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INFLUENCE OF HOT DIP GALVANIZING ON THE FATIGUE BEHAVIOUR OF T-STUDS FOR A GROUTED JOINT OF INTEGRAL COMPOSITE FRAME BRIDGES

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Abstract. *Recent investigations paved the way for hot dip galvanizing in the field of bridge constructions [1]. Due to technical limitations of hot dip galvanizing, steel girders longer than 16 meters have to be divided and connected afterwards. The Chair of Metal Structures works in cooperation with the Chair of Concrete Structures of the Technical University of Munich (TUM) and the Bundesanstalt für Materialforschung und -prüfung (BAM), to develop a grouted joint connection, which is suitable for hot dip galvanizing. In contrast to welding the segments on site or using prestressed bolt connections, the hot dip galvanization of the grouted joint remains undamaged for the whole service life time of the structure. As part of this research, the main component of the grouted joint - a hot dip galvanized T-stud - is investigated regarding its fatigue behaviour. The present paper outlines the idea and the structure of the grouted joint and classifies the hot dip galvanized T-stud according to a fatigue class given in EN 1993-1-9 [2]. Furthermore, the fatigue assessment for the grouted joint and an optimization of the geometry of the T-Studs is carried out using finite element analysis.*

1 INTRODUCTION

As a result of the increasing traffic and the rapid growth in heavy goods, many highways are currently being widened to six lanes and will be in the near future. Considering the large number of bridge structures along highways, it is obvious that a correspondingly number of new bridges will have to be built in the course of this expansion. For the required span of 35 m to 45 m and up to 60 m, the integral frame bridge in composite construction (see figure 1) has proven itself in recent years [3]. The whole span can be built without the need of a center bearing. This results in short assembly times as well as a minimal impact on traffic. It also provides low maintenance costs, as a transition construction or elastomeric bearings are not required as well.

Figure 1. integral composite frame bridge

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Except for the organic corrosion protection system of the steel girder, all main components of this bridge type can be built maintenance free over the whole service life time of 100 years. As the share of integral composite frame bridges in the road and highway system has been growing over the last decades, it is a serious approach to further reduce maintenance and therefore face the service life time of the corrosion protection system.

As one of the most sustainable options to protect steel from environmental influences, hot dip galvanizing (HDG) is able to last more than 100 years even in heavily polluted environments. During the process of HDG, the steel parts are immersed into tanks with approximately 450°C hot liquid zinc. Due to limitations in the process, the length of common zinc tanks is up to 16 m and in rare cases up to 19 m [4]. Hence, longer girders have to be divided into segments and connected afterwards. Welding is possible but comes along with disadvantages regarding maintenance. Since the welding can only take place after the process of HDG, the area close to the seam has to be protected by zinc thermal spraying, which has a service lifetime of 25 to 35 years and therefore does not offer an overall solution. While I-girders of composite sections can be connected using prestressed bolt connections, there is no suitable solution for box girders so far. A research project concerning the highway systems of South-Germany showed that box girders are favoured over I-Sections, due to several advantages. Box girders have a 25 to 30% smaller surface exposed to chloride contaminated spray, which is accompanied by lower ablation rates of the zinc coating. Furthermore, box girders offer a higher torsional stiffness, which make them suitable for curved layouts in top view.

For this reason, the Chair of Metal Structures works in cooperation with the Chair of Concrete Structures at the Technical University of Munich (TUM) and the Bundesanstalt für Materialforschung und -prüfung (BAM), to develop a joint for box-girders, which meets the technical, structural and architectural requirements. After preliminary studies, a grouted joint connection turned out as the most suitable.

2 GROUTED JOINT

Considering the length of the zinc tank to be 16 m, the required bridge span of 35 to 45 m results in three segments with lengths from 12 to 16 m (figure 2).

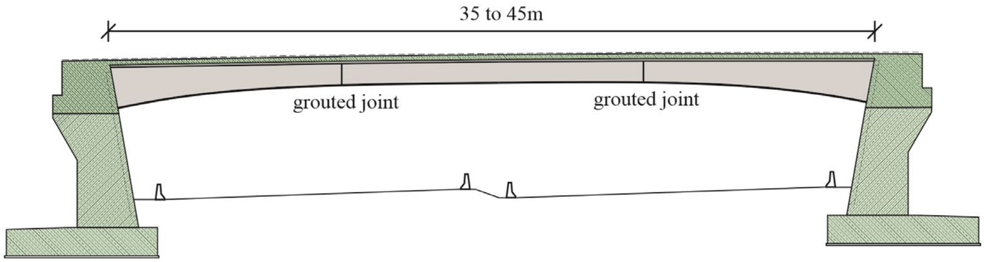


Figure 2. positioning of grouted joints

To avoid temporary propping towers at the joint position and therefore ensure a short assembly period without affecting traffic, the grouting of the joint and the pouring of the prefabricated slab should take place before the girder is lifted into place. To reduce the bending moments acting on the joint, the position of the joint should be chosen as close as possible to location where the global bending moment is zero. For this reasin, the middle segment has to be designed as long as possible. Considering the global position of the joint, mainly positive bending moments occur during assembly and over service lifetime. Nevertheless, the joint is also capable of transmitting negative bending moments. The design of the grouted connection consists of interlocking steel T-studs, diaphragms and longitudinal plates embedded in a box girder. To ensure a form- and force-locking connection, the hollow space surrounding the T-studs is grouted with Ultra-High-Performance Fiber-Reinforced Concrete (UHPFRC). Figure 3 shows the grouted joint before and after connecting the steel segments. The diaphragms, which are closer to the T-studs, enclose the space for grouting.

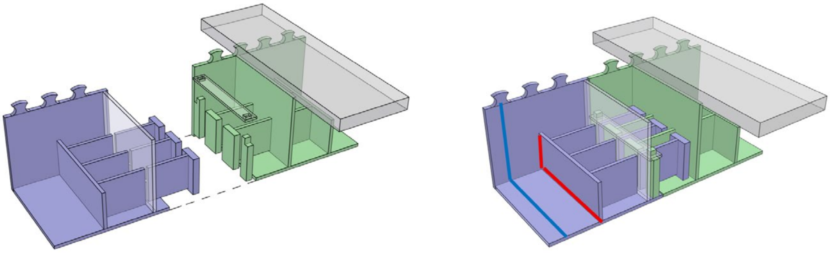


Figure 3. grouted joint before (left) and after assembly (right)

In the cross section outside the joint (figure 3; blue line), the tension force resulting from the bending moment can be assigned to the steel box section, while the compression force acts on the concrete slab, which consists of a prefabricated slab and an in-situ-concrete slab. At the transition area to the joint (figure 3; red line), the tension force is transmitted to the webs of the T-studs through the extended webs, welded to the lower steel chord of the box girder. At the end of the T-studs, the flanges of the T-studs forward the tension force to the UHPFRC, where it is transmitted to the flanges of the opposite segment by compression struts. Since two compression struts are acting on the T-stud flanges symmetrically from both sides, there is no transverse bending. At the special position of the outer flange, where the half-sized flange is welded to the web of the steel box, the compression strut is acting asymmetrically on the flange. By using a transversal steel plate (see figure 4), which is connected to the upper ends of the flanges, the outer compression struts are short-cutted.

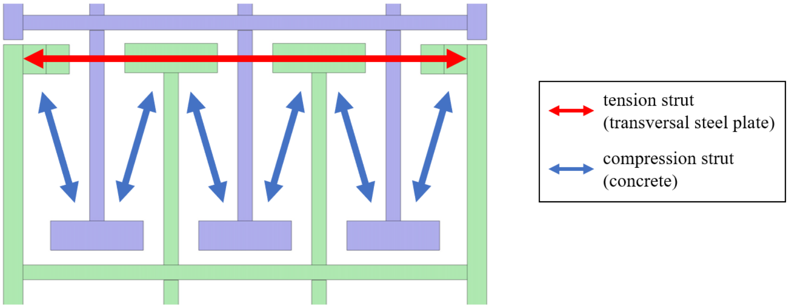


Figure 4. force transmission – top view

The bottom surfaces of the T-stud flanges are placed on top of the lower box steel chord of the opposite segment (see figure 3). In this way, the shear force can be transmitted to the diaphragms of the opposite segment.

The main load carrying component - the T-stud - has to transmit internal forces due to static and dynamic actions according to EN 1991-2 [5]. The resulting stresses can be determined by using simplified calculation models. These models are currently developed based on known mechanical relations and verified by practical experiments and finite element studies.

When determining the resistance of the T-studs according to EN 1993, a distinction between static and fatigue resistance is required. Since hot-dip galvanizing does not affect the static resistance of steel components, the static verifications can be carried out conventionally. For the fatigue assessment as per EN 1993-2 [6] however, the unfavourable influence of hot-dip galvanizing has to be taken into account.

3 RECENT INVESTIGATIONS ON THE FATIGUE BEHAVIOUR OF HOT DIP GALVANIZED DETAIL CATEGORIES AS PER EN 1993-1-9

During the process of hot dip galvanizing, the pre-treated steel component is immersed in a bath of zinc with approximately 450 °C. The liquid zinc reacts with the steel and forms a coating on the surface. During the subsequent cooling process, micro defects can form in the zinc coating due to the different shrinkage and cooling rates of steel and zinc (see figure 5). If the component is subjected to fatigue stresses during its service life, these micro defects can propagate into the base material and initiate fatigue failure.

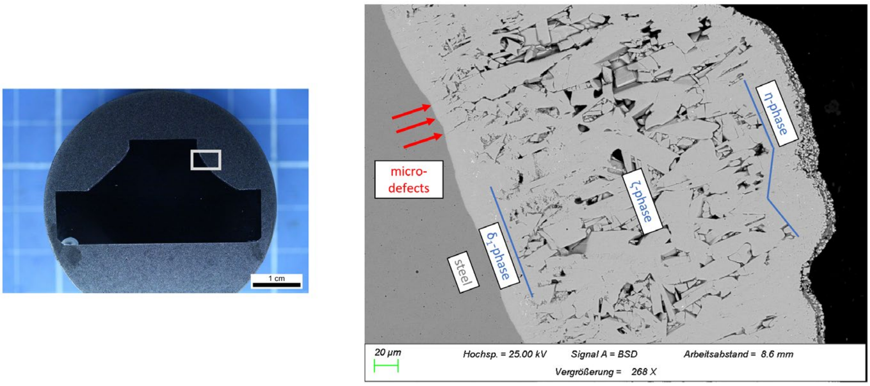


Figure 5. phases of the hot dip galvanize coating

For a long time, it was assumed that hot-dip galvanizing was generally unsuitable as a corrosion protection system for steel components which are exposed to fatigue loads. This was one of the reasons why the influence of hot-dip galvanizing on the fatigue strength was not scientifically investigated. Nevertheless, recent investigations have disproved this assumption. According to [1] the negative impact of hot dip galvanization on the fatigue resistance can be considered conservatively by the reduction of the reference detail category as per EN 1993-1-9 [2] by one category. Figure 6 shows the results from the tests carried out in [1] for a gas-cutted plate, which is assigned to fatigue class 125 (FC 125).

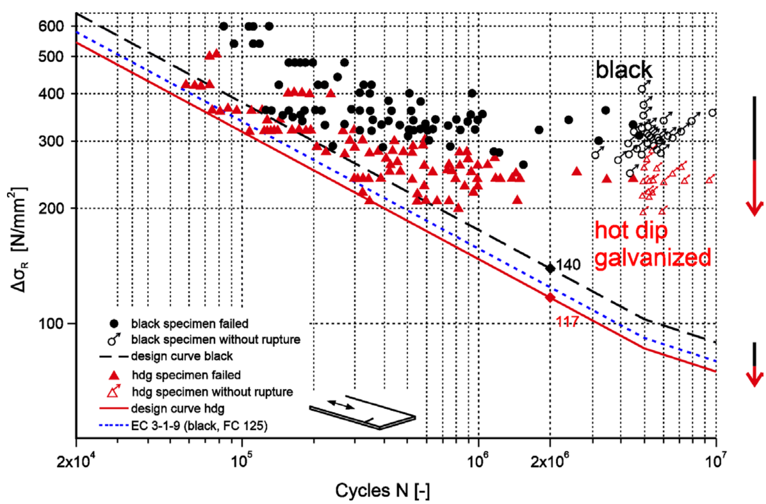


Figure 6. fatigue tests of gas-cutted plates (FC 125), black and hot dip galvanized [1]

Although a reduction of the fatigue strength is evident, the hot dip galvanized specimens achieved a reference fatigue strength $\Delta\sigma_{C,HDG}$ of 117 N/mm² at design level and are thus only 8 N/mm² below the reference detail category 125. Further detail categories investigated in [1] are shown in Table 1.

Table 1: test results for hot dip galvanized fatigue classes according to [1]

detail description acc. EC3 [2]	$\Delta\sigma_{C,EC3}$	$\Delta\sigma_{C,HDG}$
manual longitudinal fillet weld (small scale)	100	147
manual longitudinal fillet weld (big scale)	100	134
manual longitudinal fillet weld over butt weld	71	109
transverse splice in plates	112	123
full cross section butt weld of rolled section	90	85*
vertical stiffeners welded to rolled beam	80	82
Studs (shear)	90	80

*considering one specimen without rupture

All tested HDG details achieved a higher or at least a fatigue strength, which can be classified as one detail category below the reference category as per EN 1993-1-9 [2].

As the existing research does not include HDG single sided T-joints, it is part of the current investigations at the TUM. The results in [1] indicate, that the reduction of one detail category is also suitable for the single sided T-joint.

4 FATIGUE TESTS OF HOT DIP GALVANIZED SINGLE SIDED T-JOINTS

27 specimens of single sided T-joints were tested for their fatigue strength. All specimens are of the same steel material (S355) and dimensions. In order to compare to black steel, 16 specimens were hot dip galvanized while 11 remained without corrosion protection. The dimensions of the T-studs and the test setup are shown in figure 7.

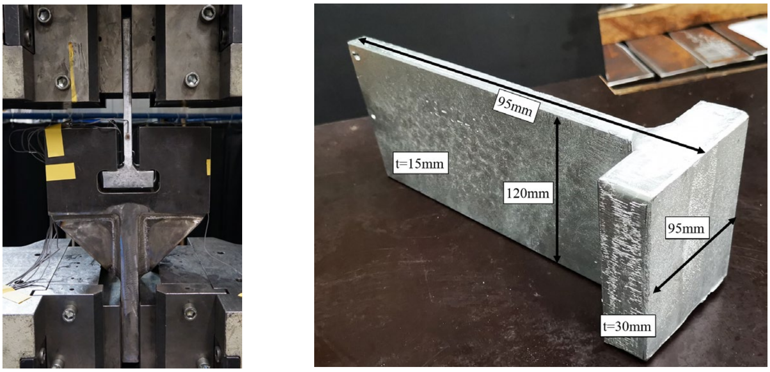


Figure 7. test configuration and dimensions of the T-stud

To measure the strains during the test, strain gauges were attached at different locations on the flange and web. To determine the structural stress according to IIW [7], strain gauges were applied to the web at a distance of 0.4t and 1.0t from the notch. Further strain gauges were applied to measure the eccentricities.

Considering the test configuration, the stress in the flange and in the transition area to the web of the T-joint, mainly depends on the geometry and the positioning of the load-carrying device - the clamp. In order to provide similar boundary conditions as for the embedded T-stud in the grouted joint, preliminary studies using finite element simulations were carried out. The stress state for the embedded T-stud was evaluated and the geometry of the clamp was iteratively adjusted until an almost identical stress state was obtained.

The specimens are organized in groups according to different stress ranges $\Delta\sigma_{R,Test}$. The estimated load cycles were calculated by using the SN curve for fatigue class 80 for black and fatigue class 71 for hot dipped galvanized specimens according to EN 1993-1-9 [2]. The SN curves as per code are given for a probability of survival of 95%. To calculate the estimated load cycles for each stress range, the SN curves was transformed to a probability of survival of 50% according to [8]. All tests were performed under sinusoidal load cycles with a stress ratio of R = 0,1 and a frequency of 2 Hz. Table 2 shows the specimen groups and estimated load cycles.

Table 2: fatigue tests of single sided T-joints with and without zinc coating

	Quantity of specimen	$\Delta\sigma_{R,Test}$ in N/mm²	Estimated load cycles (PS=50%)
black steel FC 80	2	282	100.000
	2	223	200.000
	4	165	500.000
	3	131	1.000.000
hot dip galv. FC 71	3	282	70.000
	3	223	141.000
	4	165	347.000
	5	131	694.000

The tests were evaluated on the basis of the nominal stress concept according to EN 1993-1-9 [2]. The *pearl string method* according to DIN 50100 [9] was applied to calculate the regression curve and the design curves were determined for a survival probability of 95%. Figure 8 shows the results for the specimens with and without hot dip galvanizing.

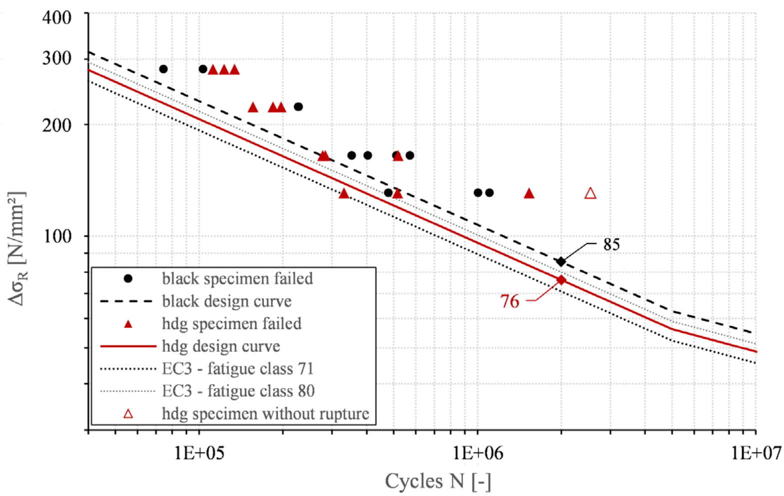


Figure 8. SN-curves of single sided T-joint with and without zinc coating

According to expectation, all specimen failed at the toe of the notch and the initial crack appeared at the edge of the web plate. The design curve of the blank steel specimens reaches a resistant stress range of 85 N/mm² for $2x10^6$ cycles and therefore is on the safe side compared to the reference detail category as per EN 1993-1-9 [2]. The design curve for HDG specimens reaches a resistant stress range of 76 N/mm² and are on the safe side as well, considering the reduction of one fatigue class according to [1].

For the stress ranges of 165 to 282 N/mm², the test results for both the black and the hot-dip galvanized specimens were close to the expected number of load cycles. For the stress range of 131 N/mm², a significantly larger scatter was observed for both groups. This, in combination with the relatively small number of specimens, has a significant effect on the design curves. It can be assumed that the test results for a higher number of specimens would be further on the safe side.

5 FATIGUE ASSESSMENT OF THE GROUTED JOINT USING THE MODIFIED NOMINAL STRESS CONCEPT

The experimental fatigue tests were performed under tension force only, which results in a constant stress distribution over the height of the T-stud. Considering the T-stud in the real joint geometry, the external forces result in an uneven stress distribution and stress concentrations, especially at the bottom of the T-stud near the transition area between web/flange and web/diaphragm as shown in figure 9. These stress concentrations have to be critically assessed regarding fatigue failure.

For the verification of the bridge joint, the modified nominal stress concept as per EN 1993-1-9 [2] is applied. This concept provides the advantage, that the stress concentration resulting from macro-geometrical effects due to the detail geometry and from load effects are considered. Furthermore, the fatigue class of the HDG T-stud is known from the experimental tests. To quantify the stresses due to macro-geometrical effects, EN 1993-1-9 [2] gives stress concentrations factors for specific details. Alternatively, a finite element analysis can be performed. Since there is no detail matching with the specific geometry of the joint, the joint was modeled as shown in figure 9. The finite element model was loaded with the fatigue load model 3 according to EN 1991-2 [5]. According to results of the experimental tests, the detail category was determined to 71.

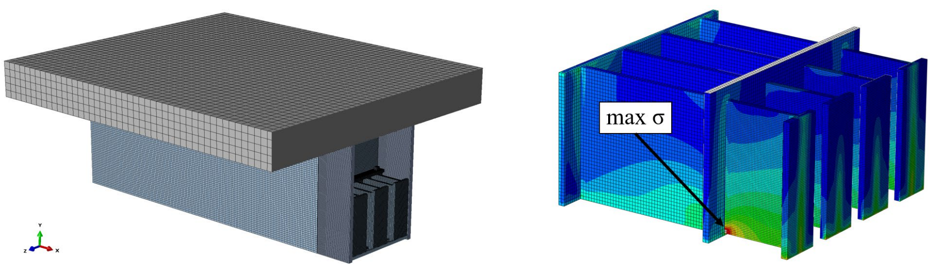


Figure 9. finite element model of the grouted joint (only left segment shown) and location of stress concentration

As per EN 1993-1-9 [2], the modified nominal stress concept includes macro-geometrical effects due to the design of the component, but excluding stress concentrations due to the weld itself. For this reason, preliminary finite element analyses on single T-studs were carried out, to determine a suitable distance from the notch where the modified nominal stresses can be evaluated. For a mesh size of 3 to 5mm, a distance of 10 to 15 mm was chosen.

The verification was carried out using the safe life method including a safety factor γ_{Mf} of 1,35, due to high consequences of failure. The stresses were evaluated from the finite element analysis and the damage equivalence factors for road bridges as per EN 1993-2 [6] as well as the ARS 22 [10] were considered. The utilization for the fatigue assessment at the guiding location – the bottom of the T-stud close to the diaphragm - is up to 95%.

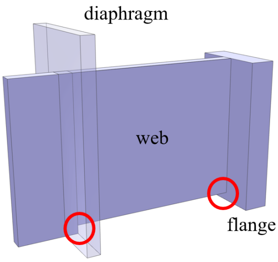
A comparison of all required verifications for the bridge joint showed that the fatigue assessment is guiding in all cases, due to distinct stress concentrations and high loads. On the one hand, the design of the joint has to be suitable for HDG which requires a geometry where stress concentrations are unavoidable. On the other hand, the load assumption is very conservative regarding the specifications given in the ARS

22. The λ_2 coefficient of the damage equivalent factors as per EN 1993-2 [6], which allows the load assumption to be adapted to the traffic volume, is prescribed with a value of 1,1 according to ARS 22 [10]. This corresponds to a crossing of $2x10^6$ vehicles with an average weight of 400 kN per year and is usually chosen for roads and highways with two or more lanes per direction and high flow rates of lorries according to EN 1991-2 [5]. The overall damage equivalent factor λ results in 2,3 with λ_{max} guiding.

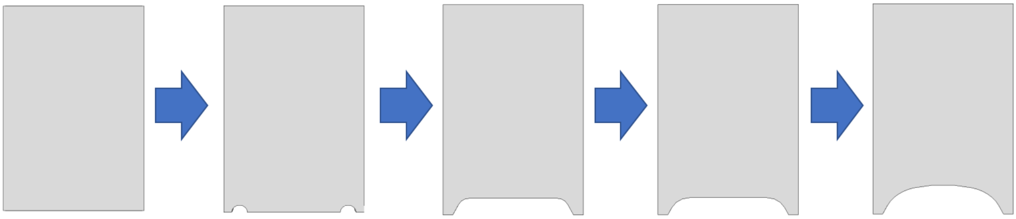
Since the present type of composite bridge including the grouted joint is designed for country roads overpassing a highway, an approach as per EN 1991-2 [5] considering medium flow rates of lorries and an average weight of 300 kN per vehicle would be sufficient. The overall λ factor would result in 1,59 which corresponds to a 30% reduction of the assumed load.

6 OPTIMIZATION OF THE FATIGUE BEHAVIOUR OF THE T-STUD

Due to the high fatigue stresses, it is of particular interest to reduce the stress concentrations and therefore improve the fatigue behavior of the T-stud. For this reason, the geometry of the T-stud was optimized at the critical locations by providing cut-outs.



(a) critical locations with stress concentration in the web



(b) geometrical optimization process of the web

Figure 10. optimization of the T-stud geometry at the location of the stress concentrations

To quantify the reduction of the fatigue relevant stresses due to the cut-out and to consider the more complex geometry, a finite element analysis using the modified nominal stress method was performed. Two models with and without cut-out were meshed and evaluated according to the corresponding concept in the IIW [7]. Figure 11 shows the principal stresses occurring at the locations of stress concentrations for the model with and without cut-out. The stresses were evaluated at a distance of 10 mm from the notch to consider the stress concentration due to the geometry but neglect the stress concentration due to the weld. The model was loaded by the fatigue load according to the fatigue load model 3 as per EN 1991-2 [5].

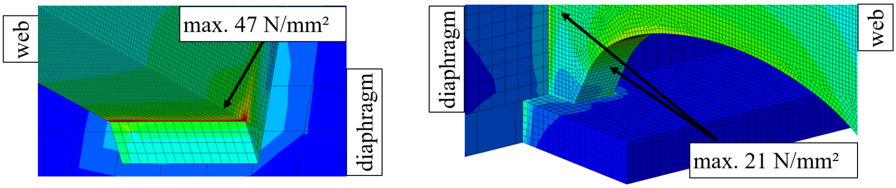


Figure 11. principal stresses at lower transition area web/diaphragm for models without (left) and with cut-out (right)

The stress concentrations in the model without cut-out are localized at one single area at the weld seam and reach a maximum of 47 N/mm² at a distance of 10 mm from the notch. In the model with cut-out, the stress concentrations are localized at the transition area to the web and to the lower cord. The maximum principal stresses are 21 N/mm². Due to the cut-out, the stresses can be distributed over a greater height at the diaphragm. Comparing the maximum stresses of both models, the reduction results in ca. 55%.

6 CONCLUSIONS

Previous research on the fatigue behavior of hot dip galvanized details recommend to classify the fatigue strength of hot dip galvanized components one detail category below the reference category as per EN 1993-1-9 [1]. The experimental tests carried out at the TUM showed, that this recommendation is also applicable for the single sided T-joint. The hot dip galvanized test specimens reaches a stress range of 76 N/mm² at $2x10^6$ load cycles and are therefore classified as detail category 71 for the specific application of the grouted joint.

The fatigue assessment of the grouted joint was undertaken applying the modified nominal stress concept according to EN 1993-1-9 [2] and considering the fatigue class of 71 for the T-studs. The relevant stresses were evaluated using a finite element analysis. The analysis showed distinct stress concentrations at the lower area of the T-studs and utilization rates up to 95%. Compared to the verifications in ultimate and service limit state, the fatigue assessment was guiding in all cases.

To reduce the stress concentrations at the notch, an optimization of the geometry of the T-studs was performed. The calculation and evaluation were performed using the modified nominal stress concept according to the IIW [7]. The stress concentrations at the critical locations could be reduced by up to 55%.

According to the investigations carried out in the present paper, the design of the grouted joint is capable of transferring the fatigue loads safely considering the current set of design rules. The idea of the hot dip galvanized grouted joint allows the corrosion protection system of composite bridges to be maintenance-free over the entire service life time. In addition, the design concept paves the way for durable and highly loadable connections in other fields of composite construction.

REFERENCES

[1] D. Ungermann e. al., „Forschungsvorhaben P 835 / IGF-Nr. 351 ZBG - Feuerverzinken im Stahl- und Verbundbrückenbau“ Verlag und Vetriebsgesellschaft mbH, Düsseldorf, 2014.

[2] DIN EN 1993-1-9:2010-12, Eurocode 3: Bemessung und Konstruktion von Stahlbauten – Teil 1-9: Ermüdung, 2010

[3] U. Kuhlmann e. al., Forschungsvorhaben P 843 / IGF-Nr. 353 ZN - Ganzheitliche Bewertung von Stahl- und Verbundbrücken nach Kriterien der Nachhaltigkeit, Düsseldorf: Verlag und Vertriebsgesellschaft mbH, 2016.

[4] D. Ungermann e. al. “Entwurfshilfe zum Einsatz von feuerverzinkten Bauteilen im Stahl- und Verbundbrückenbau” bauforumstahl e.V, Düsseldorf, 2016

[5] DIN EN 1991-2:2010-12, Eurocode 1: Einwirkungen auf Tragwerke – Teil 2: Verkehrslasten auf Brücken, 2010

[6] DIN EN 1993-2:2010-12, Eurocode 3: Bemessung und Konstruktion von Stahlbauten – Teil 2: Stahlbrücken, 2010

[7] A. F. Hobbacher „Recommendations for Fatigue Design of Welded Joints and Components, Second Edition, IIW Collection", Basel: Springer International Publishing AG, 2016 UND Genoa: International Institute of Welding, 2016

[8] Dipl.-Ing. Heise, F.-J., "Forschungsvorhaben P 778 - Bemessung von ermüdungsbeanspruchten Bauteilen aus hoch- und ultrahochfesten Feinkornbaustählen im Kran- und Anlagenbau; Rechnerische Nachweise - Kerbdetails- Lastkollektive", KIT – RWTH Aachen – Liebherr, 2013

[9] DIN 50100, Schwingfestigkeitsversuch – Durchführung und Auswertung von zyklischen Versuchen mit konstanter Lastamplitude für metallische Werkstoffproben und Bauteile, 2016

[10] Bundesanstalt für Straßenwesen, „Allgemeines Rundschreiben Straßenbau Nr. 22/2012", Bonn: Bundesministerium für Verkehr, Bau und Stadtentwicklung, 2012