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Monitoring Based Evaluation of Design Criteria for Concrete Frame Bridges

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ABSTRACT: In bridge engineering maintenance strategies are highly influenced by construction type and quality of design. Nowadays bridge designer and owner tend to include life-cycle cost analyses in their decision processes regarding the overall design by considering structural safety and durability within financial constraints.

However, efforts to reduce maintenance costs over the expected lifetime by adopting new design approaches lead to unknown risks. Monitoring solutions can reduce the associated risk of new designs by constant observation of performance of structural components during prescribed time periods. They provide essential information regarding the long-term development of time dependent processes such as creep, shrinkage, temperature earth pressure and especially boundary conditions which have high influence on the overall structural performance.

1 INTRODUCTION

Concrete frame bridges (CFB) are characterized by integral abutments associated with the lack of bearings and expansion joints. This type of reinforced concrete structures, which are restricted in their dimensions due to e.g. the soil structure interaction, are nowadays preferred to traditional structures by bridge owners mainly due to maintenance and inspection considerations and the related likely cost savings. However, concrete frame structures differ essentially in their statical characteristics and especially in their sensitivity to changing boundary and loading conditions. For example, structural loadings and effects from temperature, creep shrinkage, which in general dart no special problem for the traditional structural layout, can yield to unwanted difficulties in case of CFBs.

Furthermore one particular characteristic of concrete frame bridges is their lack of expansion joints. Temperature loads as well as creep and shrinkage thus lead to significant problems in the transition area between structure and the soil next to the abutment, which increase with the total length of the structure. Since these situations are not fully covered by codes, a research project was initiated in order to obtain real data regarding structural response under varying conditions from fiber optical sensor (FOS) systems and other sensor technologies.

In particular a recently constructed three span concrete frame bridge was instrumented with in total five different sensor systems including 54 sensors that have been combined into an integrative permanent monitoring system. Areas of interest include (a) the performance of the superstructure during construction as well as under service loads, (b) the soil-structure interaction in general and (c) the performance of the chosen slab detail to accommodate the large dilatations in particular. The data obtained by the sensor systems will be analyzed with respect to temporal processes, statistical characteristics and compared to current design assumptions. A recently performed prove loading furthermore will provide the necessary data basis to calibrate finite element models that in consequence can serve for parameter studies and the optimization of certain structural details.

2 CASE STUDY – THE MARKTWASSER BRIDGE S33.24

Due to the increase in the international eastbound transit a new highway connection bypassing Vienna in the North had to be planned. The new highway section connects the two already existing highways S33 and S5 by essentially constructing a new Danube crossing. The entire project with estimated costs of approximately 170 Mio € and a total length of 6.6 km includes the construction of 24 bridge objects ranging from small overpasses to the main bridge spanning the Danube. Marktwasser Bridge S33.24 is one of the foreshore bridges on the southern riverside .

Based on a first traditional draft by Fritsch, Chiari & Partner ZT GmbH the engineering office ZT Mayer GmbH was instructed by the Austrian Highway Financing Company (ASFINAG) to convert the original design of Marktwasser Bridge to a joint less bridge as the omission of expansion joints and bearings was expected to reduce the total costs of ownership considering necessary maintenance over the entire life-time.

Marktwasser Bridge actually consists of two structurally separate bridge objects both of which are three span reinforced concrete plate structures with span lengths of 19.50 m 28.05 m and 19.50 m, see Fig. 1. The cross-section of the wider object was designed to allow for five lanes of highway traffic. The width of the deck plate ranges between 19.40 m and 22.70 m excluding two cantilevers of 2.50 m length each. Both structures are founded on four lines of drilling piles with lengths of 12.00 m for the abutment axes and 19.50 m for the column axes.

The abutment and column axes show an inclination of 74° to the center line of the deck plate. Further design aspects include monolithical connections between bridge deck, pillars and abutments as well as haunches ranging between a constant construction height of 1.00 m in midspan and 1.60 m in the vicinity of the pillars, see also (2010a, Strauss et al. 2010a).

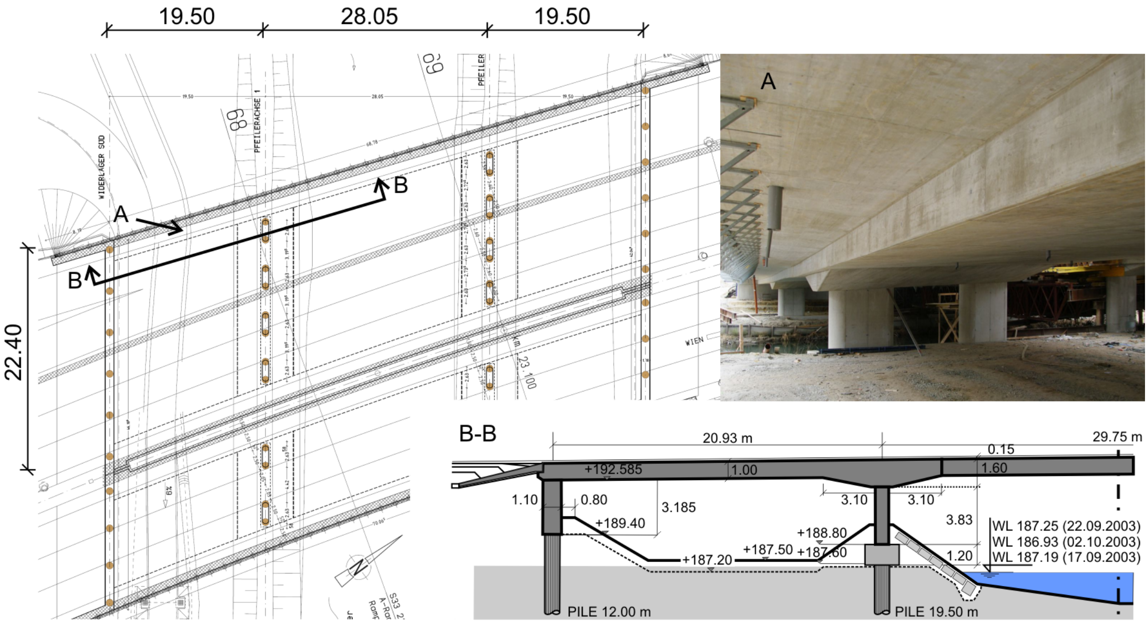

Figure 1. The Marktwasser Bridge S33.24

As the joint less design excluded the use of expansion joints a suitable detail had to be developed in order to accommodate the dilatations caused by temperature loads and time dependent creep shrinkage processes. The main challenge was associated with the goal to limit strains near the surface to levels that will not result in cracking of the pavement in the vicinity of the abutment or vertical deformations that might endanger traffic safety. Finally the design process resulted in an inclined approach slab in combination with the reinforcement of the earth dam.

This concept should in theory ensure an adequate decrease of the strain field from the tip of the slab up to the surface where main deformations should be concentrated in between two sliding surfaces that were expected to develop as sketched in Fig. 2.

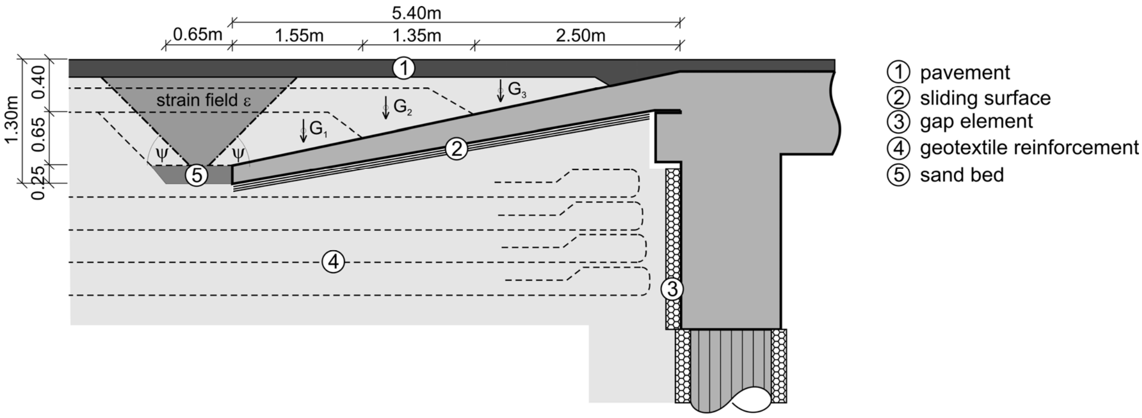

Figure 2. Approach slab detail of Marktwasser Bridge

3 MONITORING CONCEPT

Currently in Austria there is the intention to increase the design and construction experience concerning long jointless frame bridges especially with regard to the soil structure interaction and time dependent loads like temperature, creep and shrinkage. In order to accurately capture structural response and its changes over time an integrative monitoring concept was developed by BOKU University and installed by RED Bernard. The permanent monitoring system operating at a sampling rate of 1 Hz combines five different sensor systems with a total of 54 sensors divided between southern lateral field of the superstructure and the area behind the southern abutment.

The bridge deck was instrumented with two strands of fiber optic sensors as they promised to show higher durability and stability against corrosion and electromagnetic interference. Basic information regarding fiber optic sensors and their application in structural monitoring are for instance given in Glisic and Inaudi (Glisic et al. 2007). The placement of the 12 strain and 8 temperature sensors was optimized in order to provide information regarding the temperature field in the cross-section and the strain contributions from dead load, creep and shrinkage and temperature gradient. A detailed presentation of this part of the monitoring system as well as first data analyses concerning for example the creep and shrinkage processes are given in (Strauss et al. 2010b, Strauss et al. 2010c).

3.1 Slab detail and soil-structure interaction

As statically undetermined structure Marktwasser Bridge is highly susceptible to actions resulting in constraint loads such as partial settlements and the activation of passive earth pressure against the abutment walls caused by temperature loads. The latter aspect was accounted for in the design by reinforcing the earth dam and detaching the abutment using soft gap elements (see position 3 in Fig. 2). Accordingly, during calculations only the active earth pressure was considered. However, this assumption can not be guaranteed and thus had to be checked by the monitoring campaign. The second main area of interest concerns the performance of the slab solution over a full temperature cycle – one year – as a permanently crack free pavement is critical for the acceptance of the entire construction type. In consequence the majority of the sensors is targeted at capturing the deformation state above the approach slab as accurately as possible.

In addition to the sensors placed in the bridge deck four different sensor systems were installed in the area behind the southern abutment. Two extensometers with lengths of 8.0 m and 2.5 m were placed at a depth of approximately 2.0 m below the pavement aiming at the detection of both relative and absolute movements between earth body and abutment, see sensor layout in Fig. 3 (a) and the installation of extensometers in Fig. 3 (b). For the constant observation of the strain field in the area above the approach slab 10 fiber optic strain (FOS) sensors (Murray et al. 2008) and 20 electrical strain gages (DMS) divided between four layers were installed. The placement of the sensors and especially the DMS was optimized based on the location of the predicted sliding surfaces according to Fig. 2. By using two different measurement principles a cross- verification of the measurement results becomes possible and additionally redundancy is reached.

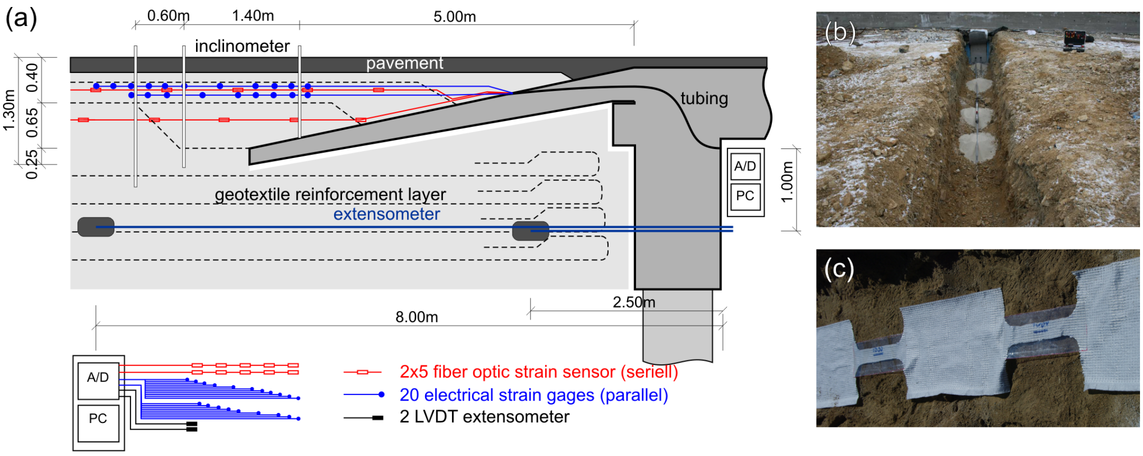

Figure 3. (a) Instrumentation of southern abutment and dam, (b) Installation of extensometer, (c) Installation of geotextile based fiber optic strain sensors

Finally three vertical inclinometers were placed in the area around the tip of the approach slab, two of which should capture the developing sliding surfaces. The location of the third inclinometer with a distance of 7.0 m from the abutment, see Fig. 3 (a), was governed by the goal to determine the deformations beyond the outer sliding surface and thus the degree of activation that can be reached by using geotextile reinforcement. Inclinometer measurements are carried out only at discrete points in time and serve for an absolute verification of the strain measurements by means of deformation measurement (Strauss et al. 2010d, Kampel et al. 2009).

4 DATA ANALYSIS

4.1 Extensometer

The extensometers were installed at the end of April 2009 shortly before the approach slabs were casted. Ever since, one sample was stored every 60 seconds by two LVDT displacement sensors. The end of the long extensometers is considered to be well outside the area that is influenced by the soil-structure interaction and thus can serve as absolute reference point. In consequence the measured data directly represents absolute movements of the abutment. The short extensometer however can only detect relative movements between earth dam and abutment.

In Fig. 4 the absolute movements of the south abutment and the earth dam behind it for the time period between sensor installation and the end of September 2009 are plotted. Since the movements of the earth dam are at all times smaller than those of the abutment it can be concluded that based on the present data (a) the earth dam is standing on its own and (b) the build-up of the passive earth pressure is not likely (Wendner et al. 2010b, Strauss et al. 2010d).

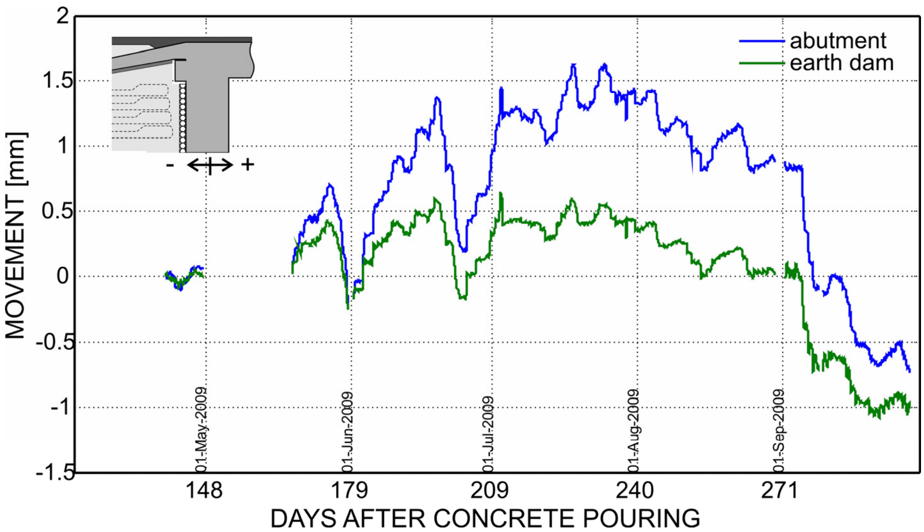

Figure 4. Movement of abutment vs. earth dam over time

4.2 Fibre optic strain data

In the area above the approach slab two strands of optical fibers woven into a geotextile for protection and bond with the earth material have been installed at two different depths. The sensors that work based on the fiber bragg grating principle (Murray et al. 2008) show a spacing of 1.0 m and have a typical resolution of 1 $\mu\varepsilon$ with a strain range of 1 %. In order to ensure representative measurements it was important to (a) limit the additional stiffness provided by the base material to a minimum and (b) provide a determined base length for the interpretation of the measured strains. The first goal was achieved by weakened the geotextiles in the strain sensitive areas of the optical fiber. Additionally this resulted in a clear differentiation between measurement area and bond area which was further improved by detaching the geotextile from the sand bed by means of a stiff and sleek sheeting of 30 cm length; see Fig. 3 (c). In consequence it can be assumed that the recorded strains represent mean strain values over a length of approximately 0.3 m (up to a maximum of 1.0 m).

The local information provided by the individual sensors can be transformed to time dependent strain fields utilizing the knowledge regarding the sensor positions. Fig. 5 shows the strain field as function of the distance to the abutment (x-axis) and time (y-axis) for the top FOS sensor layer. The temperature development over time is included on the left hand side, where the orange line denotes the mean temperature in the bridge deck and the blue line represents the soil temperature. As is obvious from the plot, no clear correlation between temperature and developing strain field can be determined. In consequence the recorded strain state is presumably caused by the ongoing construction work in the area behind the abutment. Further studies of the available data will allow the determination of dependencies between temperature, time dependent processes and strain field above the approach slab (Kampel et al. 2009, Strauss et al. 2010d).

Considering the layout of the transition area between abutment and soil body, in particular the inclined approach slab, a contraction of the CFB will result in the introduction of a horizontal displacement at the tip of the slab. This displacement causes the development of an appropriate strain field in the reinforced earth dam above the slab and possibly two sliding surface going from the tip of the slab upwards, as sketched in Fig. 2. One of these sliding surfaces corresponds to the location of the observed strain maxima in the top FOS layer and can also be detected in the respective strain field of the bottom FOS layer, thus verifying the assumptions regarding the slab detail.

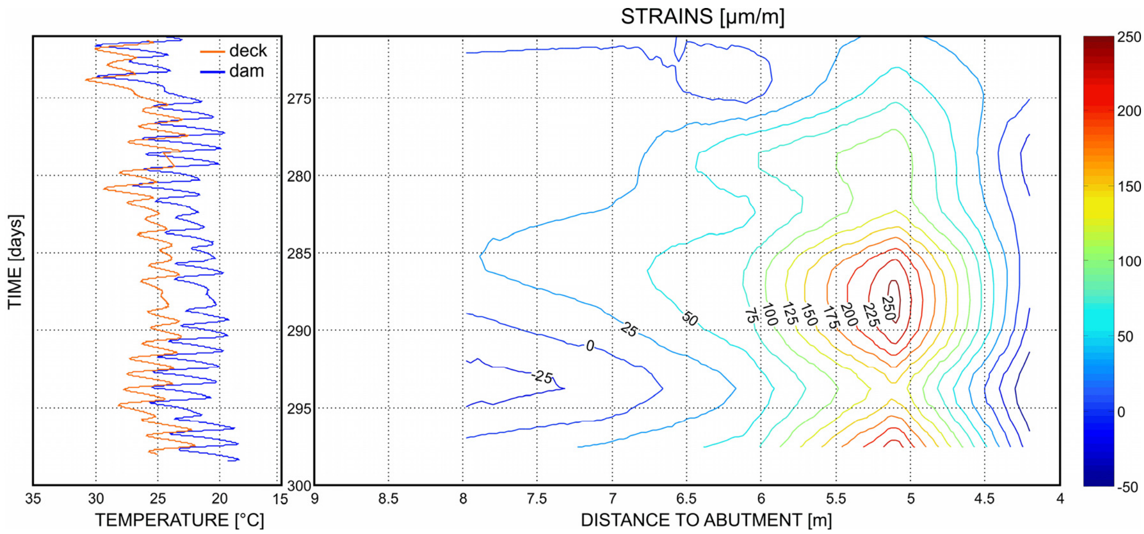

Figure 5. Strain field as function of distance to abutment and time for top FOS layer

5 STRUCTURAL ANALYSIS OF MEASUREMENTS

The nature of jointless frame structures makes the interpretation of the measured results extremely difficult and almost impossible without an adequate finite element model. However these systems are highly undetermined compared to classical bridge structures where the sub- and superstructure are separated by bearings. Constraints given by the active and passive soil pressure working against or amplifying the effects of shrinkage, creep, post tensioning and temperature extremely challenge the modeling of such systems. The required soil characteristics for such analyses (as well as for a proper design) of a jointless frame structure go beyond the information normally provided for the design of a classical bridge structure. While in general a geological survey provides only the characteristic values of e.g. the stiffness modulus and maximum allowable pressure based on the lower limit of the expected distribution, for the design of jointless frame structures such values will be required in addition for the upper range.

Contrary to most characteristics of building materials the distribution of soil characteristics show a high coefficient of variation. In case of jointless frame structures this variation complicates the analysis of the entire structure as all parts are interacting with each other and the soil body. Furthermore it is not possible to determine a characteristic value for a certain soil parameter which lies on the save side for all considerations.

The nonlinear behavior is not limited to the soil and its interaction with the bridge structure. Most jointless frame structures are fully built from concrete. The young's modulus of concrete itself shows a comparably high coefficient of variation which depends much on the type of concrete, its aggregates, cement and curing. Still much more important is the nonlinear behavior of reinforced concrete as most codes recommend applying for cracked concrete only 60% of the initial stiffness. The knowledge of the real stiffness distribution is vital for the description of the system behavior of jointless frames as this gives the basis for the force and moment distribution of the structure. As a consequence the structural analysis of a jointless frame structure will require a full nonlinear model making it a demanding task. This task is even more challenging, as the building process and sequence itself may influence the system behavior as the sequence of shrinkage will change the restrains in the frame structure causing different stiffness distributions.

In general and especially in case of CFBs meaningful measurements have to be limited to local effects like strains and deflections targeting certain questions as presented in this paper. Accordingly the measurement campaign focused on the soil-structure interaction and was able to verify the initial design assumptions (e.g. effective degree of earth pressure and performance of slab solution) which provide the basis for an overall structural analysis of the Marktwasser bridges performance. The complexity of the problem calls for further detailed measurements which will reduce the uncertainties associated with CFBs and finally assures that this design concept actually results in low maintenance requirements as suspected.

6 BESTS PRACTICE FOR THE DESIGN OF CONCRETE FRAME BRIDGES

As the soil parameters are the key for an efficient design the statistical properties for the main soil parameters should be established applying all available techniques such as exploratory drillings and insitu tests. Based on these results advanced nonlinear material and geometry analyses should be performed utilizing fitting constitutive laws.The ultimately obtained steel reinforcement distribution is to be optimized with respect to structural detailing and the reinforcement layout in order to account for the particular characteristics of CFBs and ensure an agreement with serviceability requirements such as crack width limitations.

As the performance of CFBs is highly dependent on the quality of construction works and the real soil properties at site constant supervision should be ensured during constructions. Finally integrated performance monitoring after completion guarantees the safety and serviceability of the structure and increases experience with this type of structure which will result in a more efficient future design practice.

CONCLUSIONS

A three-span jointless frame structure has been instrumented with an integrative monitoring system aiming at a better understanding of structural response under varying conditions. In particular the instrumentation regarding the soil-structure interaction and respective first analyses have been presented. Based on the available data it could be concluded that the design assumption concerning the earth pressure against the abutment was correct. The performance of the approach slab detail is basically in agreement with the initial design hypothesis. However, the complexity in structural response and especially the soil structure interaction can not be fully covered by current analytical models. Ongoing monitoring in this area in combination with numerical studies applying calibrated advanced finite element models will further improve understanding and future construction work.

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