CONTINUOUS SHEAR CONNECTORS IN BRIDGE CONSTRUCTION

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ABSTRACT

The use of continuous shear connectors is an upcoming solution in composite beams. They are characterized by a high initial stiffness, bearing capacity and ductility. With their use, new and economic constructions have been invented, e.g. the VFT-WIB construction.

This construction method is based on a rolled steel beam cut longitudinally, with a special shape, in two T-sections and a concrete top chord is concreted. The shape of the cut hereby allows for the shear transmission in the shear joint. In general pre-fabricated bridge elements are produced which are finalized on site. They are placed on the abutments and the residual superstructure is supplemented.

This paper introduces the static and fatigue design of continuous shear connections used for the VFT-WIB construction method; however the focus is on the steel design.

INTRODUCTION

Prefabricated composite bridges are regularly used in Germany since 7 years and are established as a standard solution nowadays [Schmitt and Seidl 2001]. This bridge solution is based on a prefabricated composite girder (VFT®-girder) as main bearing element of the superstructure. It consists of a steel beam with a precasted concrete flange under compression, see figure 1. A prefabricated concrete flange has many advantages. The concrete flange stabilizes the girder during transportation and in the construction stages. Braces are not longer needed for concreting the residual in-situ plate. Scaffolding for the concrete plate is unnecessary. Stiffeners are usually not required because of the high centre of gravity.

In this combination the demands of modern construction methods are met with the well planned and economic use of steel and concrete materials. However the key issues for the success of this method on the German market was achieved by the high degree of prefabrication and the design of new structural systems. Due to the high degree of prefabrication the finishes on site by the steel contractor became superfluous. All elements are manufactured under good conditions in the workshop, so that the quality of the structure increases substantially. The girders are delivered completely assembled on site and can be lifted and placed with a light-weight crane compared to the heavy weight of pre-stressed concrete girders as e.g. carried out for the Horlofftal bridge (D), see figure 2. The concrete flange is well connected to the steel girder from the beginning which means, that the composite actions is already present during the whole construction period. Continuous beam systems and frames are achieved by connecting the substructure by reinforcement and studs only. Thus new structural systems are designed, especially of frames with a new dimension in slenderness become possible. Even 1-bay frames may substitute 2-bays continuous beams with the same total span but without medium support [Schmitt et. al. 2005].

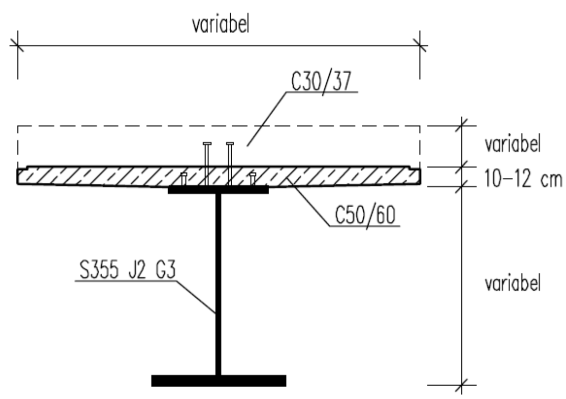


Fig. 1 – Typical cross-section of VFT®

Fig. 2 – VFT®-solution with rolled steel beam

Further, since 1997 investigations in continuous shear connectors fabricated by a single cut have been carried out (e.g. [Wurzer 1997], [P486 2000], [P612 2007]. These investigations have been based on welding a profiled steel strip on the top flange of a steel beam to achieve a shear transfer in the joint between the steel beam and the concrete chord of a composite beam. The shear bearing mechanism is equivalent to a composite dowel.

Consequently the potential of combining the use of this continuous shear connectors and the VFT®-construction technique has been identified and a new, innovative and economic construction has been invented, the VFT-WIB construction method [Seidl 2005].

This method is based on a rolled steel beam cut longitudinally in two T-sections. Further a concrete top chord is added, composed of a prefabricated part and a part which subsequently added on site to achieve the final cross section. This method is a very flexible solution offering varies cross section possibilities according to the design requirements, see figure 3.

Fig. 3 – Proposals for VFT-WIB cross-sections for bridges with implementing composite dowels

The cutting line of the rolled beam has a special shape and creates composite dowels identically to continuous shear connectors mentioned before. In figure 4 cut beams already assembled to pairs are shown during appliance of the corrosion protection. In the next step reinforcement bars are placed through the cutting shape (figure 5) and a concrete top chord is concreted to produce a prefabricated bridge element. The shape of the cut hereby allows for the shear transmission in the shear joint already in the construction stage similar to VFT®-constructions. Subsequently the prefabricated bridge elements are transported to the site (figure 6), placed on the abutments (figure 7) and, finally, the residual concrete chord is added [Schmitt et. al. 2004].

Fig. 4 – Rolled girders after cutting and coating in the shop

Fig. 5 – Reinforcement for prefabricated concrete plate

Fig. 6 – Transport of the VFT-WIB girders from the concrete plant to the construction site

Fig. 7 – Placing of the VFT-WIB girder with 32.50 [m] length

As a result VFT-WIB construction, with the use of the state of art concerning the concrete dowels technology and integrating the advantages of VFT®-constructions, are meet the following targets for competitive and sustainable construction:

- High safety standard for vehicle impact, especially for bridges with only two girders (shock),
- Reduction of coating surface,
- Shear connection without fatigue problems,
- Elementary steel construction nearly without any welding,
- Sparse maintenance and easy monitoring.

In the following more details of experimental investigations and design concept for VFT-WIB constructions, especially on the steel part of the concrete dowels, are presented.

FAILURE CRITERIA OF CONCRETE DOWELS

The bearing capacity of a composite dowel is limited by steel or concrete failure. In a good design both failures of a concrete dowel are balanced up to the maximum load.

Steel failure is limited in the ultimate limit state by a) the shear resistance, b) yielding due to bending of the dowel and in the fatigue limit state by c) fatigue cracks due to dynamic loading, see figure 8.

a) b) c)

Fig. 8 – Failure modes for steel

Concrete failure is characterized by several failure modes. Which mode finally occurs depends on the boundary conditions like geometry, concrete grade, reinforcement design, adding of fibers etc. For further information on concrete failure reference is given to [PreCo-Beam].

OPTIMISATION OF THE CONCRETE DOWEL BY EXPERIMENTAL INVESTIGATIONS

Static as well as cyclic tests on varies shapes of continuous shear connectors using the concrete dowel approach have been performed in the last years. First, tests have been conducted on the Perfobond strip, characterized by cut-outs in a steel strip used for shear connection, leading to the [DIBT 1991]. Later, tests focusing on composite dowels especially designed for the application in VFT-WIB constructions have been carried out [Schmitt et. al. 2004]. The Push-out Standard Tests (POST) according to EC4 (figure 9) and beam tests have been performed. In the tests concrete failure as well as steel failure has been observed. It has been concluded, that the ULS resistance of the steel is almost independent from the shape of the dowel. However fatigue cracks according to figure 8 c) have been observed in the POST after 2 million load cycles [P612 2007]; they have been caused by a very high level of stress amplitude in the tests. The fatigue cracks observed have a limited propagation due to the fact that the steel part is compressed in the POST (equivalent to negative bending moment region); thus a subsequent ULS-test resulted in no significant decrease of the residual strength.

Therefore the ultimate limit state design seems not to be so important for the shear connection in VFT-WIB bridges compared to the fatigue limit state.

Consequently an additional test program has been set up in the scope of [PreCo-Beam] to investigate in the following:

- Influence of the shape of dowel on the pressure profile coming from concrete to steel dowel to estimate the loading on each dowel;
- Dependency of the ultimate bearing resistance and the fatigue resistance of the steel on the shape of the dowel with regard to derive mechanical models and equations for design;
- Influence of the dowel shape, reinforcing and geometry of the composite element on the concrete failure at ultimate limit state.

One crucial aspect, especially for the fatigue verification of the steel dowels, is the superposition of stresses resulting from shear in the composite joint and global bending of the beam (normal stresses in the web). In case of a concrete dowel located in a tension zone of the web, fatigue cracks would propagate through the web and possibly into the flange which causes not only failure of the shear connection but collapse of the composite beam. Hence new POST specimens (NPOT) had to be developed to simulate the behavior of a shear connector located in the tension zone [PreCo-Beam], see figure 10, and fatigue tests have been conducted on three different shapes of shear connectors, see table 1.

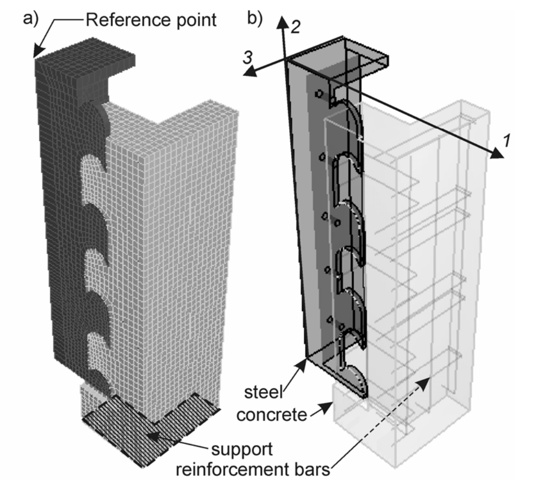


Fig. 9 – POST

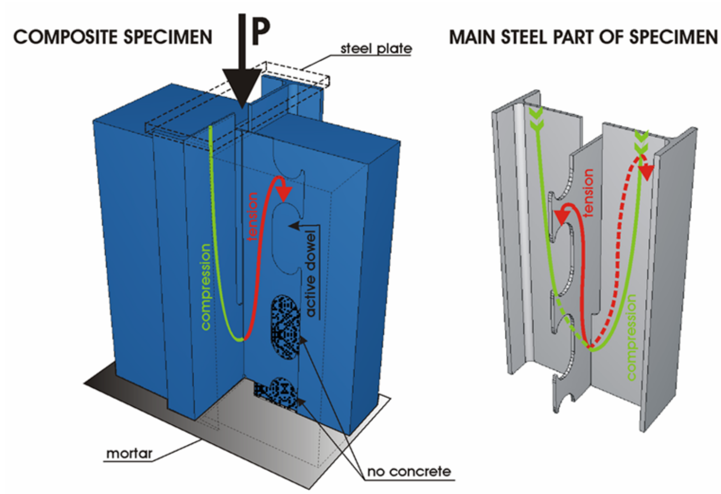
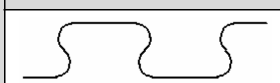
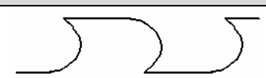


Fig. 10 – NPOT

As expected one crack in the PZ shape could be produced with the NPOT fatigue tests according to figure 8 c) and, according to the expectations, the crack propagated through the entire web. However, only one of the specimens exhibited a fatigue failure.

Table 1 – Comparison of shapes with results from the NPOT fatigue tests [PreCo-Beam]

	PZ	SA	CL
Shape			

As conclusion of the test series the puzzle shape (PZ) has been chosen to be the most promising shape considering of fabrication aspects, bearing capacity and fatigue.

ANALYSIS OF LOADING ON THE STEEL DOWEL

In addition to the fatigue test a static NPOT on the CL shape connector has been carried out with a large number of strain gauges at the steel dowel for analyzing the stresses and for the calibration of FE analyses carried out simultaneously with the tests. The results of the test have been in accordance with the numeric results and the numerical model has been modified for the PZ shape reference is given to in this paper, see Figure 11.

In the first step an analytic model for the local behaviour of a shear connector has been introduced. Here the puzzle geometry has been focused. However it is possible to transfer the approach to any geometry for each inventor of a new shape.

The local approach is based on the load introduction on single tooth. Hereby S represents the center of the projection area A_p in shearing direction; h_s is the distance from the center to the base of the shear connector. In figure 11 for example, as projection area only the area constricting a concrete block on front of the dowel should be considered.

The force on each steel tooth is composed by the shear force in the composite joint P_τ and the stress distribution due to the global loading depending on the geometry of the composite cross section; P_{up} for the uplifting forces due to the location of the shear joint in respect to the neutral axis of the cross section and σ_{en} for the notching effect from the nominal stress of the steel section, see figure 12.

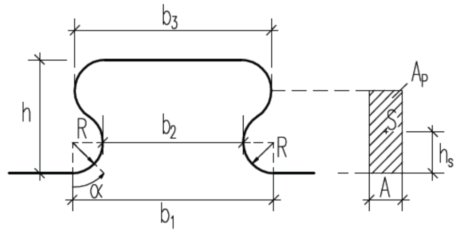


Fig. 11 – Geometry of puzzle tooth (PZ)

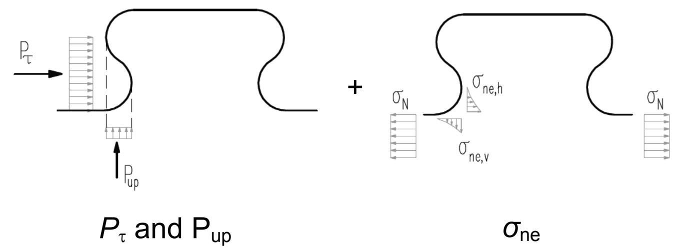


Fig. 12 – Forces on the steel dowel (PZ shape)

For the determination of P_τ it is conservatively assumed that the load distribution along the height of the dowel is constant and therefore P_τ is located at h_s.

The uplifting force P_{up} is resulting from the eccentricity h' of the shear joint to the centre of the composite compression chord, see figure 13. The trajectory generates an uplifting force on the steel dowel which would be pushed out of the concrete section if the shape of the steel dowel doesn't contain an undercut. Hence the undercut of the steel dowel implements two functions; first, it generates the 3D stress state for the kernel of the concrete dowel and second, it locks the shear connection against uplift in vertical direction.

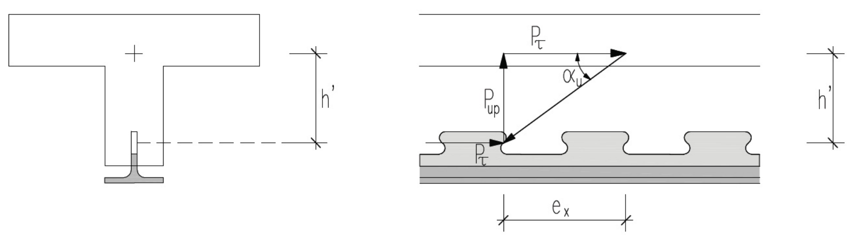


Fig. 13 – Uplifting force due to eccentricity of the shear joint to the neutral axis at support

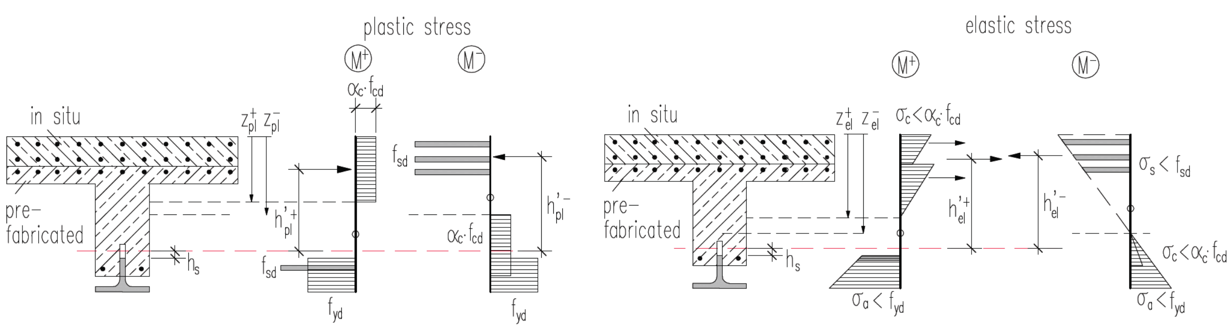


Fig. 14 – Determination of h'_i for ULS and SLS

The determination of the uplifting force is based on h'_i depending on the stress distribution of the composite section, see figure 14. In the following full shear connection is assumed. It is required to differentiate between the ULS and the SLS respectively the fatigue design. Further the construction stages have to be considered.

Generalised P_{up} is therefore calculated according to figure 13 as:

$$P_{up} = P_{\tau} \cdot \frac{h'}{e_x} \qquad \text{[kN]} \qquad \text{Eq. 1}$$

with $h' < e_x$ else $P_{up} = P_{\tau}$.

The σ_{ne} is the notching effect on the normal stresses in the web of the steel girder due to the shape of dowel. The increase in stress hereby depends on the geometry of the dowel.

On the basis of an FEA (figure 15) it has been concluded, that σ_{ne} depends directly on the ratio of the connector length to the radius of the cut out (b_1 / R), but not on length and radius separately. Moreover, the height h of the dowel is unimportant for the notching effect.

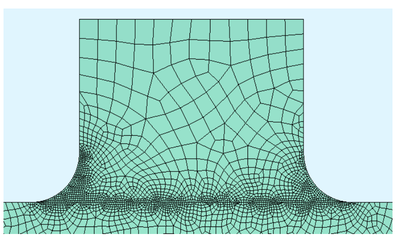


Fig. 15 – FE model for analysis

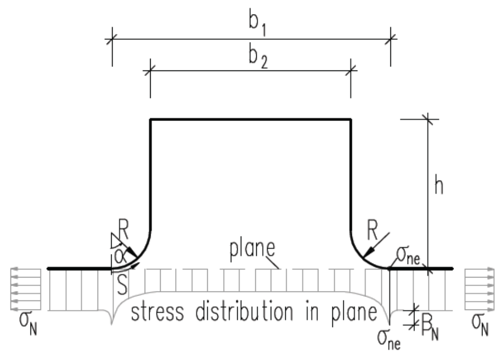


Fig. 16– Stress distribution due to notch effects

On the basis of an extensive parametric study using the FE method the notching stresses have been derived to

$$\sigma_{ne} = \beta_N \cdot \sigma_N \qquad \text{[N/mm}^2\text{]} \qquad \text{Eq. 2}$$

with the notch factor

$$\beta_N = \left[1.192 + 0.1029 \cdot \left({}^{b_1}\!/_{R} \right) - 0.0022 \cdot \left({}^{b_1}\!/_{R} \right)^2 \right] \cdot f(\alpha) \qquad [\text{ - }] \qquad \text{Eq. 3}$$

where $f(\alpha)$ expresses the decrease of the amplitude along the cut out

$$f(\alpha) = 0{,}9077 + 0{,}0104\ \alpha - 0{,}0005\ \alpha^2 \qquad [\text{ - }] \qquad \text{Eq. 4}$$

The higher the ratio b_1 / R of the tooth, the higher is the notching effect expressed by the factor β_N. Thus, not only the sharpness of the notch itself but also the increase in stiffness depending on the length of the dowel is influencing the notch effect, which is in accordance to the effect of longitudinal stiffeners.

STRESS ANALYSIS ON THE STEEL DOWEL

For the validation of the loading on a steel dowel and the resulting stresses the modified FE-analysis to the PZ shape has been consulted. For the local effects due to longitudinal shear, the model presented in [PreCo-Beam] was modified to model (M3) according to [Lorenc et. al. 2007]. The stresses due to the notching effect of the nominal stresses in the web and the uplift forces have been calculated considering only the steel part in model (M2). The geometric properties of the dowel have been chosen to b_2 = 125mm, h = 100mm; the web thickness of the steel beam has been t_w = 10.2mm.

In figure 17 the influence of the loading on the stresses of a single puzzle tooth has been evaluated along the cut. For this purpose, separate calculations have been conducted for P_τ = 50kN, P_{Up} = 50kN and the global stresses in the web σ_N = 50N/mm^2.

Consequently the influence of each loading parameter on the stress distribution along arc length has been compared with each case G,L and U (figure 17) and an analytic model has been derived for the ULS and fatigue design of a steel dowel.

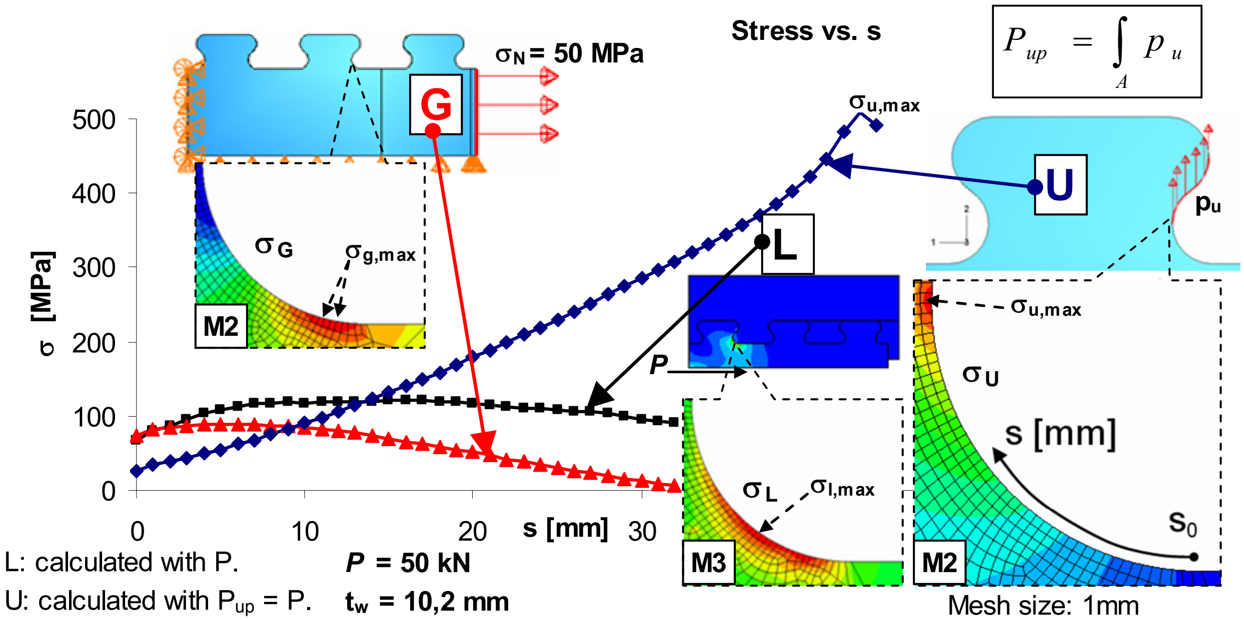


Fig. 17 – Stresses in steel dowel along arc length from specific actions: σ_{ne} ($\sigma_{g,}$ G), P_τ (L) and P_{up} (U)

ULTIMATE LIMIT STATE DESIGN OF THE STEEL DOWEL

In [P621 2007] continuous shear connectors have been experimentally investigated. Hereby cracks in the steel strip have been observed whereas the concrete matrix has not been significantly damaged (figure 18). In reference to this failure mode a steel failure criterion has been derived.

For its application for VFT-WIB bridges this formula has to be modified to cover the additional uplifting forces from the global geometry of the cross sections, see figure 19. However the overall assumption, that the resulting maximum equivalent Von Mises stresses do not exceed the yield strength is kept as basis of design.

Consequently the bearing resistance of a single steel tooth P_{Rd} is determined in dependency of the loading specified and in accordance with [P621 2007]. Hereby influence of the increase of the nominal stresses of the steel section due to the dowel geometry has been neglected as it is insignificant in the plastic design.

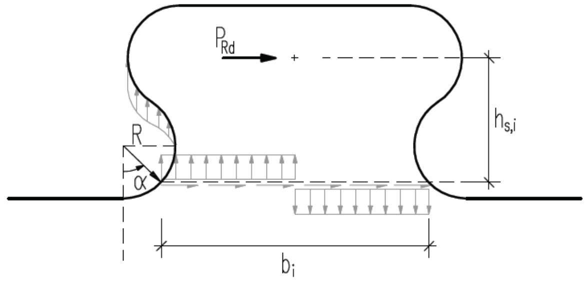


Fig. 18 – ULS-failure [P621 2007] Fig. 19 – Forces and stresses in a critical section at ULS

Thus, the following design criterion is derived:

$$P_{Rk} = \frac{f_y \cdot t_w \cdot b_i^2}{\sqrt{\left(4 \cdot h_{s,i} + \frac{h'_i\,(b_2 + b_3)}{e_x}\right)^2 + 3 \cdot b_i^2}} \quad \text{[kN]} \qquad \text{Eq. 5}$$

with

- f_y Yield strength steel [N/mm^2],
- t_w Plate thickness of web [mm],
- $h_{s,i}$ Distance of centre of gravity to critical section $= h_s - (1 - \cos\alpha) \cdot R$ [mm],
- b_i Width at critical section $= b_1 - 2 \cdot \sin\alpha \cdot R$ [mm],
- h'_i $= h' - (1 - \cos\alpha) \cdot R$,
- e_x distance between connectors, fig. 13.
- α angle along cutting edge, fig. 11,

For the puzzle shape the maximum equivalent stresses derived from equation 5 has been located to be at α = 70° for the POST which is in accordance to the tests, see figure 18.

FATIGUE RESISTANCE OF A GAS CUT EDGE

The fatigue design of the steel part is divided into two parts. One part is dedicated to the fatigue design of the web taking the increase in stress due to the notching effect of the steel tooth into account. The second part treats the estimation of the fatigue stresses along the gas cut edge of the dowel itself, considering the effect of the shear stresses as well as the nominal stresses in the steel section and their verification.

However at first, the fatigue resistance of a gas cut edge has to be specified. According to the Eurocode [EC3-1-9] the fatigue category of a gas cut edge is 140 when subsequent dressing is applied. Hereby all visible signs of edge discontinuities have to be removed. The cut areas are to be machined or ground and all burrs to be removed. Any machinery scratches, for example from grinding operations, can only be parallel to the stresses. If the cut has shallow and regular drag lines with cut quality II according to EN 1090 (for railway bridges cut quality I [DIN FB 103]) the fatigue category is reduced to 125. For both categories repair by weld refill is not allowed. Re-entrant corners are to be improved by grinding appropriate stress concentration factors.

Therefore the roughness and cutting tolerances from the oxy-cutting process have been measured in dependency to the cutting speed. The results are shown in table 2. The deviation is small and all cutting surfaces are class I.

Table 2 – Roughness in dependency of the cutting speed

Cutting speed *v*	Medium surface roughness Rz	Tolerances of rectangularity and inclination
350 [mm/min]	43 – 63 [µm]	0.10 [mm]
500 [mm/min]	20 – 74 [µm]	0.40 [mm]
650 [mm/min]	40 – 62 [µm]	0.25 [mm]

In addition knowledge on the fatigue resistance is found in the research project [P185]. In this project the influence of the cutting quality on the fatigue design made from fine-grain steels (according of today's EN10025-4) has been investigated. It has been noted, that the initial crack occurs from the blasted surface in the heat affected zone (HAZ) from cutting. However it has been noticed that short stopping of the flame cutter decreases the fatigue strength to 60%. This results from the change of the failure initiation to the cut edge. Further it has been observed that hammering (an effect which may occur due to hammering of the continuous shear connector in gaps from which plastified concrete may have disappeared), cutting speed, warming before cutting and material strength have hardly an influence on the fatigue strength.

Therefore crack initiation occurs in the HAZ along the cut. The design value is therefore conservatively derived to $\Delta\sigma_C$ = 125 N/mm^2. If stopping of the flame cutter can not be avoided it should take place at an irrelevant location in terms fatigue.

FATIGUE DESIGN OF THE STEEL WEB

Due to the notch effect of the steel tooth the stresses of the web along the cut edge are increased. The reduction due to the geometry is comparable to the effect by longitudinal stiffeners, however only the geometrical effect has to be considered as the material notch due to welding is inexistent. Therefore the fatigue verification has to be performed with the fatigue category of gas cut edges $\Delta\sigma_C$ = 125 N/mm^2 according to [EC3-1-9] and:

$$\Delta\sigma_{E,2} = \Delta\sigma_{N,w} \cdot \beta_N \qquad [\text{N/mm}^2] \qquad \text{Eq. 6}$$

with: $\Delta\sigma_{N,w}$ relevant longitudinal stresses in the web along the bottom line of the connector,

β_N according to equation 3.

FATIGUE DESIGN OF THE STEEL CONNECTOR

Fatigue crack initiation and propagation depend on the principle stresses along the cutting edge. To derive an analytic design model for fatigue verification the principle stresses have consequently to be considered, which are supposed perpendicular to the radius (figure 20).

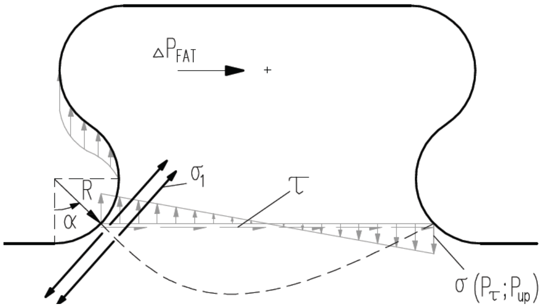


Fig. 20 – Analytic model for principle stresses (in dependency of the angular *α*)

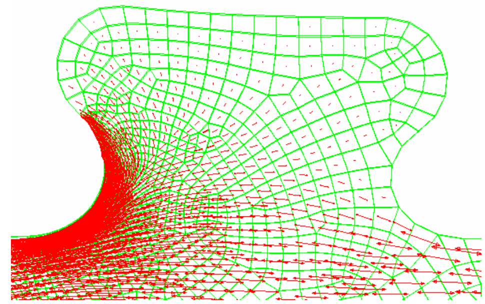


Fig. 21 – Principal stress trajectories in dowel (P_{Rd} and $P_{up} \neq 0$ and σ_N influence)

With the loading defined previously and in dependency of *α*, the following fatigue load resistance has been derived:

$$\Delta P_{FAT} = \left(\Delta\sigma_C - \beta_N \cdot \Delta\sigma_{N,w}\right) \cdot \frac{t_w \cdot b_r^2}{b_r \cdot \cos\alpha + \left(6h_{si} + \dfrac{3h'_i\,(b_2 + b_3)}{2 \cdot e_x}\right) \cdot \sin\alpha} \quad \text{[kN]} \qquad \text{Eq. 7}$$

with $\Delta\sigma_C$ Fatigue strength of gas cut edge [N/mm²],

$h_{s,i}$, t_w , h'_i , e_x , α see equation 5,

$\Delta\sigma_{N,w}$, β_N see equation 6,

b_r arc length at critical section

$$= \frac{\pi(90-\alpha)\left(\dfrac{1}{2}b_1 - \sin\alpha \cdot R\right)}{90 \cdot \cos\alpha} \quad \text{[mm].}$$

For the puzzle geometry investigated the maximum principle stresses along the cut edge have been derive at an angle *α* = 20°.

SUMMARY

In this paper, the VFT-WIB construction technique and their advantages are presented. Further the main problems for this construction in design have been identified based on experimental results from the previous years and own test results. Especially the fatigue design of the steel tooths of continuous shear connectors are here to be noticed.

Consequently a design concept for the steel part of continuous shear connectors applied in VFT-WIB constructions has been derived. Main focus has been laid on the analytic approach for hand calculation and its validation by experimental results and FEA.

However it is also possible to calculate the Von Mises and the principle stresses resulting from the local loading by FEA and to derive shape functions for each connector. These factors, $A_{el,L}$ and $A_{el,U}$ are embedded in equations 8 for the final design with global loading [PreCo-Beam].

$$\sigma = \frac{1}{I_y}\left[\mathbf{V} \cdot \frac{S_y}{t_w}\left(\frac{1}{A_{el,L}} + \frac{\tan\alpha}{A_{el,U}}\right) + \mathbf{M} \cdot z \cdot \beta_{N,w}\right] \quad \text{[N/mm}^2\text{]} \qquad \text{Eq. 8}$$

with V, M global transversal force and bending moment in beam, respectively,

z distance of bottom line of steel dowel to neutral axis,

α_u uplift angle, see Fig. 13,

t_w web thickness,

I_y, S_y second moment of area and moment of area of steel part, respectively.

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