Monitoring of Chillon viaduct after Strengthening with UHPFRC

Conference Paper

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ABSTRACT: Located at the shore of Geneva Lake, in Switzerland, the Chillon viaducts are two parallel structures consisted of post-tensioned concrete box girders, with a total length of 2 kilometers and 100m spans. Built in 1969, the bridges currently accommodate a traffic load of 50,000 vehicles per day, thereby holding a key role both in terms of historic value as well as socio-economic significance. Although several improvements have been carried out in the past two decades, recent inspections has demonstrated an alkali-aggregate reaction in the concrete deck slab reducing its strength. In order to strengthen the concrete deck and slow down further AAR, a layer of 40 mm of Ultra High Performance Fiber Reinforced cement-based Composite (UHPFRC) (incorporating rebars) was cast over the slabs, acting as a waterproof membrane and providing significant increase in resistance via the UHPFRC - RC composite action, in particular of the deck slab.

Two years after completing the works, a Structural Monitoring campaign was installed on the deck slab in one representative span, based on accelerometers, strain gauges, thermal and humidity sensors. This campaign seeks to reveal information on the behavior of UHPFRC-concrete composite systems, such as increase in stiffness, fatigue strength, durability and long-term performance. Consequently, the campaigns is expected to last for at least three years. A first insight of the analyzed results from the initial months of measurements is presented herein, along with future improvements or necessary changes on the deployment.

1 INTRODUCTION

The Chillon viaducts connect the Swiss region of Montreux and Villeneuve by means of two post tension concrete box girder structures, allocating a traffic load of 50,000 vehicles per day (Perret et al. 2014). With a length of 2 kilometers, divided in spans of 96, 98 and 104 m, these bridges along the shore of lake Leman stand not only as a critical item in the transport network, but also as a remarkable part of the cultural value of the region.

Since they were constructed in 1969, the bridges have been inspected and rehabilitated along the years, in order to maintain the safety levels in accordance with updated loads and codes. The last surveillance in 2014 revealed an Alkali-Silica Reaction (ASR) in an early stage, which motivated assessment of different solutions, both for strengthening the structure as well as diminishing the growing rate of the reaction. As established in Hobbs (1988), ARS is a combination of two phenomena: reactive aggregate and high humidity exposure. Currently, aggregates are selected carefully to avoid this type of issues, although for existing structures the only possible intervention involves minimizing the exposure of the section to water. Hence an ideal retrofitting method should serve as a waterproof membrane, while providing structural strength, as the ARS affects the concrete strength and its modulus of elasticity. In this context, an Ultra High Performance Fiber Reinforced cement-based Composite (UHPFRC) application on top of the existing deck was chosen as the most economical and convenient, based on numerous successful projects (Brühwiler & Denarié 2013, Martín-Sanz, Chatzi, & Brühwiler 2016). UHPFRC is characterized as an outstanding material for rehabilitation projects due to its exceptional mechanical and durability properties, low porosity and microcrack control. Furthermore, the works at each bridge were completed within a single month, rendering a short-term closure highly appreciated by the commuters (Bastien-Masse & Brühwiler 2015). Despite UHPFRC being used in past decades as a rehabilitation material, limited literature is available on the long term behavior for large scale projects.
Taking into consideration the age of the bridges, the present structural damaged condition and the innovative solution used for their strengthening, a monitoring deployment was envisioned, aiming to render valuable information on the remaining lifespan of the viaducts. Structural Health Monitoring (SHM) techniques based on dynamic measurements are widely spread nowadays and can provide a non-invasive and convenient solution for bridge inspection and damage detection (Ko and Ni 2005, Brownjohn 2007). In this case, modal analysis will be combined with strain gauge measurements as well as temperature and humidity information, since it has been proven that the last two factors impose a variability in modal parameters, which can mask damage effects. Even though vibration-based methods have experienced a large improvement in recent years, with new sensors technologies and advances in automation, several drawbacks must still be overcome. Those shortcomings are related to a) lack of knowledge from the excitation source, as wind or traffic is usually not measured in Operation Modal Analysis (OMA), and b) the influence of environmental factors, as established by Peeters (2000). In order to assess the condition of the structures, these methods rely on changes on modal parameters, where a loss of stiffness indicates a possible damage scenario. Prediction of these events requires further analysis, since in reality, modal parameters are sensitive to several factors as well as deterioration. Starting by temperature effects, Kim, Park, & Lee (2007) observed that higher temperatures reduce modal frequencies, with more pronounced influence on flexural modes than on lateral ones. Watson & Rajapakse discovered that asphalt stiffness varies dramatically especially around the freezing point. Moreover, steel and concrete modules of elasticity are also considerably affected (Ubertini et al. 2017). However, further studies have shown that material properties are not the only factor that is altered. Taylor & Stanton (2010) echoed that friction coefficients for teflon bearings range between 3-5% at 25°C and 15% at -20 °C, and changes on support conditions shift the natural frequencies up to 50% (Alampalli 2000). Another modifying component pertains to the applied loads, including wind speed changes, which produce variations in structural dynamics (Cross et al. 2013), or heavy traffic in short span bridges, which varies structural mass and thus modal properties. In pursuance a method which differentiates ambient variability from damage, two main strategies have been followed: first, regression models, when input is unknown and therefore Principle Component Analysis (PCA) is employed to obtain the representative components for the operational variations (Reynders & De Roeck 2014). Instead of PCA, AutoRegressive (AR) can be adopted, as demonstrated by Nandan and Singh (2011), or Polynomial Chaos Expansion (PCE) where the changes in environmental conditions can be described by stochastic propagation. In this paper, the latter will be applied following the works of Chatzi and Spiridonakos (Spiridonakos & Chatzi 2015; Spiridonakos, Chatzi, & Sudret 2016; Bogoevska, Spiridonakos, Chatzi, Dumova-Jovanoska, & Höffer 2017), where the method predicts the dynamic responses through meta-modelling. The structure of this paper is as follows: Firstly, the structural characteristics of the bridge are described, followed by a detailed explanation of monitoring implementation. Secondly, modal properties and strains are depicted, relating them with UHPFRC strengthening. The construction of a surrogate for the evolution of modal frequencies and strains, i.e., two quantities directly relating to the stiffness of the system, is described next, and finally concluding remarks are offered.

2 MONITORING DEPLOYMENT

2.1 Structural description

The viaducts of Chillon are two parallel structures with almost identical characteristics, constructed in the seventies by the same procedure. The box girder, comprising precast segments varying in height from...
5.2 to 2.2 m, was launched from the piers until midspan by means of a jacking frame and the interconnecting joints were concreted on site. Three different span lengths were included, in order to adjust the structure to the ground profile: 92, 98 or 104 m. The physical joints are located at the middle of the span, every five spans, creating a continuous deck over almost 500 m thanks to longitudinal prestress on the top section. In 2015, ARS was detected, leading to a strengthening campaign with UHPFRC. A layer of 40 mm with transverse reinforcement was applied, whereas on the supports, the thickness reached 50 mm to include additional longitudinal reinforcement up to 1/8 of the span. Eventually, the bottom zone of the supports was strengthened with conventional reinforced concrete.

2.2 Monitoring deployment

A primary objective of SHM lies in retrieving as much information as possible while using a limited number of sensors. In the case shown herein, it is not feasible to instrument both viaducts along the 2100 m they comprise. Therefore, a representative section is chosen, aiming to cover as many features as possible within the length of one span. The selected segment should gather the following characteristics: a) should be placed in the part of the viaduct closest to the lake (Viaduc du Lac), which is exposed to more wind and humidity, b) not be consecutive to the span with a cantilever joint, c) allow for a reasonable distance between power source and installation, d) maintain a straight profile to the extent possible. The span fulfilling the aforementioned requisites is number 5, with 98 m length and a non-symmetric shape (46m+52m) as depicted in Figure 2. The sensor placement is shown in Figures 4 and 5. In order to capture the mode shapes of the structure, eleven accelerometers built in ETH laboratory, are acquired by means of a GANTNER reading unit with 32 channels. Furthermore, three high precision accelerometers EPISensor type were used for a period of one month, in pursuance of a comparison between the custom and lower accuracy sensor and the higher precision (and higher cost) SED instrument. The latter were kindly offered by the Swiss Seismological Service institution for the purpose of this work. The environmental conditions were tracked by means of one humidity sensor and seven temperature sensors embedded in the concrete, located at the midspan of the bridge, on deck, walls and floor, to obtain a complete picture of temperature gradient inside the bridge. Finally, four strain gauges were installed as depicted in Figure 3, attached to the rebar. In order to limit the damage to the concrete structure to the absolute minimum, ground-penetrating radar techniques were deployed to display the reinforcement position and the cover concrete was locally removed. The cover is subsequently replaced again to ensure the protection of the steel. The latter sensors were connected to an HMB Data Acquisition System (DAQ), thus synchronization between both reading units (GANTNER and HBM) is required and relies on the PC clock to which they are connected. Datasets are stored every ten minutes, with a sampling frequency of 200Hz for the accelerometers, 100 Hz for the strains and 1Hz for environmental data.

3 MONITORING RESULTS

3.1 Temperature and Humidity

A common issue in SHM appears when part of the data is damaged or not completely recorded, due to power loss, program bugs or through the entry of dust particles inside the DAQs. In the case study shown herein, temperature, humidity and strains were missing over a period lasting 37 days (18th of March until 25th of May) and accelerations over 15 days (9th until 24th of May). In order to retrieve as much valid information as possible, humidity and temperature readings were collected from the nearest weather station, 8 kilometers from the viaducts at the same altitude. As the correlation between the bridge measurements and the station data results as adequate, the missing values were estimated by statistical methods. The procedure starts with first creating a Vector Autoregression (VAR) model, followed by the application of a Kalman filter, for the period where enough vibration data existed. The prediction provides accurate results, as observed in Figure 6, except at the point where vibration data is missing and therefore a jump on the expected trend is artificially created. Furthermore, despite the aforementioned correlation between variables, it is important to note the different behavior of each sensor depending on its location in the
bridge. For example, sensors T6 and T7, positioned on the top and bottom deck respectively, present a trend very similar to the air temperature (with delay of four hours and a half, attributable to concrete thermal inertia), whereas sensors placed on the walls experience smoother temperature change with smaller amplitudes. Since the accelerometers are installed on the top deck, sensor T6 is taken as a reference for further analysis.

3.2 Modal shapes and frequencies

The reported period herein ranges from 24th of March to June 2017. In a previous campaign, a first evaluation of the acceleration data showed a low signal-to-noise ratio (SNR) due to grounding issues. This problem was remedied and, to confirm the validity of the data, three Episensors with high accuracy were installed and compared against the MEMs, revealing a good agreement between both systems (as observed in Figure 7). However, only the vertical channels are used for the analysis, ignoring at this stage the transverse ones, since the signal was not strong enough to obtain adequate results. The identification process to obtain the frequencies over time was achieved by contrasting several subspace methods, such as Stochastic Subspace Identification (SSI), Auto Regressive Ex-
ogenous model (ARX) and Eigensystem Realization Algorithm with Natural Excitation Technique (ERA NExT), using the built-in functions provided in MATLAB. Improved results were obtained with the latter, although a high order of the state space (120) was still required to capture more stable modes. Figure 8 depicts the need for model clustering, as a large number of modes appears in the range of 1 to 7 Hz and between 15 and 21 Hz. This process, based on Modal Assurance Criteria (MAC) and damping limited to 5% maximum, renders nine traceable modes along the three months of data. The identified modal shapes can be observed in Figure 9, along with frequency and damping in Table 1. Figures 10 and 11 present the evolution of the viaduct frequencies with respect to time and temperature during the analyzed period.

Table 1: Identified modes, damping and frequency.

<table>
<thead>
<tr>
<th>Mode Number</th>
<th>Frequency (Hz)</th>
<th>Damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode 1</td>
<td>1.175</td>
<td>0.012</td>
</tr>
<tr>
<td>Mode 3</td>
<td>1.547</td>
<td>0.015</td>
</tr>
<tr>
<td>Mode 5</td>
<td>1.904</td>
<td>0.011</td>
</tr>
<tr>
<td>Mode 13</td>
<td>3.700</td>
<td>0.006</td>
</tr>
<tr>
<td>Mode 21</td>
<td>5.628</td>
<td>0.021</td>
</tr>
<tr>
<td>Mode 23</td>
<td>6.732</td>
<td>0.009</td>
</tr>
<tr>
<td>Mode 28</td>
<td>7.824</td>
<td>0.003</td>
</tr>
<tr>
<td>Mode 31</td>
<td>13.954</td>
<td>0.014</td>
</tr>
<tr>
<td>Mode 32</td>
<td>16.694</td>
<td>0.001</td>
</tr>
</tbody>
</table>

Figure 8: Stabilization diagram for reference setup.

Figure 9: From top to bottom, left to right, modes 1, 3, 5, 13, 21, 23, 28, 31 and 32.

Figure 10: Frequencies evolution for the first nine modes over time.

4 MODAL UPDATE

As stated above, environmental changes can affect modal parameters and mask the effects of damage. In the case presented herein, a frequency shift can be observed due to temperature changes, more pronounced in modes 3, 21 and 31 than in the others, whereas variations in humidity bear a smaller impact. The field data indicates that the relationship between temperature and natural frequencies is inversely linear, decreasing the latter when the former rises. Although these results are in line with several examples found in the literature ((Cury, Cremona, & Dumoulin 2012), (Moser & Moaveni 2011)), a Finite Element Model has been developed, searching for validation of this results. To this end, the model tries first to match the conditions presented at the minimum temperature and then change the parameters most affected by the 30°C variation, to retrieve the final results. Following the work of Taylor & Stanton (2010), the bearing friction coefficient must be first reduced around 2.5% to account for an increase of temperature of 30°C. Secondly, the asphalt and concrete Elastic modulus were as well diminished, according to Lytton et al. (1993) and Fernando et al. (1993), respectively. Both references show a variance in the results depending on the type of aggregates, curing methods as well as applied loads, which render a range of values implemented on the FEM model through an iterative process. Finally, with regards to what UHPFRC is concerned, limited literature can be found on thermal dependent properties at environmental temperatures. In the BETON-FIB bulletin it was established that drying shrinkage is very low due mostly to the low poros-
Figure 11: Frequencies evolution for the first nine modes over temperature.

ity of the material, whereas autogeneous shrinkage is large and may reach values higher than 1.2 µm/mm. In terms of tensile creep, Garas, Kurtis, & Kahn (2012) demonstrated that microcracking and porosity directly affect the behavior of this parameter, although heat treated UHPFRC shows a smaller compressive creep coefficient. Moreover, Dehn (2004) observed that hydrothermal reactions start to appear above 100 °C. With the aforementioned information in hand, and taking into consideration that the temperature variation reaches only 30°C, the E modulus of UHPFRC was left unchanged. At this point, it is important to note that the model exhibits a high sensitivity to friction coefficients at the bearings, noticed when the first matching process was performed. To this end, in the second round of model updating, the coefficient was changed only in a tight range in order to avoid fictitious results. Once a good agreement is achieved between the frequencies at maximum and minimum temperature for FE model and Chillon bridge, the bespoke parameters are modified following a linear law, obtaining Figure 12 which can be compared to Figure 11. The associated parameters for each temperature increment are displayed in Table 2. Although modes 5, 28 and 32 are not displayed, as the variation with temperature is almost imperceptible. Nevertheless, the remaining modes present a good agreement between FE model and reality, both at initial and final stage, proving the influence of environmental factors over the structure.

Figure 12: Frequencies evolution versus increment of temperature, obtained with FEM model.

5 STRUCTURAL IDENTIFICATION FRAMEWORK

The main goal of SHM is to predict damage or deterioration events at an early stage, by establishing certain operational characteristics that should remain constant if the structural safety is not in danger. As explained in the previous chapter, environmental conditions produce an effect on modal parameters which could conceal healthy states. Therefore, the framework presented herein, based on the works of Chatzi & Spiridonakos (2015), aims to describe dynamic characteristics varying with operational conditions by means of a statistical procedure based on PCE and Independent Component Analysis (ICA). The interested reader is referred to these sources for further details on the workings of these methods.

5.1 Polynomial Chaos Expansion

PCE permits to represent a random variable as a function of another random variable with a given distribution, as a polynomial expansion over multivariate products of polynomials that are orthonormal to that distribution. This method predicts the evolution of uncertainty in dynamical systems, which pertains to the case of SHM. Consider a system $F$ conformed by several independent variables $\Xi_1,...,\Xi_N$, being $N$ the number of variables. In the case explained herein those inputs will be the temperature and humidity, with a joint probability function (PDF) $p_{\Xi}(\xi)$, while the output $Y = f(\Xi)$, with finite variance, will be also random and correspond to the structure frequencies. This output can be expressed as in Equation 1.

$$Y = f(\Xi) = \sum_{d(j)\in\mathbb{N}^N} \theta_j \phi_{d(j)}(\Xi)$$

being $\phi_{d(j)}$ the polynomial basis functions orthonormal to $p_{\Xi}(\xi)$, $d$ the multi-indices vector related to the multivariate polynomial basis and $\theta_j$
Table 2: Parameter values for FE model at different temperatures.

<table>
<thead>
<tr>
<th>∆T</th>
<th>Friction (vert)</th>
<th>Friction (hfinal)</th>
<th>E mod. Asphalt</th>
<th>E mod. Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>KN/m</td>
<td>KN/m</td>
<td>KN/m²</td>
<td>KN/m²</td>
</tr>
<tr>
<td>∆T=5°C</td>
<td>2725000</td>
<td>217000</td>
<td>15000</td>
<td>35000000</td>
</tr>
<tr>
<td>∆T=5°C</td>
<td>2713645</td>
<td>216096</td>
<td>14625</td>
<td>34504167</td>
</tr>
<tr>
<td>∆T=10°C</td>
<td>2702291</td>
<td>215191</td>
<td>14250</td>
<td>34008333</td>
</tr>
<tr>
<td>∆T=15°C</td>
<td>2690937</td>
<td>214287</td>
<td>13875</td>
<td>33512500</td>
</tr>
<tr>
<td>∆T=20°C</td>
<td>2679583</td>
<td>213383</td>
<td>13500</td>
<td>33016666</td>
</tr>
<tr>
<td>∆T=25°C</td>
<td>2668229</td>
<td>212479</td>
<td>13125</td>
<td>32520833</td>
</tr>
<tr>
<td>∆T=30°C</td>
<td>2656875</td>
<td>211575</td>
<td>12750</td>
<td>32025000</td>
</tr>
</tbody>
</table>

Figure 13: PCE natural frequencies estimates for modes 1, 3, 5 and 13, for both training and validation set versus SSI-based estimates.

5.2 Results

Figures 13 and 14 present the results for the training and validation sets, contrasted with the SSI-based frequency estimates. It may be observed that the method provides accurate predictions, even for mode 31 where the scatter is more pronounced. Nevertheless, damage prediction should be performed with a dataset comprising a longer time interval, where a full cycle of seasonal variations is present. This preliminary study shows that the method can be implemented for larger datasets, aiming to identify damage scenarios based on abnormal trends outside the estimated range. It should be mentioned that the process presented above was also performed for the strain values obtained from the four strain gauges installed on the bridge. However, the correlation between the environmental variables and the two longitudinal strain gauges, as well as for the two transverse, was discrepant and presented a value smaller than 0.75. In Figure 15 the results from PCE analysis can be observed, showing that the method is not able to accurately capture the variations, in opposite to the previous example.

Figure 14: PCE natural frequencies estimates for modes 21, 23, 28 and 31, contrasting training and validation set with respect to SSI-based estimates.

Figure 15: PCE strains estimates for the four strain gauges installed on the viaduct.
6 CONCLUSIONS

In this paper, the SHM deployment on Chillon viaduct is presented and the analysis of three months of data is explained. Natural frequencies and damping is obtained through a SSI methodology, and contrasted with the environmental conditions, which are proven to constitute a relevant factor on modal analysis. Therefore, a physical model is implemented to validate the results obtained from the Subspace method, exhibiting a good agreement between them. Finally, a PCE methodology is applied in order to accurately describe the natural frequency fluctuations, achieving promising results for the current data. Further investigation should be conducted on the PCE outcome from strains and ambient data.

7 ACKNOWLEDGMENTS

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