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SEMI-INTEGRAL FULL-SCALE STUTTGART TIMBER MODEL BRIDGE

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ABSTRACT: This paper reports on the on-set, design and realization of the Stuttgart Timber Model Bridge (STMB), which represents worldwide the largest full-scale demonstrator of a novel concept for timber bridges. The demonstrator bridge, erected at the Campus of University of Stuttgart in 2016, has been developed within two consecutive EU-research projects. It comprises several so far novel solutions concerning supports and superstructure. All derived novelties aim at a new generic type of robust timber bridges being significantly superior vs. premature degradation experienced at many of today´s timber bridges. In pursuing of the project aim first an extensive assessment of present deficiencies of mostly untreated timber bridges including a field study at 120 bridges has been performed. Based hereon, new construction concepts and detailing for the supports and the superstructure were derived. Major novelties are the realization of an integral fully rigid timber-reinforced concrete (RC) joint and of detached member-embankment connections. The build-up of the superstructure includes a double-water shielding and the implementation of a most advanced remote-controlled moisture monitoring system. The robustness of the novel bridge concept has been positively evaluated for a period of seven years with planed partly over-excessive mechanical and water loadings.

KEYWORDS: timber bridge, integral timber-RC-joint, detached cantilever support, novel robustness concept, superstructure with double water-shielding, advanced remote moisture monitoring

1 INTRODUCTION

The aim of the research task, pursued within two consecutive EU-research projects at Timber Department of MPA University Stuttgart consisted in the development of a visionary, robust timber bridge of highest possible durability. The novel design should overcome the weakness of the existing bridges significantly without implementation of any wood preservative treatments. In order to achieve the envisaged goal in a first step a thorough analysis and field study was conducted at 120 timber bridges of different build-ups and ages. Based hereon in total 16 recurring damage scenarios were identified and attributed to different generic bridge types such as e.g. truss, trough and deck bridges [1], [2].

Based on the classification and understanding of the damage on-sets a novel bridge type was conceived, detailed, designed and finally a full-scale prototype, the so-called Stuttgart Timber Model Bridge (STMB) was built (Figure 1). The demonstrator, representing the world´s largest research demonstrator bridge, erected at the Campus of Stuttgart University, was inaugurated in June 2016 by the state minister of rural affairs and is publicly accessible since then. The bridge serves for continued extensive measurements of strains, displacements, forces and moisture evolution based on latest measurement technology. The bridge can also be used on request by external researchers for specific investigations, e.g. regarding static and dynamic loading or advanced sensorics.

Figure 1: Rendering view of the Stuttgart Timber Model Bridge (STMB)

2 INTEGRAL AND SEMI-INTEGRAL BRIDGES

2.1 GENERAL

So-called integral structures, here in specific integral bridges, are designed without joints and bearings as defined e.g. in the German guideline for the design, construction and equipment of civil engineering structures, part 2, section 5 [2]. According to the definitions in [2], the superstructure of an integral bridge is continuous without joints along the entire length of the bridge and is not separated from the piers, abutments and pillars by joints or bearings. All components are monolithically connected to each other. Figures 2 a) – c) [3] schematically illustrate the essential differences of a common bridge structure with bearings and expansion

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joints in the super-structure-abutment-transition as well as in the sliding pillar-superstructure connection (Figure 2a) vs. a frame-like integral bridge structure (Figure 2b).

The definitions for semi-integral bridge constructions are not throughout consistent in North America and Europe. Here the definition acc. to [3] is used whereby a semi-integral bridge features a monolithic connection of the super structure with the middle support(s) whilst the end supports rest on bearings and can slide as in a usual bridge construction (Figure 2c). This definition can be extended, when e.g. looking at the symmetry conditions at the middle pillar, to a semi-integral bridge construction where abutment and superstructure are rigidly connected at one (end) pier whilst the other end and eventual intermediate supports allow expansion and sliding in longitudinal direction and partly orthogonal hereto. The latter definition can then be immediately applied to the constructed Stuttgart Timber Model Bridge (STMB). The fact that the STMB comprises two sliding supports is owed to the curvature of the bridge in the ground elevation.

Integral bridge structures, which up to now are constructed almost exclusively in reinforced concrete and steel composite construction show a number of significant advantages over conventional bridges with sliding bearings and joints, whereby some specifics apply when using timber for the superstructure. It should be mentioned that the concept of integral and semi-integral bridges is (almost) not considered in the present or newly drafted timber bridge design codes EN 1995-2 [4], [5]. Draft EC5-2 mentions integral abutment bridges in chapter 7.5, hereby referring exclusively to draft EC0 [6], which addresses soil aspects. However, integral timber-concrete joints being a prerequisite for integral and semi-integral timber bridges are not addressed at all.

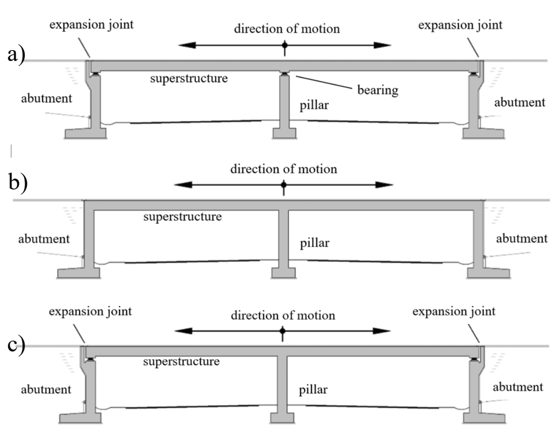

Figure 2: *Bridge construction concepts a) standard bridge superstructure with bearings and expansion joints, b) integral bridge with monolithic connections of superstructure, pillar and abutments, c) semi-integral bridge with rigid connection at the middle pillar*

2.2 ADVANTAGES OF INTEGRAL BRIDGES

The most important advantage of an integral bridge consists in the complete elimination of bearings and expansion joints, which are particularly susceptible to damage and are highly maintenance-intensive. For this aspect precise data is available for the area of steel and concrete construction. The maintenance costs of the two groups of components mentioned are about 10% of the total maintenance costs, while their share of the pure construction costs is only 3% [7]. Furthermore, bearings and expansion joints often cause damage to the overall structure, too, which also applies to a considerable extent to timber bridges [1]. A second significant advantage resulting from the monolithic connection between superstructure and the abutments consists in the significantly smaller section forces due to the clamping effect, which enable very slender, more aesthetic and more cost-effective constructions.

The magnitude of section force reductions of integral and semi-integral bridges can be illustrated in approximative manner by a comparison of a both- or one-sided clamped beam vs. a simple supported (bridge) beam. It should be noted that the comparison of rigid clamping vs. sliding supports represents a theoretical limit case as the abutment shows a finite rotational stiffness depending on soil conditions and abutment wall height. In the following a uniform loading is assumed.

In case of a both-sided clamped beam (i.e. integral bridge) the extreme moment ($ql^2/24$) at the support is 1,5 times smaller as compared to the field moment ($ql^2/8$) of a simply supported beam. In case of the semi-integral substitute model (one-sided clamped, opposite side simply supported) the clamping moment equals the field moment of the simply supported situation. However along most of the beam length a significantly reduced moment exists, whereby the extreme field moment ($ql^2/14,2$) reduces by a factor of 1,8. Additionally to section forces from external gravity loading (self-weight, traffic, …) stresses from constrained internal forces, not occurring in simply supported bridges have to be considered, what is regarded below.

Irrespective of the mentioned superimposed eigenstress-related forces integral structures are more robust overall and have significant load redistribution reserves as a result of the static overdetermination.

2.3 DISADVANTAGES OF INTEGRAL BRIDGES

For integral RC and steel composite bridge structures, the constrained internal forces resulting from concrete shrinkage, temperature and non-uniform lowering of the supports, which interact to a large extent with the ground conditions, i.e. the ground stiffness, have a disadvantageous effect. In the case of integral bridges, a realistic consideration of the soil parameters is therefore of significantly greater importance than for bridges with sliding bearings. Hereby it is not sufficient to work with characteristic lower parameters [2], [8]. Depending on the respective foundation and pier stiffnesses, integral structures suffer comparable longitudinal deformations as

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bridges with sliding bearings. These displacements and rotations are passed on directly to the backfill via the monolithically connected abutments. In the Northern Hemisphere the annual climate change results in positive and negative constrained internal forces. This leads to monotonic and cyclic deformations of the abutment wall that act on the backfill (see Figure 3). In case of integral timber bridges the constrained internal forces result from a superposition of changes in temperature but most important from moisture variations.

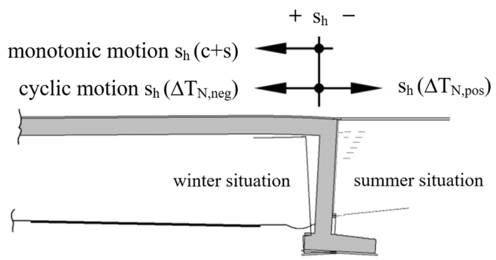

Figure 3: *Displacements of an integral abutment during seasonal change*

2.4 INTERNAL FORCES FROM TEMPERATURE AND MOISTURE CHANGES

In the design of timber structures, the effect of temperature induced movement (expansion / contraction) as opposed to the moisture induced movement (swelling / shrinking) is rarely considered. However, in structures with hindered movment possibilities as in case of integral bridges this feature has to be taken into consideration as being beneficial.

The temperature expansion coefficient of softwoods (spruce and fir) parallel to fiber can be roughly assumed as $\alpha_{T,0} = 0{,}4 \cdot 10^{-5}$ [1/K], hence being about one third as compared to steel. The moisture expansion coefficient parallel to fiber is generally taken as $\alpha_{u,0} = 10 \cdot 10^{-5}$ [1/1%Δu]. Although this value is 25 times higher as $\alpha_{T,0}$, this is partly compensated by the much higher temperature ΔT than moisture Δu changes.

In case of integral RC bridges the two general design scenarios of a uniform and linearly varying temperature distribution along cross-section depth of the superstructure are regarded. In a well-ventilated timber pedestrian bridge, it is considered sufficient to assume a rather symmetric parabolic temperature and moisture distribution over the beam cross-section as illustrated in Figure 4 (see also [9]).

For an exemplary illustration of the temperature-moisture expansion interaction in a timber bridge the parabolic distribution is approximated by a linearly stepped function. It is assumed (common for climate in Germany) that the conditions at bridge erection are: T_0 = 10°C and u_0 = 12%. In the summer load case the outer 1/6 of the cross-section depth d experience at the centroid a reduced moisture content of 8% (Δu = 4%) while in the center part comprising 4/6 of d the reduced moisture content amounts to 10% (Δu = 2%). The Temperature the centroid of the outer parts is assumed to be 30°C (ΔT = 20°C) and at the center at 25°C (ΔT = 15°C). The ratio of the restrained normal forces resulting from temperature (N_T) vs. moisture (N_u) generally reads

$$\frac{N_T}{N_u} = \frac{\varepsilon_T}{\varepsilon_u} = \frac{\alpha_T\,\Delta T_N}{\alpha_u\,\Delta u} \tag{1}$$

For the outer beam parts the summer situation then delivers

$$\left[\frac{N_T}{N_u}\right]_{Sommer} = \frac{0{,}4 \cdot 10^{-5} \cdot 20}{10 \cdot 10^{-5} \cdot (-4)} = -0{,}2 \tag{2}$$

which means that the tension force at the joint location induced by the moisture depended wood shrinkage is reduced by the counteracting temperature extension by 20%. The resulting restrained normal force of the outer cross-section parts then reads

$$\begin{aligned} N_{T,u,summer} &= (\alpha_T \Delta T + \alpha_u \Delta u)\,EA \\ &= (0{,}4 \cdot 20 + 10 \cdot (-4)) \cdot 10^{-5} \cdot EA \\ &= |-32 \cdot 10^{-5}| \cdot EA \quad \text{(tensile force)} \end{aligned} \tag{3}$$

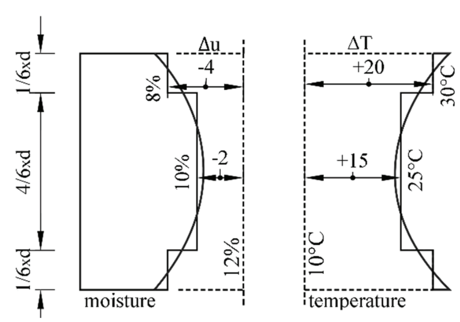

Figure 4: *Assumed moisture and temperature gradient of the timber (GLT) superstructure member in the summer period (erecting state: T = 10°C, u = 12%)*

3 THE STUTTGART TIMBER MODEL BRIDGE

3.1 DURABILITY ORIENTED DESIGN AND DETAILING APPROACH

Architecturally driven the Stuttgart Timber Model Bridge (STMB) was developed as a so-called deck-bridge, i.e. an open construction without a weather sheltering roof, considered outdated in modern architecture. The structural concept and the details of the build-up of the STMB was then in particular driven by an intention to avoid any of the previously identified 16 generically different durability damage scenarios ([1] and [10]). To do so and to deliver a trustworthy, reliable show case of the effectiveness of all newly or stringent consistently pursued ideas it was considered inevitable to verify all detailing in a full-scale demonstrator building with dimensions immediately transferable to practical use. In more detail, the following constraints were considered mandatory for the bridge layout:

- No penetration of covering and superstructure by any mechanical fastener from top to lower or bottom surfaces. This aims to avoid an extremely often occurring damage / decay situation where water penetrates – gravity wise – along the periphery of nails, screws, dowel type fasteners as the surrounding hole in the wood gets enlarged over time. The interacting mechanical, moisture and temperature loading causes small cavities which enable unhindered and hardly to detect water penetration.

- Avoidance of any access and accumulation of decay generating greenery, dirt, split, water and snow at the supports. This necessitates a revision to the typical bridge end-support-abutment detailing shown in Figure 5, where the end-grain face(s) of the superstructure beam(s) or plate ends close to the rear wall of the abutment. The cavity between the timber end grain face and the abutment rear wall, unless cleaned frequently, fills with greenery or dirt over time causing numerous bridge damages. An enlargement of the cavity width *a* to a sufficiently high distance of about 0,5 to 1 m is in general considered being uneconomic as this necessitates a much larger support bank / bridge benching of the abutment. In order to overcome the sketched decay prone support situation two new on-sets were developed, outlined below.
- Realization of a redundant two-layer water shielding of the main superstructure beam
- Segmented, easy demountable and repairable bridge pavement elements made from durable wood species. Note: Due to funding issues Douglas fir boards of rather low durability (class 3-4 according to EN 335 [11]) were used in the STMB.
- Enabling of easy inspection access especially of the superstructure from the narrow faces by shutters which can be opened by hinges.
- Continuous monitoring of the moisture evolution at discrete points.
- Segmented linear sensors for leakage detection at the lower sealing layer.

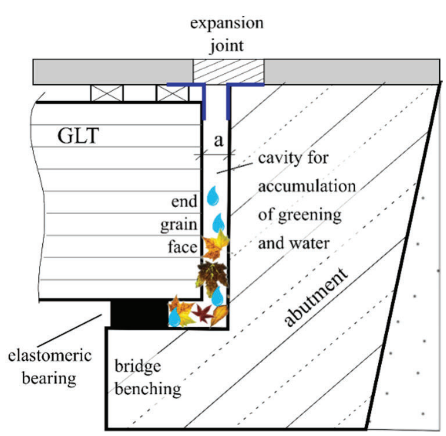

Figure 5: *Frequent damage / decay prone detail of superstructure – abutment transition of bridges with sliding end supports where the narrow cavity between GLT end grain face and abutment wall is subject to accumulation of greening and water*

3.2 BASIC CONSTRUCTION DETAILS

The Stuttgart Timber Model Bridge (STMB) was realized as a pedestrian and cycling bridge, but the principle is applicable to heavy traffic purposes, as well. Generically, the bridge can be termed a semi-integral deck bridge. The bridge is curved in plan view with a length of 22 m. The clear width of the superstructure is 2 m (see Figures 6, 7 and 8). The main load carrying member is a block-glued GLT beam (GL 24h) with a cross-section of 40 cm x 120 cm, which is supported at both ends and at mid-span at the location of highest curvature.

The bridge design considered a uniformly distributed characteristic load of 5 kN/m^2, a characteristic point load of 10 kN as well as front and rear axle loads of a service/rescue car of 40 kN and 80 kN, respectively.
At the north-east (NE) end the GLT superstructure member is rigidly clamped to the concrete abutment by a novel integral joint, specified in more detail below. The abutment is backfilled with a paved sloping ramp enabling car access to the bridge.

The supports at mid-length and at the south-east (SE) end are constructed as sliding supports with two rubber bearings, each. At the mid-span support sliding is unrestricted in any horizontal direction whilst at the SE support the sliding is prevented normal to beam longitudinal axis. The rubber bearings which are positioned at the quarter-points of the cross-sectional width of the GLT member rest on a horizontal steel frame which itself is supported by four steel screw piles at the mid-span and at the SE end support, respectively. It should be mentioned that screw piles present excellent options for cheap but robust sustainable foundations of light-weight (timber) bridge structures which can be erected and remounted very fast.

The SE end support was constructed in view of the above outlined end-support decay scenario by a so-called detached transition from the embankment to the bridge superstructure.

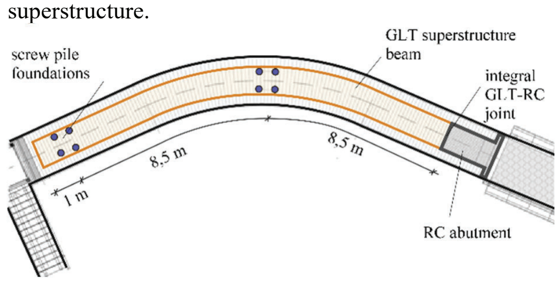

Figure 6: *Plan view of the Stuttgart Timber Model Bridge*

4 DETAILS OF SUPPORT CONSTRUCTIONS

4.1 INTGRAL RIGID TIMBER-CONCRETE ABUTMENT JOINT

The north-east (NE) support was realized for the first time in timber bridge construction as a fully integral - moment, torsion, shear force rigid timber-reinforced concrete (RC) joint (see also [10] and [12]). The connection is based on rebars (n = 34, d_{rod} = 16 mm, strength grade B500) glued

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into the GLT beam with an epoxy resin, then protruding by about 1,2 m from the end grain face (see *Figure 7* a) – c)). The GLT rebars were aligned/connected with the rebars of the reinforcement cage cased by the concrete formwork and then the assembly of the GLT and the cage bars was cast in a single step to form a rigid connection without any gap. This novel support construction, fully apart from mechanical considerations, provides the possibility to overcome in conjunction with a continuous decking the present onsets of timber decay starting from the GLT end grain face through greenery/earth and hence moisture accumulation between GLT and abutment wall as illustrated schematically in Figure 5.

The load capacity of such a joint has been verified in previous investigations reported in [13]. These tests demonstrated a superior moment and shear force transfer. The bending stiffness and capacity results in a first conservative assumption exclusively from the normal forces in the rods glued-in parallel to grain at distance e = 284 mm. This holds entirely true for the rebars located at the bending tension side, here located at the top side, due to the negative bending moment at the clamped joint, at the top. At the bending compression side, however, it is reasonable to assume an additional load transfer via contact compression between the GLT end grain face and the concrete abutment. The load sharing between rod and contact forces can in first instance be assessed by a parallel system model which undergoes significant variations over time depending on moisture changes and relaxation. This aspect is quantified separately.

The shear force transfer evolves somewhat more complicated as outlined below. As a full understanding of the mechanism and hence of a reliable capacity assessment of the shear force transfer in the GLT was on-going at time of the bridge construction, the shear force transfer was realized in redundant manner via two complementary approaches A and B:

Mechanism A is represented by the behaviour of a ribbed dowel type fastener glued-in parallel to grain and loaded laterally. The capacity may then be assessed by a Johansen model for a dowel type fastener loaded in single shear between GLT and a thick steel plate. For a laterally loaded dowel type fastener a high risk of splitting parallel to fiber and premature failure of the total joint exists in case the dowel / rebar is positioned close to the tension loaded GLT edge $a_{4,t}$. This, however, cannot be avoided in the given case as a high moment capacity necessitates a large internal lever arm e (see Figure 7 c) of the rebars glued-in the GLT cross-section. In the STMB the edge distances of the rebars (d = 16 mm) were chosen as $a_{4,t}$= 58 mm = 3,6 · d. To overcome the splitting risk the surrounding wood was reinforced by fully threaded self-tapping screws inserted from the bottom side, i.e. bending compression edge of the superstructure GLT.

The second shear force transfer mechanism B was based on some rebars at the bending tension and bending compression edge which were bonded-in with an inclination angle of 20°. These inclined bears then transfer the shear force by tension in the bar and embedment compression stresses in the timber. A details analysis of both shear force transfer mechanism is presented in a separate contribution.

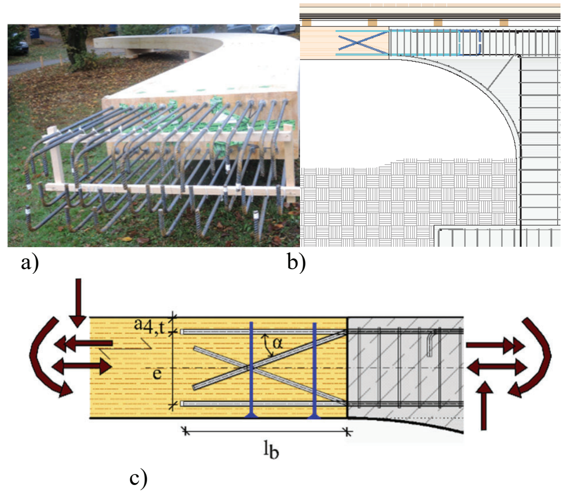

Figure 7: *Integral GLT-RC support embankment a) primary superstructure GLT beam with glued-in rebars; b) scheme of the integral joint/support solution, c) detail section of the integral joint*

4.2 DETACHED CANTILEVER-TYPE BRIDGE-EMBANKMENT TRANSITION AND CENTER SUPPORT

The SE end support consists of a plain rubber bearing where a translation displacement is restricted only in direction perpendicular to the bridge axis. The bridge related novelty consists in the fact that the GLT-RC joint exceeds the NE support cantilever-like by about 1,5 m. At the SE end the transition to the embankment is fully decoupled by means of a perforated steel grid plate supported by L-shaped steel brackets fixed to the pier front. This solution again enables a full detachment of the end grain face of the timber / superstructure from the embankment and hence exclusion of biodegradation.

The mid-span and the SE flexible supports consist of screw-piles, which are especially apt for light-weight, easily demountable sustainable foundations.

5 CROSS-SECTIONAL-BUILD UP OF THE BRIDGE

The cross-sectional-build-up of the superstructure is shown in Figure 8 a). In brief, two decoupled water shielding planes were constructed (Figure 8 b) and c)). The cross-ply LVL plate was then covered by a bonded fiber reinforced roofing membrane layer inclined at a small angle (6°) vs. the horizontal plane to enable a water drainage to the lower bridge edge. The inclination is then compensated with regard to a horizontal plane by tapered support beams for the pavement. An important feature of the superstructure build-up is that no mechanical fastener penetrates any structural and water proofing component / layer from top. The connection of the cross-ply LVL plate as well as for the hand rail is realized by angle brackets connected to the narrow faces of the structural GLT beam.

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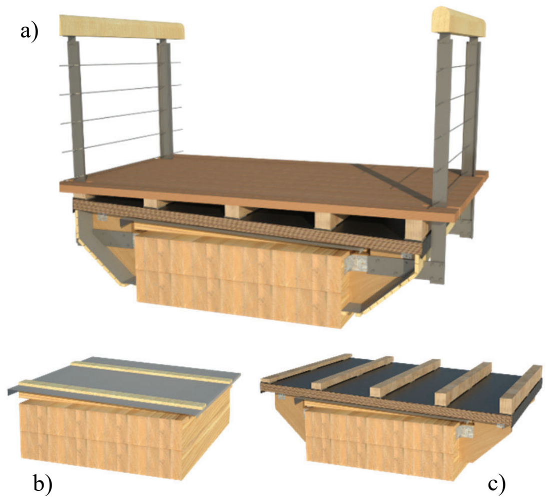

Figure 8: *build-up of STMB a) total cross-section, b) lower water shielding, c) upper water shielding*

6 INSTALLED MONITORING SYSTEMS

The monitoring of the bridge comprises the mechanical aspects (displacements, strains, forces) and hygro-thermal issues such as moisture and temperature evolutions. All data are sampled at multiple points within the cross-section and along the bridge length and are recorded continuously remote. Special novel line sensors are applied to register eventual water penetration despite the double water proof shielding.

The detailing of the Stuttgart Timber Model Bridge (STMB) results from above mentioned research project [1]. To ensure and prove the function of the implemented construction preservation measures a monitoring concept with the following components was installed (see Figure 9):

- 16 sensors for punctual moisture content (u) measurement in different depths (electrical resistance method)
- 3 sensors for climate (*cli*) located beside and under the GLT superstructure beam and between the GLT structure and the sealing layer
- 3 sensors for wood temperature (T_{timber})
- 11 sensors for leakage detection at the second sealing layer (L)
- 1 displacement transducer for temperature and moisture dependent length variation of the GLT (d_L)
- 5 displacement transducers for vertical and horizontal relative displacements at the integral GLT-concrete abutment connection (d_v, d_h)
- 8 strain gauges on the surface of the GLT beam to obtain the stresses at the integral support (ε_{Timber})
- strain gauges on 8 of the glued-in rods at the integral GLT-concrete connection (ε_{rod})

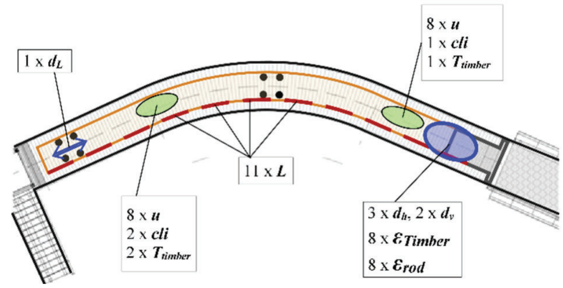

Figure 9: *Overview and locations of the different monitoring sensor-types*

All systems are running continuously and send wireless their data in a regular interval to a server. A big challenge of the combination of five different data logging systems consists in the consolidation and evaluation of the data. Five data loggers of different manufacturers are necessary for the functioning of the of the installed systems. Every system has its own method to record, transcript and send the data to the web. Some are sending the files via email protocol, others are using the File Transfer Protocol (FTP) to a local server and others are sending the data via FTP to a remote server of the manufacturer. At the homepage related to STMB (www.stuttgarter-bruecke.de) results of the monitored values are shown and updated continuously. Further investigations [14] at MPA University Stuttgart aim to compile a monitoring system with one centralised logger and computing unit which can handle all of the installed sensors.

7 RESULTS

7.1 CLIMATE AND MOISTURE MEASUREMENTS

In a rough summary of the climate measurements throughout seven years it can be stated that the temperature (daily mean values) varied over a year's season in the range of -5°C and +26°C. The relative humidity daily mean values varied from 45% in summer to 90% in winter. Based on the climate recordings the resulting surface moisture content derived by [15]

$$u(T, rH) = 10^{-2}\left[\frac{-T\cdot\ln(1-rH)}{0{,}13\cdot\left(1-\frac{T}{647{,}1}\right)^{-6{,}46}}\right]^{\frac{1}{110\cdot T^{-0{,}75}}} \tag{4}$$

is shown in Figure 10. It can be summarized that the calculated surface moisture (daily means) varied from 8% to 20% with a mean value over seven years of about 14,5%.

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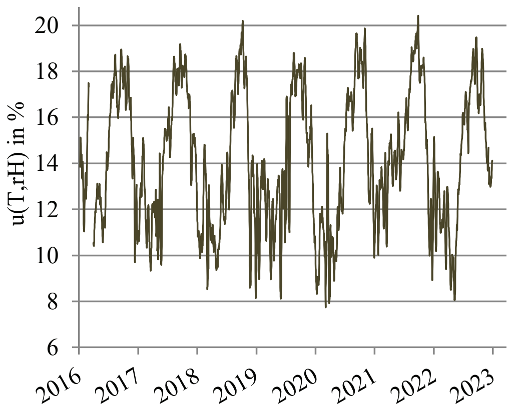

Figure 10: *Theoretical surface moisture content calculated from monitored air temperature and humidity by Eq. (4)*

The recording of the moisture content in different locations of the superstructure GLT beam by electrodes positioned at different depths of 1 cm, 3 cm and 6 cm is depicted exemplary in Figure 11. The graph shows that the moisture in 1 cm depth conforms rather well with computed surface moistures presented in Figure 10. Further it can be seen that the moisture amplitudes closer to mid-depth are significantly reduced. In a rough approximation the moisture contents vary in the cross-sectional center part in the range of 13,5 % to 16 % with a mean value of roughly 15 %. Further details and improvements regarding the moisture monitoring are presented in [14].

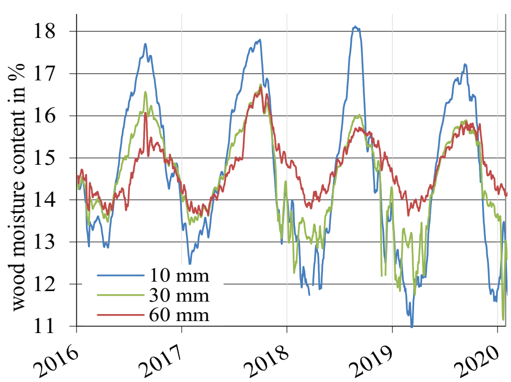

Figure 11: *Results of wood moisture monitoring of the GLT superstructure beam in cross-sectional depths of 10 mm, 30 mm and 60 mm*

7.2 MECHANICAL LOAD TESTS

In order to acquire a thorough understanding and experimental verification of the load carrying mechanisms of the integral joint of GLT and RC-abutment different static and dynamic load tests of the STMB were conducted. Due to page limitations, here one exemplary test is presented.

To activate a high bending moment in the rigid integral joint, a dead load in form of steel plates and concrete blocks were located at the mid-point of the first bridge field for a long-term test lasting six months. The 8.000 kg dead load was carried up in eight steps by a fork lift with an empty weight of 2.000 kg, shown in Figure 12.

Figure 12: *Load application for a long-term test with a fork lift*

The results of the measurements during this step-wise load application represented by rod forces calculated from strain gauges applied on the glued-in rebars are given in Figure 13. The correlation of strain and force was calibrated for all of the monitored rebars in a test machine before bonded into the GLT. The graph shows the results of four rebar forces, whereby two rebars are located at the upper bending tension edge and two at the lower bending compression edge of the joint. The tension rebar forces after full load application amount to 12,7 kN and 11,2 kN whereby the difference of the values results from the additional torsion moment because of the curved geometry of the GLT superstructure (Figure 13). In contrast hereto, the compressive forces show a much lower value of 4,3 kN in both cases being roughly 35% of the tension forces. The reason for the significantly smaller compression rebar forces can be attributed to the superimposed contact pressure of the GLT end-grain face to the RC-abutment whereas the tensile forces have to be transferred exclusively via the rebars. Figure 13 also shows the force peaks of the additional weight of the fork lift itself. The characteristic load carrying capacity in tension of a single rebar glued into the GLT is about 80 kN hence much higher than the forces resulting of the here applied bending moment.

A detailed analytical and numerical analysis of the integral joint, presented separately, reveals a satisfactory agreement between calculated and measured rebar forces.

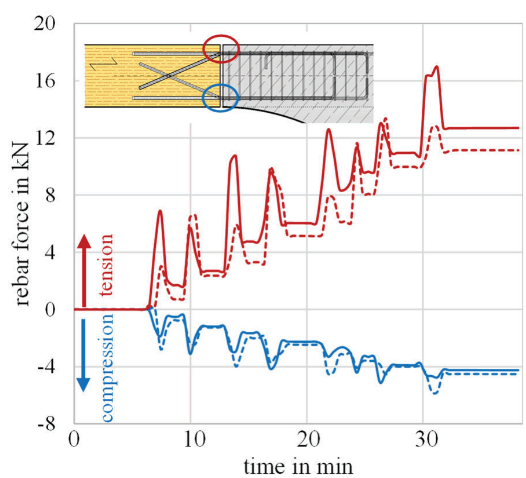

Figure 13: *Tensile and compressive forces in the rebars of the integral joint during stepwise application of 8.000 kg dead load*

8 EXPERIENCE AFTER SEVEN YEARS OF SERVICE LIFE

So far, all recorded mechanical and hygro-thermal results conformed well to the expectations/calculations. The agreements of the measurements are in all cases superior to the anticipated values [3]. In the follow-up of the STMB three public pedestrian bridge with both-sided integral timber-RC joints were realized and are monitored by MPA University of Stuttgart.

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- MERK Timber GmbH
- Metsä Wood Deutschland GmbH
- Paul Bauder GmbH & Co.KG
- Rettenmaier Holzindustrie Hirschberg GmbH
- Schaffitzel Holzindustrie GmbH + Co. KG
- SIMPSON STRONG-TIE GmbH
- Ulrich Lübbert Warenhandel GmbH & Co. KG

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