Design of composite dowels as shear connectors according to the German technical approval

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Abstract: Composite dowels are known as powerful shear connectors in steel-concrete-composite girders. More and more they are used in practice especially for prefabricated composite bridges. Advantages over headed studs are in particular the increased strength, the sufficient deformation capacity even in high strength concrete and the simple application in steel sections without upper flange. However, missing provisions in standards for composite dowels with the economic clothoid and puzzle strip have led to retentions of clients and delays in the approval process. Hence, the aim of the recently finished German research project P804 [15] founded by FOSTA- Research Association for Steel Application was to solve open questions concerning these innovative shear connectors and to prepare a general technical approval available for any design office and construction company. In this paper design concepts for ultimate limit state and fatigue limit state, structural design principles and instructions for production and construction are presented and background information are given.

1. Introduction

Composite dowels are shear connectors for composite beams, which consist of openings in steel plates, casted with concrete. They are either made of steel plates welded on the upper flange of the steel beam or are fabricated directly out of the web of steel beams. Main advantages, compared to headed studs, are a higher bearing capacity and a sufficient deformation capacity even in high strength concrete to be classified as ductile shear connectors acc. to EN 199411. Furthermore, composite dowels are particularly suitable and economic for composite sections made of steel sections without upper flange, as steel parts next to the neutral axis are reduced ("Fig. 1 a"). However, they have been successfully applied for *VFT-Rail*® girders as well, where the compression zone is additionally reinforced by external reinforcement, consisting of composite dowels. Another economic application area is the arrangement of composite dowels in concrete tee-beams as external reinforcement ("Fig. 1 b"). In "Fig.2" headed studs and composite dowels are compared in view of longitudinal shear capacity as well as composite bending capacity and bending stiffness.

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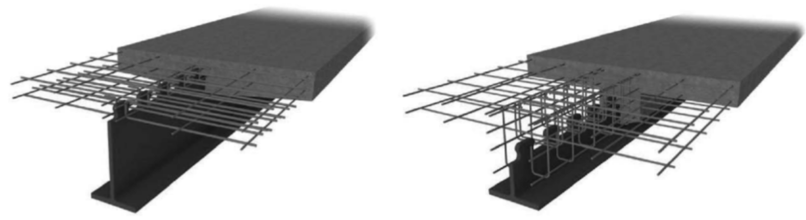

Figure 1: Application examples for composite dowels [3]

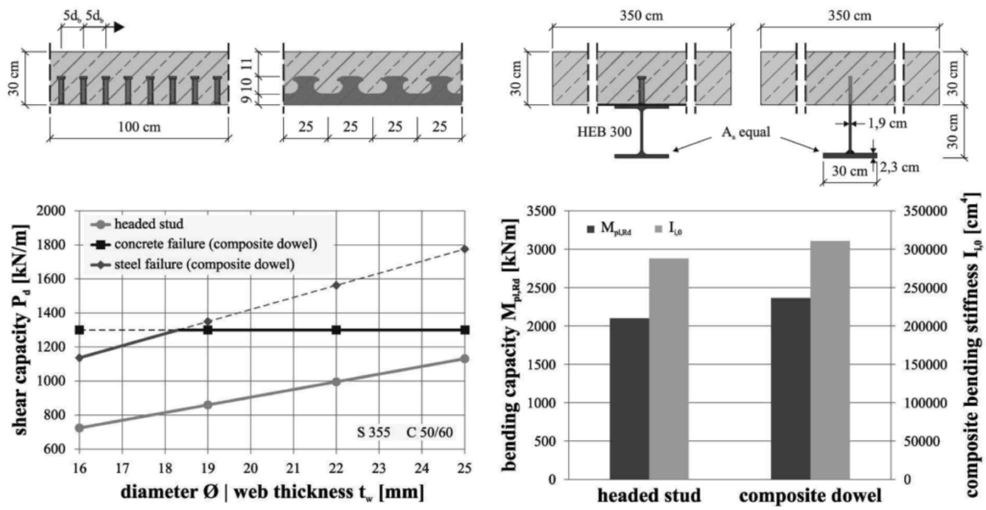

Figure 2: Comparison of longitudinal shear capacity between headed studs and composite dowels (left), comparison of moment capacity and stiffness between conventional composite section with headed studs and upper flange less steel section and composite dowels (right)

Up to now, the lack of technical rules for composite dowels led to delays in the approval process and to retention of clients. However, due to economic and technical advantages in more and more often composite dowels are used for road and railway bridges in Germany with approvals in the individual case. For example, the specific prefabricated composite bridge type *VFT-Rail®* with a composite dowel in clothoid strip was approved by the German railway authority (EBA) and applied e. g. in the railway bridge Simmerbach, Germany ("Fig. 3").

The starting point in the development of composite dowels can be traced back to research of Andrä and Leonhardt, which led to the development of the Perfobond strip [2], [11]. At the same time Bode developed the Kombidübel [1], [6]. Both design concepts are based on the mechanical model for shear failure of the concrete dowel. In the following years important knowledge about the bearing behavior of composite dowels was gained at the University of German Armed Forces [12], [13]. From this, mechanical models for the exceedance of the partial area pressure of concrete in the opening and concrete pry-out for composite dowels next to the concrete surface were obtained. The development of composite dowels with puzzle strip were pushed by research projects at RWTH Aachen University [8]

[10] [14] and by HOCHTIEF [9]. They led to a further development of the pry-out model and new models for steel failure. Seidl [16] developed design models for distance effects on concrete pry-out and vertical splitting of composite dowels in concrete tee-beams. The beginning of the clothoid strip – an optimization for fatigue loading – can be found in [4], [5]. This strip was finally used for the prefabricated composite bridge type *VFT-Rail*®, which was approved by the German railway authority (EBA) [7].

Figure 3: Bridges with composite dowels: road bridge in Pöcking (left) and Simmerbach (right) (photos: SSF Ingenieure AG, Munich)

2. Scope of the general technical approval

The new German general technical approval Z-26.4-56 [3] regulates composite dowels in clothoid (CL) and puzzle (PZ) strip ("Fig. 4"). The geometry can be scaled in dependence of the distance of the openings e_x between $150 \leq e_x \leq 500$ mm (notations see "Fig. 5"). The lower bound ensures a sufficient shear area for a ductile failure mode in the failure mode concrete shearing. The upper bound limits the maximum distance between composite dowels to prevent an unacceptable curtailment of the dowel capacity. The plate thickness can be varied between $6 \leq t_w \leq 60$ mm with a ratio of thickness to height between $0.08 \leq t_w / h_D \leq 0.5$. However, in the design formulas a thickness of up to 40 mm only is allowed to be considered. The minimum perpendicular distance of two steel plates is defined as 120 mm to ensure a sufficient installation of the reinforcement in between. For composite dowels structural steel in grade S235, S355 and S460 acc. to EN 10025 can be applied.

A minimum distance of 20 mm between concrete surface and top edge, respectively base, of the composite dowel has to be kept ("Fig. 5"). The distance from the opening to the concrete edge has to be in longitudinal direction more than 2.5 times the concrete pry-out cone h_{po} and in perpendicular direction more than 5.0 times of h_{po}. This ensures the full development of the concrete pry-out cone, if pry-out failure occurs. The minimum distance in perpendicular direction can be neglected, if in beam type sections the concrete reaches to the steel flange and confinement stirrups are installed (see below). This prevents pry-out at the lower concrete surface. A minimum width of beam type sections of 250 mm is required. For the composite section, concrete strengthC20/25 to C60/75 can be chosen (equal to the range given in EN 199411).

Composite dowels can be used under sagging and hogging moment for static as well as fatigue loading. However, structural members with centric tension forces perpendicular to the composite dowel under fatigue loading are not covered by the general technical approval as in this case. Unacceptable deterioration of the shear connection may occur.

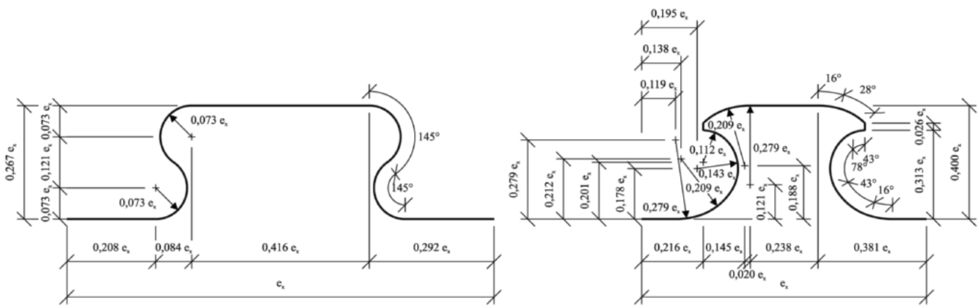

Figure 4: Definition of the composite dowel geometry puzzle strip (left) and clothoid shape (right)

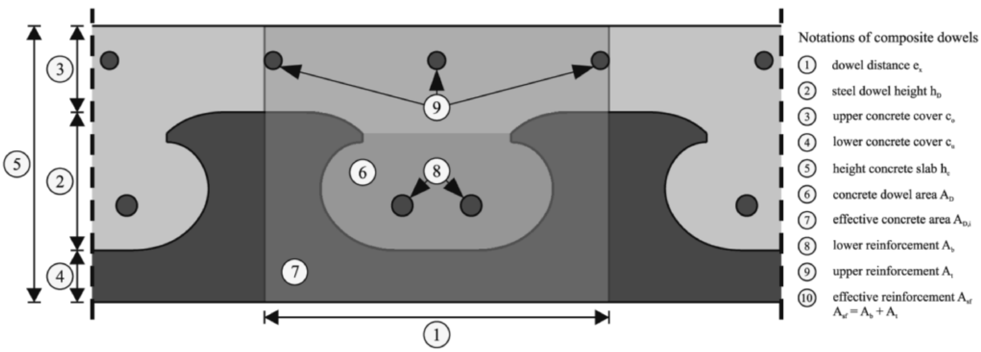

Figure 5: Notations of composite dowels

3. Design Concept

The design of composite beams with composite dowels is carried out in accordance with EN 1992, EN 1993 and EN 1994. The German general technical approval only regulates parts, which are not dealt with or which are different to the given European standards.

Besides the high bearing capacity, the major innovation leap of these composite dowels is:

- applicability in composite sections with steel beams without upper flange with equally spaced and partial shear connection;
- the ductility criterion according to EN 1994 is met to utilize plastic redistribution (for static loading only);
- a complete and consistent design concept for static and fatigue loading is provided for practical application.

4. Longitudinal shear capacity (static loading)

Possible failure modes of composite dowels subjected to static loading are
concrete shearing,

(i) concrete pry-out and
(ii) steel failure.

The characteristic longitudinal shear capacity P_{rk} is determined as the minimum capacity of the aforementioned failure modes. The design value is calculated as the characteristic bearing capacity divided by a partial safety factor of $g_v = 1.25$.

The design formulas for these three failure modes are derived from existing, modified or new developed mechanical models, backed by a statistical evaluation of test results. The quality of different design concepts is compared by the ratio between experimental and theoretical values of a huge test data base [15]. Criteria are the shift of mean value and the coefficient of variation. The mechanical models with the highest quality are used to derive design formulas by the statistical evaluation procedure according to EN 1990 Annex D.

(i) Concrete shearing

In particular for small openings and large steel plate thicknesses the dominating failure mode is double shearing of the concrete dowel. Therefore, the main parameters for the bearing capacity are the shear area of the concrete dowel and the shear strength of the concrete. Furthermore, the bearing capacity is affected by the transversal reinforcement within the opening due to an additional doweling. In large openings the two shear areas merge together, which is considered by a geometry depended reduction factor η_D ($\eta_{D,CL} = 3\text{-}e_x / 180$, $\eta_{D,PZ} = 2\text{-}e_x / 400$). For sufficient large openings, which are guaranteed by the application range of the general technical approval, concrete shearing is a ductile failure mode.

$$P_{sh,k} = \eta_D \cdot e_x^{\;2} \cdot \sqrt{f_{ck}} \cdot (1 + \rho_D) \qquad \text{in [N]} \tag{1}$$

with

$$\rho_D = \frac{E_s \cdot A_b}{E_{cm} \cdot A_D}$$

$$A_{D,CL} = 0{,}20 \cdot e_x^{\;2} \text{ and } A_{D,PZ} = 0{,}13 \cdot e_x^{\;2}$$

(ii) Concrete pry-out

Low distances between concrete dowel and concrete surface (top or bottom concrete cover) may provoke a failure mode, which is similar to the concrete pry-out of anchors subjected to shear forces. The hydrostatic pressure condition in the load introduction zone generates transversal tension forces, which lead – for insufficient concrete covers – to a cone-shape concrete pry-out (“Fig. 6”). The pry-out can occur for top or bottom concrete cover depending on the position of the concrete dowel. The result is a loss of hydrostatic pressure condition and therefore a reduction of concrete strength, which causes a secondary concrete pressure failure. This failure mode is ductile.

$$P_{po,k} = 90 \cdot h_{po}^{1,5} \cdot \sqrt{f_{ck}} \cdot \left(1 + \rho_{D,i}\right) \quad \text{in [N]} \tag{2}$$

$$h_{po} = \min\left(c_o + 0{,}07 \cdot e_x ; c_u + 0{,}13 \cdot e_x\right)$$

$$\rho_{D,i} = \frac{E_s \cdot A_{sf}}{E_{cm} \cdot A_{D,i}}$$

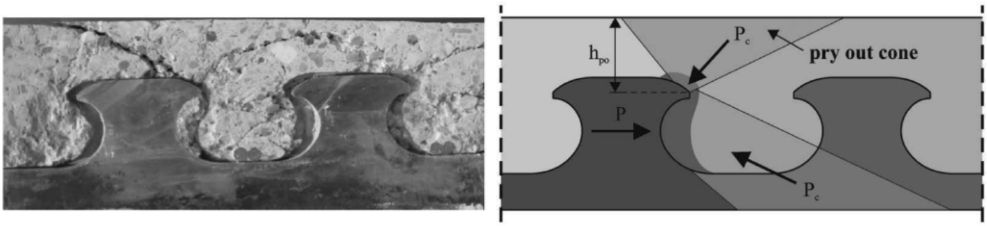

Figure 6: Failure mode pry-out: test specimen (left) and schematic illustration (right)

For composite dowels, where the distance in longitudinal direction is below $e_x < 4{,}5 \ h_{po}$, the bearing capacity is reduced due to an overlapping of the concrete cones. In this case $P_{po,k}$ has to be reduced by χ_x.

$$\chi_x = \frac{e_x}{4{,}5 \cdot h_{po}} \tag{3}$$

The same effect occurs in the case of parallel arrangement of composite dowels with a distance smaller than $e_y < 9 \times h_{po}$. For this the following reduction factor is used:

$$\chi_y = \frac{1}{2}\left(\frac{e_y}{9 \cdot h_{po}} + 1\right) \leq 1 \tag{4}$$

In plate type sections, stirrups ø8 mm have to be used to guarantee for a ductile behavior. A maximum spacing of $4.5 \cdot h_{po}$ and 300 mm has to be kept to ensure that at least two reinforcing bars are in each concrete cone (“Fig. 7”).

The design against concrete pry-out can be neglected, if the concrete is covered by a steel flange and confinement stirrups are applied (e. g. beam type sections with external reinforcement).

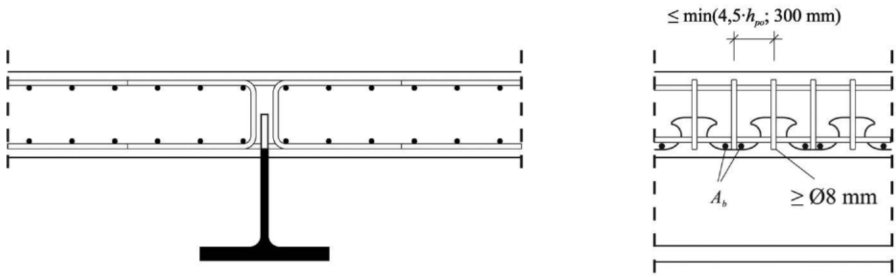

Figure 7: Detailing of reinforcement in composite girder with RC-slab and composite dowels

(iii) Steel failure

For small plate thickness and low steel strength, steel failure of the steel dowel can occur. This failure mode is caused by a combined shear-bending mechanism, which leads to a horizontal crack in the steel plate. Due to the ductile behavior of structural steel, this failure mode goes along with large plastic deformations and is therefore ductile ("Fig. 8").

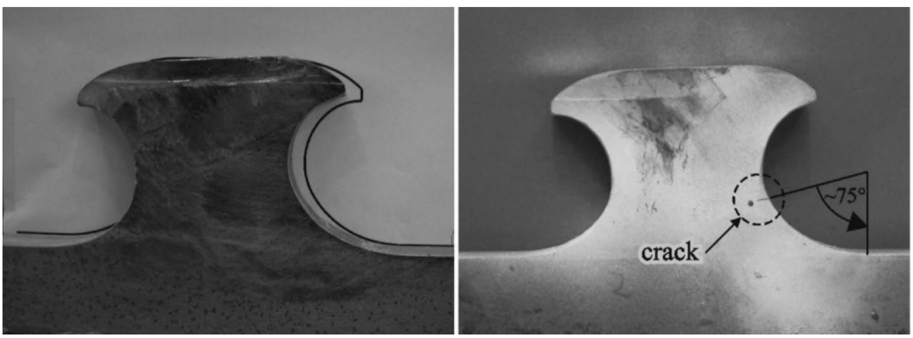

Figure 8: Test specimen with steel failure: plastic deformation (left) and static crack (right)

The basis of the mechanical model is von Mises yielding in a critical section, where stresses due to the shear force P_2 and the bending moment due to $P_2 \cdot z_{p2}$ occur. The position of the critical section is an extremum problem of decreasing cantilever z_{p2} and decreasing load P_2 as well as a decreasing section area ("Fig. 9"). Based on this mechanical model the characteristic bearing capacity is expressed by:

$$P_{pl} = \frac{h_{eff}}{h_{eff} - h_{krit}} \cdot \frac{{b_{krit}}^2}{\sqrt{16 \cdot {h_{s,krit}}^2 + 3 \cdot {b_{krit}}^2}} \cdot t_w \cdot f_y \tag{5}$$

with $h_{krit}(\alpha)$, $h_{s,krit}(\alpha)$, $b_{krit}(\alpha)$, where $\alpha = f(\partial P/\partial \alpha \rightarrow \min)$

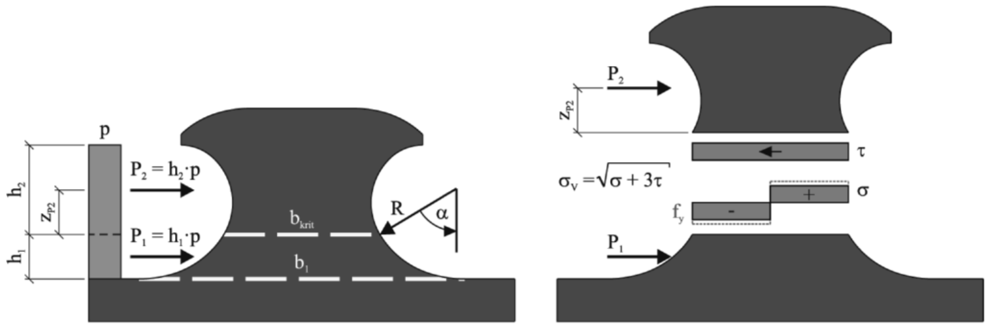

Figure 9: Schematic illustration of failure mode steel failure

Using the specific geometrical parameters for the clothoid and puzzle shape the design formula can be rewritten as a function of e_x:

$$P_{pl,k} = 0{,}25 \cdot e_x \cdot t_w \cdot f_y \tag{6}$$

(iv) Additional verifications for beam-type sections

In beam-type sections with composite dowels as external reinforcement, failure of the concrete web can occur. For thin concrete webs splitting tensile forces can exceed the concrete tensile stress, which results in a horizontal crack at the height of the composite dowel ("Fig. 10"). This failure mode is non-ductile and has to be prevented by sufficient confinement stirrups. The required reinforcement $A_{s,conf}$ is determined in the line with models for pre-stressed concrete by a strut-and-tie model with an aperture angle of 33° (equ. 7). The confining stirrups have to enclose the concrete strut, which is guaranteed by a spacing smaller than (e_x; 300 mm) ("Fig. 11"). Furthermore, the stirrup has to reach at least $\Delta = 0{,}15 \cdot e_x$ below the dowel base. For a ductile behavior, a minimum of two stirrups (ø10 mm) for each opening is required.

$$A_{s,conf} = 0{,}3 \cdot P / f_{sd} \tag{7}$$

Additionally, shear forces in the concrete web have to be verified according to EN 199211. In this design check, the effective depth d_v starts from half the dowel height.

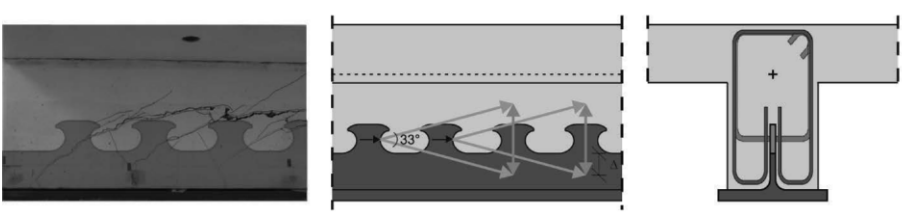

Figure 10: Failure mode vertical splitting: test specimen (left), strut-and-tie model (middle) and required stirrups (right)

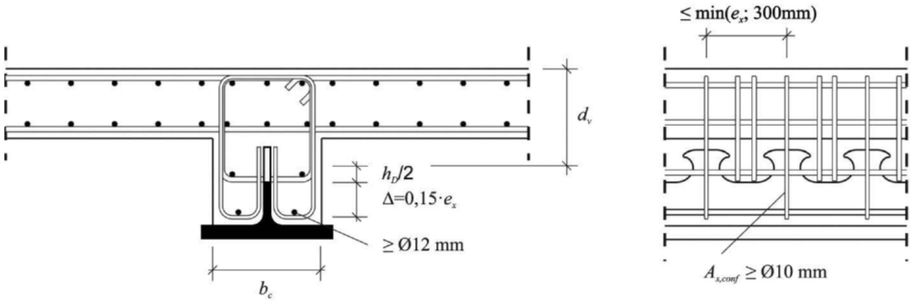

Figure 11: Detailing of reinforcement in composite girder with RC-web and composite dowels

(v) Shear connection

The verification of the longitudinal shear connection has to be proofed in accordance with EN 199411. The number of required shear connectors equals the number of openings in the steel plate. The required transversal reinforcement is determined based on a 45° strut-and-tie model:

$$A_{s,conf} = 0{,}3 \cdot P / f_{sd} \tag{8}$$

Equal spacing of shear connectors is allowed for hogging and sagging moment, if the minimum shear connection degree acc. to EN 199411 is met. In contrast to EN 199411, equal spacing is also allowed for steel sections without upper flange, if the following requirements are met:

- the shear connection degree is $\eta \geq 0.5$;
- the span is $L \leq 18.0$ m;
- the plastic moment capacity of the composite section must be equal to or smaller than 10-times the plastic moment capacity of the steel profile;
- the curtailment of the dowel capacity must not incise the design longitudinal shear force more than 25 %.

Equally spacing for steel sections without upper flange extend the range of EN 199411 and opens new interesting application fields for composite dowels.

For fatigue loading the longitudinal shear force has to be determined by elastic theory and the curtailment of the dowel capacity must not incise the design longitudinal shear force. Reason for this is the prevention of unacceptable deterioration of the connection due to plastic force redistribution.

5. Fatigue strength

The fatigue design has to be carried out according to the fatigue load model of appropriate standards. The fatigue design concept comprises

(i) steel fatigue design,
(ii) concrete fatigue design and
(iii) securing of a rigid shear joint.

Here, the interaction between the fatigue behavior of different components has to be considered (“Fig. 12”): e. g. degradation of the concrete dowel leads to decrease of the shear connection. This affects the stress distribution over the cross section and therefore the steel fatigue design.

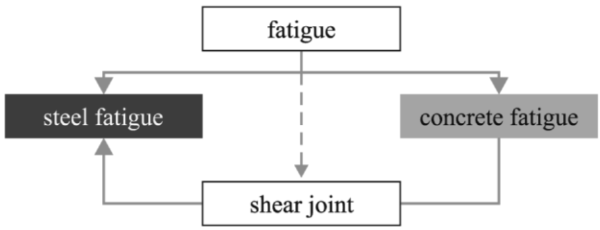

Figure 12: Schematic illustration of the interaction of the fatigue verifications

(i) Steel fatigue

The steel fatigue design is based on the geometric stress approach. The stress amplitude at the hot spot is determined for the fatigue load model and compared with the material fatigue strength. The fatigue strength (resistance) is described by the fatigue strength curve of detail category 125 (machine gas cut edges having shallow and regular drag lines) or of detail category 140 (machine gas cut edges with subsequent dressing) in accordance with EN 199419. The geometric stress amplitude (action) is the sum of stresses due to longitudinal shear forces (local effects) and bending of the composite beam (global effects). Both parts are amplified by stress concentration factors depending on the geometry of the composite dowel. The nominal stresses are defined as longitudinal shear stress (local) and normal stress (global) at the dowel base.

$$\Delta\sigma = k_{f,L} \cdot \frac{\Delta V \cdot S_y}{I_y \cdot t_w} + k_{f,G} \cdot \left(\frac{\Delta N}{A} + \frac{\Delta M}{I_y} \cdot z_D \right) \tag{9}$$

with $k_{f,L,CL} = 7{,}3\,;\quad k_{f,G,CL} = 1{,}5$ (10)

$$k_{f,L,PZ} = 8{,}6\,;\quad k_{f,G,PZ} = 1{,}9$$

The stress concentration factors kf,L (local) and kf,G (global) for the clothoid and puzzle shape are determined by finite element analysis and verified by strain measurements in cyclic push out and beam tests [15] (“Fig. 13”). The stress concentration factors are applicable for steel sections with a lower flange and concrete strength C20/25 and higher. To exclude low cycle fatigue, the geometric stress amplitude is limited to $2 \cdot fy$ and the upper geometric stress is limited to $1.3 \cdot fy$.

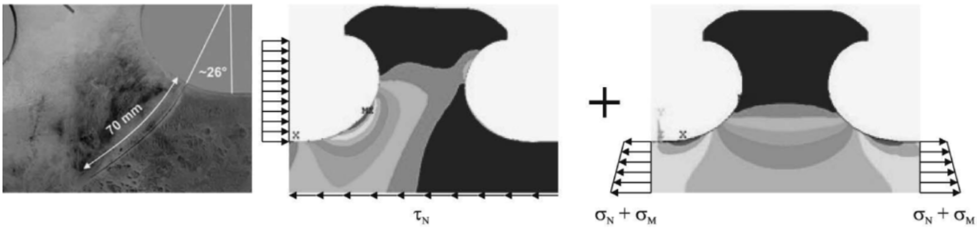

Figure 13: Steel fatigue failure: test specimen (left), FE-analysis of stresses due to local and global loading (middle and right)

In general tension stiffening has to be considered in the stress determination for regions where concrete cracking is expected. On the safe side the influence of tension stiffening (cracked section) can be neglected for the determination of stresses due to global bearing. For stresses due to longitudinal shear transfer the more unfavorable value from calculations with a cracked and an un-cracked section should be used. The design rule in EN 199411, which allows for the conservative determination of the fatigue stress in the composite joint with an un-cracked section, can be unsafe for composite dowels under hogging moment.

(ii) Concrete fatigue

Two types of concrete fatigue failure are known from tests:

- the loss of bearing capacity due to trickling of crushed concrete out of the composite joint;
- cyclic concrete pry-out of composite dowels with insufficient concrete cover subjected to high upper loads.

The first failure mode can be prevented by a limitation of the crack width to 0.15 mm in regions, where the composite dowel is in the concrete tension zone. This has to be considered for bending action in the concrete chord in longitudinal as well as perpendicular direction. Cyclic pry-out can be prevented by limitation of the upper load to 70 % of characteristic static bearing capacity according. to equation 1 and 2. Up to this load level the bearing behavior is mainly elastic.

(iii) Securing rigid shear connection

The basic requirement for the determination of stresses for the composite cross-section based on elastic theory is the assumption of a rigid shear connection between steel section and concrete section. However, results of cyclic beam tests show that this assumption is not justified a priori, which can lead to higher geometric stresses in the steel section (Fig. 14).

The loss of rigid shear connection is caused by a degradation of the concrete dowel due to cyclic loading. The consequence is a forceless slip before the composite dowel is activated (Fig. 14 bottom). This can be avoided by a limitation of the upper force. As criterion for a relevant degradation of the composite joint, the strain shift between steel and concrete section is used. Zero shift is equal to full shear connection; the maximum strain shift corresponds to a composite beam without shear connection. The threshold for an unacceptable degradation is assumed at 90 % shear connection. This criterion is applied to evaluate the condition of the composite joint in cyclic beam tests (acceptable or not acceptable). Afterwards the condition of the composite joint is correlated with the partial area pressure at the concrete dowel. Therefore, the partial area pressure ratio $E_{cd,3D}$ for upper and lower load from beam tests are plotted into a Goodman diagram, which is often used for concrete fatigue design ("Fig. 15" left). Tests with a residual shear connection of more than 90 % are located in the green area (acceptable), while tests with an unacceptable degradation are plotted outside. This behavior is shown in "Fig. 15" (middle and right). The strain distribution in test series 7 shows a relatively large strain discontinuity Δε compared to test series 8, where the loss of rigid shear connection was not that distinctive. The evaluation of the tests shows that the available concrete compressive strength under multi axial stress condition is up to 7.5 times higher than the uniaxial compressive strength. Based on the assumption that at least 10 % of the total loads are dead loads, the upper load is limited to 55 % of the available multi axial compression strength.

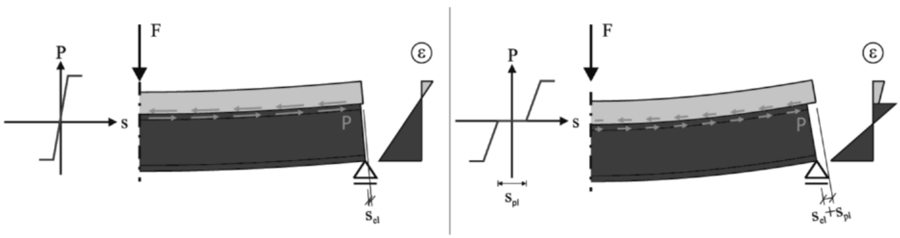

Figure 14: Deformation behavior and strain distribution of composite girder with intact shear connection (left) and damaged shear connection (right)

$$P_{cyc} = 3{,}1 \cdot t_w \cdot h_D \cdot f_{ck} \tag{11}$$

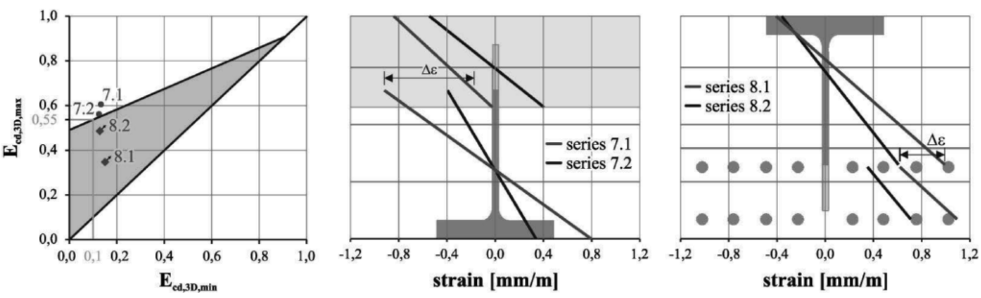

Figure 15: Fatigue strength of cyclic loaded beam tests as a function of partial area pressure ratio $E_{cd,3D}$ for upper and lower load

6. Fabrication and construction

Composite dowels have to be fabricated by gas cutting or a cutting-process, which is similar in terms of strength and fatigue. The nominal geometrical values are given in Fig. 4, where tolerances of +2/-4 mm are acceptable ("+" is an enlargement of the steel shape). The cutting quality has to be in accordance with EN 10901 and EN 10902 and depends on the execution class. For fatigue loading the quality has to meet the requirements for detail category 125, 140 respectively, according to EN 199319. Checking and documentation of the cutting quality is very important as inspection of the shear connector is not possible after casting. To prevent blowholes in the concrete next to the composite dowel, the maximum grain size is limited to 16 mm and the consistence of the fresh concrete should be soft to flowable.

7. Outlook

The general technical approval is a consistent continuation of the development of standardization of composite dowels: it started with approvals for single cases and should become a German steel guideline (DASt) in the future. For this, the presented design concept is a strong basis. Due to the given regulations uncertainties in design of composite dowels are eliminated. This enables an economic and timesaving application of these powerful shear connectors. Additional costs for approvals for single cases are omitted. For the future, it is planned to implement composite dowels in Eurocode 4 as alternative shear connector besides headed studs.

8. Notations

$P_{sh,k}$	[N]	Characteristic value of the concrete shearing resistance
$P_{po,k}$	[N]	Characteristic value of the concrete pry-out resistance
$P_{pl,k}$	[N]	Characteristic value of the steel resistance
e_x	[mm]	Distance of recesses in longitudinal direction
η_D	[-]	Reduction factor depend on the dowel geometry
η_D	[-]	Reinforcement ratio for transversal reinforcement
$\eta_{D,i}$	[-]	Effective reinforcement ratio for transversal reinforcement
E_S	[N/mm²]	Young‘s modulus of reinforcing steel
E_{cm}	[N/mm²]	Secant modulus of elasticity of concrete
A_b	[mm²]	Lower reinforcement in the range of A_D
A_t	[mm²]	Upper reinforcement in the range of e_x
A_D	[mm²]	Concrete dowel area
$A_{D,i}$	[mm²]	Effective concrete area (ex · hc)
A_{sf}	[mm²]	Effective reinforcement ($A_b + A_t$)
c_o	[mm]	Thickness of upper concrete cover
c_u	[mm]	Thickness of lower concrete cover
h_{po}	[mm]	Pry out cone height
f_{ck}	[N/mm²]	Characteristic value of the cylinder compressive strength of concrete
f_y	[N/mm²]	Nominal value of the yield strength of structural steel
χ_x	[-]	Reduction factor for concrete pry out in longitudinal direction
χ_y	[-]	Reduction factor for concrete pry out in transversal direction

e_y	[mm]	Distance between parallel arrangement composite dowels
t_w	[mm]	Web thickness
$A_{s,conf}$	[mm²]	Required transversal reinforcement in composite girders with RC-web
f_{sd}	[N/mm²]	Characteristic value of the yield strength of reinforcing steel
$\Delta\sigma$	[N/mm²]	Stress range
$k_{f,L}$	[-]	Stress concentration factor for longitudinal shear force (local)
$k_{f,G}$	[-]	Stress concentration factor for bending and axial force (global)
ΔV	[N]	Design value of the shear force range
S_y	[mm³]	First moment of area of the effective composite section
I_y	[mm⁴]	Second moment of area of the effective composite section
ΔN	[N]	Design value of the axial force range
A	[mm²]	Cross-sectional area of structural steel
ΔM	[Nmm]	Design value of the bending moment range
z_D	[mm]	Lever arm
P_{cyc}	[N]	Maximum force for cyclic loading
h_c	[mm]	Concrete slab height

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