NOVEL TIMBER-CONCRETE COMPOSITE SLABS USING BEECH DOWELS AS CONNECTION ELEMENTS

Michael Mikoschek-Muggendorfer[1], François Colling[2]

ABSTRACT: Timber-concrete composite slabs are state of the art since many years and may now also find their way into the upcoming EUROCODE 5 with the technical specification CEN/TS 19103. In particular, dowel-type fasteners and notches as a composite solution with screws as lift-off protection are described in detail in this standard for design and application. However, the large number of screws required is leading to an increased effort and higher building costs. An alternative to this is a novel timber-concrete composite system using beech wood dowels. This composite solution enables a high degree of prefabrication and more efficient use thanks to shorter construction times. Further increases in efficiency are achieved by dispensing reinforcement of the concrete layer, which also leads to ecological advantages and should make it easier to dismantle. Since the rules for dowel-type fasteners given in CEN/TS 19103 apply only to metallic fasteners, extensive experimental and numerical investigations were carried out with regard to the load-bearing behaviour of the novel composite solutions.

KEYWORDS: timber-concrete composite, TCC, beech dowels, dowel laminated timber, DLT

1 – INTRODUCTION

Timber-concrete composite (TCC) construction is an efficient building method that combines timber and concrete to leverage the best properties of both materials. Timber offers sustainability, light weight, and flexibility, while concrete provides stiffness, durability, and fire resistance. In TCC systems, a concrete layer is connected to timber elements, such as beams or plates. This combination results in enhanced structural performance, reduced material usage, and improved stiffness and acoustic properties, making TCC an attractive solution for modern slab constructions.

TCC systems have evolved over the past century as a solution to improve structural performance in construction. The concept of combining timber and concrete dates back to the early 20th century, when engineers sought to replace expensive steel in reinforced concrete. Later TCC constructions were used to enhance the load-bearing capacity and durability of existing timber structures, particularly in slab systems. During the mid-to-late 20th century, research into TCC systems expanded significantly in Europe, particularly in countries like Germany, Switzerland, and Austria. These nations pioneered the development of modern TCC techniques, leading to optimized structural designs and improved load transfer efficiency.

The experiences gained from research and development over the past decades have led to the first European design standard for TCC constructions, which was published in 2022 as the technical specification CEN/TS 19103 [1] and is intended to be included as another part of the EUROCODE 5 [2].

2 – BACKGROUND

Different connection types, such as notched connections, glued interfaces, and mechanical fasteners (e.g., screws or rods) are commonly used as connection, securing the load transfer between timber element and concrete. The choice of connection type depends on factors such as construction requirements, load conditions, and economic aspects. However, the large amount of steel connectors used in TCC slabs also emphasizes ecological aspects. For this reason, the utilisation of shear connection using dowels made from beech wood is an essential motivation of the presented

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research. A further aim is the reduction of steel reinforcement, since the timber elements mostly carry the tensile forces. Furthermore, the overall absence of steel components could allow for a reduction in the concrete layer. Therefore, new types of timber-concrete composite systems were developed and first presented in [3, 4], which are described below.

2.1 NOVEL TIMBER-CONCRETE COMPOSITE SYSTEMS

The basis of the investigated TCC systems is a dowel laminated timber (DLT) element whose timber profiles are milled to a trapezoidal shape on the top side. This special design allows beech wood dowels to be inserted perpendicular to the span direction, resulting in a grid-like shape. Casting of the timber grid structure with concrete results in a form-fitting bond.

Two types of TCC systems were examined in this project: Type DB1, where beech wood dowels are responsible for the transfer of shear forces between timber and concrete, and type DB2, where beech wood dowels are only used to secure the uplift force of notches that are milled into the top of the DLT element. Both types are shown in Fig. 1.

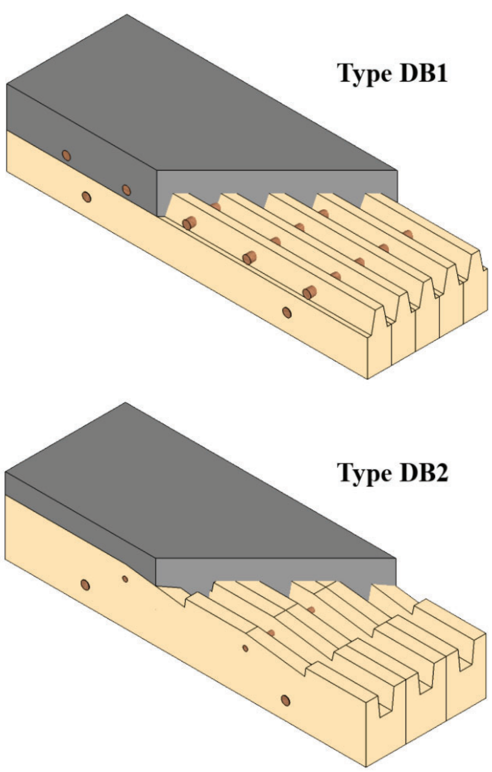

Figure 1. TCC systems with beech wood dowel connections.

In type DB1 the dowel diameter d_D is 20 mm, with a 10 mm vertical gap between the lower edge of the dowel and the deep groove of the timber profile. For type DB2, a dowel with 12 mm diameter takes the uplift force of the wedge-shaped notch (150 mm length and 20 mm depth). Each notch pair has a load-bearing area A_n of 1200 mm².

2.2 MATERIALS

The timber profiles of the DLT elements are made out of solid wood (spruce, picea abies) with an average density of 441 kg/m³ (COV = 11.5 %) classified as strength class C24 according to EN 338 [4]. The dowels made of defect-free and knot-free beech wood (fagus sylvatica) had an average bulk density of 714 kg/m³ (COV = 4.1 %). Since in Europe no strength classes are defined for hardwood dowels, strength class D45 could be applicable according to [5] based on the bulk density.

To minimize the swelling of the timber profiles and the beech wood dowels in fresh concrete, they were hydrophobized before concreting. This treatment roughly halved the wood swelling during the first 12 hours in fresh concrete.

For the concrete layer, a flowing concrete with a maximum aggregate size of 8 mm was used. This was processed as in-situ concrete by mixing a dry concrete mixture with the prescribed amount of water (w/c ratio = 0,64) via silo mixing pumps. According to the manufacturer, the concrete is classified as strength class C25/30 [6]. However, in tests often significantly higher average compressive strengths (up to 54 N/mm²) were obtained. The modulus of elasticity of the concrete was in the range of 25,000 N/mm². Due to its excellent flow properties (consistency class F5 according to [7]), all deep grooves of the timber profiles were completely filled, and the dowels were seamlessly encased. Practical trials have shown that manual compaction of the concrete surface using trowels or "wobble bars" is usually sufficient.

3 – PROJECT DESCRIPTION

The stiffness properties and load-bearing capacity of the connection significantly influence the stress distribution in the cross-sections and the ultimate load of the whole construction. Thus, these properties are fundamental for the design of TCC constructions. Since beech wood dowels have not yet been used for such applications and only limited fundamental data is available, their properties have to be experimentally determined and verified.

As previously mentioned, the concrete slab is constructed without any steel reinforcement. Only in particularly

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highly stressed areas and at the slab edges appropriate reinforcement meshes should be installed, depending on structural requirements. According to EUROCODE 2 [8], the tensile strength of concrete in unreinforced concrete components may be considered in the ultimate limit state, provided that it can be demonstrated - either through calculation or testing - that brittle failure is excluded and sufficient load-bearing capacity is ensured. To what extent this applies and whether the novel TCC system can be designed using the established methods and regulations of the Technical Specification CEN/TS 19103 [1] (hereinafter abbreviated as "TS") and other standards, was examined within the following tests:

- Bending tests with beech dowels after storage in fresh concrete.
- Push-out shear tests with single dowels in small timber-concrete-timber specimen.
- Slip-block shear tests with TCC specimen.
- Three-point bending tests with TCC elements.

The experimental investigations were accompanied by calculations. Overall, this paper focuses on the bending load-bearing behaviour and presents only a segment of the whole project. The shear load-bearing behaviour is part of further projects.

4 – EXPERIMENTAL PROGRAM

4.1 DOWEL BENDING TESTS

Beech wood is known for its high bending strength [9], but it is also particularly susceptible to significant swelling and shrinkage deformations. For this reason, a hydrophobization process was developed to reduce water absorption from the fresh concrete. This approach halved the swelling rate within the first 12 hours. However, moisture measurements on treated beech wood dowels embedded in fresh concrete showed that the MC can still rise to up to 20 % within the first 3–4 days and then only slowly return to a lower MC level. Therefore, the impact of moisture exposure on the strength was examined in dowel bending tests. The tests were carried out according to DIN 52186 [10] on beech wood dowels with a diameter of 12 mm and 20 mm. For each diameter a total of eight hydrophobized dowels were placed in a mortar tray and embedded in fresh concrete. After four days, the dowels were broken out of the concrete and tested immediately. The testing setup is pictured in Fig. 2a).

4.2 DOWEL SHEAR TESTS

Since the dowels are primarily subjected to shear stress rather than bending stress, small-scale shear tests were conducted to determine the slip modulus k_s and load-bearing capacity F_{max} according to EN 26891 [11]. A push-out test setup was chosen (see Fig. 2b), in which two timber boards (t = 33 mm) were placed on the outside, with concrete cast in the middle (t = 34 mm). Each test specimen contained a single dowel. Dowels with diameter d_D = 12 mm and 20 mm were used. The load was applied once perpendicular to the grain (α = 90°) of the timber boards and once parallel to the grain (α = 0°), with six specimen for each dowel diameter and load direction. The timber boards were coated with industrial grease at the contact surfaces with the concrete to minimize bond adhesion and friction. The tests were conducted after a concrete curing period of 3 to 4 weeks.

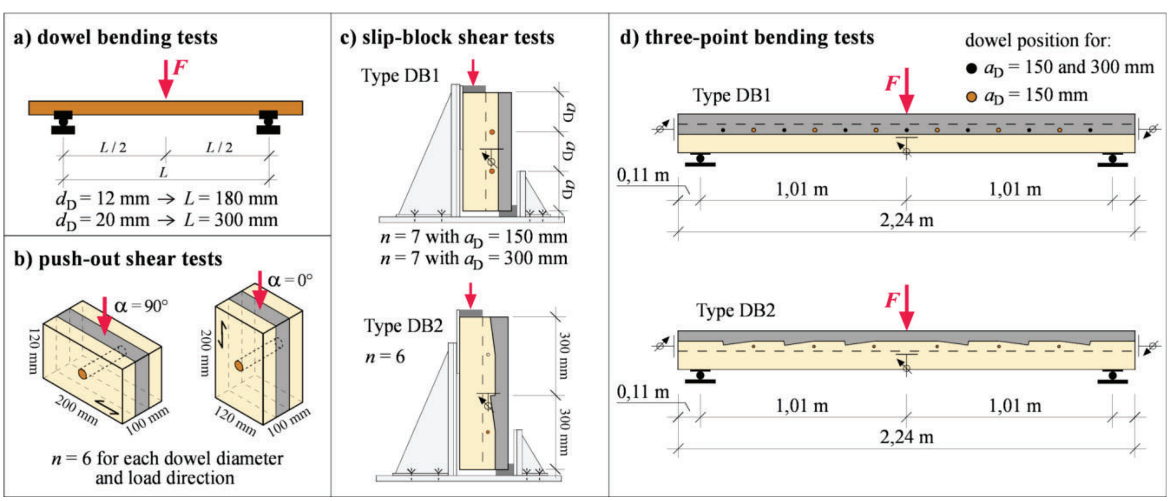

Figure 2. Overview of the testing program and test setups.

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4.3 SLIP-BLOCK SHEAR TESTS

To investigate the load-bearing and slip behaviour of the two TCC types, larger shear tests were conducted. For the sake of simplifying the specimen production, the same timber profiles used for type DB2 were employed for the specimens of type DB1. For type DB1, two dowels were used per specimen with dowel spacing a_D = 150 mm and 300 mm. In the test specimens of type DB2, each dowel segment was combined with a notch pair. A potential weakening effect due to an adjacent notch pair was accounted for by cutting the timber profile (creating a wedge shape) and pre-drilling a dowel hole with 300 mm spacing. The thickness of the concrete layer (excluding deep groves) was 50 mm and the specimen width 10 cm. Upon delivery, the timber profiles had a MC of approx. 14.5 %. At the time of testing (14 days of concrete curing), MCs of 12.3–15.2 % were measured in the lower wooden areas, while MCs of 16.4–19.4 % were recorded in the composite zones. In order to minimize the adhesive bond between wood and concrete as much as possible, the contact areas of the wooden profiles were treated with teflon spray and then coated with industrial grease before concreting. For each test variant, six specimens were produced and tested. The shear tests were carried out using a vertical slip-block test setup (see Fig. 2c) following the loading regime of EN 26891 [11].

4.4 THREE-POINT BENDING TESTS

In addition to the shear tests, smaller bending tests were conducted as three-point bending tests (see Fig. 2d). The aim was to examine the behaviour of the components and to assess the influence of the unreinforced concrete. Additionally, the results of the shear tests were to be verified, as every type of shear-test setup has its weaknesses and inaccuracies [12]. Another reason is that, compared to large-scale bending tests (usually four-point bending tests), less material is required in a three-point bending test, which allows a shorter span width. However, this load distribution increases the risk of bending-shear failure, but it also provides more consistent results regarding the effect of the fasteners' stiffness, as the shear flow is uniformly distributed across the entire shear plane between concrete and the DLT elements. For this reason, an even distribution of the dowels was chosen along the entire length.

The width of the DLT elements was 36 cm (= six timber profiles) for type DB1 and 40 cm (= four timber profiles) for type DB2. The concrete height was 5 cm above the timber profiles, which had a height of 14 cm. After a concrete curing time of two weeks, one specimen with dowel spacing a_D of 150 mm and 300 mm was tested for type DB1. For type DB2, two tests were performed.

5 – RESULTS

The key results of the conducted investigations are presented below, categorized into:

- Local load-bearing behaviour (based on dowel bending tests and small-scale push-out shear tests).
- Global load-bearing behaviour (based on slip-block shear tests and three-point bending tests).

5.1 LOCAL LOAD-BEARING BEHAVIOR

The bending tests showed a significant influence of moisture exposure from the fresh concrete (see results in Fig. 3). With an average bending strength of 94.6 N/mm² (for d_D = 12 mm) and 98,9 N/mm² (for d_D = 20 mm), the achieved strengths were approx. 20 % lower than the expected value of 120 N/mm² specified in [9] under standard climate conditions. With a coefficient of variation (COV) of approx. 15 %, the values are within a typical range. Further insights into the influence of concrete on the load-bearing capacity of beech dowel connections were gained from the dowel shear tests. The results of the dowel shear test are summarised in Tab. 1 and Fig. 4 (only for slip modulus k_s).

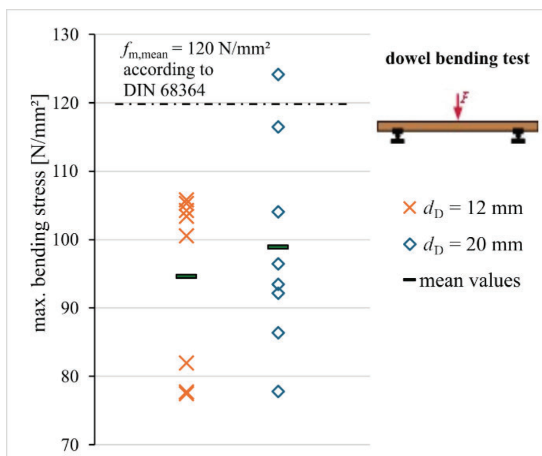

Figure 3. Results of the dowel bending tests.

Table 1: Results of the dowel shear tests (COV in brackets).

d_D [mm]	α [°]	$F_{max,mean}$ [kN]	$k_{s,mean}$ [N/mm]
12	0	6.42 (0.09)	5630 (0.34)
	90	6.13 (0.11)	4616 (0.31)
20	0	14.21 (0.15)	12740 (0.34)
	90	12.43 (0.10)	15521 (0.25)

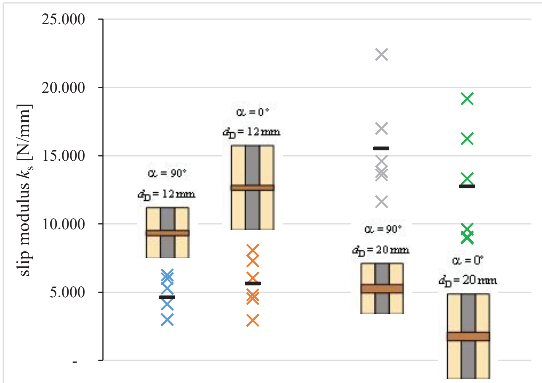

Figure 4. Results of the dowel shear tests.

It is clearly visible that the slip modulus and load-bearing capacity increases significantly with a larger dowel diameter d_D, while the load direction has no major impact. This indicates that the transverse compression and shear deformation of the dowel influenced the results mainly. All dowels exhibited plastic hinges, significant shear deformations and transverse compression indentations along with bearing deformations in the timber boards (see Fig. 5). In some cases, signs of bending failure on the dowels were also observed.

For the calculation of the characteristic dowel load-bearing capacity R_k, a design equation for oak wood dowels and nails is given in (1) according to [13]. It is limited for dowel diameters from 20 to 30 mm.

$$R_k = 9.5 \cdot d_D^2 \tag{1}$$

From (1), it is evident that the influence of the angle between the force and the wood grain can be neglected, confirming the observations from the shear tests. However, the equation was apparently derived empirically for timber-timber connections, meaning that the higher bearing strength of concrete is not considered, making it unsuitable for the design of timber-concrete connections.

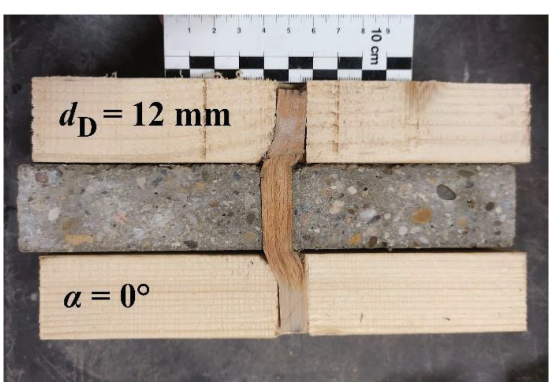

Figure 5. Deformations after a dowel shear test.

In [14], several tests were carried out on connections with wooden dowels, and design equations (see Eq. 2–4) were derived based on Johansen's theory [15].

$$R_k = \sqrt{\frac{2 \cdot \beta}{1+\beta}} \cdot \sqrt{2 \cdot M_{u,k} \cdot \delta \cdot f_{h,1,k} \cdot d_D} \tag{2}$$

$$\beta = f_{h,2,k} / f_{h,1,k} \tag{3}$$

$$M_{u,k} = \frac{f_{m,k} \cdot \pi \cdot d_D^3}{32} \tag{4}$$

Here, the dowel load-bearing capacity R_k depends primarily on the dowel diameter d_D, the bending capacity $M_{u,k}$ of the dowels, and the embedment strength $f_{h,1,k}$ of the timber boards. The coefficient β accounts for differences in embedment strengths (see Eq. 3). In (4) $f_{m,k}$ is the characteristic bending strength of the dowel. The coefficient δ (= 0.75) considers the uneven stress distribution, the incomplete utilization of embedment strength due to premature dowel failure, as well as the effects of friction and rope mechanisms. The embedment strength $f_{h,1,k}$ can be derived according to (5) as specified in [14], while the TS [1] prescribes the determination of the embedment strength of concrete according to (6).

$$f_{h,1,k} = \rho_{D,k} \cdot \rho_{H,k} \cdot 10^{-4} \cdot 1.1 \cdot (1 - 0.01 \cdot d_D) \tag{5}$$

$$f_{h,2,k} = 3 \cdot f_{ck} \tag{6}$$

In contrast to the equations of EUROCODE 5 [2], the density of the dowel $\rho_{D,k}$ is considered in addition to the density $\rho_{H,k}$ of the timber boards. The TS [1] simplifies the embedment strength $f_{h,2,k}$ as three times the characteristic compressive strength of the concrete. Due to the high concrete embedment strength, a reduction of the load-bearing capacity R_k may be necessary for a dowel diameter of 20 mm if the board thickness t_1 is insufficient. The required thickness $t_{1,req}$ is to be calculated according to [14] with (7).

$$t_{1,req} = \left(2 \cdot \sqrt{\frac{\beta}{1+\beta}} + 2\right) \cdot \sqrt{\frac{M_{u,k}}{\delta \cdot f_{h,1,k} \cdot d_D}} \tag{7}$$

Considering the determined densities of the dowels and wooden boards, as well as the average bending strengths from the dowel bending tests, the calculated load-bearing capacities were compared with the max. loads from the dowel shear tests. For f_{ck}, the mean concrete compressive strength of 33 N/mm² (for C25/30) was assumed according to EUROCODE 2 [8]. The comparison of the values is illustrated in Fig. 6.

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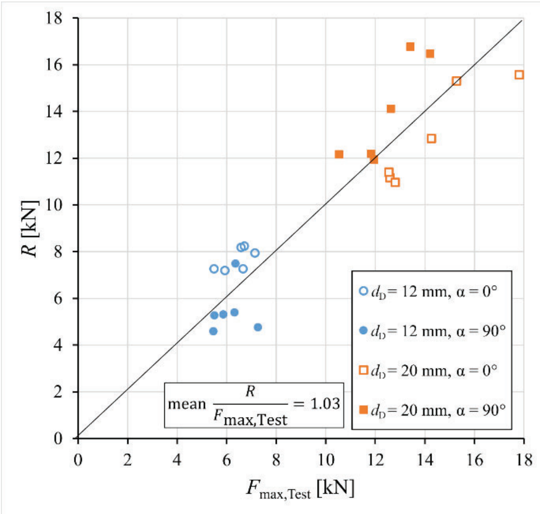

Figure 6. Comparison of the calculated load-bearing capacities R and the maximum values F_{max} from the dowel shear tests.

The good agreement of the values demonstrates the suitability of equations (2–7) for the design the dowel load-bearing capacities. However, these equations do not account for potential concrete failure due to splitting. This issue was addressed in the further investigations described below.

5.2 GLOBAL LOAD-BEARING BEHAVIOR

To assess the load-bearing and deformation behaviour of entire TCC components for types DB1 and DB2, the slip-block shear tests shown in Fig. 2 and described in chapter 4.3 were conducted. The load-slip curves of the tests are summarized in Fig. 7. For comparability, the curves of all tests in a series were averaged into a single curve. More detailed results are listed in Tab. 2.

In type DB2, nearly twice the load-bearing capacity F_{max} and slip-modulus k_s was achieved compared to type DB1. However, it must be noted that significant friction effects were observed in the test series of type DB1 up to a test load of 5 kN. For this reason, the slip-modulus k_s of type DB1 has been evaluated in the range of 6 to 12 kN.

Table 2: Results (mean values) of the slip-block shear tests (COV in brackets).

Type	$F_{max,mean}$ [kN]	$k_{s,mean}$ [N/mm]
DB1, a_D = 150 mm[1)]	22.2 (0.18)	31442 (0.23)
DB1, a_D = 300 mm[1)]	23.7 (0.09)	25031 (0.22)
DB2	41.2 (0.09)	60917 (0.12)

[1)] The slip-modulus k_s was evaluated in the range of 6 to 12 kN.

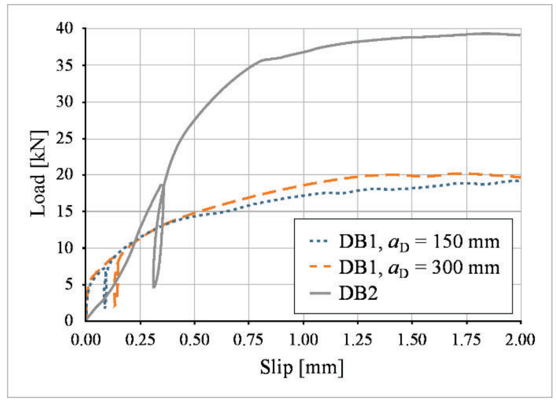

Figure 7. Averaged load-slip curves of the slip-block tests.

In test series DB2, no friction effects were observed, which may be attributed to a smoother wood surface. In both test series of type DB1, surface irregularities were noted after testing, leading to an adhesive bond between wood and concrete. Two additional tests with sanded wood surfaces confirmed this presumption.

Since two dowels were installed for each specimen in the test series of type DB1, it is plausible that the evaluated slip-modulus is roughly twice as high as the slip-modulus of a single dowel with a diameter of 20 mm (see Tab. 1). However, this does not apply to the load capacities. The average load-bearing capacity from the dowel shear tests was utilized to only approx. 80 %. This can be attributed to the observation that premature concrete splitting (see Fig. 8a) at the dowels typically occurred before the load-bearing capacity of the beech dowels could be fully utilized. Nevertheless, significant dowel deformations were evident (see Fig. 8b).

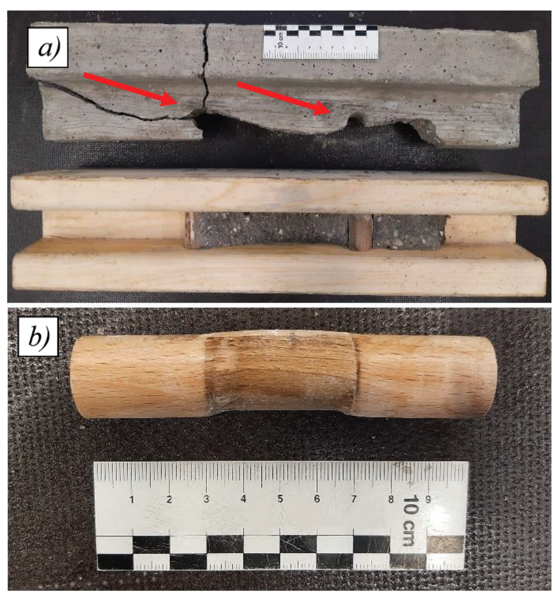

Figure 8. Signs of failure in test series DB1.

Although concrete splitting failure is considered brittle and no reinforcement is present at the relevant locations, the load-slip curves showed no significant load drops. For this reason, the exact moment of splitting failure in the shear tests is difficult to determine. To define the concrete splitting load-bearing capacity more precisely, additional concrete splitting tests were conducted on steel dowels ($d_D = 20$ mm). At a concrete age of 14 days and a dowel spacing of 150 mm (equal to the splitting length), an average load-bearing capacity of 15.77 kN (COV = 0.37) was determined. With a dowel spacing of 300 mm, the average load-bearing capacity was 13.45 kN (COV = 0.35). Apparently, the dowel spacing has no significant impact on the load-bearing capacities. It has to be noted, that the high scatter of the results even exceeds the usual coefficient of variation of 0.30 for concrete tensile strength published by the JCSS [16]. This indicates challenges in designing the connection. EUROCODE 2 [8] provides an option (see Eq. 8–9) for the design of compression struts that takes into account the presence of transverse tension (= splitting) and cracks in the concrete.

$$\sigma_{\mathrm{Rd,max}} = 0.6 \cdot \nu \cdot f_{\mathrm{cd}} \tag{8}$$

$$\nu = 1 - f_{\mathrm{ck}} / 250 \tag{9}$$

The reduction factor ν depends on the characteristic concrete compression strength f_{ck} (= 25 N/mm²) and has to be multiplied with the design compression strength f_{cd}. To determine the concrete load-bearing capacity of a dowel, the permissible concrete stress $\sigma_{Rd,max}$ must be multiplied by the dowel compression area $A_{D,c}$. Based on measurements of tested dowels from the shear tests, it is suggested to assume half the dowel diameter as the height of the compression area. In type DB1, the dowels are embedded in the concrete over a width of 28 mm, which, when multiplied by half the dowel diameter, results in a dowel compression area $A_{D,c}$ of 280 mm². Using the concretes mean compression strength f_{cm} (= 33 N/mm²) instead of the design value in (8), this results in a mean concrete dowel load-bearing capacity of 4990 N. Comparing this value with the load capacities from the splitting tests, this approach appears to be rather conservative. However, due to the high scatter, this may be appropriate.

The failure modes in the shear tests for type DB2 were predominantly characterized by wood failures. Initial compression wrinkles in the wood fibres within the notch area typically appeared at a test load of approx. 35 kN (see Fig. 9a). Although minor cracks were observed around 32 kN in the concrete near the notch flanks (Fig. 9b), they did not lead to failure.

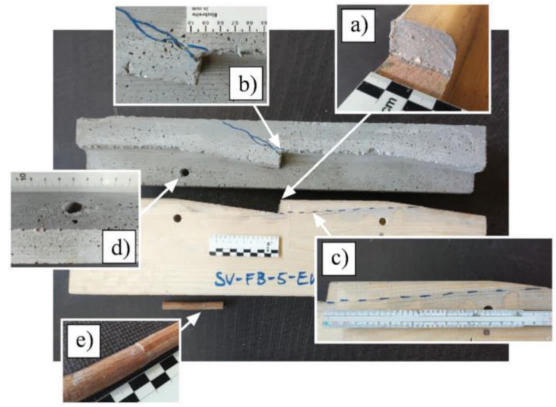

Figure 9. Signs of failure in test series DB2.

As loading progressed and displacement increased significantly, final failure due to wood shear occurred in some cases (see Fig. 9c). The concrete surrounding the dowel was undamaged in all cases (Fig. 9d), and only the dowels showed signs of deformation (Fig. 9e). However, the potential lifting of the concrete due to the eccentric moment was prevented by the experimental setup. For this reason, additional transverse tension tests were conducted, which are described more detailed in [4]. The comparison of the experimental results with the calculated load-bearing capacities based on the so-called CC-method [17] (also referenced in EUROCODE 2 – Part 4 [18]), showed that it provides a very suitable approach for the design of the dowel/concrete against uplift forces. Also described in [4] is the good agreement between the shear test results of type DB2 and the design approaches from the TS. For example, the slip-modulus for notches proposed in the TS corresponds accurately with the determined value k_s in Tab. 2. The wood compression failure at the notch could also be precisely calculated. Multiplying the notch load-bearing area A_n of 1200 mm² with the assumed wood compression strength of 29 N/mm² (mean value with a MC of 20 % according to [19]) gives the failure load 34.8 kN, which aligns well to the observations of the slip-block shear tests. Whether this also applies to larger TCC components, was examined in the three-point bending tests (TPBT) described below.

The load-deflection curves of the TPBT after a concrete curing time of two weeks are shown in Fig. 10. Additionally, the calculated load-deflection lines for a rigid bond ($\gamma = 1$) and a timber only cross-section (b/h = 400/140 mm) are shown with black dashed and dotted lines. Furthermore, a square marker indicates the load at which the timber only cross-section reaches a bending stress σ_M of 24 N/mm², corresponding to the characteristic bending strength of C24 timber.

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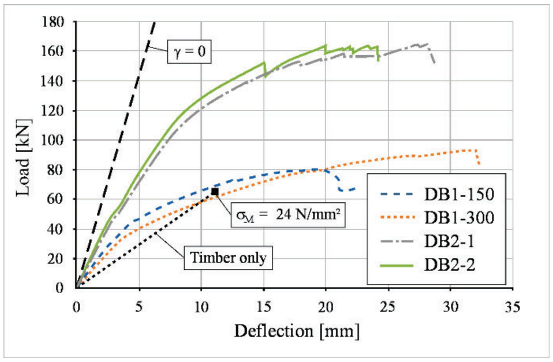

Figure 10. Load-deflection curves of the TPBT.

As in the shear tests, type DB2 exhibited significantly higher stiffness and load-bearing capacity. To better classify and compare the effective bending stiffness EI_{eff}, it was evaluated based on the test results in the range of 10 to 30 kN and converted to a one-meter slab width. Additionally, the composite factor γ (derived from the γ-method in EUROCODE 5 [2]) was evaluated based on the bending stiffness EI_{i} and axial stiffness EA_{i} of each partial cross-section (i = 1 for the concrete cross-section, i = 2 for DLT cross-section) according to (10). The authors acknowledge that the γ-method has certain limitations and may produce inaccurate results for a point load and short spans. However, for the comparability of the composite effectiveness and the assessment of the test results, this method is sufficient. A γ-value of 1 corresponds to a rigid connection, while a value of 0 represents two separate parts without composite action.

$$\gamma = \frac{-EA_2 \cdot \left(EI_1 + EI_2 - EI_{\mathrm{eff}}\right)}{EA_1 \cdot \left(a^2 \cdot EA_2 + EI_1 + EI_2 - EI_{\mathrm{eff}}\right)} \tag{10}$$

For the DLT elements, a Young's modulus E of 11000 N/mm² (according to strength class C24) was assumed. Tests on concrete cylinders showed a Young's modulus around 25000 N/mm². The variable a represents the distance between the centroids of the partial cross-sections. The results of tests and the analysis are summarized in Tab. 3.

Table 3: Results of the three-point bending tests.

Specimen	$F_{\mathrm{max,mean}}$ [kN]	EI_{eff} [1)] [kNm²]	γ
DB1-150	80.8	5008.4	0.41
DB1-300	93.1	4249.4	0.31
DB2-1	164.7	7104.5	0.57
DB2-2	163.8	7722.8	0.70

[1)] EI_{eff} was evaluated in the range of 10 to 30 kN and converted to a slab width of one meter.

The tests on type DB2 show a typical composite factor γ for notches, which usually ranges between 0.6 and 0.8. Specimen DB1-150 shows, as expected, a more effective composite action than DB1-300, as more dowels were installed in the specimen. The composite factor γ for type DB1 may be compared to that of common TCC screws.

For the test specimen DB1-150, initial concrete cracks at the dowel were observed at a load of 45 kN due to tensile splitting stress. As the test progressed, this led to shear failure of the concrete and lifting of the concrete slab (see Fig. 11a). DB1-300 exhibited similar behaviour from a test load of 35 kN onward, but without significant lifting of the concrete slab (see Fig. 11b). However, more pronounced signs of stress on the dowels were visible after the test. The failure mechanisms of the concrete were also evident in the flattening of both load-deflection curves and slowly aligned to the one of the fictitious line for timber only. Additionally, the measured deformation at the front faces increased significantly. The max. test load was determined by the bending failure of the DLT elements and was only slightly higher than the marked point indicating the exceedance of the char. bending strength (= 24 N/mm²). This clearly demonstrates the complete failure of the DB1 dowel connection.

In both tests of type DB2, initial concrete cracks at the notch flank were observed from a test load of 115 kN onward. These cracks continued to widen as the test progressed, and lifting of the concrete slab was also visible (see Fig. 12a). This suggests a failure of the concrete around the dowel responsible for uplift prevention. Nevertheless, significant compression of the wood fibres in the notch area occurred as the test continued, and in some cases, the concrete failed under compression (see Fig. 12b).

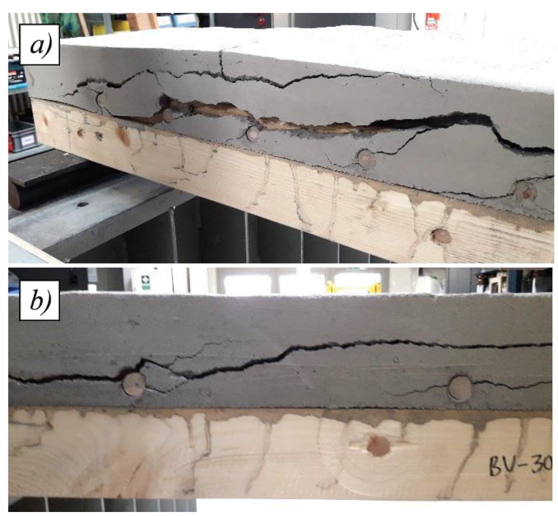

Figure 11. Concrete failure of specimen DB1-150 a) and DB1-300 b).

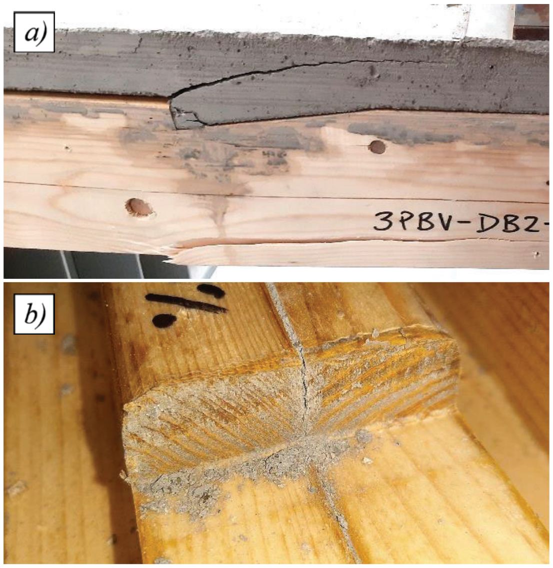

Figure 12. Signs of failure in the TPBT specimen DB2-2.

Since the γ-method may produces inaccurate results, a framework model according to [20] was applied for the analytical estimation of the dowel forces of the DB1-type tests. The calculation showed that in both tests, at the critical loads (35 and 45 kN), the max. dowel force ranged between 4.6 and 5.1 kN. This corresponds to the average load-bearing capacity of a dowel section calculated according to (8) and (9).

The determination of the notch forces was carried out using the truss model described in [4], which also allows for the calculation of the uplifting forces in the dowels. The calculations showed that at the critical load of 115 kN, the max. notch force is approximately 30 kN, and the max. dowel force is 2.6 kN. This dowel load exceeds the maximum load-bearing capacity determined in the investigations described in [4], showing a very good correlation with the observed test results (lifting of the concrete slab). The calculated notch force is close to the max. loads determined from the shear tests and may explain the flattening of the load-deflection curves.

6 – CONCLUSION

The connection type DB2 exhibited significantly superior load-bearing and deformation behaviour compared to type DB1. Additionally, the predominantly ductile wood compression failure at the notches resulted in a more gradual load-bearing response after reaching the maximum capacity. In contrast, the dowels in DB1 could cause brittle splitting and shearing of the concrete. While the damaged concrete areas were still able to "hook" into the dowels and maintain some residual composite action, this is considered critical from a design perspective, especially in combination with unreinforced concrete. For this reason, it is recommended to use type DB1 only for verification of the serviceability limit state (deflection and vibrations). The verification of the ultimate limit state could, for a certain span, be made solely based on the DLT elements only.

Type DB2, on the other hand, exhibits a very ductile bending load-bearing behaviour despite the unreinforced concrete design, with a load increase of more than a factor of 2.5 compared to timber only. According to EUROCODE 2, the tensile strength of unreinforced concrete may be considered in the ultimate limit state if it is demonstrated either by calculation or experiment that brittle failure can be excluded, and sufficient load-bearing capacity is present. As proven in the described investigations, TCC system type DB2 can be verified without reinforcement of the concrete. In any case, the tensile zone in the concrete layer must be neglected according to the TS [1], so only the compression area is considered in the calculations. Moreover, reinforcement in a bending-stressed TCC slab is rarely activated when considering the maximum strains [21]. For the design of the notches and the uplift prevention using beech wood dowels, the rules of the TS [1], EUROCODE 2 [18] and the truss model presented in [4] can be applied. However, in the case of concentrated loads, local reinforcement of the concrete slab or additional screws for uplift prevention may be necessary [22].

Although the load-bearing capacity of the unreinforced concrete is decisive for both composite types, the design concept from [14] based on Johannsen's theory provides sufficiently accurate results for the load-bearing behaviour of the beech wood dowel in TCC connections. However, it is recommended to adjust the strength values of the beech wood dowels (e. g. according to service class 3 of EUROCODE 5 [1]), since the wood moisture content decreases only slowly below 20 %.

Overall, significant frictional effects can arise due to the profiling of the DLT elements. In [23], for example, DLT elements with alternately high boards and a sawed surface are intentionally used for the friction connection to the concrete slab. According to the TS [1], this is not permitted, and long-term tests have shown that over time, the friction connection can weaken.

Three-point bending tests offer several advantages over shear tests for determining the bond stiffness and load-bearing capacity. They provide a more realistic load distribution, leading to a better simulation of the bending stresses in TCC structures. The deformation can be directly measured through deflection, making it easier to evaluate bond stiffness. Additionally, the shear stress is

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distributed more evenly, leading to more representative results. These tests also allow for observation of concrete cracking, giving better insight into failure mechanisms. Furthermore, they require a simpler test setup compared to shear tests. However, shear tests remain more suitable for isolating pure shear strength, so the choice of method depends on the specific research objective.

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