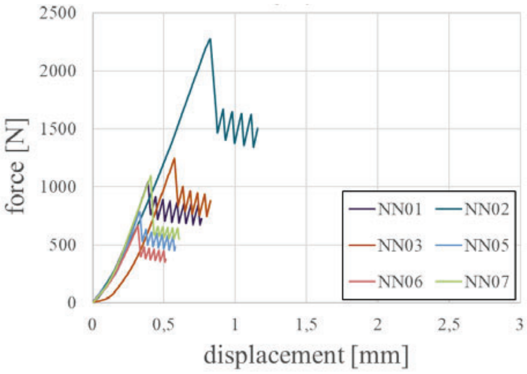

***Figure 14:** Withdrawal strength test results with new nails*

Embedment strength

The test specimens for the embedment strength test are shown in Figure 15. As specified in the standard, all tests were performed perpendicular to the grain. The characteristic value of the embedment strength is almost identical for the chord and the strut, but the strut shows a significantly higher scattering. Results are summarised in Table 2.

***Figure 15:** Embedment strength: specimens after the test*

3.2 TESTS ON JOINTS

Regarding the experimental tests on the connections, eight different nodes were cut out of two beams (A and B), and the posts and the struts were tested in each case. The results are summarised in Tables 6 to 9.

The connection of the struts could generally take a higher load per nail than the connection of the posts. This was most probably due to the larger contact area and the greater friction associated with it. Additionally, the spacing and the edge-distance of the fasteners were larger compared to the posts. Typical load-displacement curves of connections of struts and posts are depicted in Figure 16.

***Table 6:** Test results for Type I-new, Serie SV (PF = post)*

	α [°]	n_{Nail} [-]	F_{max} [N]	F_{max}/n_{Nail} [N]
SV-1947-PF-01	90	12	15010	1251
SV-1947-PF-02	90	12	13152	1096
SV-1947-PF-03	90	12	13447	1121
SV-1947-PF-04	90	12	14956	1246
SV-1947-PF-05	90	12	15248	1271
mean		**12**	**14363**	**1197**
SV-1969-PF-01	90	12	15748	1312
SV-1969-PF-02	90	12	13349	1112
SV-1969-PF-03	90	12	13084	1090
SV-1969-PF-04	90	12	18317	1526
SV-1969-PF-05	90	12	14041	1170
mean		**12**	**14908**	**1242**
SV-EC5-PF-01	90	14	18915	1351
SV-EC5-PF-02	90	14	21198	1514
SV-EC5-PF-03	90	14	19335	1381
SV-EC5-PF-04	90	14	21258	1518
SV-EC5-PF-05	90	14	22648	1618
mean		**14**	**20671**	**1476**

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Table 7: *Test results for Type I-old, Serie AH-PD-PF (PF = post)*

	n_{Nail} [-]	F_{max} [N]	F_{max} $/n_{Nail}$ [N]
AH-PD-PF-B04	12.0	26224	2185
AH-PD-PF-B06	12.0	25772	2148
AH-PD-PF-B07	12.0	18586	1549
AH-PD-PF-B08	12.0	19096	1591
AH-PD-PF-B09	12.0	16769	1397
AH-PD-PF-B10	12.0	23337	2122
AH-PD-PF-B11	12.0	18432	1536
AH-PD-PF-B12	12.0	13917	1265
mean-PF-B	**11.8**	**20267**	**1725**

Table 8: *Test results for Type I-new, Serie SV (ST = strut)*

	α [°]	n_{Nail} [-]	F_{max} [N]	F_{max} $/n_{Nail}$ [N]
SV-1947-ST-01	35	18	25274	1404
SV-1947-ST-02	35	18	26754	1486
SV-1947-ST-03	35	18	28799	1600
SV-1947-ST-04	35	18	24068	1337
SV-1947-ST-05	35	18	23161	1287
mean		**18**	**25611**	**1423**
SV-1969-ST-01	35	24	35145	1464
SV-1969-ST-02	35	24	37762	1573
SV-1969-ST-03	35	24	32429	1351
SV-1969-ST-04	35	24	35774	1491
SV-1969-ST-05	35	24	37358	1557
mean		**24**	**35694**	**1487**
SV-EC5-ST-01	35	22	27781	1263
SV-EC5-ST-02	35	22	31744	1443
SV-EC5-ST-03	35	22	31405	1428
SV-EC5-ST-04	35	22	31305	1423
SV-EC5-ST-05	35	22	32058	1457
mean		**12**	**30859**	**1403**

The comparison between the old joints and the new joints are shown in Figure 17. The average load-bearing capacity across all the tests of a series evaluated here can be considered similar for the old and the new connections. Nevertheless, the different deformation behaviour of the individual tests is clearly visible. The new nodes have significantly higher stiffnesses than the old ones. In addition, a hardening can be seen in the old nodes after about 5 mm of deformation. This could be due to a larger hole clearance in the old nodes or corrosion of the nails.

Table 9: *Test results for Type I-old, Serie AH-PD-ST (ST = strut)*

	α [°]	n_{Nail} [-]	F_{max} [N]	F_{max} $/n_{Nail}$ [N]
AH-PD-ST-B04	32	13,0	30703	2362
AH-PD-ST-B06	37	13,0	26925	2071
AH-PD-ST-B07	30	12,0	21789	1816
AH-PD-ST-B08	42	13,0	29097	2238
AH-PD-ST-B09	35	12,0	27206	2267
AH-PD-ST-B10	47	14,0	20094	1435
AH-PD-ST-B11	39	12,0	22935	1911
AH-PD-ST-B12	60	12,0	15701	1308
mean-B-ST		**12.6**	**24306**	**1925**

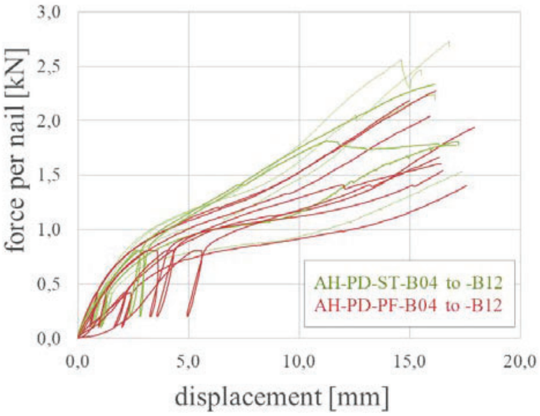

Figure 16: *Load-displacement curve – comparison between old strut and post*

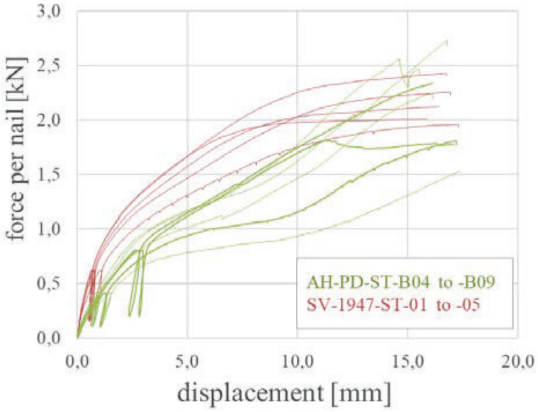

Figure 17: *Load-displacement curve – comparison between old and new joints*

The comparison between the load-bearing capacity per nail for Type I-old and I-new is presented in Table 10. The evaluation shows the higher load-bearing capacity of

Type I-old. Although the values of the old joints are scattered to a greater extent, the capacity of I-old connections exceeds that of the I-new connections slightly. The larger variation of Type I-old can be explained by different degradation conditions. Otherwise, it became apparent that Type I-new performed better in terms of initial stiffness.

The initial assumption that the “ageing” of joints could improve the load-bearing capacity due to nail corrosion seems to be partially confirmed, especially regarding the significantly lower tensile strengths and yield moments of the old nails in the testing of the individual parameters. Despite these lower strengths, higher load-bearing capacities can be achieved than with the new joints. However, it can also be seen that the assumed effect of the load increase does not seem to be significant at all. There is a clear scattering of the results, which refers mainly to the parameter testing. further investigations are still required to be able to assess whether aged constructions with corroded nails always have a higher load-bearing capacity.

Table 10: *Load capacity per nail for Types I-old and I-new with characteristic and mean values*

Type		nails [-]	$F_{u,m}$ [N]	n [-]	COV [%]	$F_{u,k}$ [N]
I-old	post	11.8	1724	8	21.4	920
	strut	12.6	1926	8	20.2	1078
	total	12.2	1825	16	20.1	1109
	EC5					969
I-new	post	12.7	1305	15	6.8	1066
	strut	10.7	1438	15	8.7	1257
	total	11.7	1371	30	7.7	1175
	EC5					836

4 CONCLUSIONS AND OUTLOOK

Test on connections which were taken from old, nailed trusses showed a good agreement between experimental results and characteristic resistance according to EC 5. This also applies to the relevant parameters, which are the tension strength and yield moment of the nails and embedment strength of the timber. Nevertheless, a positive effect of aging, such as an increase of the rope effect due to the slightly corroded nails, could not be confirmed. On the other hand, no negative effect of aging could be detected.

The large scattering of the withdrawal strength for the old connections indicates that this phenomenon should be studied in more detail. Another observation concerns the different condition of the joints in terms of spacing and edge distance, as well as cracks in the timber. An assessment schema to consider different forms and levels of irregularities is presented by Völlmecke et al. [11].

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https://doi.org/10.52202/069179-0529