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Monitoring the static and dynamic behavior of The New Svinesund Bridge

R. Karoumi[1] and P. Harryson[2]

[1] The Royal Institute of Technology (KTH), Stockholm, Sweden
[2] The Swedish Transport Administration, Sweden

ABSTRACT: The New Svinesund Bridge across the Ide Fjord between Norway and Sweden is a structurally complicated bridge. Due to the uniqueness of the design and the importance of the bridge, an extensive monitoring program was initiated. The installed monitoring system continuously logs data (accelerations, displacements, strains, hanger forces, temperatures, wind speed and wind direction) from 72 sensors and has gathered data since the casting of the first arch segment in the spring of 2003. As part of the monitoring programme, comprehensive static and dynamic load tests have also been undertaken just before bridge opening. This paper describes the instrumentation used for monitoring the structural behavior of the bridge and presents interesting results such as measured strains, displacements and dynamic properties. Results are compared with theoretic values based on the FE-model of the bridge.

1 INTRODUCTION

The New Svinesund Bridge is structurally complicated. The design of the bridge is a result of an international design contest. The bridge forms a part of the European highway, E6, which is the main route for all road traffic between the cities Gothenburg in Sweden and Oslo in Norway.

Due to the uniqueness of design and the importance of the bridge, an extensive monitoring program was initiated. The monitoring project, including measurements during construction phase, testing phase, and the first 5 years of operation, is coordinated by The Royal Institute of Technology (KTH) which is also responsible for data analysis, verification, and reporting. Instrumentation and operation of the monitoring system has been carried out in cooperation between KTH and the Norwegian Geotechnical Institute (NGI).

In cooperation with the monitoring project advisory group and the bridge design engineers from the contractor Bilfinger Berger AG, the monitoring program was designed to monitor critical construction stages as well as to acquire data needed for design verification studies and long-term performance assessment.

This paper describes briefly the instrumentation used for monitoring the structural behavior of the bridge and presents some interesting results such as measured strains, displacements and dynamic properties. Additional information and reports are available on the monitoring projects homepage, www.byv.kth.se/svinesund.

2 DESCRIPTION OF THE BRIDGE

The New Svinesund Bridge, Figure 1, has a total length of 704 m, and was built in only 36 months. The main span of the bridge between abutments is approximately 247 m and consists of a single ordinary reinforced concrete arch which carries a multiple-cell steel box-girder: double-cell on either side of the arch. The concrete arch has a rectangular hollow cross-section that tapers in two directions reducing the section of the arch from the abutment to the crown in both width and height. The superstructure is joined to the arch at approximately half its height. The steel bridge deck is monolithically connected at the junction to the arch and assists in providing lateral stability to the arch.

The construction of the arch used a climbing formwork and was done in parallel on the Norwegian and Swedish sides. During the construction phase, the arch was supported by cables which were anchored to temporary towers, see Figure 2 (left picture).

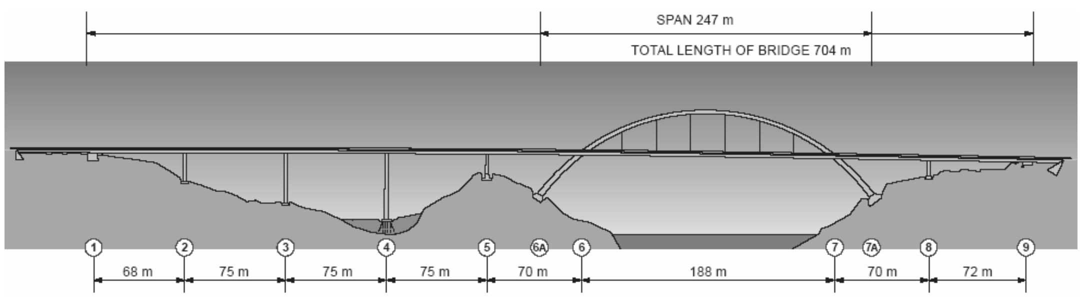

Figure 1. Sketch of the New Svinesund Bridge in its entirety, showing gridline numbering and dimensions.

Figure 2. The New Svinesund Bridge joining Sweden and Norway. The main span is a single arch with a bridge deck on either side of the arch. Photographs of the bridge under construction.

3 THE MONITORING SYSTEM

The data acquisition system consists of two separate data sub-control units built up of basic MGC Digital Frontend modules from HBM (Hottinger Baldwin Messtechnik). The units are located at the base of the arch on respectively the Norwegian and Swedish side. The sub-control system on the Swedish side contains the central rack-mounted industrial computer and is connected with ADSL line for data transmittal to the computer facilities at KTH for further analysis and presentation of data. During construction of the bridge, the logged data on the Norwegian side was transmitted to the central computer on the Swedish side via a radio Ethernet link.

The selected logging procedure provides sampling of all sensors continuously at 50 Hz with the exception of the temperature sensors which have a sampling of once per 20 seconds (i.e. 1/20 Hz). At the end of each 10 minute sampling period, statistical data such as mean, maximum, minimum and standard deviation are calculated for each sensor and stored in a statistical data file. Raw data, taken during a 10 minutes period, is permanently stored if either of the programmed "trigger" values for the calculated standard deviations of acceleration or wind speed is exceeded.

The instrumentation of the arch is composed of, see also James & Karoumi (2003):

- 16 vibrating-wire strain gauges, 4 at arch base and 4 just below the bridge deck, on both the Norwegian and the Swedish side.
- 8 resistance strain gauges, 2 at arch base, 2 in a segment just below bridge deck, and 4 at the crown.
- linear servo accelerometers installed pair-wise. 2 accelerometers at the arch mid point and 2 at the arch's Swedish quarter point.
- 28 temperature gauges, at the same sections as the strain gauges.
- 1 outside air temperature gauge, and 1 3-directional ultrasonic anemometer for measuring wind speed and direction at deck level close to the first support on the Swedish side.

All the 24 strain gauges and 28 temperature gauges are embedded in the concrete section. In some sections both vibrating-wire and resistance strain gauges are installed side by side for instrument verification and quality control purposes.

The suspended part of the bridge deck is instrumented with 6 linear servo accelerometers: 3 at mid point and 3 at quarter point. At each section, 2 of the accelerometers will monitor vertical deck acceleration and 1 for horizontal (transverse) deck acceleration. The forces in the first hangers on the Swedish side are monitored using 4 load cells. Furthermore, displacement sensors (LVDT) are installed at the first bridge pier supports on both sides of the arch to monitor the transverse movement of the bridge deck.

In addition to the above listed permanent sensors, the hangers were instrumented with accelerometers and extensometers during the static and dynamic load tests, to measure the forces and strains in the hangers due to self weight and traffic loadings.

4 TYPICAL RESULTS OF MEASUREMENTS

In the following section some illustrative examples of measured results are presented, to show the possibilities and usefulness of bridge monitoring in increasing our understanding of actual bridge behaviour.

4.1 Monitoring the behaviour during construction

As example of measured response is shown in Figure 3. This figure shows the strains (10 minutes mean strain) at the top and bottom of arch segment S1 close to the arch base on the Swedish side. Casting of this segment was done in June 2003. It can be seen that the strains measured using the resistance strain gauge (RS) agree very well with the ones measured with the vibrating-wire gauge (VW).

The events on-site obviously play an important role in interpreting the results from the strain gauges. The casting of each subsequent segment, the tensioning and removal of the temporary back-stay cables and the lifting of steel deck sections can easily be followed in this diagram. Furthermore, Figure 3 verifies that the assembling of side span deck sections resulted in high concrete tension at the top of segment S1 and therefore one could observe cracking of the concrete. However, lifting of the 1450 tonnes main span deck on the 27^{th} of July caused these cracks to close. The owner can now be sure that, as the asphalt layer is in place, the concrete arch is fully in compression due to the dead load.

Throughout the entire construction process it has also been possible to follow the changing natural frequencies of the bridge, by analyzing data from the accelerometers. Very good agreements were obtained when the measured frequencies from the different construction stages were compared with the theoretical ones calculated by the consulting engineers. For more information see Karoumi et al. (2005).

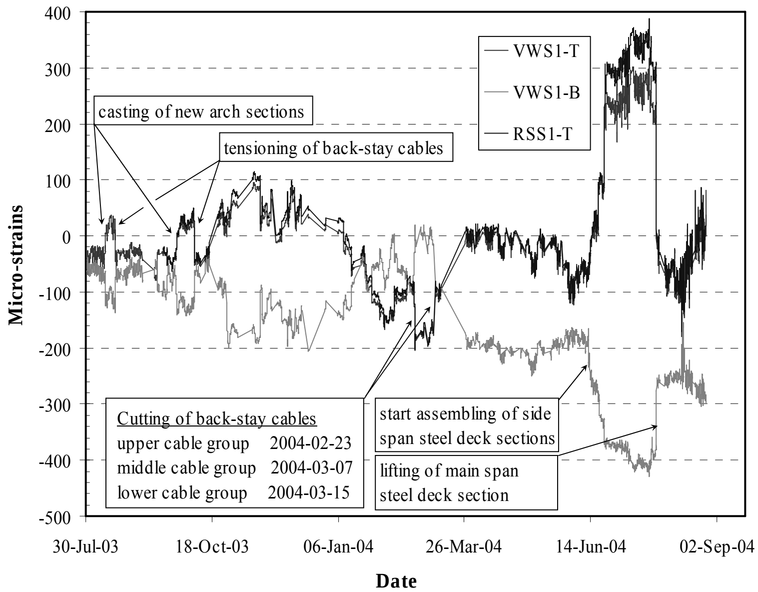

Figure 3. The above figure shows how the work on site is mirrored by the measured strains. The gauges are installed at the top and bottom of arch segment S1 close to the arch base on the Swedish side. VWS1-T vibrating-wire gauge at the top of S1, VWS1-B vibrating-wire gauge at the bottom of S1, RSS1-T resistance strain gauge at the top of S1.

4.2 The behaviour of the completed bridge

4.2.1 Static and dynamic load tests

Comprehensive static and dynamic load tests were performed on 18^{th}-19^{th} of May 2005, just before bridge opening. Detailed program and results are presented in Karoumi & Andersson (2006). The collected data has been used to understand the static and dynamic behaviour of this unique structure as well as to produce an initial database (a footprint) of the undamaged structure that can be used for future condition assessment of the bridge.

In the static tests, eight 25-ton lorries (see Figure 4) with known dimensions and axle weights were positioned according to 7 different loading patterns. Table 1 gives the order in which these were carried out. As seen in Table 1, important loading patterns were repeated 3 times to verify the reliability of the results. In addition, many unloaded bridge readings were collected during the test to be able to remove temperature effects from the readings.

For the dynamic tests, two identical lorries, one leaf suspended and one air suspended, were driven with or without a road bump on the eastern bridge deck. The lorries were driven with different constant speeds starting from a crawling speed of 10 km/h and up to the maximum allowable speed of 90 km/h.

Table 1. The order in which loaded and unloaded bridge tests were carried out.

Test no.	Loading type
O1	Unloaded bridge
A1	Load pattern A
O2	Unloaded bridge
A2	Load pattern A
D1	Load pattern D
O3	Unloaded bridge
D2	Load pattern D
A3	Load pattern A
D3	Load pattern D
O4	Unloaded bridge
B1	Load pattern B
C1-1	Load pattern C (1/2 of C)
C1-2	Load pattern C (1/1 of C)
O5	Unloaded bridge
C2	Load pattern C
B2	Load pattern B

Test no.	Loading type
O6	Unloaded bridge
Lunch	
O7	Unloaded bridge
B3	Load pattern B
C3	Load pattern C
O8	Unloaded bridge
E1	Load pattern E
AE1	Load pattern AE
O9	Unloaded bridge
E2	Load pattern E
AW1	Load pattern AW
O10	Unloaded bridge
E3	Load pattern E
AE2	Load pattern AE
AW2	Load pattern AW
O11	Unloaded bridge

Figure 4. Loading pattern C with 8 lorries (left photo) and loading pattern E with 6 lorries (right photo).

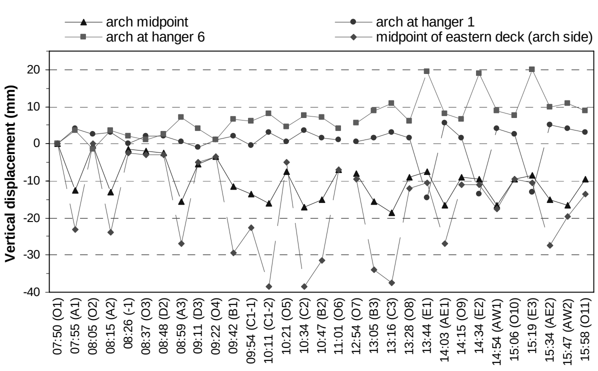

Figure 5. Arch and bridge deck vertical displacements. Hanger 1 corresponds to arch ¼-point on Swedish side and hanger 6 to arch ¼-point on Norwegian side.

Figure 5 presents example of results from the static tests. This figure shows the vertical static displacement of the arch and bridge deck. The measured displacement of the arch mid point under load pattern C is 10 mm (temperature effect removed). This is lower than what was predicted by the FE-model (14 mm) which indicates that the real bridge is much stiffer than what was assumed during design.

Looking at the unloaded readings (O1-O11) in Figure 5, the effect of increasing temperature during the test is clear. While the quarter points of the arch moved upwards, the arch mid point dropped approximately 10 mm. This strange behavior is believed to be the result of the elongation of the bridge deck which is rigidly connected to the arch.

The dynamic tests with one leaf suspended and one air suspended lorries have clearly shown the big influence different vehicle suspension systems have on bridge structures. Results show that air suspended lorries are more bridge friendly than leaf suspended ones as they cause lower dynamic effects. However, the type of vehicle suspension system has shown to be of greater importance for the forces in the hangers and the vibrations of the bridge deck than for the strains in the arch, see Karoumi & Andersson (2006) for more details.

4.2.2 Statistical data from long-term monitoring

Continuous data have been collected since bridge opening. Figure 6 shows an example of two years of registered strain in segment S1 versus the concrete temperature. The yearly variation of the strain due to temperature is clearly seen.

Such long-term statistical data is carefully studied sensor by sensor in order to identify any possible changes in bridge behavior. Thus, for this particular strain sensor at S1, a warning is given to the bridge owner if the strain is not within the range of 110 to -350 Microstrain.

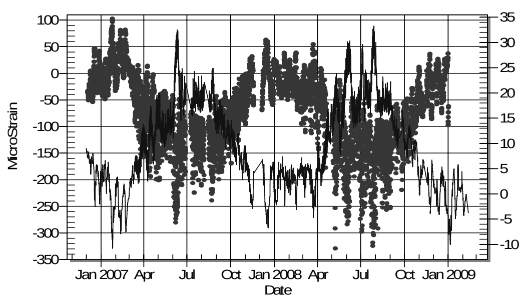

Figure 6. Temperature effect (solid line) on the strain (dotted) at top of arch segment S1.

4.2.3 Dynamic properties

The first six vibration modes (in terms of natural frequencies and mode shapes) of the completed bridge have been identified. The first, third and fourth modes are transversal with respect to the movements of the arch whereas the second mode is vertical (the arch takes the shape of its first antimetric mode) and causes longitudinal movements of the bridge deck. The theoretical mode shapes of the first two modes are shown in Figures 7.

Comparing the results from measurements with those from the FE-model used for design show very good agreement expect for the second mode. The designers FE-model gave a frequency of about 0.5 Hz, for that mode. The measurements on the other hand result in a frequency of 0.87 Hz which indicates that the bridge and the arch are much stiffer than what was assumed in design. The cause of this discrepancy is believed to be the arrangement of the bearings at grid lines 5 and 8, see Figure 1. It is believed that these supports act as fixed bearings. When these constraints were introduced in the revised FE-model, the theoretical natural frequency of the second mode was increased from 0.51 Hz to 0.84 Hz. Details regarding the dynamic properties and the influence of temperature, wind and damages on the dynamic properties of the bridge are presented in Ülker & Karoumi (2006), where it is e.g. concluded that increasing outside air temperature will lead to decreasing natural frequencies.

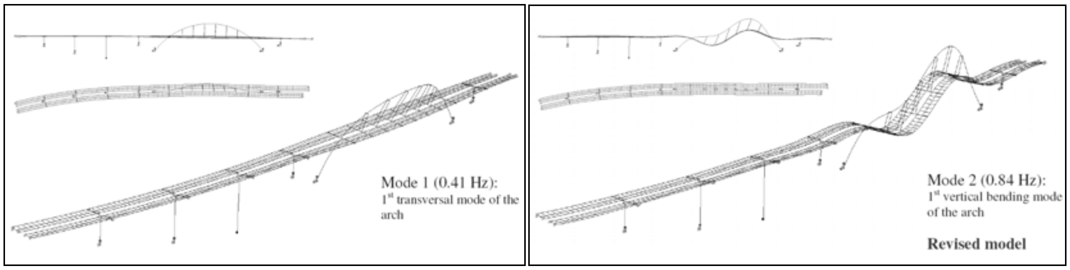

Figure 7. First and second modes of vibration.

5 CONCLUSIONS

This paper describes briefly the instrumentation used for monitoring the structural behaviour of the Svinesund Bridge and presents some illustrative examples of measured results.

On the whole, the well planned monitoring project and the load testing that took place about one month before bridge opening are a complete success. Since bridge opening in June 2005, huge amount of interesting data have been collected. The data have increased the authors understanding of the real static and dynamic behaviour of this unique structure. Measured data is also used to produce an initial database (a footprint) of the undamaged structure that can be used to identify any possible changes in bridge behavior and for future condition assessment.

Results from measurements have been compared with theoretic values from the initial FE-model that was used for the structural analysis of the bridge. The assumptions and approximations in the FE-model have been carefully looked over and improved, as measurements indicate among others that the bridge is stiffer than what was assumed during design.

Detailed results of measurements and analysis are presented in several reports from The Royal Institute of Technology (KTH) and Chalmers University (CTH). These are available on the monitoring projects homepage at www.byv.kth.se/svinesund.

6 REFERENCES

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