Static and Dynamic Load Testing of the New Svinesund Arch Bridge

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Summary

The New Svinesund Bridge at the border between Sweden and Norway has recently been opened to traffic. Due to the uniqueness of design and the importance of the bridge, an extensive long-term monitoring program was initiated. This paper briefly describes the instrumentation of the bridge and focuses on the comprehensive static and dynamic load tests that were performed just before bridge opening. Some interesting results are presented and compared with those predicted by theory.

Keywords
Monitoring, Load Test, Bridge, Vehicle, Dynamic, Dynamic Amplification Factor

1. Introduction

The New Svinesund Bridge is structurally complicated as it carries the highway over the Ide Fjord through a single concrete arch with a span length of 247 meters. The design of the bridge is a result of an international design contest. The bridge will form 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 4 years of operation, is coordinated by The Royal Institute of Technology (KTH) which is also responsible for analysis and documentation of monitored results.

Static and dynamic load tests have frequently been used before the opening of new large bridges to verify the actual structural behaviour of the bridge compared with that predicted by theory. Consequently, as part of the developed monitoring programme, comprehensive static and dynamic load tests were performed just before bridge opening in June 2005. As a whole, the primary objectives of the load tests are to better understand the bridge's response to static and dynamic loadings and to produce an initial database (a footprint) of the undamaged structure that can be used for future condition assessment.

This paper describes the load tests that were performed before bridge opening and presents some interesting results. All presented results are verified by those predicted by theory. The monitoring programme is very briefly presented in the paper however additional information can be found in the reports [JAMES et al, 2003] and [Karoumi et al, 2006b]. The dynamic properties of the bridge, both measured and theoretical, are presented in [KAROUMI et al, 2005].

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.

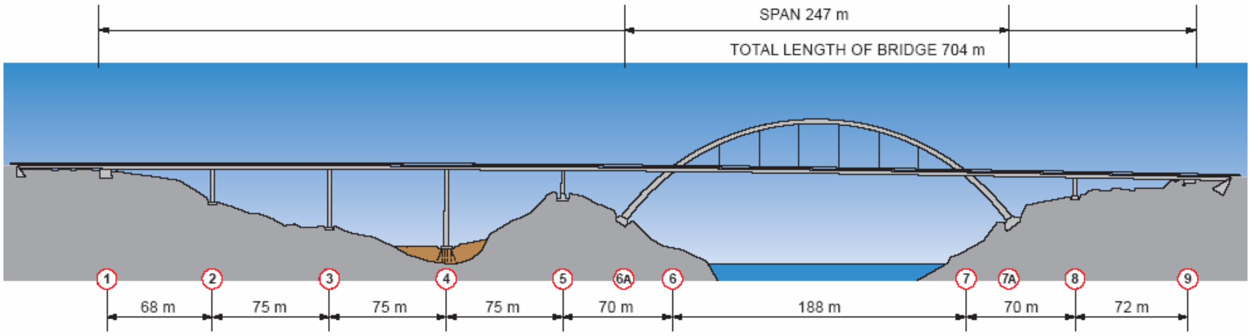

Fig.1 Sketch of the New Svinesund Bridge in its entirety, showing gridline numbering and dimensions.

Fig.2 A photograph of the bridge during construction. The temporary towers (can be seen at both arch bases) have not yet been completely removed.

The construction of the arch uses a climbing formwork and is 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. The towers were dismantled after completion of the arch. More information on the bridge structure and the construction process can be found at http://www.byv.kth.se/svinesund.

3. Description of the instrumentation and data acquisition 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 or 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 having a file name that identifies the date and time period when the data was recorded. Raw data, taken during a 10 minutes period, is permanently stored in a buffer 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:

- 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.
- 4 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, LVDTs 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. Load tests

The bridge was tested on 18th-19th of May 2005. The programme for testing and results are presented in [Karoumi et al, 2006a]. There were 3 phases to the testing. First, on May 18th the hanger forces due to dead load were measured in all 12 hangers by evaluating the natural frequencies of the hangers from acceleration measurements. Each hanger was tested at least 3 times in order to verify the reliability of the evaluated hanger forces. The excitation was achieved by hitting the hangers with a soft sledgehammer. The accelerations were collected using an accelerometer positioned at a location where the first 5 modes of vibration show theoretically a none zero contribution. This method of measuring forces in cables/hangers is described in detail in [ANDERSSON et al, 2006].

The next day, 19th of May, both phase 2 and phase 3 of the testing were carried out. Phase 2 was another static phase where eight 25-ton lorries with known dimensions and axle weights were positioned according to 7 different loading patterns. Figure 3 illustrated the 5 main loading patterns and Table 1 gives the order in which these where carried out. In this table, load pattern AE corresponds to load pattern A but with only lorry 1-4 on the eastern deck. Similarly, load pattern AW corresponds to load pattern A but with only lorry 5-8 on the western deck. The aim of these two load patterns AE and AW is to check if superposition of results is possible to obtain results for load pattern A.

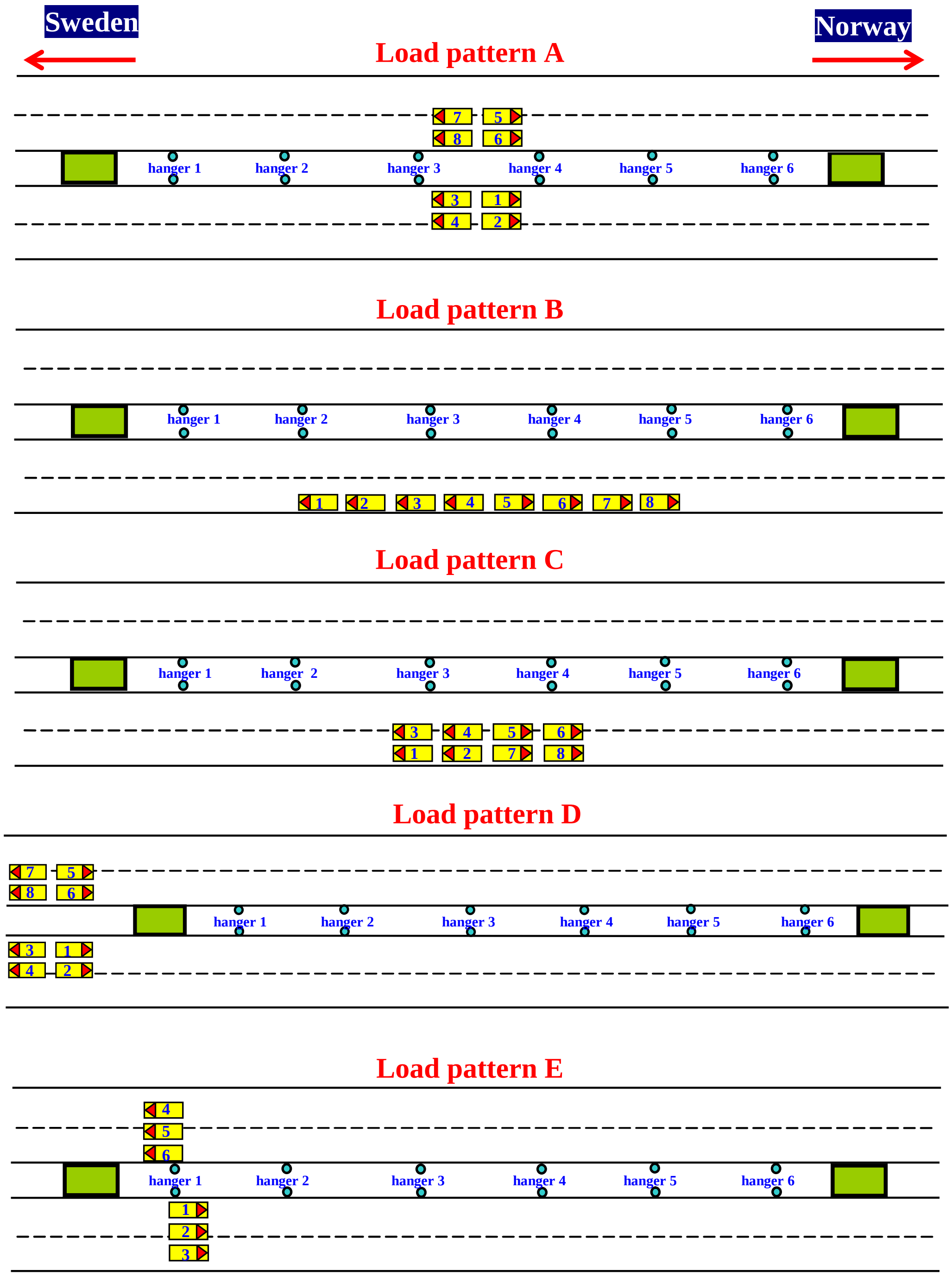

Fig.3 The 5 main loading patterns A-E showing lorry positions on the bridge. As can be seen, the lorries were positioned back-to-back in order to have a symmetric loading and for maximum load effects.

As seen in Table 1, important loading pattern 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.

Since loading pattern C is the most severe one, it was the first time applied in two steps (see Table 1) C1-1 which is half of C and C1-2 which is the full C loading. For C1-1, a check of arch mid point displacements and strains in hangers 3 and 4 was done before proceeding to full C loading.

During the test, forces in the first hangers on the Swedish side (hanger 1E and 1W) and strains at 5 locations in the arch were collected using permanently installed sensors. In addition, air and road surface temperatures, strains in hangers 3 and 4 as well as displacements at 3 point on the arch and 30 point on the bridge deck where measured for each test outlined in Table 1 below.

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
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

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

Fig.4 Loading patterns C with 8 lorries (left photo) and E with 6 lorries (right photo)

The last phase of the testing, Phase 3, was a dynamic phase. Here 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 about 90 km/h. For each run, the real speed was calculated from the measured crossing time. Braking tests were also included in this phase. Arch and bridge deck accelerations, forces in hanger 1 and strains in the arch where continuously monitored. A sampling rate of 100 Hz and a low-pass filter set at 20 % of sampling rate (cut-off 100×0.2=20 Hz) were used.

The two lorries consisted of a front axle and a bogie where the distance from the front axle to centre of the bogie was 4.6 m and the bogie distance was 1.37 m. Table 2 presents the measured axle weights from lorry 1 and 2 along with values used in the FE-model simulations (see Section 5 of this paper).

	Front axle (ton)	**Bogie (ton)**	
	Axle 1	**Axle 2**	**Axle 3**
Lorry 1 leaf suspension	7.02	9.52	8.48
Lorry 2 air suspension	6.86	11.12	6.96
FE-model constant moving forces	7.00	9.00	9.00

Table 2 Measured axle weights of the two lorries used in the dynamic test and the corresponding values used in the FE-model simulations

5. Results

In preparation for the static and dynamic load tests, theoretical predictions were achieved using the developed 3 dimensional FE-model of the bridge in *ABAQUS*, see [PLOS et al, 2004]. In the following subsections, some measured results are presented and compared with those obtained from the FE-model. For the interested reader, full test details and measured results can be found in [Karoumi et al, 2006a].

5.1 Static test

Since traffic load effects are of major interest in this study, many unloaded bridge measurements were made during the day (see Table 1) and temperature effects are removed by linear interpolation between these.

The vertical displacement of the arch and bridge deck is shown in Figure 5. 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 indicate that the real arch is stiffer than what was assumed during design. This has also been concluded earlier by the authors when the dynamic properties of the bridge were investigated, see [KAROUMI et al, 2005] and [KAROUMI et al, 2006b]. That earlier study showed that the first measured vertical bending frequency of the arch (0.89 Hz) is much higher than the corresponding theoretical value (0.42 Hz).

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 behaviour is believed to be the result of the elongation of the bridge deck which is rigidly connected to the arch.

Figure 6 and Table 3 show the strain variation at the top of arch segment S6, i.e. an arch segment just below the bridge deck on the Swedish side. Before noon, this segment is in the shadow of the bridge deck and for that reason no temperature effect on the strain is noted. Table 3 show perfect agreement between measured and theoretical results except for loading pattern E where the developed FE-model underestimate the response. The reason for this is not yet fully investigated but it is clear that some approximations in the FE-model need to be improved.

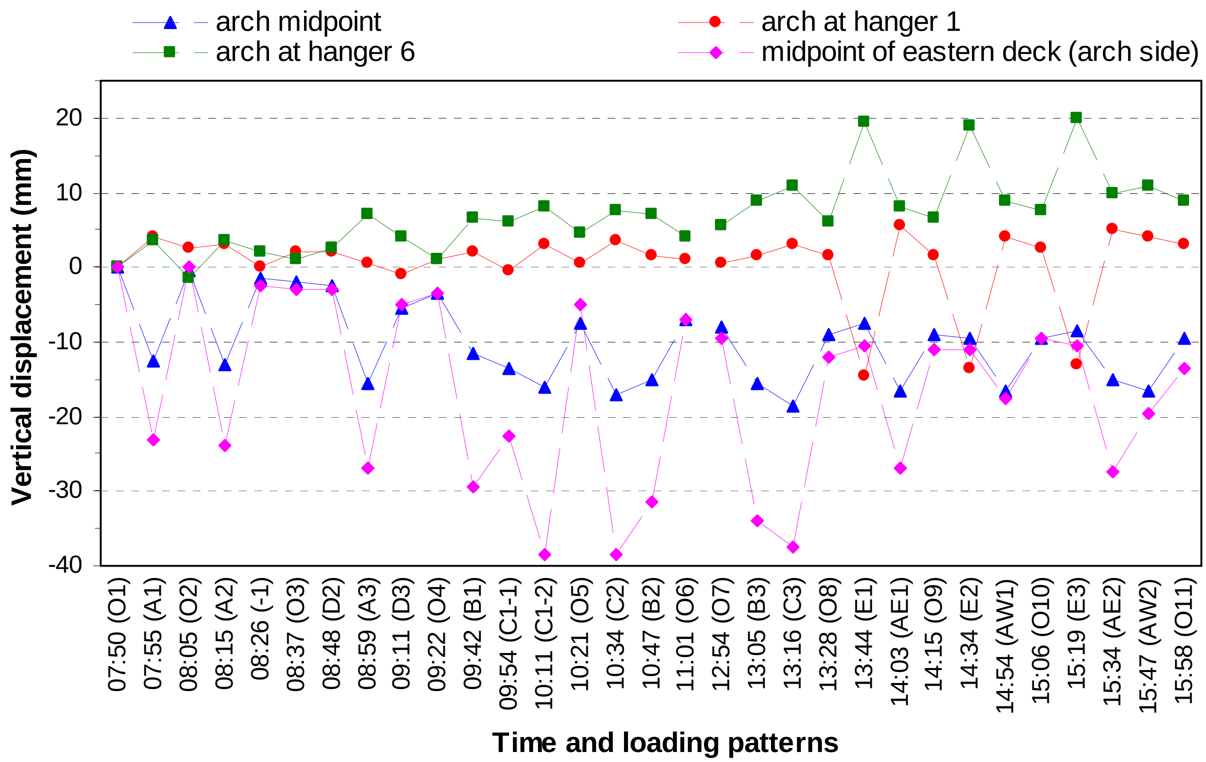

Fig.5 Arch and bridge deck vertical displacements. Hanger 1 corresponds to arch ¼-point on Swedish side and hanger 6 to arch ¼-point on Norwegian side.

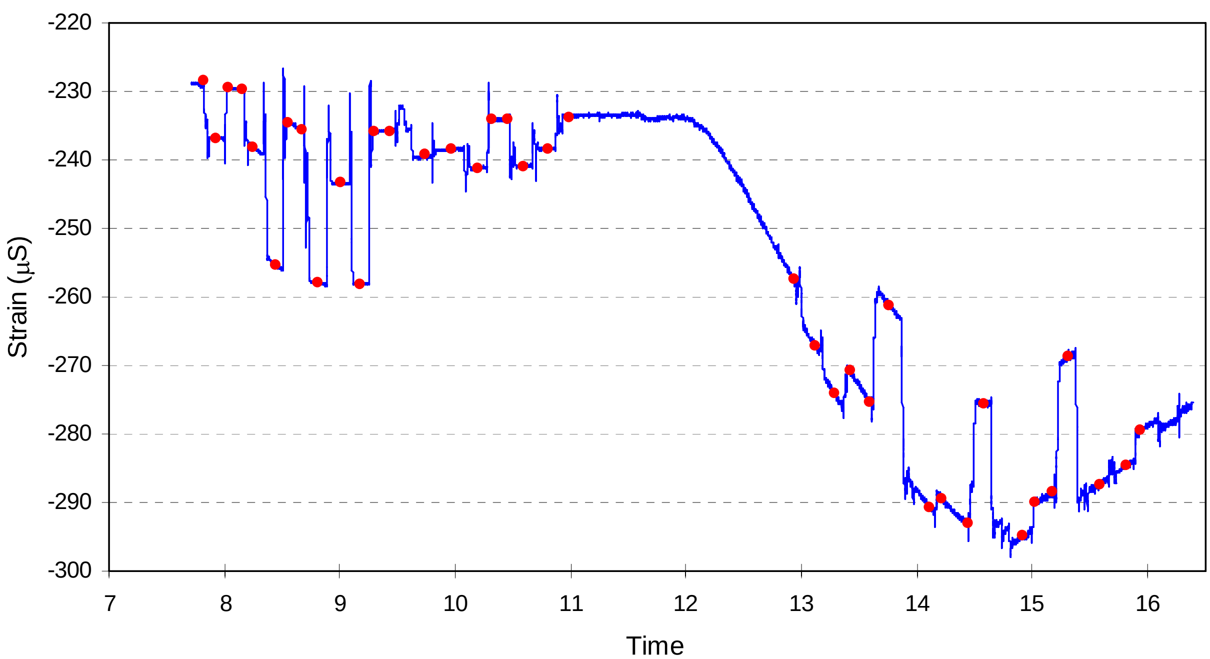

Fig.6 Strain at top of arch segment S6 which is just below the bridge deck on the Swedish side. The red dots indicate the different loaded and unloaded readings made according to Table 1.

Load case	Strain (µS)			measured / calculated
	measured (average)	measured (std)	calculated (FEM)	
A	-7,6	0,4	-7,5	**1,01**
Asuperposition (AE+AW)	-7,7		-7,5	**1,02**
B	-4,3	0,4	-4,7	0,93
C	-6,9	0,1	-6,9	1,00
D	-22,3	0,2	-21,0	**1,06**
E	17,6	0,7	12,9	**1,37**
AE	-3,7	0,1		
AW	-3,9	0,3		

Table 3 Comparison between measured and theoretical strains at the top of arch segment S6

Of course, measured values for all measurement points and load patterns not always agree with theoretical values as good as shown in Table 3 above. This is because some load patterns are not really relevant for comparison at a certain sensor location as these patterns produce low signals compared to the background signal noise etc.

5.2 Dynamic test

As mentioned before, two lorries were used for this test; lorry1 with leaf spring suspension and lorry2 with an air suspension system. Figure 7a shows the axial force in hanger 1E (1st hanger on Swedish side supporting the eastern deck) due to passage of lorry1 at different constant speeds. The signals in Figure 7 have been subjected to filtering using a 2nd order Butterworth low pass filter. The static response was obtained using results from lorry1 passing at 10 km/h and using a cut-off frequency at 0.5 Hz. The responses at 50 km/h up to 90 km/h were filtered with the cut-off frequency of 10 Hz, mainly to remove high frequency vibrations due to wind etc. The frequency spectra from the unfiltered signal show no distinct frequencies beyond this level.

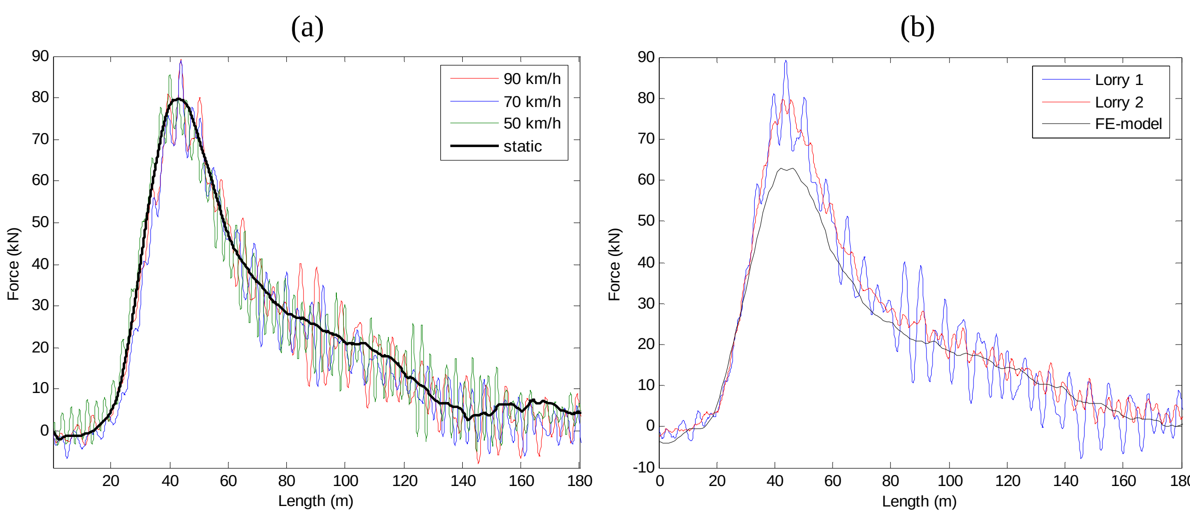

Fig.7 (a) axial force in hanger 1E during passage of lorry1 at different speeds; (b) lorry1 and lorry2 at 90 km/h compared with FE-model simulation result

As expected, Figure 7b shows that air suspended lorries are more bridge friendly than leaf suspended ones as they cause lower dynamic effects. Figure 7b shows also that almost no dynamic amplification of the hanger force is obtained from the FE-model simulations. In addition, the FE-model gives a static response of 63.5 kN, which is 20 % less than what is measured for lorry1. For the FE-simulations in *ABAQUS*, the lorries have been modelled using constant moving nodal loads without considering the dynamic properties of the lorries. This together with other approximations made in the FE-model is believed to be the reason for the differences.

Table 4 summaries the maximal measured axial force in hanger 1E at different speeds for lorry1 and lorry2. The results show that lorry1 gives a measured dynamic amplification factor of 1.12, i.e. dynamic response 12 % larger than the static. This occurs at the speeds 70 and 90 km/h. Lorry2 on the other hand, result in maximal dynamic amplification factor 1.07, occurring at 50 km/h. The measured static response is 7 % higher from lorry1 than for lorry2, even though they have approximately the same total weight (see Table 2). The reason for this may be the distribution of the load between the axles, but may also depend on the transverse position of the lorries on the deck during the passages.

Speed (km/h)	**Lorry1 (leaf)**		**Lorry2 (air)**	
	Force (kN)	**DAF**	**Force (kN)**	**DAF**
static	79.7	1.00	74.5	1.00
50	85.6	1.07	79.7	1.07
70	88.9	1.12	76.6	1.03
90	89.2	1.12	78.1	1.05

Table 4 Maximal axial force in hanger 1E during passage of Lorry 1 and Lorry 2 along with the dynamic amplification factor DAF

6. Conclusion

The static and dynamic load testing of the New Svinesund Arch Bridge is presented. On the whole, the well planned load testing that took place about one month before bridge opening was a complete success and huge amount of interesting data was collected. This data is used by the authors 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.

Analysis of the measured data is still ongoing. Nevertheless, some interesting results are presented in this paper and are verified by comparing with theoretical predictions. Preliminary results indicate that the bridge’s arch is stiffer than what was assumed during design and when developing the FE-model. Because of this and as the theoretical predictions achieved using that FE-model are in some cases in bad agreement with the measured response, the approximations in the FE-model need to be looked over and improved.

Finally, 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 strain in the arch.

Acknowledgements

The financing of this project is provided by the Swedish National Road Administration (Vägverket) and is gratefully acknowledged. The authors of this paper would also like to thank the following members of the monitoring project advisory group for their input to this work: Lars Lundh, Robert Ronnebrant and Peter Harryson (Vägverket), Ian Markey (Statens vegvesen), Thomas Darholm (FB Engineering AB) and Frank Myrvoll (NGI).

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