Hot-Dip Galvanizing in Bridge Construction

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Abstract The research project "Hot-Dip Galvanizing in Steel and Composite Bridge Construction" has demonstrated, among other findings, that hot-dip galvanizing of bridge structures can achieve a maintenance-free corrosion protection duration of 100 years. To achieve this protective effect, constructive and manufacturing requirements must be considered. The selection of steel grades is determined by the static requirements of the bridge. In addition to corrosion protection design in accordance with DIN EN ISO 12944-3 and DIN EN ISO 14713-1, additional requirements corresponding to the design for hot-dip galvanizing according to DIN EN ISO 14713-2, Annex A, and DASt Guideline 022, as well as manufacturing suitable for hot-dip galvanizing, must be taken into account.

Bridge main beams with component lengths over 16–18 meters must be segmented and joined by weldable assembly joints. The location of the weld joints should be chosen considering possible maintenance work.

As part of a research project, the Chair of Metal Structures, in collaboration with the Chair of Concrete and Masonry Structures of the Technical University of Munich and the Bundesanstalt für Materialforschung und -prüfung (BAM), developed a new type of galvanization-compatible grouted joint connection using ultra-high performance, fibre-reinforced concrete. In contrast to existing solution as welding the segments on site or using prestressed slip-resistant bolt connections, the hot dip galvanization of the grouted joint remains undamaged for the whole service life time of the bridge structure.

This paper aims to explain the constructive and manufacturing requirements for hot-dip galvanized bridge structures. Additionally, examples of existing hot-dip galvanized bridges were summarized.

1 Introduction

A significant disadvantage of steel bridges with organic coatings as corrosion protection systems is the need for recurring maintenance of the coating. However, through modern hot-dip galvanization with coating thicknesses starting at 200 μm, the corrosion protection system can be provided largely maintenance-free over the 100-year lifecycle of the bridge. This offers significant financial benefits and a considerable reduction in economic costs due to traffic disruptions. Currently, steel components up to 16 meters in length (19 meters in specific cases) can be hot-dip galvanized, which requires segmentation and subsequent joining for larger spans. I-beam sections can be joined using slip-resistant bolted connections, though these are aesthetically controversial and critical regarding the integrity of the zinc coating. Joints for box sections can be provided by welding and using thermal zinc spraying as corrosion protection in the area of the joint. However, this solution is not entirely maintenance-free due to the service life of thermal zinc spraying. Therefore, a robust, maintenance-free connection system is of great interest. For this reason, a grouted joint, which provides a continuous hot-dip galvanization without the need for subsequent bolting or welding after the assembly of the steel segments has been developed in the course of the research project IGF No. 20312N.

2 Hot-dip galvanizing

The process of hot-dip galvanizing is described in DIN EN ISO 14713-2 [4] as the application of a zinc or zinc-iron alloy coating to iron or steel products by immersion in molten zinc. This process is crucial for corrosion protection, with the formation of the coating being influenced by various factors, including the chemical composition of the steel, the zinc bath, the galvanizing temperature, and the technology used. A distinction is made between continuous and batch hot-dip galvanizing processes. The latter, such as piece galvanizing, are widely used in Germany, particularly in metal and steel construction, as well as in the infrastructure sector. In piece galvanizing, the steel component undergoes several preparatory processes before being immersed in the zinc bath at approximately 450°C, where a zinc-iron alloy layer forms through diffusion. The process can be carried out as dry galvanizing, where the component is dried before immersion, or as wet galvanizing, where this drying step is omitted.

The standard DIN EN ISO 1461 [2] specifies the requirements for the composition of the zinc bath, which must consist of at least 98% zinc, as well as the testing of the coatings. Additionally, the DASt guideline 022 [5] further specifies the chemical requirements for load-bearing steel components to prevent cracking. Thorough surface preparation is essential for the quality of the zinc coating, as insufficient cleaning or drying can lead to defects such as uneven layers or sticky zinc ash. Moreover, the uninterrupted inflow and outflow of the zinc bath and surface preparation fluids are crucial for achieving optimal galvanizing results.

The layer formation in batch hot-dip galvanizing is based on the reaction between zinc and iron, where intermetallic Fe-Zn phases are formed through thermal diffusion. The nature of these phases and the thickness of the zinc coating are primarily determined by the chemical composition of the steel, the temperature of the zinc bath, and the immersion time. At approximately 450°C, a robust zinc coating forms, which can be divided into four phases and is characterized by a final pure zinc layer. The alloying elements silicon (Si), phosphorus (P), and aluminum (Al) in the steel significantly influence the coating thickness, adhesion, and phase growth. Scanning electron microscopy studies have shown that the hydrogen diffusion during the pickling process varies depending on the silicon content of the steel. Based on silicon content, steel is classified into four types (see Figure 1): low-silicon steel (<0.03% Si), Sandelin steel (0.03–0.14% Si), Sebisty steel (0.14–0.25% Si), and high-silicon steel (>0.25% Si). This classification affects the zinc coating thickness, with temperature also playing a crucial role.

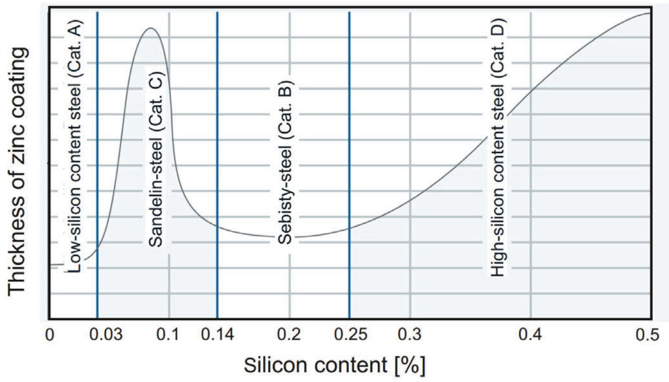

Figure 1: Schematic zinc coating thickness in dependence of the temperature and the silicon content of steel [6]

The DASt guideline 022 [5], first introduced in 2009 and revised in 2016, is mandatory for hot-dip galvanized, load-bearing steel structures. It aims to prevent cracking caused by the galvanizing process and includes a simplified testing procedure without complex calculations. Steel components are initially classified into construction classes (I, II, III) and detail classes (A, B, C) based on specific material and component parameters. Depending on the classification, post-galvanizing inspection falls into one of three trust zones, involving visual inspections and, if necessary, magnetic particle testing. The guideline also sets the maximum immersion time for components in the zinc bath, limiting it to 27 minutes for material thicknesses over 30 mm, although this duration is rarely reached.

3 Design of galvanized bridges

When designing galvanized bridges, several key factors need to be considered to ensure long-term durability and effectiveness of the corrosion protection: material selection, design considerations, construction details, segmentation and joints, galvanizing thickness, inspection and quality assurance and maintenance. By addressing these aspects during the design phase, the effectiveness of the galvanizing process can be maximized, leading to a durable and maintenance-free bridge structure. A practical implementation of the latest scientific findings is provided by a working aid from the Hot Dip Galvanizing Institute. In [8, 22] the key factors for the design of hot-dip galvanized bridge structures are outlined:

- Corrosion protection verification
- Steel selection and construction guidelines
- Fatigue resistance verification
- Execution and inspection of hot-dip galvanizing
- Execution of assembly welds
- Repair of zinc coating damage

The research project “FOSTA P835 – Hot-Dip Galvanizing in Steel and Composite Bridge Construction” [7] has demonstrated that hot-dip galvanizing of bridge structures can achieve a maintenance-free corrosion protection duration of up to 100 years. In environments corresponding to Corrosivity Category C4 and with increased exposure to de-icing salts, a zinc coating thickness exceeding $250\mu m$ is required to attain this level of protection. For bridges with average or lower exposure to de-icing salts, a coating thickness of more than $200\mu m$ is sufficient to ensure a protection duration of 100 years.[8]

The selection of steel grades is dictated by the structural requirements of the bridge. When considering hot-dip galvanizing, the steel’s chemical composition must be specified. Steels compliant with DIN EN 10025-2, Section 7.4.3 “Hot-Dip Galvanizing,” [9] must be used, featuring the following silicon and phosphorus content: $0.14 \leq$ Si ≤ 0.35 and P ≤ 0.035 percent by weight.

In addition to corrosion protection design according to DIN EN ISO 12944-3 and DIN EN ISO 14713-1 [10], further hot-dip galvanizing requirements outlined in DIN EN ISO 14713-2, Annex A [4], and DASt guideline 022 [5] must be taken into account. Primary bridge girders exceeding 16-18 meters in length must be segmented and connected with weldable assembly joints. The placement of weld seams should be chosen with future maintenance work in mind. For weld seams, only repair by thermal spraying is allowed.

Unlike organic coatings, galvanizing protects internal surfaces from corrosion and eliminates the need to ensure the airtightness of hollow sections. This allows for more cost effective designs, such as omitting the top steel flange and using shear connectors instead. Sufficient openings for ventilation and fluid flow must be provided to ensure uniform immersion in the zinc bath and avoid temperature variations for proper hot-dip galvanizing. DIN EN ISO 14713-2 [4] provides guidelines for these openings, which should be large enough to prevent air entrapment and ensure the removal of treatment fluids. Openings should be designed to balance technical and structural requirements, with rounded cuts preferred over straight cuts to facilitate zinc flow. Attention should be paid to edge quality and surface preparation in accordance with DIN EN 1090-2 [11] and DIN EN ISO 14713-2 [4]. Burned edges should be smoothed or flame hardened to reduce the risk of cracking. Limits on sheet thickness and immersion times to prevent local overloading are suggested in DASt Guideline 022 [5]. Typically, steel thicknesses up to 30 mm are manageable without further verification, while thicker plates require controlled immersion times. The dimensions of the components must match the size of the available galvanizing bath, with single dips being preferred to avoid distortion and cracking. Surface quality should be in accordance with DIN EN 10163-2 [12] and preparation grade P3 of DIN EN ISO 8501-3 [13], including rounding of thermally cut edges and smoothing of welds.

The verification of fatigue resistance must be conducted in accordance with DIN EN 1993-2 [14], DIN EN 1993-1-9 [15], and DIN EN 1994-2 [16], taking into account the notch cases for hot-dip galvanized construction details as specified in [22]. These guidelines already account for the fatigue strength of hot-dip galvanized steel, including the reductions associated with the galvanizing process.
Corrosion protection through hot-dip galvanizing (batch galvanizing) must be carried out in accordance with DIN EN ISO 1461 and DASt guideline 022, ensuring that the specified minimum zinc coating thicknesses for the bridge structure are achieved. The verification of zinc coating thickness must follow the procedures outlined in DIN EN ISO 1461. Per the requirements of ZTV-ING, control surfaces should be established at specified locations as detailed in the aid manual. Besides visual inspections, systematic testing using the magnetic particle method, as prescribed in Annex 3 of DASt guideline 022, is required for bridge components.

4 Application examples of hot-dip galvanizing in steel composite bridge construction

There are only a few examples of hot-dip galvanized bridges in Germany, and these have had relatively short service lives. Long-term experience comes mainly from bridges abroad, some of which are over 70 years old. Studies show that hot-dip galvanized bridges can last more than 100 years without maintenance. One example is the Ehzer Bridge in the Netherlands, built in 1945, which in 2014 still had good zinc coating thicknesses ranging from $69\mu m$ to $200\mu m$. In addition, the 90 metre long Lier Bridge in Belgium, built in 1993, could last another 150 years based on the zinc coating thickness of over $300\mu m$ measured in 2014. Other examples can be found in Japan, England and the USA. [17]
In 2014, the Short Span Steel Bridge Alliance (SSSBA) and West Virginia University collaborated on the development of a modular, hot-dip galvanized, curved bathtub girder (PBFTG) in the USA. It is designed for steel bridges with spans of up to approximately 18 metres. According to the manufacturer, Valmont, the hot-dip galvanization is expected to last at least 60 years [18]. The steel section of the girder consists of flat trapezoidal boxes made of cold-formed steel sheet. Headed stud shear connections are attached to the top flange for connection to the concrete deck. The concrete can be cast either in-situ or at the factory, with the longitudinal grouting joints filled with UHPC on site if cast at the factory.
The 2009 DASt guideline 022 had a limit of 2% for cold forming before hot dip galvanizing. However, this has been removed in the revised version and instead only the minimum bending radius according to DIN EN 10025 and DIN EN 10219 has to be respected, depending on the steel grade and material thickness. This means that the use of modular PBFTG beams could, in principle, also be implemented in Germany.

The first hot-dip galvanized integral frame bridge in composite construction was built in 2016 in North Hessen as part of a pilot project to cross the A44 motorway between Kassel and Herleshausen (Figure 3). The single-span bridge with a span of 40 metres was developed in cooperation with research institutes and DEGES (Deutsche Einheitsstraßenplanung und -bau GmbH). The aim was

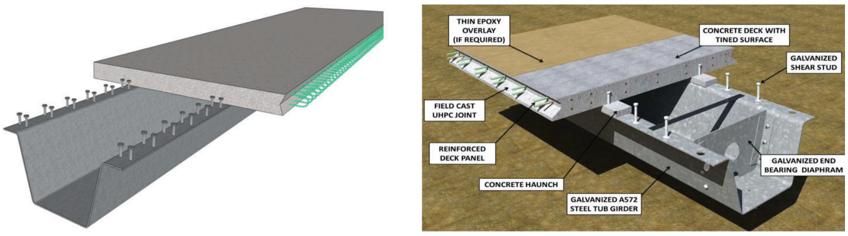

Figure 2: Press-Brake-Formed Tub-Girder - Konzept [19, 20]

to quickly apply and test research results in practice. For comparison, adjacent structures of similar design were coated with organic corrosion protection. The bridge has a two-girder plate girder cross-section with a variable construction height ranging from 1.40 m in the centre to 2.10 m at the supports. The deck has a total thickness of 37 cm, with 12 cm of precast and 25 cm of in-situ concrete reinforced with flat steel. S355 steel girders were used, while C35/45 concrete elements and B500B reinforcing steel were used. The 36 metre long steel beams were divided into three segments for hot-dip galvanizing to allow future repairs at the joints without closing the road. The segments were welded on site and protected against corrosion by thermal spray galvanising and sealing. A detailed field report of the pilot project is available in [21].

Figure 3: First hot-dip galvanized composite frame bridge in Germany over the BAB 44 motorway [21]

5 Joining techniques for hot-dip galvanized steel beams in bridge construction

5.1 Bolt joint

Both welded and bolted connections are available for open sections. The advantage of bolted joints is that the hot-dip galvanization of the components is maintained throughout. This ensures that there are no gaps in corrosion protection compared to welded joints. Bolted connections must use high strength bolts or non-slip pre-tensioned bolts due to fatigue stresses, with slip or clearance connections being unacceptable.

The coefficient of friction (μ) between surfaces is critical to slip resistance and is defined by DIN EN 1090-2. The 2018 revised standard includes values for hot-dip galvanized surfaces. The application of a zinc-silicate coating (e.g. ASi to TL 918300 sheet 85) with a thickness of 50-80 μm can improve performance. Recent research by the Technical Universities of Dortmund and Darmstadt and the

Institute for Corrosion Protection in Dresden has shown that zinc coatings of $\geq 200\ \mu$m lead to higher than expected pre-stress losses and lower coefficients of friction. This is attributed to increased creep effects with thicker zinc coatings. Despite this, bolted joints can still be viable in bridge construction with appropriate design adjustments and additional bolt tensioning.

Figure 4: Bolted plate joint of a composite bridge in Istanbul, Turkey [22]

5.2 Welded joint

One method of joining galvanized steel bridge segments is by welding. This process damages the hot dip galvanized coating in the area of the joints. Thermal spray galvanizing is a suitable repair method that has been proven effective in bridge construction [23, 24]. The process for thermal spray galvanizing of welds is described in DIN EN ISO 2063 [3] and involves the application of a zinc coating with adequate adhesion to blasted steel and swept galvanized surfaces.
Welded joints in bridge construction are typically designed as butt joints to resist fatigue and reduce notch effects in accordance with the requirements for notch category 112 according to DIN EN 1993-9:2010, Table 8.3 [15]. Welding and post-welding procedures include the grinding of welds flush with the plate surface in the direction of loading, the use and removal of weld metal and run-off pieces and grinding of plate edges flush in load direction and the non-destructive testing (NDT) of welds on both sides.
To ensure protection against corrosion in the area of the weld seams, a process was developed on the basis of and in addition to section 5.5 of ZTV-ING [25] - Weld seams on construction sites - in which the weld seams are sprayed in accordance with the specifications of DIN EN ISO 2063 [3] and then sealed pore-tight. The surface preparation and the work instructions for carrying out the spray metallisation according to [7] are shown in Figure 5:

- Area D: Cover unprocessed and intact hot-dip galvanizing with suitable work equipment when working in the weld seam area

- Surface C: Sweep the intact hot-dip galvanizing, average roughness depth min. Rz = 40 μm (G), coat with porous filler

- Surface B: Sweeping of the intact hot-dip galvanizing, average roughness depth min. Rz = 40 μm (G), thermal spraying with Zn or Zn/Al > 200 μm layer thickness and coating with pore filler

- Surface A: Blasting the steel to surface preparation level: Sa 3, degree of roughness: coarse (G), average roughness depth min. Rz = 85 μm (G), thermal spraying with Zn or Zn/Al > 200 μm and coating with pore filler.

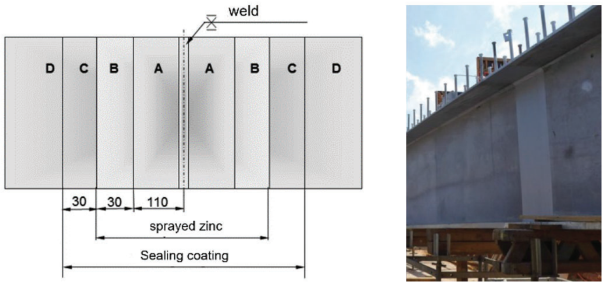

Figure 5: Illustration of the repair area (left); example of a repair in the weld seam area (right) [21]

The professional execution of assembly welds for corrosion protection of hot-dip galvanized steel components is described in detail in [17] and in the design aid for the use of hot-dip galvanized components in steel and composite bridge construction [22]. The hot-dip galvanized composite frame bridge over the BAB 44 motorway, which was implemented as a pilot project, serves as an application example.

The thermal spray galvanizing process has significant drawbacks. The process is labour intensive and requires extensive quality control. Unlike hot-dip galvanizing, thermal spraying does not form a metallic bond with the steel, resulting in a porous zinc layer that must be further coated with an organic filler to ensure durability. This additional coating must be an epoxy-based system, as polyurethane systems are not suitable for alkaline environments such as concrete. However, the use of aluminium in high alkali environments like fresh concrete is a problem because of excessive hydrogen production. Even with a pore filler, coating defects can occur during concrete placement, leading to potential delamination due to cathodic reactions.

According to DIN EN ISO 12944-5 [1], the protection times for this system are categorised as H and VH, corresponding to 15-25 years and over 25 years respectively. However, the standards do not address the duration of protection for areas that overlap with reinforced concrete bridge decks. Therefore, it is not possible to achieve lifetime corrosion protection at welded joints.

Maintenance and repair of spray-galvanized coatings would be complex and require road closures due to the size constraints of the galvanizing bath, negating many of the benefits of hot-dip galvanizing. In addition, the technology does not allow for the elimination of post-weld sealing of box sections, and achieving a visually consistent appearance with the zinc surface can be challenging.

5.3 Grouted joint for box girder cross-sections

As part of the research project IGF-Nr. 20312N [26], the Chair of Metal Structures, in collaboration with the Chair of Concrete Structures of the Technical University of Munich (TUM) and with the Bundesanstalt für Materialforschung und -prüfung (BAM), has developed an innovative, galvanisation-compatible grouted joint that features ultra-high performance fibre-reinforced concrete (UHPFRC) as grouting material. In contrast to conventional methods for connecting bridge

segments, such as welded joints or slip-resistant pre-stressed bolted connections, the grouted joint allows the entire steel surface to be hot-dip galvanised, preserving its integrity throughout the service life of the bridge structure. The connection technique developed is a grouted joint where galvanized bridge beams are joined to form a continuous box girder using an interlocking steel T-stub chamber system. The chambers are grouted with ultra high performance fibre reinforced concrete (UHPFRC). This approach is inspired by the 'grouted joints' used in offshore wind turbine technology. The key to the design of this joint was to ensure that the existing zinc coating on the steel components remains undamaged throughout the assembly process. In addition, the technique eliminates the need for bolting or welding after galvanizing, thus avoiding potential corrosion protection weaknesses from secondary fastening methods or subsewuent thermal spray galvanizing. Figure 6 illustrate the basic concept of this innovative grouted joint.
Hot-dip galvanizing as a corrosion protection system, eliminates the requirement for airtight welding, thereby negating the necessity for fully enclosed box sections. Consequently, the structurally inefficient top steel flange can be omitted, allowing for the use of a U-section with a top composite dowel instead. This partially compensates for the additional material and costs of the joint. To close the U-section for casting the half-finished slab (HFT slab), a prefabricated concrete slab, approximately 40 to 50 mm thick, is installed as a lost formwork between the steel web sections.

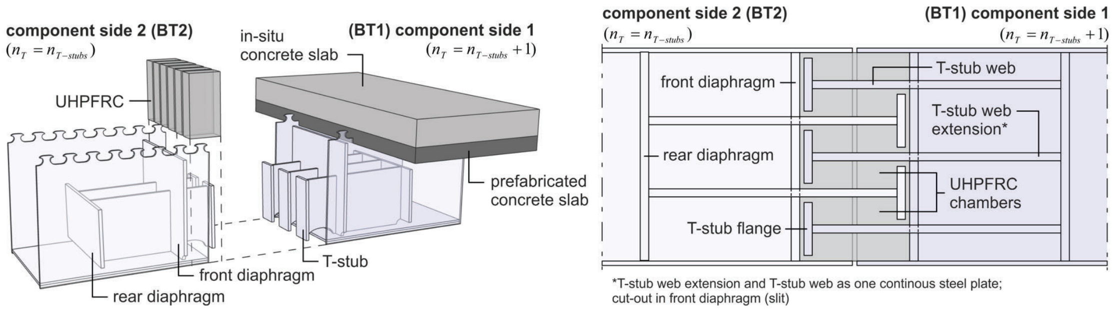

Figure 6: Grouted joint – isometry and top view [26]

The primary internal force transferred of the grouted joint is the bending moment. This bending moment is divided into compressive and tensile forces, which are transferred by the concrete chord and the grouted joint consisting of the T-stubs and UHPFRC chambers. The grouted joint features different load-bearing mechanisms for positive and negative bending moments. For positive moments, the tensile force is transferred via the T-stub webs and flanges and diagonal compression struts in the UHPFRC chambers, while for negative moments, the compression force is directly transferred through the T-stubs to the front diaphragm of the opposing segment. The load transfer mechanisms are detailed in Figure 7, with F_{joint} representing the total longitudinal force and σ_T the stress distribution in the T-stub web.

The arrangement of the segment sides is of particular importance for optimum load-bearing behaviour and favourable shear force transfer. In order to ensure the shear force transfer via segment side 1, which features one T-stub more than segment side 2, segment side 1 must be arranged towards midspan. Figure 8 shows the arrangement.

Depending on the span of a girder, there are different options for the number of grouted joints required. For a bridge span of approximately 30 metres and an assumed maximum segment length

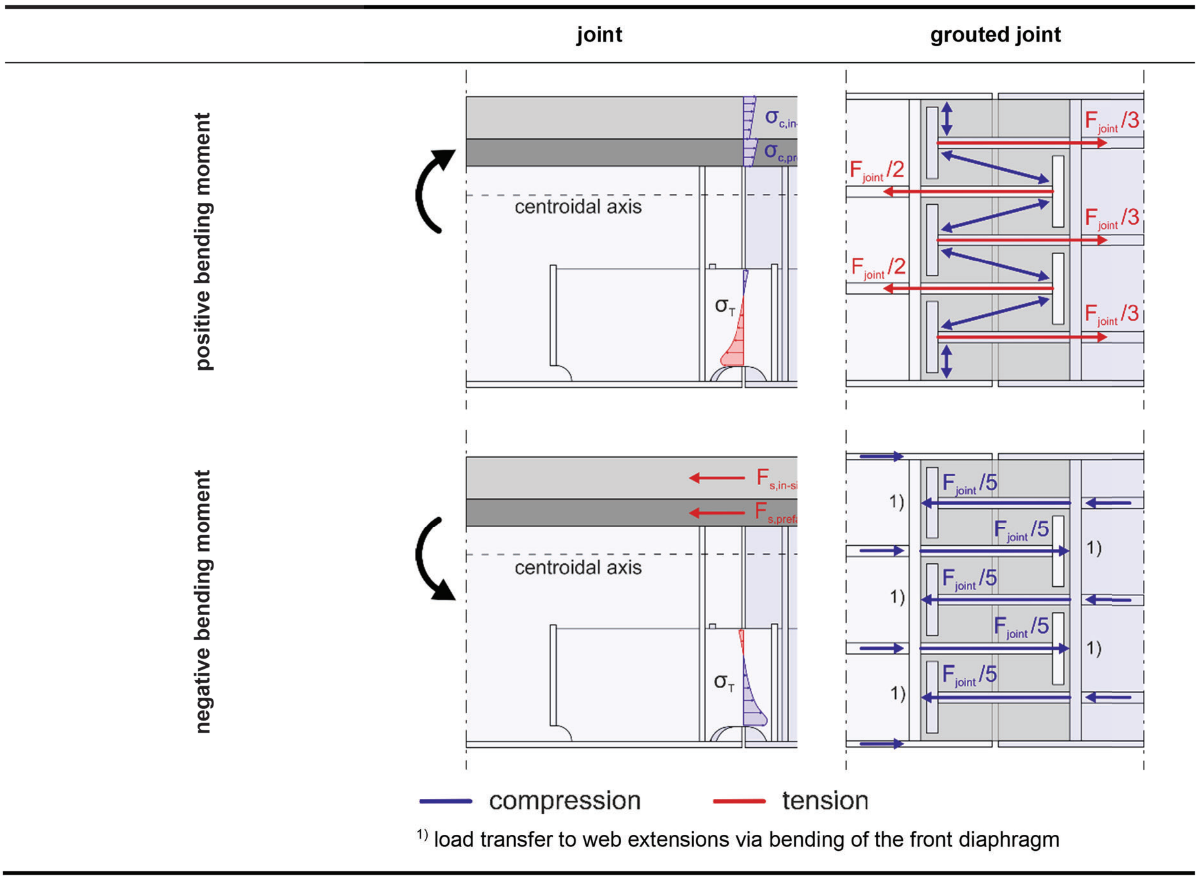

Figure 7: Transfer of a bending moments at the joint [26]

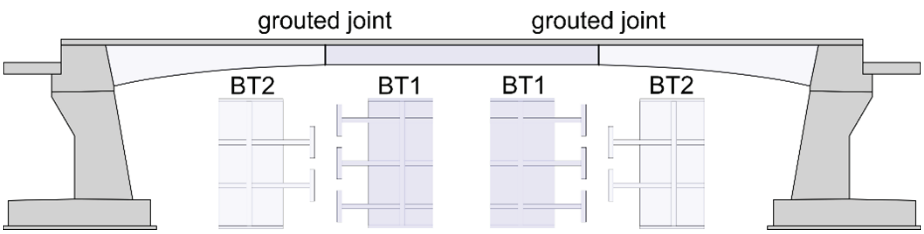

Figure 8: Arrangement of the component sides of the grouted joints [26]

of 16 metres due to hot-dip galvanizing, the following configurations are possible: the placement of one joint at the centre of the span (minimum possible number of joints) and the placing two joints at the zero moment points (statically more optimal number of joints). In general, the most economical solution is to use the minimum number of joints. For spans between 30 and 45 metres, the minimum number of joints required coincides with the most favourable static position, as at least two joints are required. However, for spans between 17.5 and 29 metres, the minimum number of joints may require an increase in cross-section or the addition of extra T-stubs due to static requirements. In such cases, the placement of a second joint could provide a more advantageous position along the moment distribution. Therefore, these options should be carefully evaluated for cost effectiveness. The positioning of grouted joints must take into account the bending moments, shear forces and their superposition. Due to limitations of the developed design concept, the grouted joint shall be positioned where, in the ultimate limit state and under fatigue loading models, the absolute value of the minimum bending moment $M_{Ed,min}$ (negative) does not exceed the maximum bending moment $M_{Ed,max}$ (positive). In spans where flexible joint positioning is possible, this criterion can be easily met. In cases where positioning is constrained by the maximum possible segment lengths, placement will naturally occur in regions where $|M_{Ed,max}| > |M_{Ed,min}|$. If the joint is grouted in the shop in a stress free state, and the bridge girder is transported to site in one piece, the positioning of joints does not affect the internal force distribution of the bridge structure. Consequently, the joint placement can be made after the internal forces have been determined. This simplifies the design process as the internal forces can be calculated for the whole structure without considering the joints.

For the design and dimensioning of the joint, the influence of hot-dip galvanizing on the fatigue behaviour of the steel components must be considered. During the galvanizing process, the steel components are immersed in a zinc bath at approximately 450°C. Due to the different cooling rates between the base steel material and the zinc coating, micro-defects form in the zinc layer near the base material. When the component is subsequently subjected to fatigue loading, these micro-defects can propagate into the base material and initiate fatigue failure [17]. The effect of hot-dip galvanizing is generally recognizable for fatigue class >80 with a small not effects due to the geometric notch of the detail. For fatigue class 80 and lower the influence of hot-dip galvanizing is neglectable compared to the influence of the geometric notch Therefore, no reduction of the reference fatigue class of ungalvanized steel details according to DIN EN 1993-1-9 [15] could be observed in previous tests from the literature. As part of the research project [26] and to investigate the fatigue behaviour of the T-stubs of the grouted joint, 27 T-stubs - 16 galvanized and 11 non-galvanized - were tested with regard to their fatigue strength. According to the test results and in consistency with the literature, the T-stubs can be assigned to the reference detail category 80 of ungalvanized Tee joints as per DIN EN 1993-1-9 [15].

To investigate the load-bearing and deformationof the joint, two large-scale static tests were carried out in the course of the research project IGF-Nr. 20312N [26]. The dimensions were maximized within the constraints of the available testing equipment to minimize scaling effects and ensure realistic results. The 20-ton girders measured 12 meters in length, with a cross-sectional height of 80 cm and a width of 1.90 meters.

The steel components were manufactured in accordance with execution class EXC 3 which is common in bridge construction and in accordance with EN 1090-2. All components, spacings and welds were manufactured based on a realistic composite bridge structure in order to identify critical points. The main girders were made of S355J2 steel, the reinforced concrete top chord of normal concrete of strength class C35/45 and the UHPFRC of concrete strength class C130/145.

The galvanized segments were assembled and strain gauges were attached at the relevant locations in the test hall at the Technical University of Munich. Before grouting, the load-transferring surfaces of the steel components were coated with an epoxy resin to prevent hydrogen formation resulting from the fresh UHPFRC phase in contact with galvanized steel surfaces. The casting was carried out with the fibre-reinforced compound Effix-Plus from Heidelberg Cement. With the formwork, reinforcement work and casting of the concrete top chord, the test specimens were finalized.

The static system was a single-span beam with cantilever, which was loaded via hydraulic cylinders at the end of the cantilever. The distance of the load application point to the centre of the grouted joint was chosen to approx. 4.56 m, taking into account the moment-shear force ratio of real bridge structures (Figure 9).

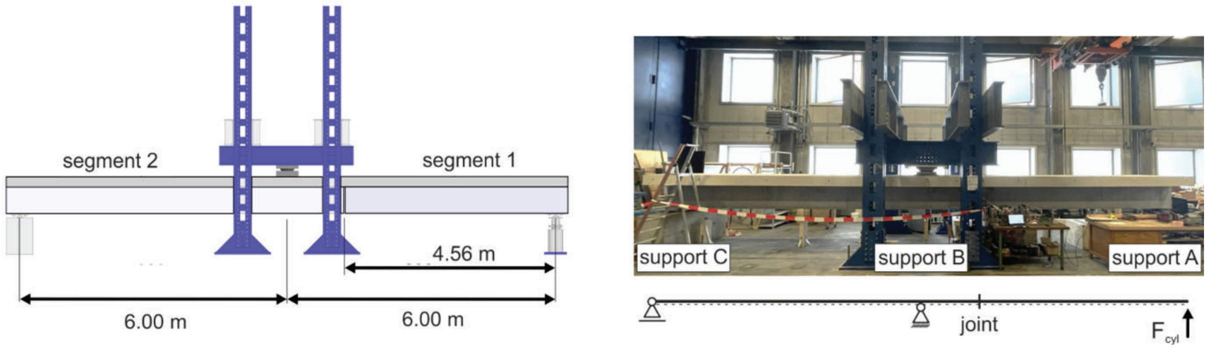

Figure 9: Large beams and test frames [26]

The tests showed that the decisive components of the grouted joint are the steel T-stubs, which failed due to plasticizing in the lower area of the T-stub webs. When the T-stubs reach a maximum principle plastic strain (neglecting the elastic strain component) of approx. 10 ‰ to 15 ‰, a rapidly increasing and disproportionately deformation of the joint could be observed. However, the experimental tests showed a high rotational and a high load-bearing capacity even after reaching a maximum principle plastic strain of 15 ‰ in the T-stubs. In the tests, the maximum load-bearing capacity of the joint could not be reached due to the stroke limitation of the cylinders. After reaching a maximum principle plastic strain of 10 ‰ to 15 ‰ in the T-stubs, the test load could be further increased by up to 50 % without failure of the joint. In order to avoid significant plastic deformations in the bridge structure, the limit criterion for the ultimate limit state of the grouted joint is defined on the basis of the test results using a limit of 10 % maximum principle plastic strain in the T-stubs. Up to this value, the deformation and load-bearing behavior of the joint is predominantly elastic. The UHPFRC showed no signs of damage or failure in both of the tests, even at the maximum test load. Besides the investigation of individual components of the joint, the rotational stiffness of the grouted joint was investigated by measuring the deflection along the girder as well as the gap opening between the girder segments. The evaluation showed a kink in the bending curve which results from the lower bending stiffness of the joint compared to the regular cross-section. Although the kink in the bending curve could already be observed in the elastic state of the girder, the deformation is again moderate up to the limit criterion of the ultimate limit state.

6 Conclusion

When designing galvanized bridges, several key factors presented in this paper need to be considered to ensure long-term durability and effectiveness of the corrosion protection system: material selection, design considerations, construction details, segmentation and joints, galvanizing thickness, inspection and quality assurance as well as maintenance.

The joining techniques for hot-dip galvanized steel beams in bridge construction were presented for bold and welding joints. As part of the IGF-Nr. 20312N research project, a grouted joint was developed that allows frame bridges in prefabricated composite construction to be hot-dip galvanized and remain entirely maintenance-free over the entire service life of the structure. The innovative joint eliminates the need for subsequent thermal zinc spraying or the use of slip-resistant prestressed bolted connections in hot-dip galvanized bridges with small to medium spans.

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