Continuous monitoring of the Vlassenbroek bridge

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ABSTRACT: The Vlassenbroek bridge near Dendermonde (Belgium) is a post-stressed box girder bridge, now some 25 years in service. The largest spans have been constructed using the balanced cantilever method. Due to above normal deflections, the bridge is under close observation, with a continuous monitoring system in operation since 2005. Although number and location of the various sensors are not optimal, the first findings do not point towards abnormal behaviour of the structure. A more in-depth analysis of the available data, supported by a calibrated finite element model, will constitute a basis to judge the effectiveness of the system in its current state.

1 VLASSENBROEK BRIDGE

The Vlassenbroek bridge (Figure 1) is located near the town of Dendermonde in northern Belgium. It leads the busy trunk road N41 over the tidal river Schelde. As such, it is an important north-south traffic link for the entire region. Its importance may even increase should the plans to transform the N41 into a dual carriageway become actual again.

Figure 1 The Vlassenbroek bridge (viewed from the south-west).

1.1 General characteristics

The bridge has a total length of 252,60 m. It consists of 2 parallel adjacent bridges (eastern and western bridge) each 15,70 m wide. At present only the eastern bridge is in use, but should the N41 become a dual carriageway, then the western bridge will also come into service. In the longitudinal direction the bridge has 2 distinctive parts: a northern part of 60 m long and a southern part of 192,60 m long, with a transversal joint in between. This means that from a

structural point of view, the entire construction can be subdivided into 4 separate independent bridges.

1.2 Northern bridges

These bridges are continuous single cell box girder bridges with 2 spans of 30 m each. The box girder has a constant height of 2,13 m and was cast in situ. The post-stressing was applied through longitudinal, transversal and oblique Dywidag steel bars. Abutment and piers rest on pile foundations of concrete precast prestressed piles. The construction crosses the wet lands on the left bank of the river Schelde.

1.3 Southern bridges

The southern bridges are also continuous single cell box girder bridges. They consist of 3 spans: 1 central span of 96,6 m and 2 side spans of 48 m each. The box girder has a variable height of 2,13 m at the extremities, 4,75 m over the central piers and 2,375 m at the centre of the mid span. The central span and 1 side span were built through the balanced cantilever method with prestressed concrete segments of 5 m long, starting from the central piers. Dywidag steel bars (longitudinal, transversal and oblique) assured the continuity post-stressing. The southern bridges cross the actual river. The river Schelde flows from Saint-Quentin in France to Vlissingen in The Netherlands. The river has a large estuary and is tidal in its lower reaches; at the location of the Vlassenbroek bridge the amplitude between high and low tide is still of the order of 4,25 m.

2 BRIDGE HISTORY, MANAGEMENT AND ACTUAL SITUATION

2.1 Bridge history

By early 1979 the construction of the Vlassenbroek bridge came to an end. In September 1981 the bridge was proof loaded with a satisfactory outcome (see further). Subsequently, early 1982, the Vlassenbroek bridge (eastern bridge parts only) came into service and still is, up to this day.

2.2 Bridge management

The bridge is owned and managed by the Flemish Public Authority. On entering into service, the bridge was proof loaded for up to around 90% of its design load. At the same time a general inspection (visual) was held and a basic levelling was conducted, in order to establish a reference to measure future bridge deformations. From then on, the general visual inspection is repeated every 3rd year as well as a check-up levelling. Based on the outcome of these periodic activities, the frequency of the check-up levelling can be increased and/or a particular, in-depth inspection can take place.

2.3 Actual situation

Bridges built through the balanced cantilever method are known to have larger deformations. This is mainly due to the particular nature of the construction method, where the originally isostatic construction “creeps” over time towards its hyperstatic state. However, since mid 1987 the southern bridges have been put under close surveillance as the vertical deflections at mid span were judged to be more than average. The frequency of the check-up levelling was increased from once every 3 years to several times a year. In-depth inspection did not reveal any substantial deterioration. Particular detail: traffic loads did not seem to trigger the increased deflections as the south-eastern bridge, which is in service, does not show larger deformations

than the south-western bridge, which is not yet in use. After a period of stabilization during 2000-2003, the vertical deflection started to increase again over the last years (Figure 2). As of February 2005 a continuous monitoring system has been installed inside the box girder of the south-eastern bridge.

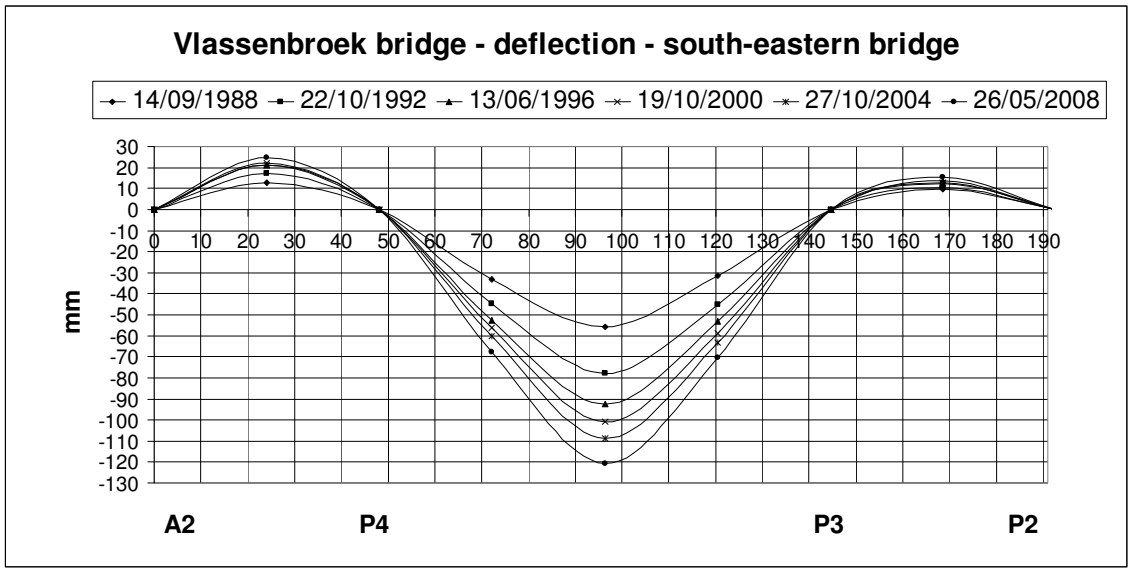

Figure 2 Deflection of the south-eastern bridge over the last 20 years.

3 OVERVIEW OF THE CONTINUOUS MONITORING SYSTEM

3.1 General information

The continuous monitoring system has been set up by Stamotec (Stamotec (2009)), a local Belgian firm, using the OSMOS technology. OSMOS (Osmos (2009)) is a German-French industrial group focusing on integrated safety for structures. Deformations and corresponding temperatures are measured on a continuous basis. Results are stored on-line and can be visualized, compared and treated graphically. As only the south-eastern bridge has been equipped with monitoring sensors, the remaining part of this article will deal with the behaviour of this bridge part only.

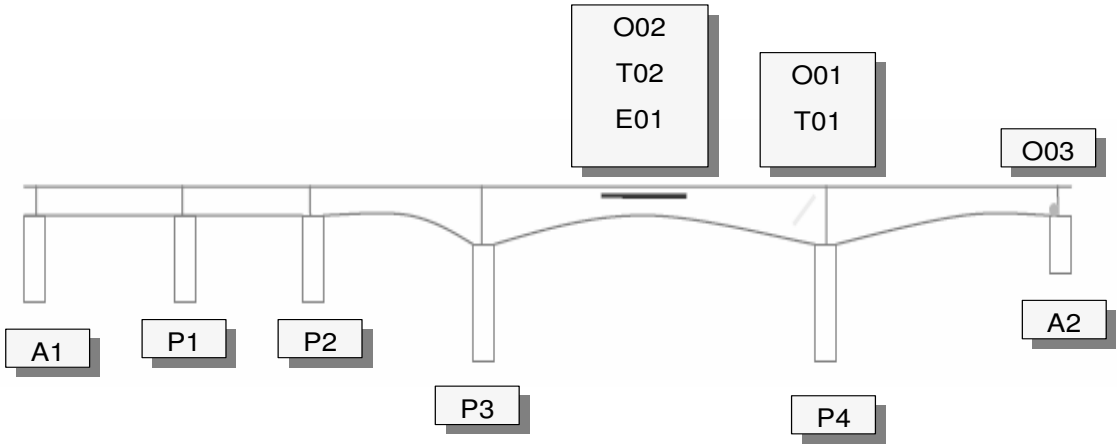

Figure 3 Location of sensors (A = abutment, P = pier).

3.2 Sensors and locations (Figure 3)

3.2.1 Deformation sensors

These are 2 optical sensors, located at 2 places inside the box girder along the central span: 1 at the bottom of the top flange (O02) at mid-span and 1 at the eastern web next to the pier P4 (O01). O02 is a horizontal optical cord of 5 m length, while O01 is an optical cord of 2 m long, placed under an angle of 45°. The sensors are light intensity based according to the OSMOS technology. Light intensity changes are a measure for changes of length of the cords. The precision is said to be of the order of 1µm.

3.2.2 Temperature sensors

Each optical cord is accompanied by a temperature sensor located in the immediate neighbourhood, inside the box girder (T02 next to O02, T01 next to O01). The temperature sensors are supposed to be of a commonly used type as no further specifications have been made available by the implementing firm.

3.2.3 Weigh-in-motion sensor (WIMS)

This sensor is located at the most eastern one of the 3 teflon surfaced elastomeric bearings of the southern abutment (A2). The WIMS (O03) is also an optical sensor of the same kind as O01 and O02. As this sensor has been installed upside-down, an impression of the bearing produces a positive reading.

3.2.4 Accelerometer

An accelerometer (E01) of the capacity spring-mass type is located at a web near the centre of the mid span. It can measure accelerations in the frequency range of 0 Hz to 160 Hz with a resolution of approximately 0,001 m/s².

3.3 The monitoring system

Individual readings are performed in real time with a frequency of 100 Hz. A monitoring station receives these measured data, averages them over a preconfigured period (10 ms to 1000 ms) and stores these average values in a memory with a capacity of 300 values. Out of this memory 2 types of daily files are fed: a static file and a dynamic file. Via modem, these files are transferred to a database on a central server. By logging on to this database through a simple internet connection, data can be retrieved and analyzed on a local PC.

3.3.1 Static data

The daily file with static data contains mean values obtained by continuously averaging the 300 value memory over a chosen interval between 100 s and 1 day. Static data can give a good insight in the longer term behaviour of a structure (year, month, week, day).

3.3.2 Dynamic data

Data are sent to the dynamic file only when a predefined threshold value is crossed. This threshold value is expressed as a specific measurement variation over a specific time variation (e.g. $\Delta l/\Delta t \geq x$). At that time, the memory values are emptied into the daily dynamic file, together with the 300 subsequent values. If the threshold value is not reached, no data is written to the file, so there can be days with no dynamic file. This type of data is suited for very short

term analysis (seconds) and detection of eigenfrequencies. The monitoring system can be configured in such a way that the dynamic data trigger certain alarm levels, which cause a signal to be sent to a visual control panel on a PC connected to the system.

4 THE FINITE ELEMENT MODEL

4.1 Justification and use

As the monitoring system came into operation before the authors of this article were actually involved, some shortcomings were detected while getting familiar with the system. The absence of temperature sensors for the outside temperature makes it more difficult to establish a relationship between the temperature of the environment and the temperature of the structure itself. Additionally, the presence of more deformation and temperature sensors (e.g. at the bottom flange at the same horizontal position as the sensors O02 and T02) would have given valuable information about the curvature of the bridge and the temperature gradient through the bridge deck. An adequate finite element model (FE model) should be able to counter some of the problems related to this lack of information. On top of that, the FE model will certainly lead to a better understanding and interpretation of the measured data.

4.2 Set up of the model

At present, the initial FE model is a 1D beam model (x/z coordinates) composed of 42 variable single box girder sections along the x-axis as close as possible to reality (Figure 4). The software used is Scia Engineer 2008 from the Nemetschek Scia Group (Nemetschek Scia (2009)). As in the Vlassenbroek bridge, the fixed point is located at the pier P4 (ux and uz fixed, φy free). The support P3 is fixed for uz and free for ux and fy, while the 2 outer supports (elastomeric bearings at P2 and A2) are set up with the appropriate stiffness for translation (kux, kuz) as well as for rotation (kφy). The concrete is supposed to be equivalent to class C30/37, derived from an effective concrete compressive strength of 450 kg/cm² (cube 200 mm) at the time of the construction of the bridge in 1978-1979. At this point in time, the objective was to create a simple but reliable model, hence the 1D model was chosen as a starting model. A 2D, or even 3D model, will certainly be considered in a further stadium, if necessary.

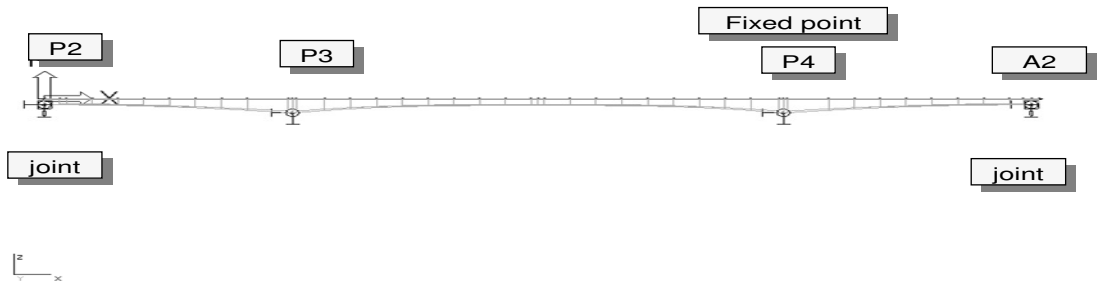

Figure 4 1D beam model (FE model).

4.3 Calibration of the model

The model was calibrated using the data of the proof load from September 1981. During this proof load several loading phases took place: loading of the central span only (phase 1), loading of the central span and 1 side span (phase 2) and loading of the 2 side spans (phase 3). Measured

deflections and rotations were compared to the calculated values by the FE model. The E-modulus (secant) of the concrete was artificially increased by 10 % to 36300 MPa to account for the aging of the concrete between end of construction and proof load (+6,5 %) and to cope with the presence of the post-stressing steel (+3,5%). When taking into account this adjustment, the fit is more than reasonable (see Table 1).

Table 1. Deflections (upward is +) and rotations (clockwise is +): proof load versus FE model

Location		P2	P2-P3	P3	P3-P4	P4	P4-A2	A2
		mrad	mm	mrad	mm	mrad	mm	mrad
Phase 1	Proof load	-1,13	+17,00	1,24	-69,60	-1,24	+17,89	1,13
	FE-model	-1,26	+18,54	1,30	-68,90	-1,29	+18,58	1,25
	Deviation (% abs)	*11,50*	*9,06*	*4,84*	*1,01*	*4,03*	*3,86*	*10,62*
Phase 2	Proof load	-0,99	+15,30	1,14	-59,50	-0,73	+6,52	0,23
	FE-model	-1,15	+16,92	1,19	-59,84	-0,80	+8,18	0,40
	Deviation (% abs)	*16,16*	*10,59*	*4,39*	*0,57*	*9,59*	*25,46*	*73,91*
Phase 3	Proof load	1,07	-13,50	-0,60	+18,60	0,56	-12,95	-0,96
	FE-model	1,08	-13,92	-0,67	+19,61	0,60	-12,63	-0,98
	Deviation (% abs)	*0,93*	*3,11*	*11,67*	*5,43*	*7,14*	*2,47*	*2,08*

Calculations performed with the calibrated FE model also indicate that a temperature gradient of 1 °C through the bridge deck (top slab 1°C warmer than bottom slab) accounts for a -1,85 mm deflection of the centre of mid span. Subsequent check-up levelling performed at several hours on a hot sunny day in July 1995 gave a 7 to 8 mm vertical downward movement of the mid span centre, which would then be equal to a daily thermal gradient of 4,5 °C.

5 OVERVIEW AND DISCUSSION OF THE MONITORING RESULTS

At present, the collected data are being analyzed and compared to the results of the FE model. In the framework of this article, a selection of the first findings is presented.

5.1 Deformation sensors

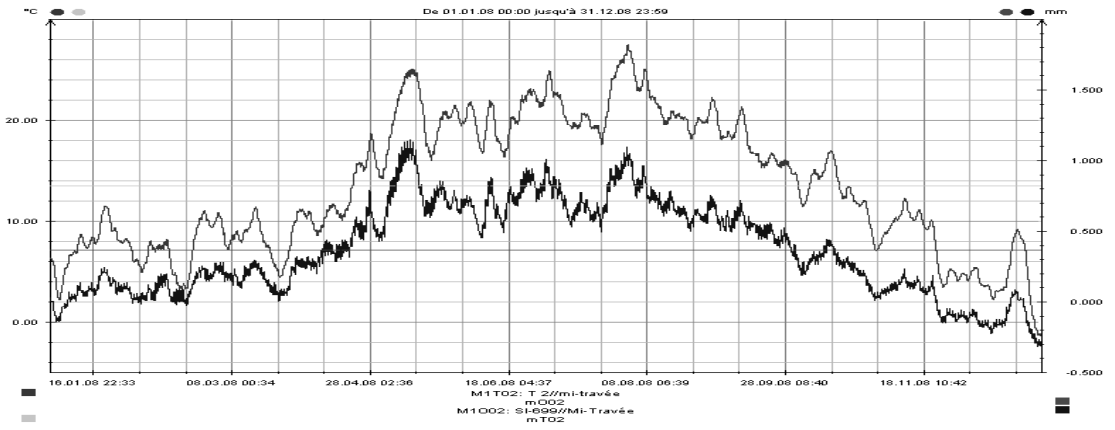

Figure 5 T02 (upper curve) and O02 (lower curve) during 2008 (static data).

As can be expected, figure 5 shows that the deformation at mid span (O02) remains in close correlation with the temperature (T02). The same phenomena can be observed between O01 and T01. This could be considered as an indication that the structure is still behaving in a normal way as far as temperature is concerned. However, a comparison of the evolution of this deformation over the last 3 years shows a diminishing average deformation of the cord O02 (Table 2), which can only partly be explained by a lower mean temperature. As this cord is attached to the bottom of the top slab, this contraction has to be related to the progressive deflection. The FE model will be used to establish a numerical relationship between cord contraction and mid span deflection.

Table 2. Mean deformation of sensor O02 during recent years

Year	mean temperature °C	mean deformation µm	Δ due to temperature µm	Remaining Δ µm
2006	14,2	554		
2007	14,1	453	-5 *(0,00001 x -0,1°C x 5m)*	-96
2008	13,5	368	-30 *(0,00001 x -0,6°C x 5m)*	-55

5.2 WIMS

5.2.1 Long term analysis

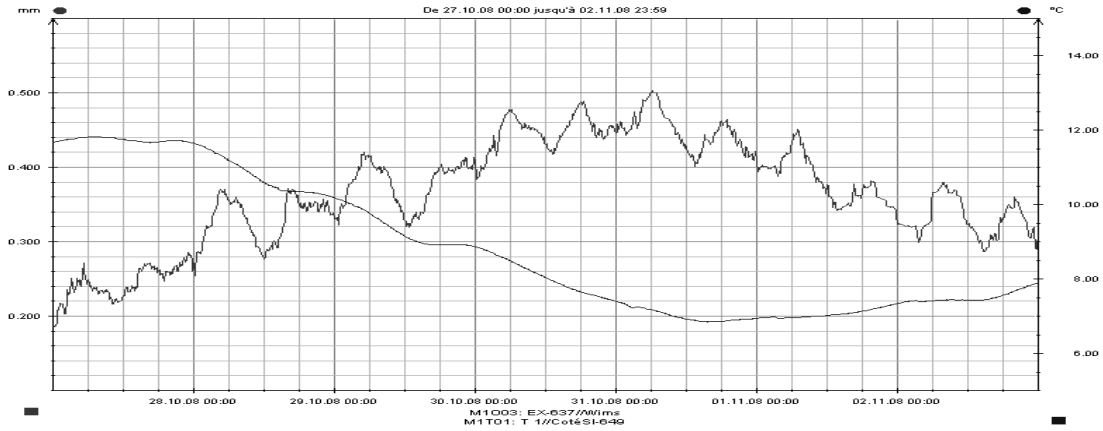

Figure 6 O03 (sawtooth curve) and T01 (fluent curve) during week 2008/44 (static data).

Figure 6 depicts the evolution of the WIMS (O03) and the temperature of the eastern web of the box girder above pier P4 (T01). Given the location of T01, the temperature indicated by this sensor could by approximation be considered as the „bridge temperature“. The analysis of the static data of the WIMS shows that the deformation moves in the opposite direction of the temperature. As the WIMS is installed upside down, this means that the impression of the bearing increases as the temperature decreases and thus as the bridge deck shrinks. Additionally, during each day, sunny or not, the curve „peaks“ twice with an interval of about 12 hours, shifting slightly from day to day. Examination of the tides of the river Schelde during week 2008/44 shows that this movement is definitely connected to the tidal flow. Both phenomena are currently being examined and quantified by means of the FE model.

5.2.2 Short term analysis

An element which has yet to be examined, is the ability of the WIMS to determine from its measured impression the load which has caused this impression. At the time of the installation, the WIMS has been calibrated by driving a truck of known weight over the traffic free bridge and measuring the matching deformation at various speeds of the truck. The impression of the WIMS can then also be compared with the deformation of the sensors O01 and O02 under the same load. A possible problem, however, is that the calibration is based on a single truck on the bridge, which is hardly the standard live situation. Analysis of the WIMS data by a 3D FE model might be required.

5.3 Accelerometer

For the accelerometer (E01) only dynamic data are of use. Figure 7 depicts a dynamic recording at a specific point in time (27 October 2006, from 7:28:45 till 7:29:15). Fourier analysis of this vibration came up with a first eigenfrequency of the construction of 1,22 Hz. This frequency is in line with the eigenfrequency calculated through the FE model (1,19 Hz). Analyses of recent dynamic data show that this eigenfrequency has not changed over the years as their range was still within the interval of 1,20 to 1,25 Hz. As this outcome points to unchanged material properties, which is positive for the structure as a whole, the importance of these results must not be valued too high. Tests on post-stressed concrete girders at Ghent University indicated that their eigenmodes only changed significantly when the girders reached the yielding stage (Matthys et al. (2003), Matthys et al. (2005)). Therefore, modal analysis on pre-stressed/post-stressed elements in serviceability conditions does not appear to be very effective, while the stressing forces prevent practically all cracking.

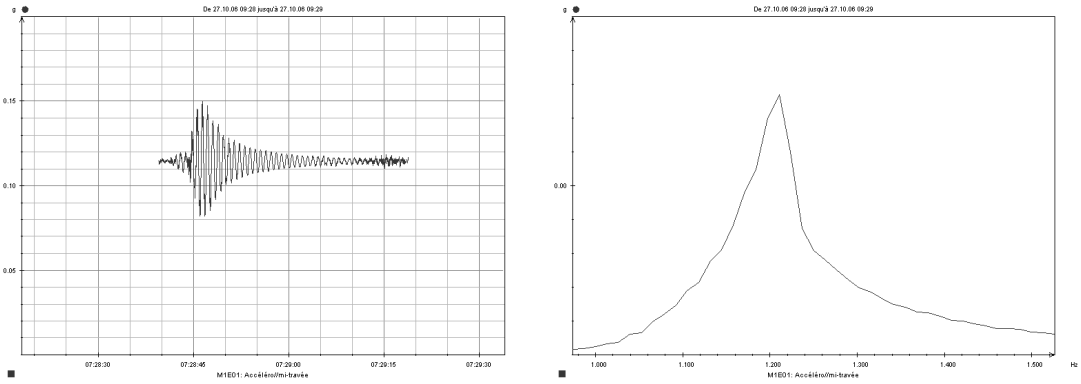

Figure 7 Vibration recorded by accelerometer E01 and resulting eigenfrequency.

6 CONCLUSIONS

While the continuous monitoring of the Vlassenbroek bridge is working properly, its effectiveness remains still to be proved. The fact that deformation data are real time on-line available, is clearly an advantage. However, the low number of sensors and their location are not in favour of straightforward results. As the first findings do not point towards abnormal behaviour of the structure, only in-depth analysis of the available monitoring data can lead to sound conclusions. For that purpose, a calibrated finite element model of the bridge has been set up. This will lead to a better evaluation of the measured data in particular and the monitoring system as a whole.

7 REFERENCES

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