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Long term monitoring of a UHPFRC-strengthened bridge deck slab using strain gauges

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ABSTRACT: The Chillon viaducts, in service since 1969, are two parallel structures with a total length of 2.1km each and spans varying from 92 to 104 meters. This post-tensioned precast concrete structure was strengthened with UHPFRC (Ultra High Performance Fiber Reinforced Cementitious composite) in 2014/2015 as, among other aspects, fatigue behavior was of concern. A monitoring campaign was commenced in May 2016 in order to verify the effectiveness of the UHPFRC-strengthening.

The representative span of one of the viaducts was instrumented with strain gauges and thermocouples. The strain gauges were installed directly on the reinforcement bars in the longitudinal and transversal directions. This paper presents the main outcomes after more than 2 years of monitoring. The considerations on traffic-induced stress variation and thermal strain effects are presented. The results from the monitoring confirm the expected effectiveness of the UHPFRC strengthening.

1 INTRODUCTION

The Chillon viaducts, in service since 1969, are two parallel structures with a total length of 2.1km each and spans varying from 92 to 104 meters. This post-tensioned precast concrete structure was strengthened with UHPFRC (Ultra High Performance Fiber Reinforced Cementitious composite) in 2014/2015 as, among other aspects, fatigue behavior was of concern (Brühwiler et al (2015)). A monitoring campaign was commenced in May 2016 in order to verify the effectiveness of the UHPFRC-strengthening. The monitoring also has the objective to better understand the nature of the motorized traffic on this heavily loaded road section.

2 MONITORING DESCRIPTION

The monitoring system discussed in this paper is composed of four strain gauges, eight thermocouples and one humidity meter.

The strain gauges are glued in two locations (Figure 1). Group 1 is located at mid span, on the central axis of the slab, where the longitudinal and transversal bars are crossing. Group 2 is placed in the distance of around 50cm from the first one, at another rebars' crossing point. At each of the two locations, one gauge is glued to the longitudinal rebar, while the other to the transversal rebar. To do so, the rebars where detected and then the concrete cover carefully removed to expose the reinforcement.

The seven thermocouples are glued to the concrete surface on the perimeter of the box girder from inside. Additionally, the air temperature and humidity inside of the box girder are registered.

The signals from strain gauges were recorded with frequencies of 50Hz, 100Hz or 200Hz, while from thermocouples and humidity sensor with frequencies of 0.1Hz, 1Hz or 5Hz, depending on the monitoring period. The frequency was varied to get the minimum file size while not losing any important strain peak due to the traffic. Still, about 200MB of data is collected daily.

3 RECORDED STRAINS

The signal registered with the DAQ (Data Acquisition System) is composed of the traffic-induced strain variation and the thermal wave, as presented in Figure 2. To analyze the monitoring results, the two kinds of signals need to be separated. Since the variation of thermal wave is much slower than of traffic-induced strains, a running average function can be used. The signal resulting from this operation presents only the thermal response of the structure. If this signal is subtracted from the original one, only the structural response due to the traffic action is obtained (Treacy (2014)). In this paper, the two parts of signal shall be discussed separately.

4 TEMPERATURE VARIATION

The monitored part of the viaduct is oriented approximately along the north-south direction. From the east, it is close to the slope of a mountain and the neighboring viaduct, while the western part is fully exposed due to the presence of lake. The effect of this orientation will be discussed below on the example of a randomly chosen day (10.04.2017). Due to its position, the structure remains shadowed in the morning, while being exposed to the sun in afternoon, until the sun sets. Additionally, the thermal inertia of concrete causes a delay in the strain thermal wave. Since the temperature is measured on the bottom face of concrete, additional delay due to the heat transfer across the slab is observed. Thus, the lowest registered temperature occurs at around noon (Figure 3).

The largest temperature variation is registered by thermocouples T6 and T7, which are located respectively on the upper and lower slab of the box girder. This comes from the difference in concrete thickness, i.e. the upper slab thickness is 22cm and the lower slab is 16cm, while the walls are 40cm thick. Additionally, the void in the cantilevers acts as a thermal insulator. Due to that, the wall temperature starts to rise approximately 2 hours later than temperature of the slabs.

Within the thermocouples on the walls, the highest temperature is recorded by thermocouple T5, then T4 and T3 respectively. This is due to the exposition of this wall to the west, where the sun may operate approximately from 3PM until sunset (8PM) on the discussed day (April 10th).

The most stable temperature is the one recorded inside of the box-girder. It is also lower than the temperature of the walls. This depends on the external air temperature during the couple of previous days and is expected (Treacy (2014)).

5 IMPORTANCE OF THERMAL WAVE

As mentioned previously, the recorded signal is composed of the strain from two sources: thermal and traffic actions. When only the vehicle traffic loading is of interest, the thermally induced strain wave needs to be removed. However, with proper instrumentation, the thermal wave can carry information as well. In this monitoring campaign, the Poisson half-bridge was used, which is a type of the Wheatstone Bridge circuit. It is composed of two active gauges, measuring the strain perpendicularly in relation one to another (Figure 4).

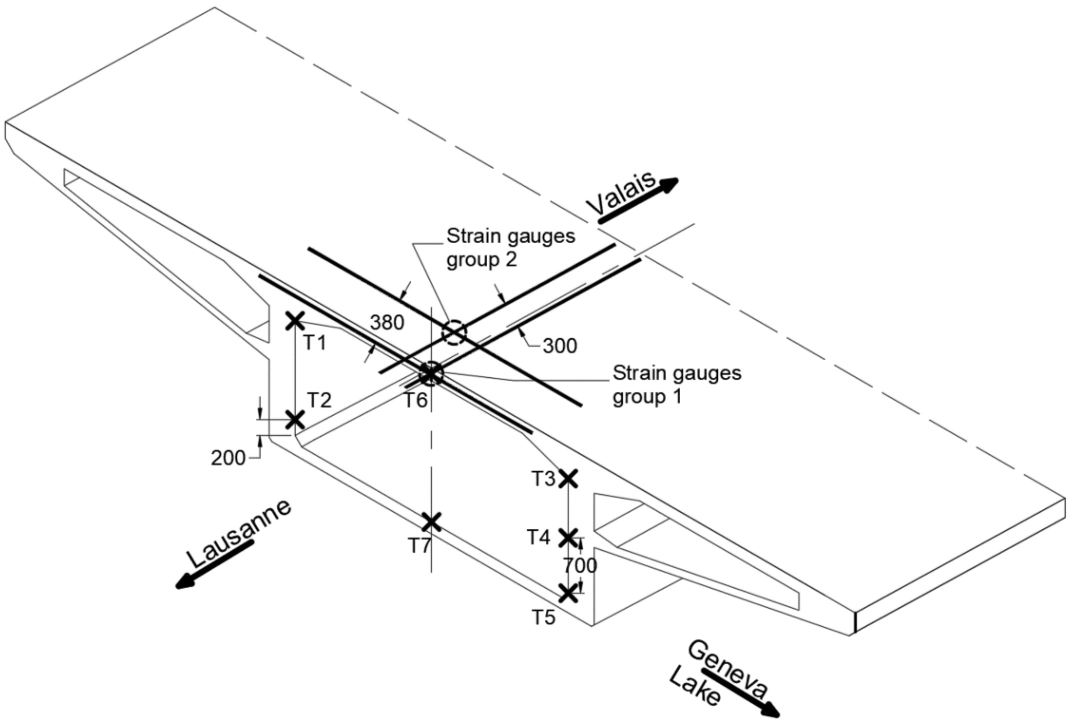

Figure 1. Scheme of monitoring; T1 to T7 - thermocouples; dimensions in mm.

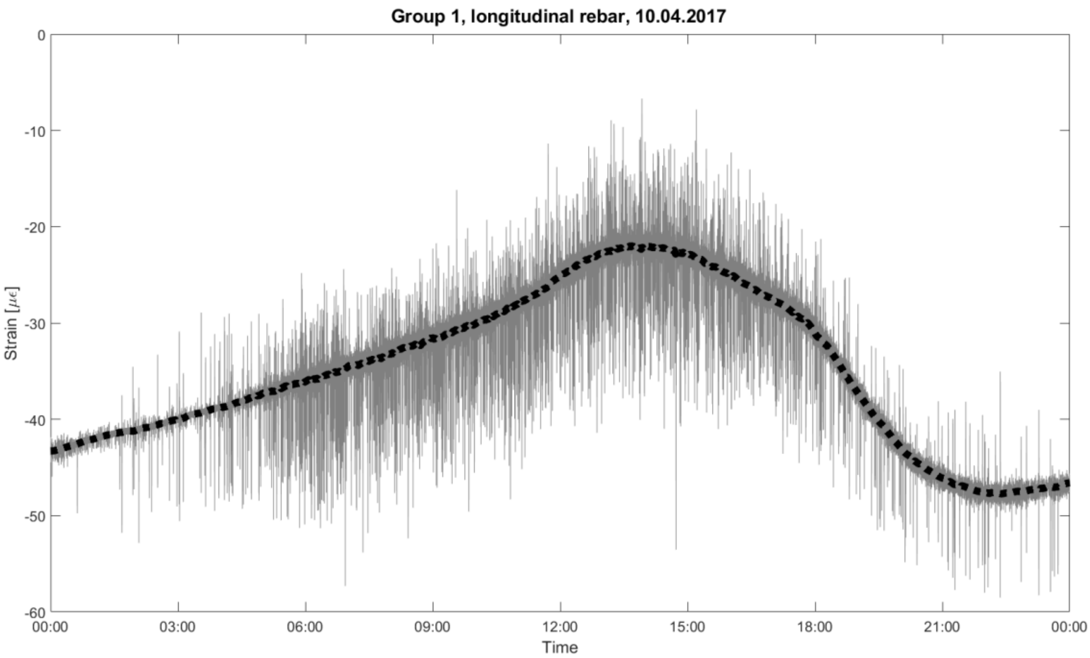

Figure 2. Strain registered during one day on longitudinal rebar, group 1; filtered-out thermal wave shown with dotted line.

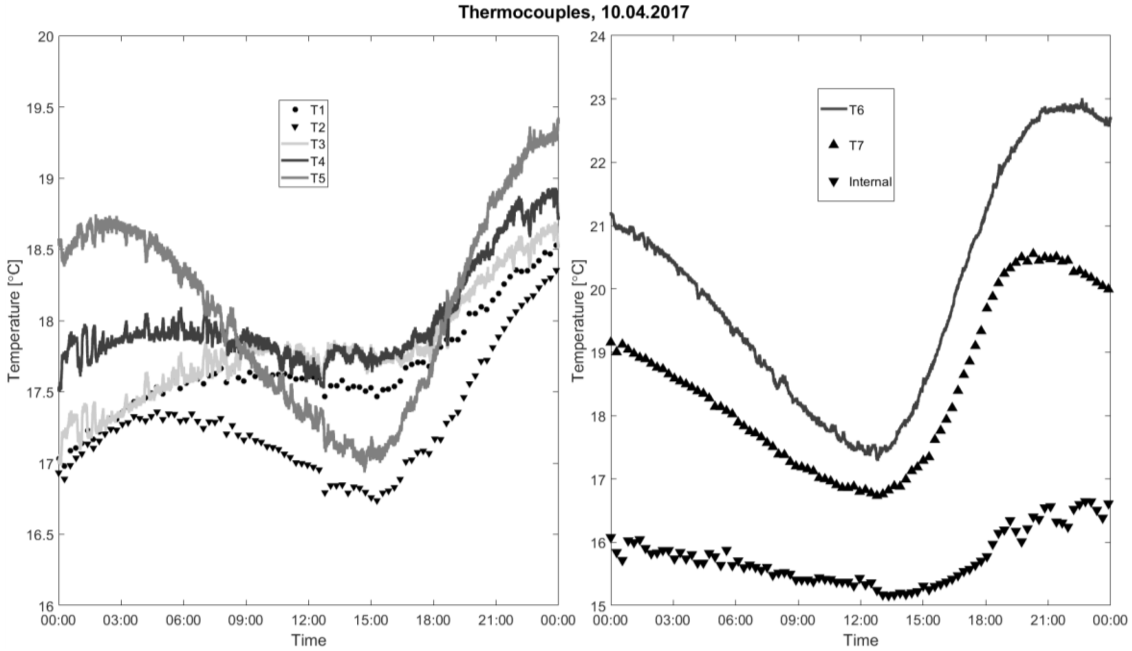

Figure 3. Temperature registered during one day with all thermocouples.

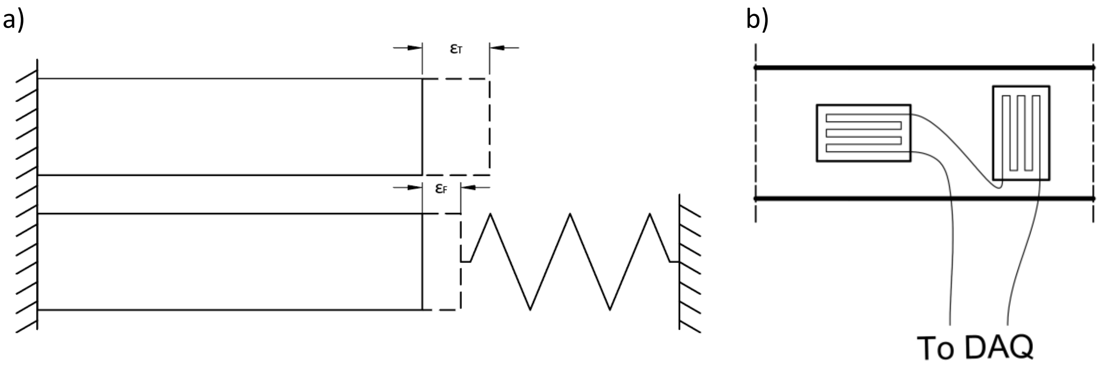

Figure 4. a) Free and partially restrained thermal expansion; b) Scheme of strain gauges glued on rebar in half-bridge.

The gauge oriented along the rebar measures both the thermal expansion of the slab and the deformations due to the traffic action. Since the concrete cover of the rebar is locally removed, the section of interest is free to expand in the direction perpendicular to bar axis. Thus, the perpendicular gauge is recording only the free thermal expansion of steel and the strain variation due to Poisson's effect. Thanks to the half-bridge connection, the signal recorded with perpendicular gauge is subtracted from the signal recorded by the longitudinal gauge taking into account the Poisson effect.

The DAQ automatically cancels the variation of electrical conductivity of cables and measurement unit due to the changes of temperature. Thus, the only source of this difference originates from the strain gauges.

The upper portion of Figure 4 a) presents situation of the gauge which is perpendicular to the rebar axis. If the Poisson's effect is disregarded, it measures only the free body expansion according to the formula (1):

$$\varepsilon_T = \alpha_T \cdot \Delta T \qquad (1)$$

where ε_T is a free thermal expansion, α_T is a coefficient of thermal expansion and ΔT is the temperature variation.

The measurements taken by the gauge parallel to the rebar's axis are affected due to the partial restrain of the slab. However, as the restrain is partial, there is still some free expansion possible, noted with ε_F. The effect of Wheatstone half-bridge can be described by the relation:

$$\varepsilon_F - \varepsilon_T = -\frac{\sigma_T}{E} \qquad (2)$$

where σ_T is the stress due to partially restrained thermal expansion and E is the modulus of elasticity of steel. The right part of equation (2) is recorded by the DAQ as a thermal wave, with the dotted line in Figure 2. Thus, the monitoring system allows for an indirect measurement or the residual thermal stress variation in the structure.

The variation of the residual thermal stresses in longitudinal and transversal rebars are presented in

Figure 5. In the longitudinal rebar, the structural response is delayed by 1.5 hours with respect to the slab temperature. The transversal rebar stresses are further delayed, in total by 4 hours. The stress variation in longitudinal bar is mostly dependent from the temperature of slab, while the transversal rebar responses to the temperature distribution on the whole box-girder perimeter. Importantly, the stress variation in the transversal reinforcement is much larger than that in longitudinal reinforcement.

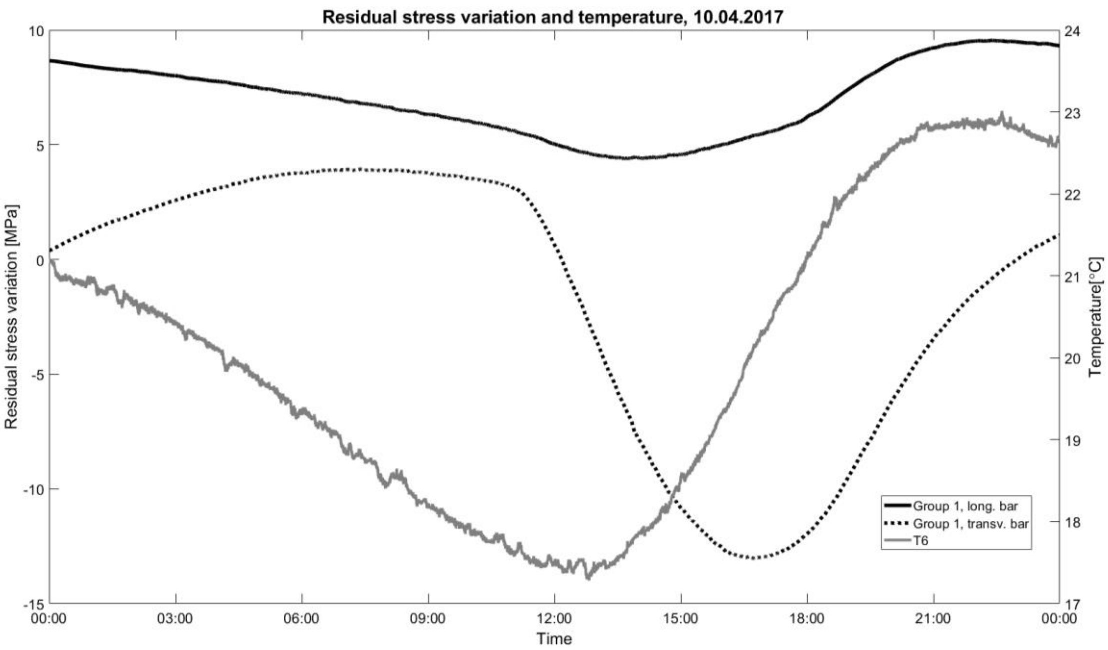

Figure 5. Residual thermal stress variation and temperature registered during one day.

6 HISTOGRAMS OF STRESS VARIATIONS

In Figure 6, the histograms of stress variation originating from temperature variation and traffic loading are presented. The histograms were prepared on the basis of measurements between May 2016 and end of 2018. For the transversal rebar of group 1 and longitudinal rebar of group 2 the data was available for 713 days in total, while for longitudinal bar of group 1 and transversal bar of group 2 it was 514 and 568 days, respectively. These differences are due to the failure of two gauges after around 2 years of measurements.

The thermal stress variation histograms were prepared by taking the thermal wave signals of each day and composing them together. To such a data, representing the whole period of interest, the rainflow algorithm was applied. In this way, the day-to-day jumps are not considered as stress variation and the effect of windowing is avoided.

Figure 6 reveals that, firstly, the maximum stress variation ranges due to both the partially restrained thermal expansion and the traffic action are on the similar level, namely 13MPa for longitudinal and 25MPa for transversal rebar. Obviously, the number of cycles due to the traffic is much higher than the one due to the temperature variation. However, the number of cycles of highest amplitudes are similar.

Secondly, the stress variations in the reinforcement bars are far below the Constant Amplitude Fatigue Limit (CAFL), which in Swiss standard for existing structures SIA 269 is equal to 120MPa. Thus, the slab is not vulnerable to fatigue.

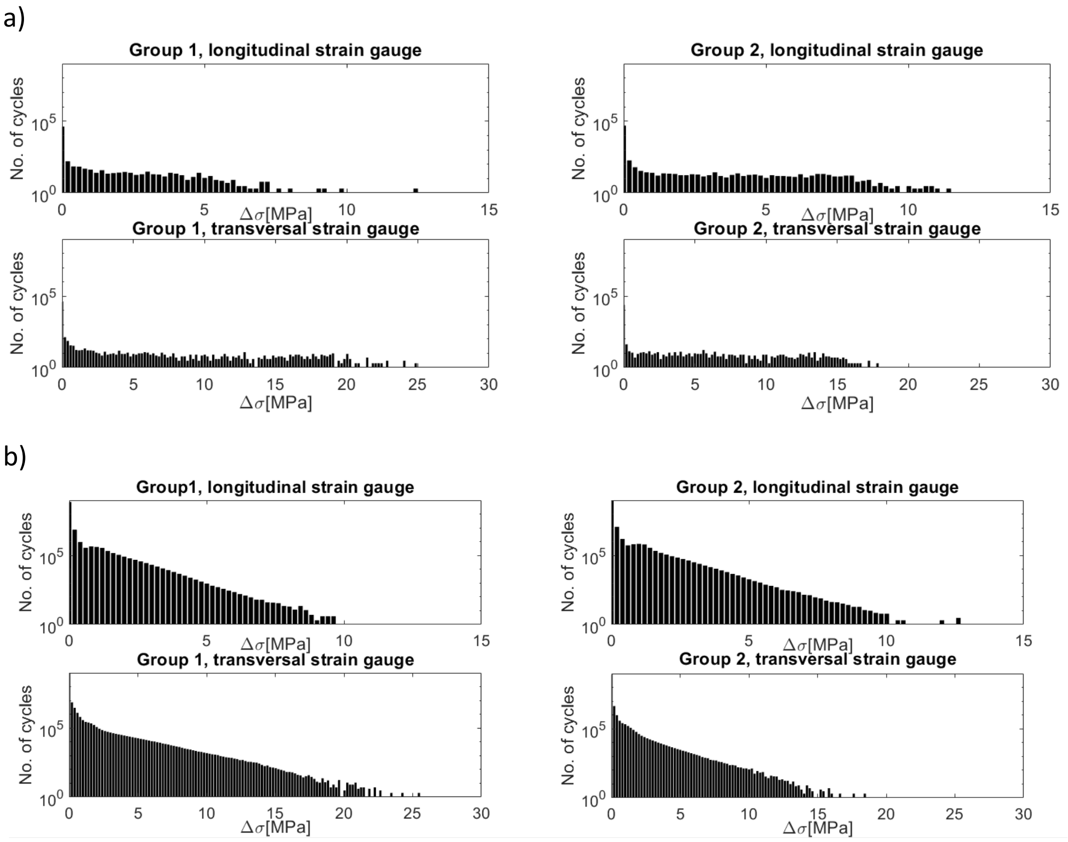

Figure 6. a) Temperature and b) traffic induced stress variations histograms for the whole monitoring period.

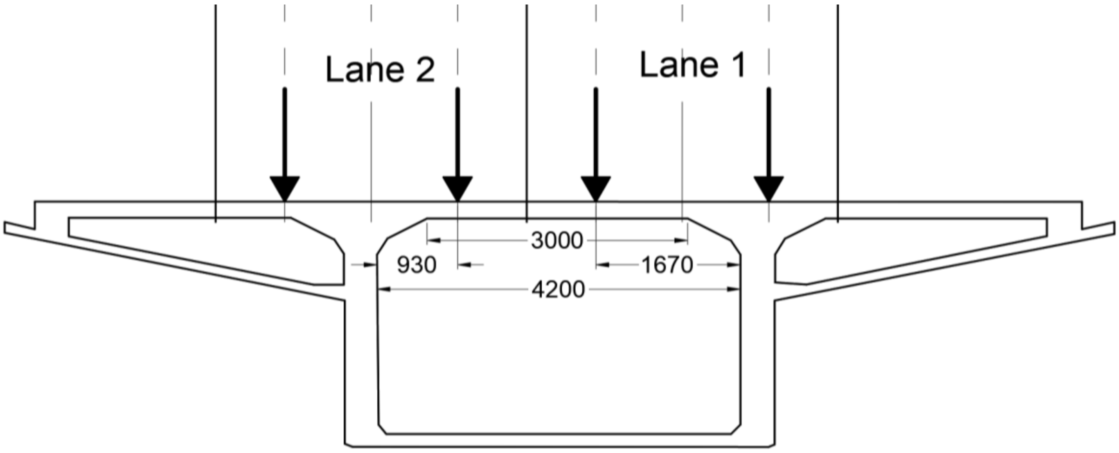

Figure 7. Cross-section of the superstructure with layout of the fatigue load model (dimensions in mm).

7 STRESSES FROM MONITORING VS CALCULATED

Since the upper slab of the box-girder was relatively thin (18cm), its fatigue performance was of concern before the strengthening with UHPFRC. Below, a simplified fatigue analysis of the UHPFRC strengthened (22cm thick) slab in transversal direction will be presented.

The slab (Figure 7) can be represented simply by an elongated plate fixed along its longer sides. As the haunched parts are much stiffer than the slab itself, a span of 3m shall be adopted for this calculation. A shell numerical model of plate was prepared to get the bending moments distribution.

According to the Swiss standard for existing structures SIA 269, the fatigue load model consists of a double-axle with 1.2m spacing and the following axle load:

$$Q_{fat} = Q_{k,1} \cdot \alpha_{Q1,act} \cdot \gamma_{Ff} = 300kN \cdot 0.7 \cdot 1.0 = 210kN \qquad (3)$$

where $Q_{k,1}$ is the characteristic axle load on Lane 1; $\alpha_{Q1,act}$ is the updating factor for road traffic; γ_{Ff} is the partial load factor for fatigue. The fatigue load model is positioned on the real, rightmost lane of traffic (Figure 7) and wheel force is distributed on the square area of edge length 0.4m.

The calculated maximum positive bending moment is equal to 24kNm, and the computed strain and stress distributions due to this moment are given in Table 1. The maximum stress variation in reinforcement is just below the CAFL. This is because the strengthening of the structure was designed using the method presented here.

Table 1. Stress distribution in slab under positive bending moment 24kNm/m

Layer	Strain [‰]	Stress [MPa]
UHPFRC	-0.13	-7
Reinforcement in UHPFRC	-0.04	-9
Concrete	N.A.	N.A.
Upper reinforcement in concrete	0.09	18.2
Lower reinforcement in concrete	0.53	108

However, the stress variations calculated using the structural code load model are four times bigger then measured. This big difference comes, among others, from exaggerated dynamic factor implicitly present in the load model (Ludescher (2003)) leading to overestimating the load nearly by factor of two, which in turn leads to uneconomical design and verification of structures.

8 CONCLUSIONS

This paper presents the main results from the two-and-a-half-year-long monitoring campaign of the deck slab of the Chillon viaduct in Switzerland. The measured stress variations due to the traffic loading are much lower than the fatigue endurance limit (Constant Amplitude Fatigue Limit) of the investigated reinforcement bars. Furthermore, the traffic-induced stress variation is of the same order as the one due to the partially restrained thermal expansion of the structure.

Since, as expected, the measured stress variations are much smaller than the Constant Amplitude Fatigue Limit, there is no fatigue issue anymore for the deck slab of the viaduct.

9 REFERENCES

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