#### NEW DEVELOPMENTS OF STEEL AND COMPOSITE BRIDGES

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#### 1. ABSTRACT

Building bridges nowadays is dominated by several tendencies such as the overall lack of financial resources and increasing traffic volume on one hand, and demands from ecological and environmental views and questions as how to build within an existing infrastructure and how to improve durability on the other. Some new developments of steel and composite bridges may give answers. Within the scope of three research topics report is given on first results and tendencies which may lead to more economic, more durable aesthetic steel composite bridges.

In the first topic steel composite bridges are considered under a sustainability and holistic approach, including life cycle assessment, life cycle costs and life cycle performance. In the second steel-concrete composite bridges with horizontally lying shear studs and corru-

gated webs are within the focus, managed through numerous experimental and numerical investigations. In the third the fatigue behaviour of steel-composite bridges made of hollow sections is investigated by large and small scale testing and numerical studies considering the welded joints. All projects are realized in cooperation with research partners and financed by different research associations to whom appreciation deserves.

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# 2. INTRODUCTION

Building bridges nowadays is dominated by several tendencies: There is the overall lack of financial resources that leads to a strong urge for economic solutions. Additionally traffic volume has dramatically increased in recent years so that there is a need for building new infrastructures as quickly as possible. On the other hand society often is no longer satisfied to have a bridge but increasing demands from ecological and environmental views ask for solutions that are integrated in the surrounding landscape and are of aesthetical appearance. Furthermore, meanwhile there is an existing infrastructure which has become "old" especially in view of the increased traffic, so that questions as how to renew existing bridge structures with reduced impact on the running traffic and how to improve durability and maintenance requirements have become relevant.

In this light, the following new developments of steel and composite bridges will be focused:

Sustainaiblity as a new general demand on structures today leads to fundamentally different considerations for bridges than for buildings. A new running European project on "Sustainability of steel-composite bridges" aims at a holistic approach dealing with bridges in view of their whole lifespan. A combined analysis of Lifecycle Assessment (LCA), Lifecycle Costs (LCC) and Lifecycle Performance (LCP) should allow to bring out the advantages of steel- concrete composite bridges in view of durability and similar aspects considering also the important issue of costs.

In order to improve the behaviour of composite bridge structures also in view of sustainability, new ideas such as forming a composite connection by studs directly welded to the web and thus lying horizontally in the slab or the application of corrugated webs for long spanned slender bridges may deliver a contribution. Recent results on "Steel-concrete composite bridges with horizontally lying shear studs and corrugated webs" will be reported in the 4<sup>th</sup> chapter.



Fig. 1 Highway A73 in Germany "St. Kilian bridge", A7 "new and old Sinntal bridge"

The demand for aesthetical bridges such as "Steel-composite bridges made of hollow sections" seems not be affordable in many cases. However, welding the braces directly on the chords may form an interesting economic solution in comparison to cast steel joints. In order to realize this type of structures also for "real highway bridges" the fatigue verification should be feasible without special expert knowledge. Recommendations on welded KK-joints [15] developed by the authors on order of the Bundesanstalt für Straßenwesen on behalf of the Bundesministerium für Verkehr, Bau- und Stadtentwicklung form a first step in this direction. Further running investigations within a cooperative research project should complement and enlarge this also by experimental results. More explanations will be given chapter 5.

# 3. SUSTAINABLITY OF STEEL-COMPOSITE BRIDGES

# 3.1. General

The current situation in the European bridge market is dominated by concrete bridges. Steel and steel-composite bridges only form an interesting alternative if additional criteria count such as e.g. aesthetics, construction time or reduced overall height. That is why the choice of orders is mostly only made according to minimum costs. However, with rising traffic volume and increasing vehicle gross weight this approach does no longer seem to be adequate, especially considering that bridges are in general long-living structures where the lifecycle is planned for more than 100 years.

Therefore, a new holistic approach is investigated in a running European research project by combining analyses of Lifecycle Assessment (LCA), Lifecycle Costs (LCC) and Lifecycle Performance (LCP). For steel-composite bridges innovative solutions are elaborated to give alternatives to concrete bridges. Throughout the project the approach is applied to three realistic case studies representing standard situations of steel-composite bridges. Variations and optimizations are performed in a second step. By improving the durability of details affected by degradation and reducing lifecycle costs the sustainability of composite bridge structures is increased.

# 3.2. Sustainability by lifecycle design of bridges

Looking at bridges with a view to sustainability, not only the construction stage must be taken into account but the entire lifecycle of 100 years. These long-living structures are facing different degradation processes througout the years. Degradation can be divided into several processes such as fatigue, corrosion and carbonation having an impact on various details. The structural function of the details, and therefore the structure itself, can be pre-

served and improved by maintenance and/or renewal actions concerning defects discovered during inspections see Fig. 2.



Fig. 2 Schematic representation of the lifecycle of a bridge

The lifecycle performance of steel-composite bridges is analysed from the production of raw materials and the construction over the operation of the bridge (including maintenance etc.) till the demolition at the end of life. In the frame of the running European research project SBRI [1] (see also [2]) under the participation of scientists, bridge owners, consultants and industry the lifecycle performance of steel-composite bridges is analyzed with a holistic approach as described in the following.

# 3.3. Holistic Approach

# 3.3.1. General

Lifecycle analysis aiming at sustainable bridge structures is divided into three main categories of consideration, see Fig. 3. First, the environmental quality represents the analyses of emissions within the lifecycle assessment (LCA). Parallel, the economical quality comprises costs occurring during the entire lifecycle (LCC). The social and functional quality is involved as the third main category of lifecycle analyses. Applying this holistic approach to the entire lifecycle of bridges influences affecting the structure at all stages are covered and no single design criterion is focussed.



Fig. 3 Holistic approach to lifecycle analyses

The description of the lifecycle performance (LCP) of the structure and its details is the allembracing condition to determine any measurement during operation needed to guarantee a functional structure. Possible effects of degradation and renewal actions may lead to additional emissions (LCA), costs (LCC) and restricted social and functional quality. The application of this holistic approach promises to point out the advantages of steelcomposite bridges compared to concrete bridges regarding construction time, durability and exploration of material properties in an efficient way.

# 3.4. Case Studies

# 3.4.1. Selection of bridge types

To apply the holistic approach to realistic steel-composite road bridges, it was taken advantage of the experience of involved research partners and typical bridges were selected. A complete design and calculation is performed for the representative bridges. In order to achieve a reasonable comparability, the bridges were divided into three types according to their functionality and span lengths.

Small motorway bridges spanning around 50 to 60 meters are considered as bridges of Type A. Bridges of Type B are crossings of motorways. Span lengths up to 120 meters are reached by big motorway bridges and are assigned to Type C, see Fig. 4.

For achieving an optimization of the sustainable behaviour over the lifecycle variations are analysed throughout the case studies. Material properties will be changed in Case A2 and an allowance for increased traffic volume is given by an additional lane on the cross section in Case A3. A comparison between integral abutments, a standard two-span bridge and a clamped centerspan by two outer spans is made in Case B. Innovative solutions are applied to enhance the structural capacity and improving fatigue details of the box girder composite section in Case C.



Type C

![](_page_4_Figure_9.jpeg)

![](_page_4_Picture_10.jpeg)

![](_page_4_Picture_11.jpeg)

![](_page_4_Picture_12.jpeg)

Fig. 4 Bridge Types

#### 3.5. Lifecycle Performance and optimisation of inspection intervals

3.5.1. Lifecycle performance

![](_page_5_Figure_2.jpeg)

Fig. 5 Degradation processes

Steel-composite bridges are affected by various degradation processes such as carbonation, corrosion and fatigue, see Fig. 5. For each bridge component the long term behaviour must be described to be included in LCA and LCC analyses. A scheduling of inspections and maintenance actions should be done based on a detailed description of the lifecycle performance of the affected details. Thus, lifecycle costs and

emissions can be reduced. Intervals of bridge inspections can also be optimized by the knowledge of the adequate non-destructive testing methods to early detect defects. A comparison of these methods is done during fatigue tests on a standard detail of steel girders, the transverse stiffener.

#### 3.5.2. Fatigue tests and non-destructive testing on steel girders

The detail of transverse stiffeners is a typical decision detail to guarantee the safety of steel and steel-composite bridges, especially in regard of fatigue. An optimization of this detail in regard of a long-living structure is to be achieved. To extend the service life of welded details under fatigue loading promising results have been gained by Dürr [3] with the application of ultrasonic impact treatment (high frequent hammering) as post-weld treatment method. Therefore, the lifetime of the welded detail is investigated in tests applying postweld treatment. By this high frequent hammering on one hand the geometry at the weld transition is improved and on the other hand the notch effect is reduced by introduction of compressive stresses at the surface. In a test series of 2 standard girders and 2 girders with

![](_page_5_Figure_8.jpeg)

#### Fig. 6 Test set-up for fatigue tests

improved detail the fatigue behaviour of transverse stiffeners is compared to existing data in the commentary to [4]. The set-up of the fatigue tests is shown in Fig. 6. For the standard girders (T1 and T2) a number of cycles N = 500,000 was calculated and post-weld treatment by high frequent hammering was expected to double the number of cycles (T3 and T4). With N1= 520,200 and N2= 702,000 cycles the test reproduced well the tests found in the commentary to [4] (3 4 NCL 16 – 18). The weld of T3 was improved by high frequent hammering, here pneumatic impact treatment was used [5], lead to a breakthrough of the flange after N3= 1,929,000. After N4= 4,500,000 cycles the test of T4 had to be stopped without any fatigue occurring at the transverse stiffeners. Fig. 7 illustrates in the S-N curve of the transverse stiffener the improved fatigue behaviour of T3 and T4 due to the post-weld treatment.

![](_page_6_Figure_0.jpeg)

Fig. 7 Test results compared with S-N curve EN 1993-1-9 (m=3 and  $\Delta \sigma_c=80$ N/mm<sup>2</sup>)

Cracks due to fatigue may result in the loss of structural capacity and may cause costly maintenance actions if they are not detected in time. Inspection intervals are closely linked to the probability of detection of defects and thus influence the sustainability analysis. Non-destructive testing methods (NDT) were applied to the steel girders during the fatigue tests in order to detect cracks and compare the methods regarding application to conditions such as coating. Cracks were made visible with magnetic particle testing (MT) comparing the use of a black suspension, a red powder and fluorescent particles under ultraviolet light. Penetration testing (PT) was performed with a red colour and yellow fluorescent colour under ultraviolet light. An optimization of detection was performed by applying the innovative solution of ultrasonic testing by Phased Array technique.

![](_page_6_Picture_3.jpeg)

Fig. 8 Crack detection with Phased Array

![](_page_6_Picture_5.jpeg)

Fig. 9 Crack detection with PT and MT

It was shown that by the method of Phased Array (Fig. 8) it is possible to detect cracks earlier than by the common methods. For the penetration testing the crack has to be open towards the surface to enable any penetration. An idea of the crack depth due to the amount of ink leaking out was gained; see Fig. 9 at the top. Especially the magnetic particle testing under ultraviolet light showed cracks in a very precise and thin way. Its limitation was reached with the coating as no detection was possible there; see Fig. 9 at the bottom. These first results will be introduced to the improved lifecycle performance of the fatigue detail and just be one of the analysed detail throughout the project.

## 3.6. Summary

From the performed analysis in the frame of a running European research project it can be seen that bridges need to be optimized in a holistic way combining the environmental aspects, costs and social and functional factors throughout the entire lifecycle. Assumptions were made for the lifecycle scenario and maintenance actions. These influence not only the lifecycle assessment but also lifecycle and user costs. Optimized crack detection during inspections results in minimisation of maintenance needed. The application of post-weld treatment may increase the fatigue resistance of critical details remarkably.

These items are of great importance in order to come along with the future demands on bridge structures resulting from the ever increasing traffic volume within the European road network. The research work therefore aims to point out all the benefits of steelcomposite road bridges regarding sustainability.

The authors as project coordinators express their gratitude to the Research Fund for Coal and Steel of the European Commission for the project funding. They also like to underline that the presented results are based on the work of the whole project team and like to thank the partners coming form the University of Coimbra, Dillinger Hütte, Arcelor Mittal, Laboratoire Central des Ponts et Chaussees, Rambøll Denmark, Brisa Engenharia e Gestão, SETRA and BASt for their collaboration.

![](_page_7_Picture_4.jpeg)

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# 4. STEEL-CONCRETE COMPOSITE BRIDGES WITH HORIZONTALLY LYING SHEAR STUDS AND CORRUGATED WEBS

# 4.1. General

Headed studs arranged close to the concrete edge, especially loaded in direction to the free edge, show a lower static resistance and partly a different behaviour than studs further away from the concrete edge. Whereas the static resistance for both load directions and the fatigue strength for a loading parallel to the free edge are well investigated in earlier research works, there is no design rule for the fatigue strength for the load direction orthogonal to the free edge. Based on push-out tests, some information is given about the fatigue strength for a loading orthogonal to the concrete edge subjected to the edge distance.

For large span bridges the use of deep plate girders results in slender webs, thus making the web buckling problem more relevant in design. The use of corrugated steel plates as plate girder web is a possible way of achieving adequate out-of- plane stiffness and buckling resistance without the need to use stiffeners or thicker plates. An interesting development is the application of corrugated steel webs in composite girders. Omitting the steel flange and using the corrugation and e.g. headed studs close to the surface as shear connectors leads to economic and constructional advantages.

# 4.2. Horizontally lying shear studs

# 4.2.1. State of the art

Independent of a vertical or a horizontal arrangement, see Fig. 10, so called horizontally lying shear studs feature a small edge distance  $a_r$  of the headed stud to the concrete surface. They allow for new interesting cross sections and applications. For bridges some examples are given in Fig. 11.

![](_page_8_Figure_0.jpeg)

Fig. 10 Vertical or horizontal arrangement of headed shear studs with edge influence

![](_page_8_Figure_2.jpeg)

Fig. 11 Some examples for horizontally lying shear studs in bridges, a) Connection between longitudinal steel girder and concrete cross girder, b) Edge girder of an arch bridge, c) Application in composite girders with corrugated steel webs

Depending on the load direction parallel or orthogonal to the free concrete edge the behaviour and resistance differ. Shear in longitudinal direction parallel to the edge occurs in general due to beam action and shear in transverse direction orthogonal to the concrete surface may arise due to self weight and direct loading e. g. by the wheel of a vehicle. Substantial fatigue loads in vertical direction may derive from concentrated traffic loads like wheel loads.

Numerous experimental and numerical investigations [6], [7] have lead to design rules for static resistance for both load directions and furthermore for fatigue resistance under longitudinal shear which meanwhile have been implemented in [8].

![](_page_8_Figure_6.jpeg)

Fig. 12 Geometrical parameters of shear connections with horizontally lying shear studs

The resistance is influenced by the reinforcement. Therefore, beside the load direction the resistance is specified depending on the effective edge distance  $a_r'$ , which is the clearance between the headed stud and the outer reinforcement layer, see Fig. 12. While the effective

edge distance  $a_r$  for longitudinal shear is interesting on both sides of the studs the edge distance for transverse shear is mainly important on the side of the loading point. With decreasing of the effective edge distance cracking of the concrete gets more important and stud failure is dominated by the concrete cracking. An arrangement in middle position leads to a higher resistance than in the edge position.

As a result of a research project [9] provides a design rule for horizontally lying headed studs under longitudinal shear and cyclic loading, eq. (1):

$$(\Delta P_{R,L})^m \cdot N_R = (\Delta P_{c,L})^m \cdot N_c$$

(1)

- with  $\Delta P_{R,L}$  Fatigue shear strength based on the range of shear force per stud in longitudinal direction [kN]
  - $\Delta P_{c,L}$  Reference value at  $N_c = 2 \cdot 10^6$  cycles depending on the effective edge distance  $a_r$

a <sub>r</sub> ' [mm]	50	≥100
$\Delta P_{c,L}$ [kN]	24.9	35.6

For  $50 < a_r' < 100 \text{ mm } \Delta P_{c,L}$  should be determined by linear interpolation.

- m Slope of the fatigue strength curve with the value m = 8
- $N_R$  Number of force range cycles

The diameter of the stud was not identified as an influencing factor. Therefore, in contrast to the design rule for headed studs without an edge influence, the design rule deals with forces and not with shear stresses. Similarly to headed studs without an edge influence, the fatigue behaviour of studs close to the edge surface under longitudinal cyclic loading features a two stage failure mechanism. Due to an early initiation of concrete crushing close to the weld-collar and concrete splitting, the stud is subjected to bending at an early stage of fatigue loading. However, forced by a splitting action, for horizontally lying studs the degree of concrete damage is higher, which leads to a slightly reduced fatigue strength compared with studs without edge influence. With increasing edge distance, the influence of splitting action decreases and the fatigue strength life aligns to that of studs without edge influence.

4.2.2. Investigation on the fatigue strength under transverse loading

Two test series on push-out specimens were performed to investigate the influence of the effective edge distance  $a_{r,o}$ . One test series had an effective edge distance  $a_{r,o}$  of 100 mm – test series QE1 – and the other one 50 mm – test series QE2.

![](_page_9_Picture_13.jpeg)

Fig. 13 Failure modes of the test specimens, a) Shearing of the studs, b) Concrete breakout, c) Stirrup failure

The existing experiences concerning the fatigue behaviour of headed studs only show shearing of the studs as stop criterion. Contrarily, in these tests three different failure modes occurred. Some specimens failed by shearing of the studs, too. But most specimens failed by concrete breakout. Two specimens showed a fatigue failure of the reinforcement. In both cases one stirrup failed in the upper bending, Fig. 13.

Contrary to headed studs without an edge influence only a slight concrete crushing at the weld collar could be observed. Concrete cracking at an early stage is essential for the fatigue behaviour. The resulting increasing slip causes bending in the stud shank.

Based on the test results and the experiences on the fatigue behaviour of horizontally lying headed studs under longitudinal loading as well as headed studs without an edge influence the fatigue strength for headed studs with edge influence under transverse loading has been derived in [10] as follows, eq. (2):

$$(\Delta P_{R,V})^m \cdot N_R = (\Delta P_{c,V})^m \cdot N_c$$

with  $\Delta P_{R,V}$  Fatigue shear strength based on the range of shear force per stud in transverse direction [kN]

 $\Delta P_{c,V}$  Reference value at  $N_c = 2 \cdot 10^6$  cycles depending on the effective edge distance  $a_{r,o}$ 

1

a <sub>r,o</sub> ' [mm]	50	≥100
$\Delta P_{c,V}$ [kN]	8.9	27.7

For  $50 < a_{r,o}' < 100 \text{ mm} \Delta P_{c,V}$  should be determined by linear interpolation.

- m Slope of the fatigue strength curve with the value m = 8
- N<sub>R</sub> Number of force range cycles

 $P_{Rd}$  Design value of the resistance of horizontally lying shear studs under vertical shear

Peak load per stud in vertical direction
Effective edge distance; $a_{r,o}' = a_{r,o} - c_v - d_s/2$ [mm]
Diameter of the stirrups
Diameter of the longitudinal reinforcement
Diameter of the shank of the studs
Characteristic cylinder strength of the concrete

For the static resistance the diameter of the stirrups in [8] when using horizontally lying studs is specified as at the least 8 mm. For the fatigue strength the minimum diameter has to be increased to 12 mm due to the two fatigue fractures of the stirrups. Based on the tested specimens the diameter of the shank of the studs is limited to at least 22 mm. Due to the force based design rule the result is on the safe side for headed studs with a shank diameter of 25 mm. For further considerations, see [11]. The fatigue strength was investigated for an edge position. It can be used for a middle position, also because these are more favourite conditions.

In parts this section underlies the research project FE-Nr. 15.0407/2004/CRB [10] on behalf of the Bundesministerium für Verkehr, Bau und Stadtentwicklung, represented by the Bundesanstalt für Straßenwesen. The authors of this paper are responsible for the content exclusively. The authors gratefully acknowledge the financial support of the research project by the Bundesministerium für Verkehr, Bau und Stadtentwicklung (BMVBS) and the technical and scientific steering of the Bundesanstalt für Straßenwesen (bast).

#### 4.3. Horizontally lying shear studs used for corrugated steel webs

## 4.3.1. General

Due to the features of the corrugation, see Fig. 14, the application of corrugated steel webs lead to advantages for composite constructions. The web has a small stiffness in longitudinal direction. In case of longitudinal prestressing the forces due to creep and shrinkage remain in concrete chords. The resistance against buckling, local and global, rises. So the number of stiffeners can be reduced significantly. In comparison to plain webs there is a high bending stiffness in transverse direction, which allows reducing the number of cross frames in box girder bridges.

![](_page_11_Figure_4.jpeg)

*Fig. 14 Load bearing of corrugated webs in dependence on load direction, a) Longitudinal direction, b) Shear, c) Transverse direction* 

#### 4.3.2. Shear connection

An important detail for functionality and cost effectiveness is the composite joint. In general, there is the solution with a steel flange and an embedded steel web in the concrete chord to differ, see Fig. 15.

![](_page_11_Figure_8.jpeg)

Fig. 15 Fundamental detailing of the composite joint, a) Steel flange, b) Embedded steel web

Omitting the steel flange leads to further advantages. The corrugation can be used as additional shear connector. By omitting the steel flange there are no work intensive weld seams necessary. Therefore, the fatigue critical details are reduced. In case of a transverse bending moment the design with a steel flange asks for shear connectors which are able to transfer tension forces too.

The composite joint is loaded in different directions: Shear in longitudinal and transverse direction and bending in transverse direction. Tests under longitudinal shear and transverse bending moment with an embedded corrugated web have been carried out [12]. Besides

headed studs concrete dowels have been investigated as additional shear connector. In order to get information about the behaviour and resistance of the corrugated web as single shear connector the test program implied some tests without any additional shear connectors. Beside the shear connector the embedment depth was the main investigated parameter.

#### Longitudinal shear transfer

Four out of altogether 22 tests were performed without any additional shear connector, eleven with headed studs and seven with concrete dowels.

The tests show the ability to carry alone considerable forces by the corrugated web. Additional headed studs or concrete dowels are able to increase the resistances. Fig. 16 shows the typical load-slip graph of specimens with the varied connectors. The maximum shear capacity of the specimen without additional connectors  $P_{e,ref}$  is normalised to 1.0. The graphs show the influence of the longitudinal shear behaviour and shear capacity of the shown arrangements of connectors.

![](_page_12_Figure_4.jpeg)

*Fig. 16 Load-slip behaviour of typical connector arrangements, a) Arrangement of studs and dowels, b) Load-slip graph* 

In general, the tests showed a high ductility. Only some tests under longitudinal shear force with the corrugated steel web without a single shear connector showed sudden slip before reaching the maximum load, which should be prevented in practice.

The embedment depth of the corrugated web features a big influence on the resistance. The design of the composite joint shows a couple of further important parameters like the web thickness, the concrete and steel strength, the amount and arrangement of reinforcement, the number and arrangement of stude etc..

#### Transverse bending moment

The nine conducted tests differ in the shear connector type, the embedment depth of the corrugated web and a supplementary reinforcement ahead of the steel web. The specimens were bolted on the strong floor and loaded with an eccentricity of 1.0 m, see Fig. 17.

![](_page_13_Figure_0.jpeg)

Fig. 17 Test setup for transverse bending moment (Dimensions in cm)

From the tests it has been obvious that resistance and stiffness are drastically increased by the additional shear connectors compared to the pure sheeting. Also a supplementary transverse reinforcement bar just above the embedded corrugated web proved to be highly efficient. The recalculation by FE models could simulate the real failure behaviour quite well. It showed that a compression zone develops at the top of the concrete chord and a tension zone exists directly above the embedded corrugated web. This might be explained with the help of a typical strut and tie model usually used in reinforced concrete design. The load transfer at the embedded web initiates a compression strut at the top of the concrete and a tension tie just above the web. As a consequence, the transverse bending moment also induces tension stresses into the headed studs used as connectors. The tension stresses have to be considered for the design.

# Conclusions

First conclusions regarding the behaviour of horizontally lying studs at corrugated webs may be summarised:

- The corrugated web itself forms an own shear connector.
- Headed studs/concrete dowels can increase the shear resistance significantly.
- The shear connection is very ductile
- and transfer of longitudinal as well as transverse shear and bending is possible.

Acknowledgement is made to the German Research Association for Steel Application (FOSTA) for the funding of this research project [12] and to the colleagues of the team of Prof. Novák for the fruitful cooperation.

# 5. STEEL-COMPOSITE BRIDGES MADE OF HOLLOW SECTIONS

# 5.1. General

Spatial trusses made of circular hollow sections (CHS) have become more and more popular in steel-concrete composite highway bridges over the last 15 years in Europe forming an innovative and aesthetic alternative to the conventional all-steel bridges. The CHS trusses of these bridges normally comprise rising and falling (not crossing) braces and a continuous bottom chord, Fig. 17. At the bottom chord the connection of the truss members forms a so called uni-planar K-joint. Multi-planar K-joints in spatial trusses are also

common. The top chords of the trusses may be integrated within the concrete slab of the bridge.

The advantages of circular hollow sections are their stiffness (independent of their orientation) and their rigidity, the low risk of buckling together with their high resistance against torsion. Truss joints have often been carried out as cast steel joints because of the complex geometry of the brace to chord intersection. With the introduction of the CNC controlled tube cut-off technology it is possible to connect the braces directly to the chord tube by welding. Today, computerized numerically controlled cutting systems (flame or plasma) are able to cut even the most difficult weld preparations. Consequently, welding the braces directly on the bottom chord forms an interesting alternative from the viewpoint of economics and ease of construction. Generally, arc welding process 135 or 136 according to [13] are preferred.

![](_page_14_Figure_2.jpeg)

Fig. 18 Steel-composite bridge in Germany (Image source: Stahl-Informations-Zentrum), K-joint geometry

In order to verify a tubular truss structure used for a highway bridge e. g. against fatigue failure the S-N method cannot be used anymore due to the complex geometry of the welded connections. The more sophisticated hot-spot stress approach has to be applied. The spots of stress concentration within the brace-to-chord intersection are referred to as hot spots. The magnitude of the stress concentration is highly dependent of the geometry of the truss members. In summary, the hot spots are crucial for the fatigue strength of the entire truss structure.

Although hollow section joints are also widely used in crane and offshore structures, the available fatigue strength values from literature cannot be used for bridges, since they are related to K-joints with relatively high  $\gamma$ -values,  $\gamma = d_0/(2 \cdot t_0)$ . Trusses in bridges show relatively low  $\gamma$ -values, normally. Furthermore, the manufacturing of the K-joints in bridges is different. While the hollow section joints in offshore structures are usually accessible and thus may be welded from both sides, this is not possible for bridge structures.

The fatigue behaviour of tubular joints is extremely depending on its proportions. For this reason a research project has recently started in order to investigate fatigue behaviour particular for welded hollow section joints typical of steel-composite bridges [14].

#### 5.2. Research project

#### 5.2.1. Scope

In cooperation with 4 research partners (Universität Stuttgart, Institut für Konstruktion Entwurf; Universität der Bundeswehr München, Institut für Konstruktiven Ingenieurbau, Fachgebiet Stahlbau; Fachhochschule München, Labor für Stahl- und Leichtmetallbau; Schweißtechnische Lehr- und Versuchsanstalt Halle GmbH) a research project on fatigue design of trusses made of circular hollow section with thick-walled chords has been launched planning over 90 large-scale tests and several small-scale tests [14]. This project is based on the finalized research project "Recommendations for welded KK-joints in highway bridge structures" [15], financed by the German Ministry of Transport, Building and Urban Affairs (BMVBS) and accompanied by the German Federal Highway Research Institute (BASt). In [15] the joint proportions typical of tubular trusses with K-joints in highway bridges have been identified, see Eq. (3) - (6). Furthermore, it is recommended in [15] to avoid joints with overlapping braces for highway bridges.

$0.50 \le \beta = d_1/d_0 \le 0.60$	(3)
$4 \le \gamma = d_0 / (2 \cdot t_0) \le 12$	(4)
$0.25 \le \tau = t_1 / t_0 \le 0.75$	(5)
$\varphi = 90^{\circ} \text{ and } 45^{\circ} \le \theta \le 60^{\circ}$	(6)
with $d_1, t_1 \dots$ diameter and wall thickness of brace	
and $d_0, t_0 \dots$ diameter and wall thickness of chord	

From the cyclic testing on uni-planar K-joints with highway bridge specific geometry in [16] it is known that the fatigue failure is usually initiated at the weld toe of the chord in the gap region, differently to cast joints exhibiting a crack initiation starting at the weld roots of the butt joints on the inner surfaces of the hollow sections. Thus, welded K-joints with gap are advantageous since they allow for a visual inspection of the weld toes in the critical zones. Additionally, such joints offer the opportunity of a future post-weld treatment [3] of the welds to extend the service life. Whether cracks still start from weld toes after being post-weld-treated, has still to be verified within the new research project. The geometry of test specimens for the planned large-scale tests under dynamic loading is illustrated in Fig. 19. The weld preparation of the test specimens is also shown in Fig. 19. For more details see [14].

![](_page_15_Figure_5.jpeg)

Fig. 19 Joint proportions of the test specimens and welding preparation for brace-chord-intersection

#### 5.2.2. Experimental investigations

For fatigue assessment particular fatigue load models [18] have to be applied to the bridges inducing internal actions of the trusses e. g. overall bending moments and shear forces. For the single K-joint the bending moment of the truss results in chord action and the shear forces of the truss in brace action, Fig. 20. It is impossible to determine the fatigue strength for each single combination of chord and brace action by testing. Hence elementary loading conditions have to be investigated: axial brace or chord loading, in plane brace or chord bending and so forth and an interaction formula for combined brace and chord action has to be developed. In order to confirm this interaction formula several joints are tested under combined loading conditions in addition to a large-scale girder test with spacial KK-joints.

![](_page_16_Figure_2.jpeg)

Fig. 20 Elementary load cases, test setup for axial brace loading

![](_page_16_Picture_4.jpeg)

Fig. 21 Welding preparation for fillet weld transition, welded joint

Due to the low frequency (1-2 Hz) of the cyclic testing, the crack propagation can be documented quite well. There are several definitions of failure criteria given in literature [16], [20]:

- N1 : Number of cycles to a 15% change in strain near the point of crack initiation
- N2 : Number of cycles at detection of crack
- N3 : Number of cycles to through-thickness cracking of the fatigue damaged member
- N4 : Number of cycles to complete loss of static joint strength

From the different failure criteria given in literature the criteria N1, N2 and N3 are recorded for the tests under brace loading for example. The cracks are always initiated at the weld toes on the chord in the gap region, Fig. 22. Criterion N1 is checked by 3mm strain gauges placed at both welding toes between the braces in the gap region. N2 is recorded through permanent inspection. N3 is checked by setting the chord under pressure. The sudden pressure release indicates the through-cracking of the chord wall.

![](_page_17_Figure_0.jpeg)

Fig. 22 Documentation of crack propagation and crack location with macrosections for the tested specimens

#### 5.2.3. Numerical investigations

Due to the complex geometry at the brace-to-chord intersection the definition of a nominal stress for the K-joint is not possible any more. The nominal stress is the most important precondition if the classification method, the standard approach of Eurocode 3 [4] is used. For that reason, the fatigue verification has to be performed on the basis of structural stresses. This approach, also referred to as hot-spot stress concept [21], accounts for the stiffening effect of the welds.

The underlying principle of the hot-spot stress concept can be summarized as follows: The verification is concentrated on the weld connection type (such as the fillet weld) rather than on the constructional joint (such as the K-joint). It is assumed that each weld detail is embedded in a structural stress field. The structural stress  $\sigma_{hS}$  and the notch stress  $\sigma_{notch}$  have a constant ratio for each weld connection type, Fig. 23.

![](_page_17_Figure_5.jpeg)

Fig. 23 Hot spot stress method

$$\sigma_{hs} = SCF \cdot \sigma_{nom} \cdot \gamma_{Ff} \le \frac{\Delta \sigma_c}{\gamma_{Mf}}$$

$$\sigma_{hs} \qquad \text{hot-spot stress} \\ \sigma_{nom} \qquad \text{nominal stress} \\ \Delta \sigma_c \qquad \text{fatigue resistance based on hot-spot stresses}$$

$$(4)$$

 $\gamma_{Ff}, \gamma_{Mf}$  partial safety factors

For the trusses of highway bridges several adjustments of Eq. (4) are necessary in order to account for the low  $\gamma$ -ratios of chord diameter and wall thickness. On one hand these joints exhibit lower hot-spot stresses  $\sigma_{hs}$  and on the other hand their fatigue strength  $\Delta \sigma_c$  is rela-

tively low compared with the fatigue strengths given by Eurocode 3 because of the so-called scale effect.

In the above mentioned, already finalized research project [15] SCF-values for multiplanar K-joints were derived for several elementary load cases. Thus, also the determination of the K-joints action can be simplified. The aim of the still-ongoing research project is to reveal the magnitude of the scale effect.

# 5.3. Summary

Steel-composite bridges made of hollow sections form an especially aesthetic solution for highway bridges. In order to verify a tubular truss structure used for a highway bridge e. g. against fatigue failure the common S-N method cannot be used anymore due to the complex geometry of the welded connections. The more sophisticated hot-spot stress approach has to be applied. The magnitude of the stress concentration is highly dependent of the geometry of the truss members. Therefore, the available fatigue strength values from literature cannot be used directly for bridges, since trusses in bridges normally show relatively low slenderness values of the chords ( $\gamma = d_0/(2 \cdot t_0)$  between 4 and 12). Furthermore, the manufacturing of the K-joints in bridges is different because the hollow section joints are not accessible from inside and therefore have to be welded from one side. For this reason, a research project has recently started in order to investigate the fatigue behaviour particular for welded hollow section joints typical of steel-composite bridges. First results and explanations of this project are given.

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# 6. CONCLUSIONS

From the performed analysis sustainability of steel-concrete bridges it can be seen that bridges need to be optimized in a holistic way combining the environmental aspects, costs and social and functional factors throughout the entire lifecycle. Assumptions were made for the lifecycle scenario and maintenance actions. These influence not only the lifecycle assessment but also lifecycle and user costs. Optimized crack detection during inspections results in minimisation of maintenance needed. The application of post-weld treatment may increase the fatigue resistance of critical details remarkably. These items are of great importance in order to come along with the future demands on bridge structures resulting from the ever increasing traffic volume within the European road network.

Omitting the steel top flange of a composite girder and using the corrugation of its web and e.g. headed studs close to the surface as shear connectors leads to economic and constructional advantages. Rules for the application of horizontally lying studs welded to plane steel webs are meanwhile given also in [8]. Additional considerations concerning the fatigue resistance under transverse shear are explained among others in this paper. Also first conclusions regarding the behaviour of horizontally lying studs at corrugated webs have been summarised. The investigations show that the corrugated web itself forms an own shear connector, that the headed studs can increase the shear resistance significantly resulting in a very ductile shear connection and that the transfer of longitudinal as well as transverse shear and bending is possible also without have a top steel flange.

Steel-composite bridges made of hollow sections form an especially aesthetic solution for highway bridges. In order to verify a tubular truss structure used for a highway bridge against fatigue failure the common S-N method cannot be used anymore due to the complex geometry of the welded connections. The more sophisticated hot-spot stress approach has to be applied. However, for the trusses of highway bridges several adjustments are necessary in order to account for the low  $\gamma$ -ratios of chord diameter and wall thickness. On one hand these joints exhibit lower hot-spot stresses  $\sigma_{hs}$  and on the other hand their fatigue strength  $\Delta \sigma_c$  is relatively lower compared with the fatigue strengths given by Eurocode 3 because of the so-called scale effect. The aim of the still-ongoing research project is to reveal the magnitude of the scale effect. In the already finalized project on recommendations for KK-joints SCF-values for multi-planar K-joints were derived for several elementary load cases. Thus, also the determination of the K-joints action can be simplified and the joint fatigue verification becomes feasible also for use in practice.

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