E. Measurements

Contents

E.1 Measurements during 2009

Figure E1.1 The bridge over Kalix River at Långforsen

E1.1 General

During October 8-9,2009 preliminary measurements were carried out by Luleå University of Technology. Material properties were determined. Six cylinders with a diameter of 95 mm were drilled out from the lower part of the arch. The results are given in Chapter 3.

Measurements were done when a locomotive moved over the bridge. Strains were studied in one of the cross beams and the curvature κ was determined in one of the main top beams. Very small strains and curvatures were observed. The maximum strain was $8 \cdot 10^{-6}$ corresponding to a concrete stress of about $\sigma = E\varepsilon = \approx 30 \cdot 10^{9} \cdot 8 \cdot 10^{-6}$ Pa = 0,24 MPa.

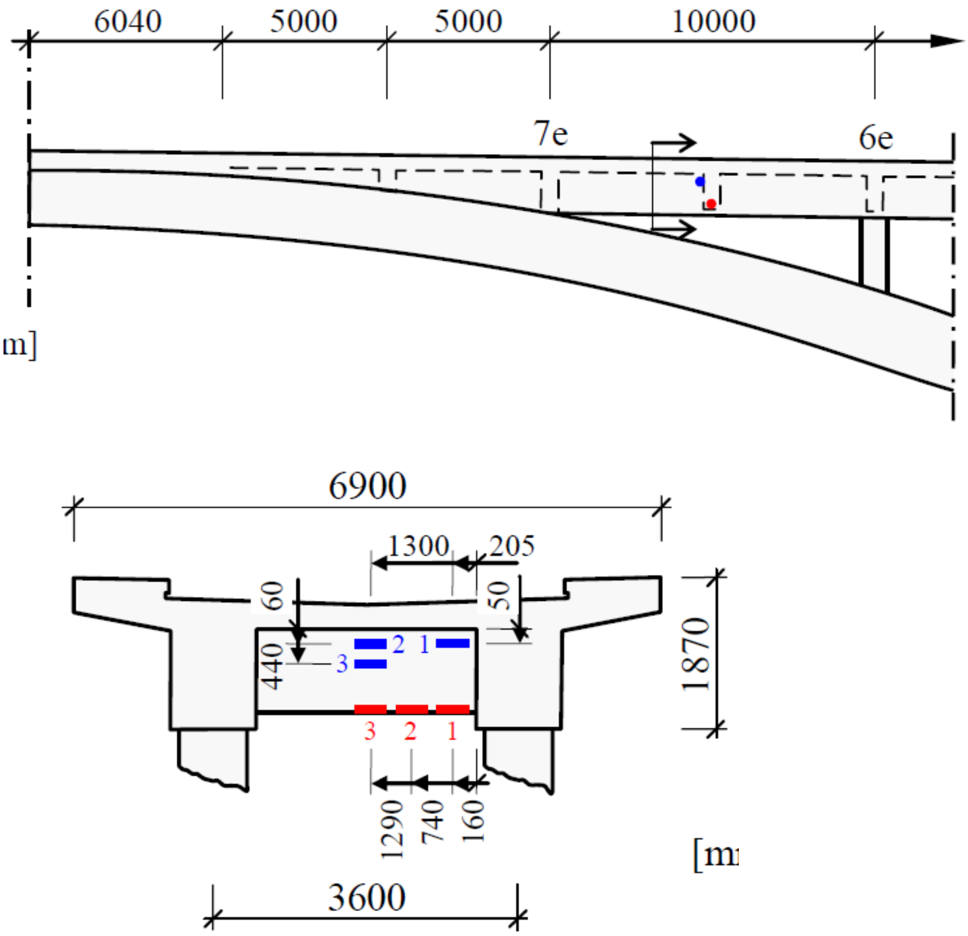


Figure E1.1 Strain gauges in a cross-beam ca 20 m from the top of the arch.
Concrete strain gauges (top, blue) and steel reinforcement strain gauges (bottom, red).
The gauges measure the strains in the longitudinal direction of the cross beam, i.e. in the transverse direction of the bridge...

E1.2 Strains

Strains according to Figure E1.1 in the concrete and in the reinforcement bars are shown in Figures E1.2 when three trains passed the bridge.

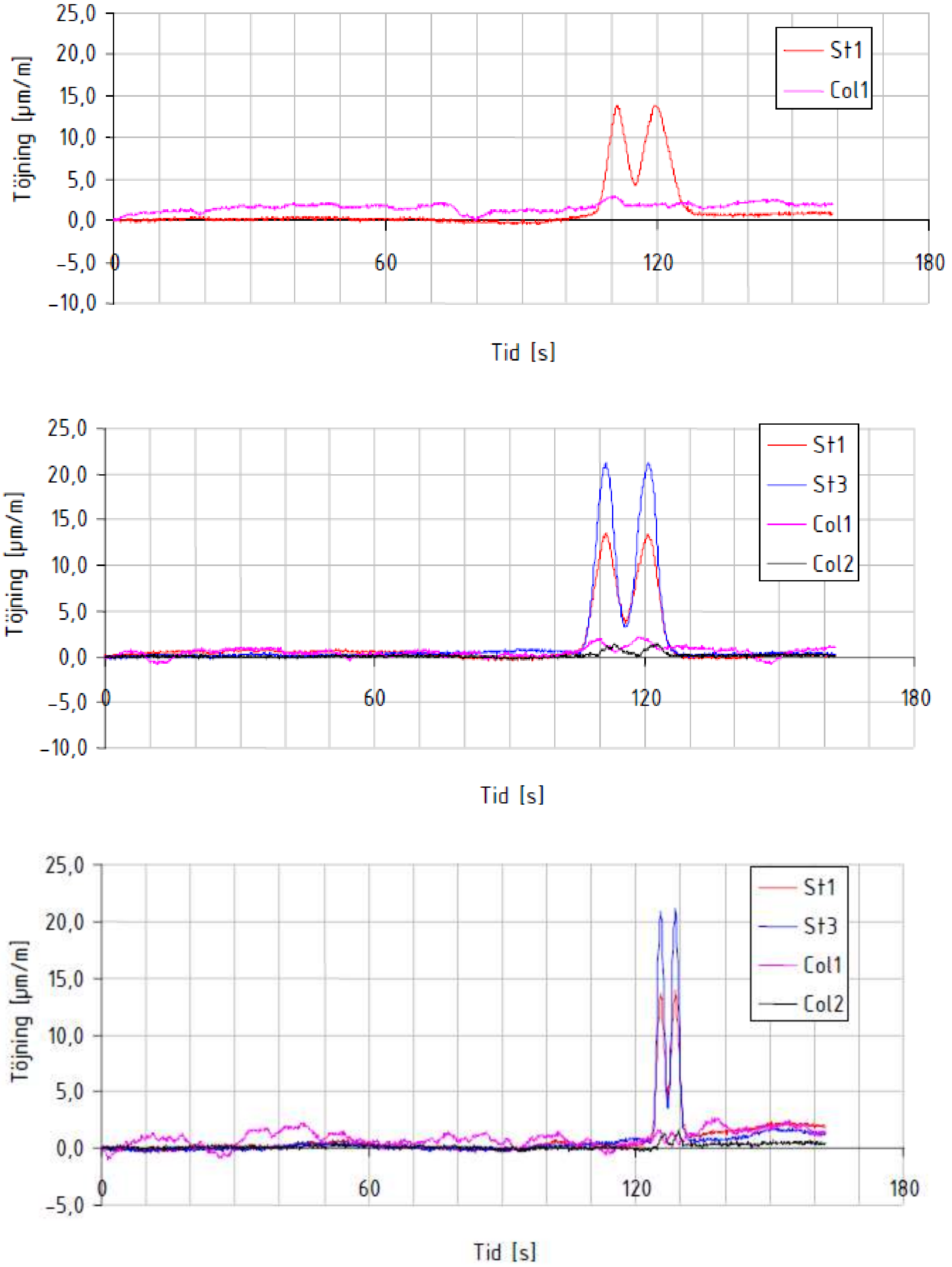


Figure E1.2. Strains in a cross-beam during passing of three train STA1-01E (top), STA2-01E (middle) and STA 3-01E (bottom). Steel strain gauges St1 and St3 and concrete strain gauges Col1 and Col 2, see Figure E1.1

The steel strains were maximally ε_s = 21 µm/m in gauge 3 in the middle of the beam. This corresponds to a steel stress $\sigma_s = E_s \cdot \varepsilon_s = 200 \cdot 10^9 \cdot 21 \cdot 10^{-6}$ Pa = 4,2 MPa.

E1.2 Curvature and moments in top beams

The curvature κ of the top beams was determined from deflections δ_i measured by seven LVDTs. relatively to a stiff beam of length ca 10 m (6 x 1,6 m). Three positions of the stiff beam were tested, see Figure E1.3.

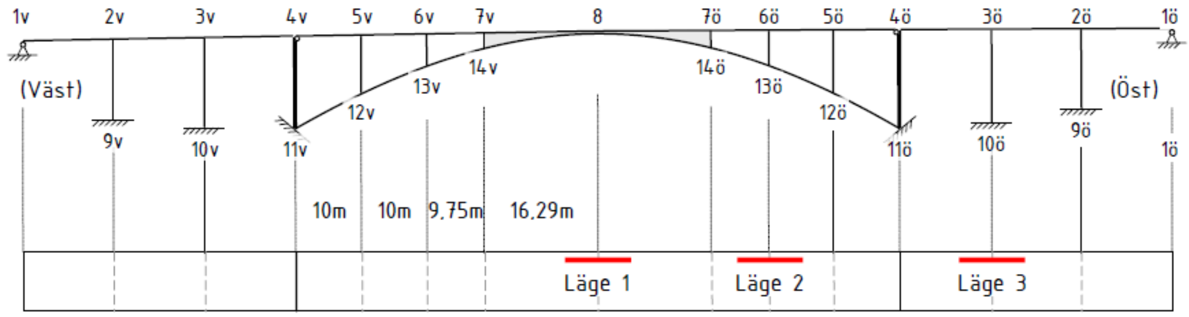
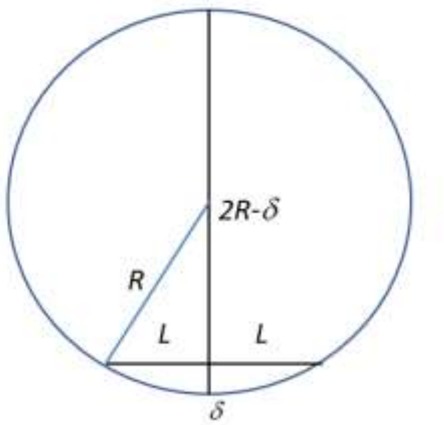


Figure E1.3. Three positions (Läge 1-3) for measurements of deflections to be used to calculate curvatures of the top beams.

The radius of the curvature R = $1/\kappa$ can be determined from the chord theorem, see Figure E1.4.



According to the chord theorem we get for a small deflection δ, a radius of curvature R and a distance L between measurement points that

$$\delta(2R - \delta) = L^2$$

$$R = L^2/2\delta + \delta/2$$

For $\delta << R$ the last term may be neglected

Figure E1.4. Radius R of curvature according to the chord theorem

A series of measurements are given in Figure E1.4.

From them it can be seen, that a typical max difference in deflection between LVDTs No 4 and Nos 3 and 5 is $\delta = 0{,}03$ mm, which with L = 1,6 m gives $R = L^2/2\delta = 1{,}6^2/(2 \cdot 0{,}00003)$ m = 42,7 km and the curvature $\kappa = 1/R = 23{,}4 \cdot 10^{-6}$ 1/m. The corresponding moment M can be calculated from $M = EI/R$. With E = 40 GPa and the moment of inertia I = 5,172 m^4 from Leander-Fredriksson (2003), p. 3.5 (or I = 5,66 m^4 from the original design, p. B188) we obtain $EI = 40 \cdot 10^9 \cdot 5{,}2$ Nm^2 = 208 GNm^2. This gives $M = EI/R$ = 208 / 42,7 MNm = 4,9 MNm.

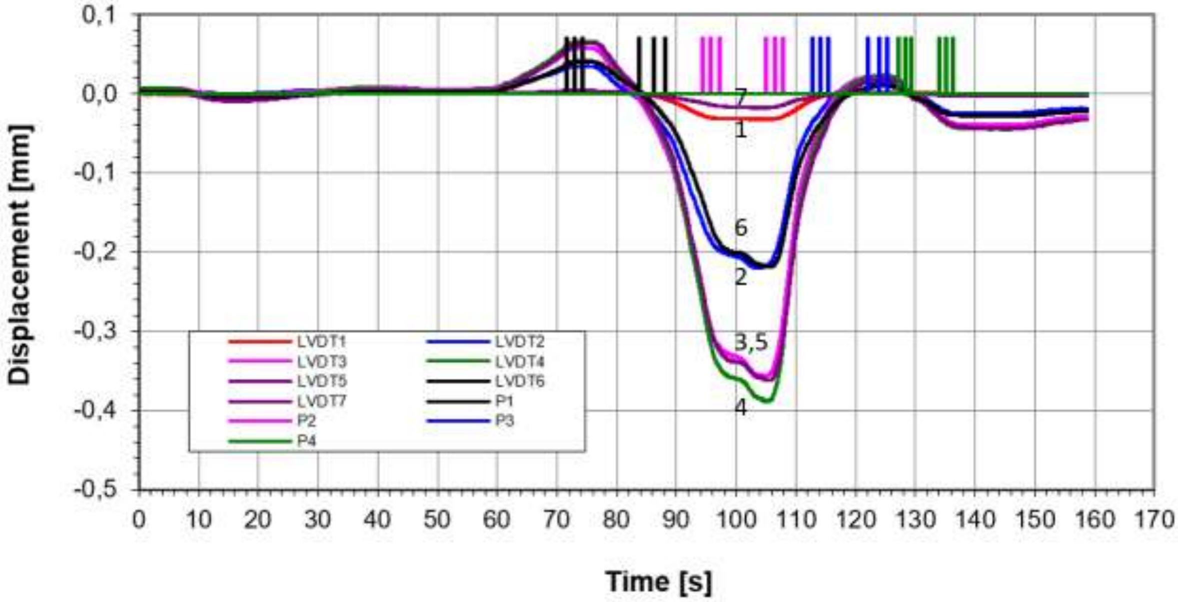


Figure E1.4. Displacements measured by seven LVDTs marked 1 – 7 in Position 1 in the centre of the arch while a train with an axle load of 210 kN is passing at a slow speed of ca 1m/sec (= 3,6 km/h). Position indicators P1-P2 show when the locomotive passes.

From Figure E1.4 the relative deflection δ_i in relation to the deflections on both sides Δ_{i-1} and Δ_{i+1} is $\delta_i = \Delta_i - (\Delta_{i-1} + \Delta_{i+1})/2$ with which the moments can be calculated in Figure E1.5.

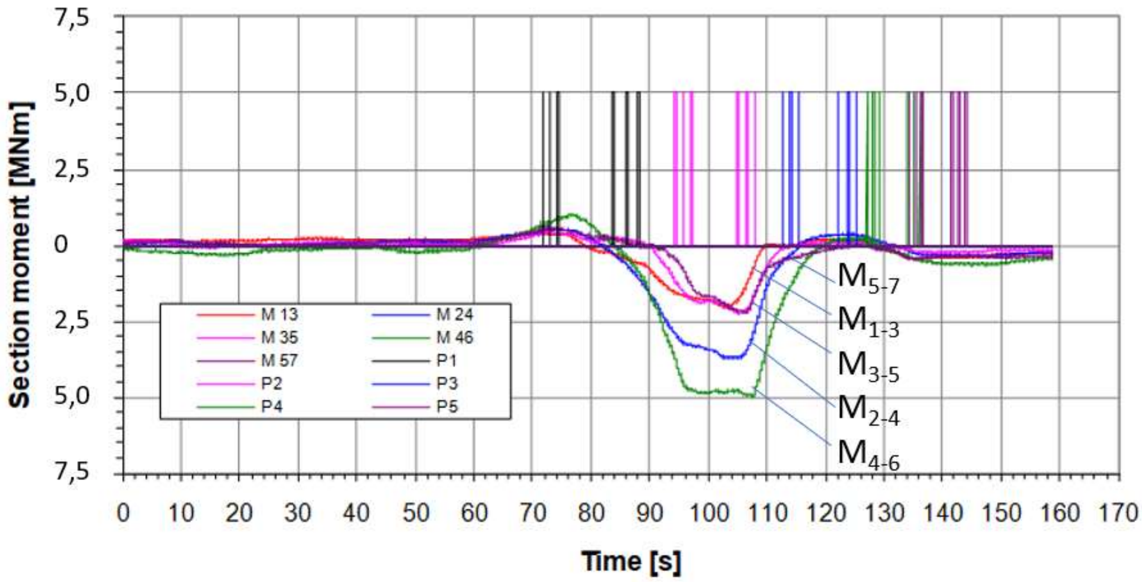


Figure E1.5. Bending moments based on the curvatures of the top beams as measured by seven LVDTs at position 1 at the top of the arch. Moment M_{1-3} indicates the moment based on the measurements in points 1, 2 and 3 and M_{2-4} is based on the points 2, 3 and 4 etc.

As a comparison, the design moment for the traffic load of a train with an axle load of 250 kN was 8 MNm, see Figure 2.5. This is of the same order as the moments in Figure E1.5. In the assessment by Leander-Fredriksson (2003) for a train load BV-2000 with an axle load of 330 kN the bending moment was ca 9 MNm, see Table D3.2

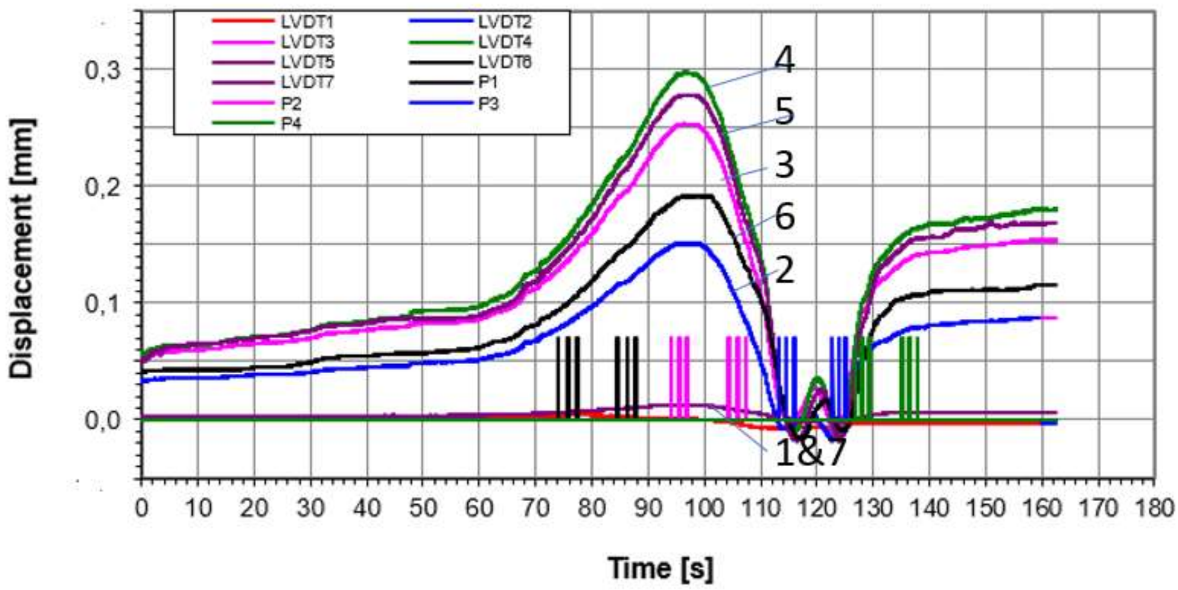


Figure E1.6. Displacements measured by seven LVDTs marked 1 – 7 in Position 2 in the centre of the arch while a train with an axle load of 210 kN is passing at a slow speed of ca 1m/sec (= 3,6 km/h). Position indicators P1-P2 show when the locomotive passes.

For position 2 on top of a column on the arch, see Figure -E1.3 measured deflections are given in Figure E1.6. From the deflections it can be seen, that a typical max difference in deflection between LVDTs No 4 and Nos 3 and 5 is $\delta = 0{,}30 - (0{,}25 + 0{,}28)/2 = 0{,}035$ mm, which with $L = 1{,}6$ m gives $R = L^2/2\delta = 1{,}6^2 /(2\cdot 0{,}000035)$ m = 36,6 km and the curvature $\kappa = -1/R = -27{,}3\cdot 10^{-6}$ 1/m. The corresponding moment M can be calculated from $M = - EI/R$. With $E = 40$ GPa and the moment of inertia of the top slab and beams $I = 1{,}43$ m^4 from Leander-Fredriksson (2003), p. 3.7 (or $I = 1{,}506$ m^4 from the original design, p. B8) we obtain $EI = 40\cdot 10^9 \cdot 1{,}45$ Nm^2 = 58 GNm^2. This gives $M = -EI/R = -58 / 36{,}6$ MNm = -1,6 MNm.

As a comparison, the original design moment for the traffic load of a train with an axle load of 250 kN was calculated to -3,5 MNm on top of the column next to the arch top (p B228). This is of the same order of magnitude as the moment estimated from the measurements.

For position 3, somewhat larger deflections (0,84 mm), curvatures and moments were obtained, see Figure E1.7. According to the original design, the maximum moments due to a train with a axle load of 250 kN were -3,74 MNm (tension in top) and 0,55 MNm (tension in bottom), page B293.

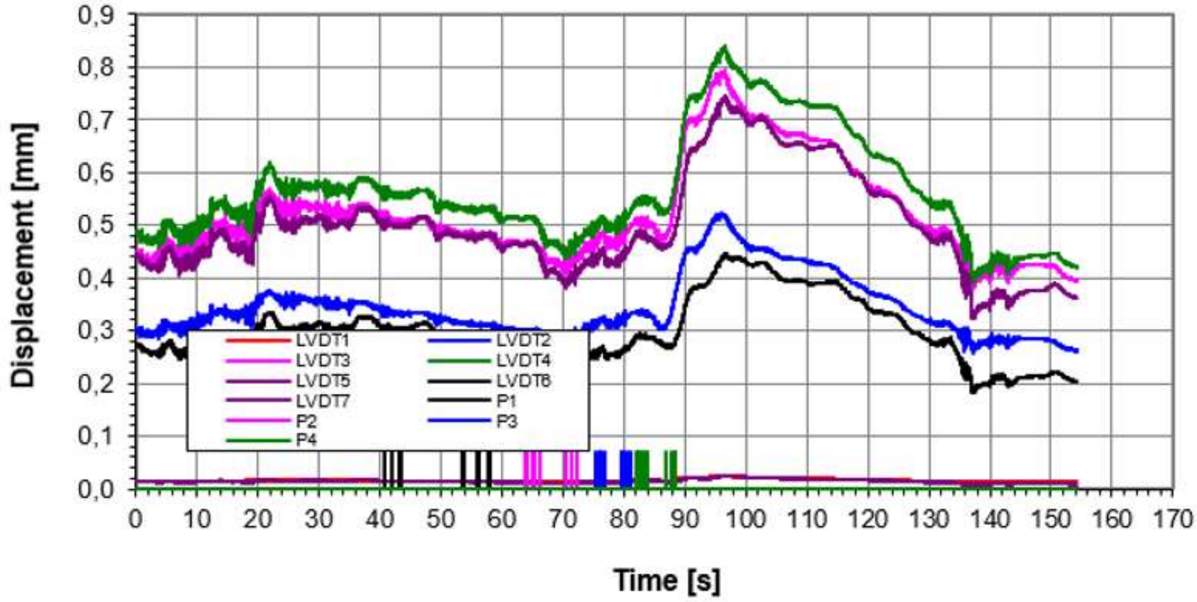


Figure E1.7. Displacements measured by seven LVDTs marked 1 – 7 in Position 3 in the centre of the arch while a train with an axle load of 210 kN is passing at a slow speed of ca 1m/sec (= 3,6 km/h). Position indicators P1-P2 show when the locomotive passes.

E.2 Measurements during 2011

E.2.1 General

In 2011 more systematic measurements were carried out during measurements on September 5^{th} - 8^{th}. In Figure E2.1 deformation measurement points (top) and acceleration measurement points (bottom) are given. In Figure E2.2 location of strain gauges for concrete and steel are indicated. More information about the measurements is given in the following sections and in Appendix F.

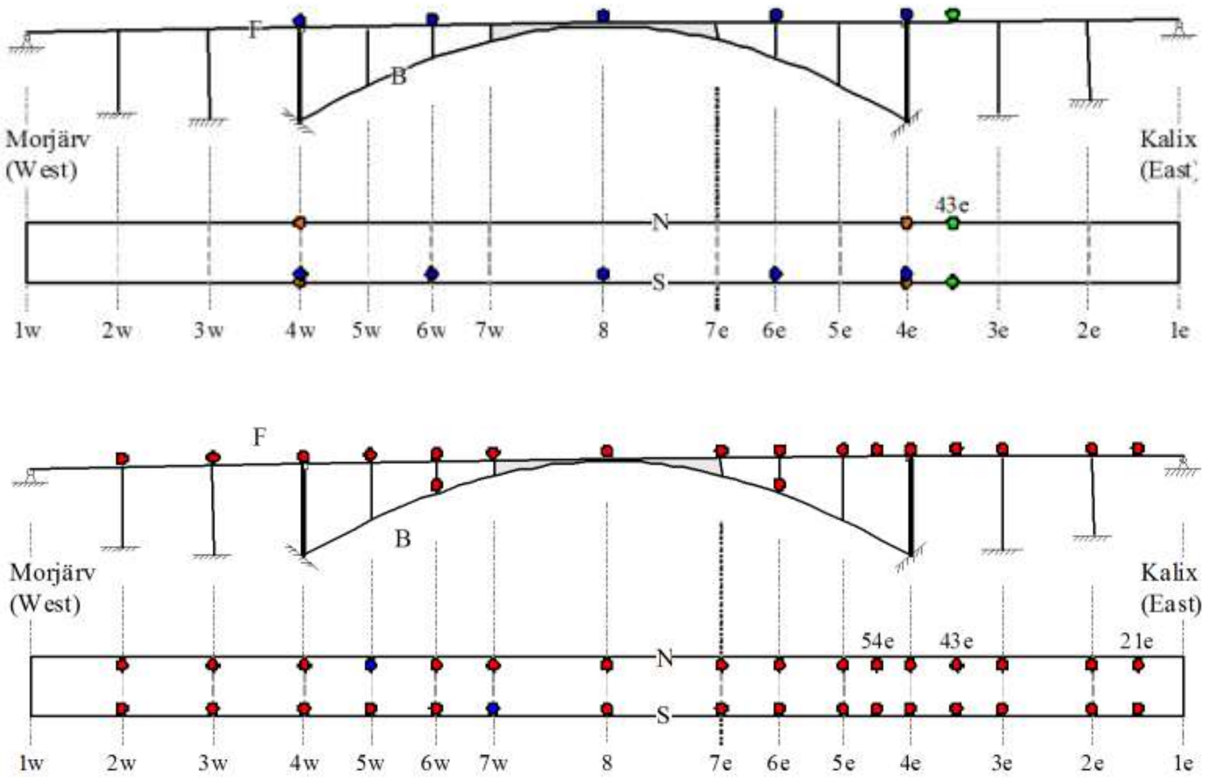


Figure E2.1. Measurements of deformations (top) and accelerations (bottom). In the top figure blue dots indicate optical measurements, green dots indicate vertical LVDTs (between 3e and 4e) and orange dots indicate horizontal LVDTs (4w & 4e). In the bottom the locations of the accelerometers are indicated

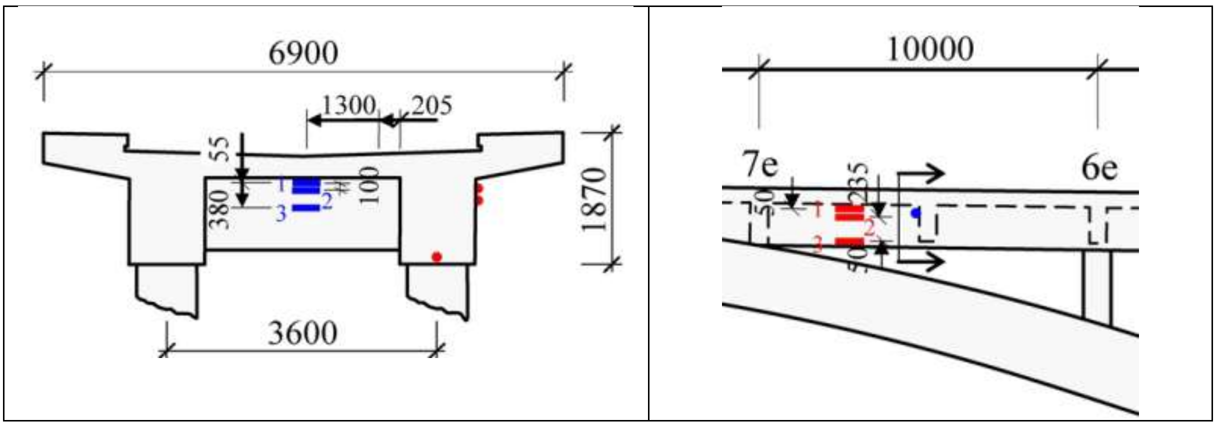


Figure E2.2. Measurements of strains. Blue gauges 1, 2 and 3 measure concrete strains in the transverse direction (CT). Red gauges 1 and 2 measure concrete strains in the longitudinal direction (CL) while red gauge 3 measures steel strains (SL) the bottom of one of the longitudinal top beams.

E2.2 Calibration and mounting of accelerometers

The calibration of the accelerometers is presented in Grip - Sabourova (2011) and in Forsberg et al (2013). General presentations of the measurements are also given in Grip (2013) and Grip et al (2016). The accelerometers were attached to the bridge and then aligned with the horizontal plane with three screws, see Figure E2.3. A sampling rate of 1200 Hz was used in most setups,

Figure E2.3. The accelerometers were firmly attached to the bridge and then aligned with a horizontal plane with three screws. The photos also show a train used for measurements

The length T of the time series was chosen as proposed in Brincker-Ventura (2012), $T > 20/(2cf_{\mathrm{min}})$, with c being the damping ratio and f_{min} the lowest natural frequency. With c = 0,15 % , according to UIC 776-2R (2009), and f_{min} = 1,75 Hz we obtained

$T > 20/(2cf_{\mathrm{min}}) = 20/(2\cdot 0{,}015\cdot 1{,}75) = 381$ s = 6 min 21 sec

E2.3 Measurements

Seven setups were used for accelerometer measurements, see Figures E2.4 and E2.5. In the setups 1 -4, the accelerometers were moved along the top beams on the arch starting from East. In the setups 5 and 6 the accelerometers were mounted on the columns on the arch and in the last setup, No 7, they were mounted on the top beams on the East side span..

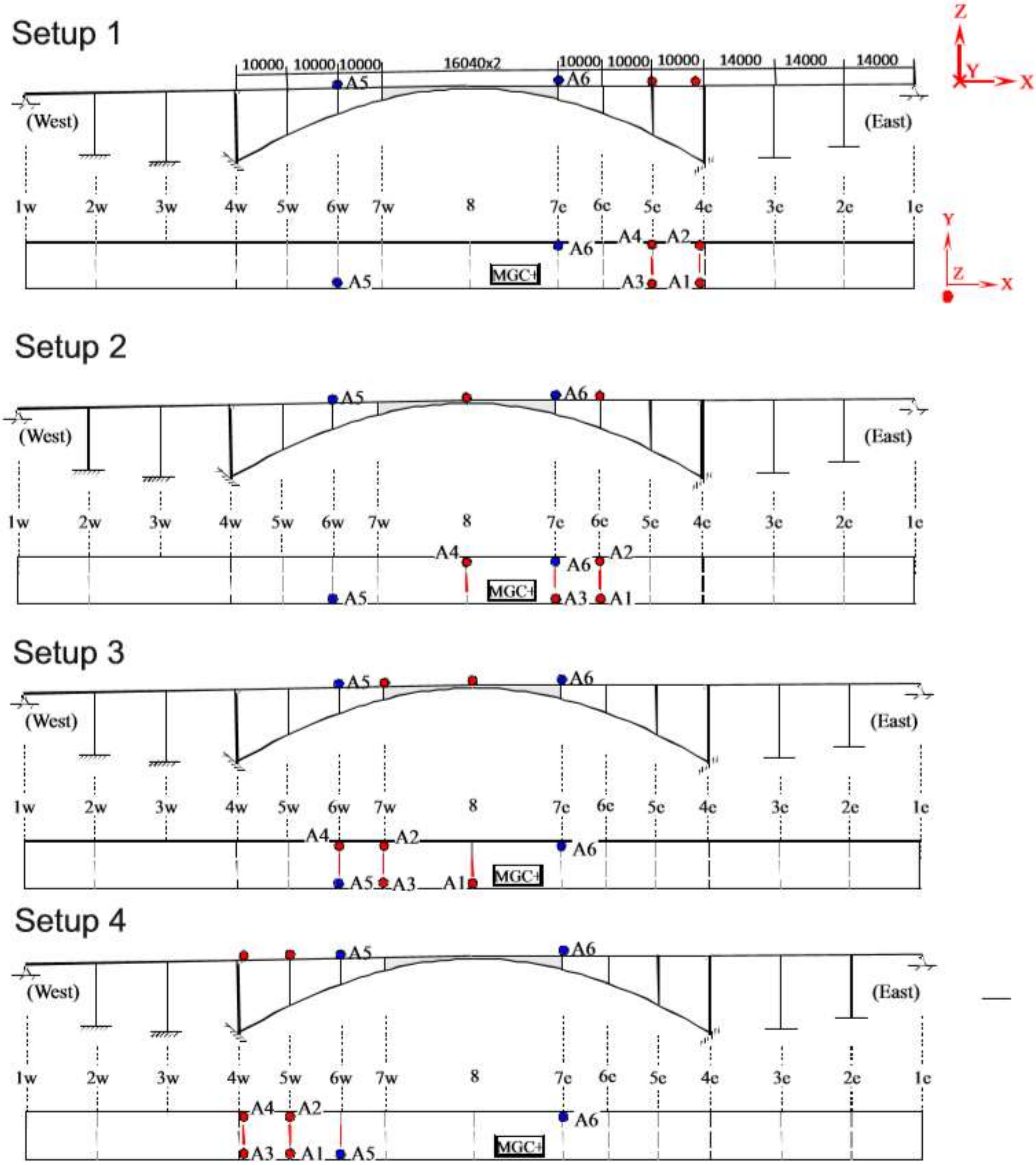


Figure E2.4. Test setups 1-4 used for acceleration measurements

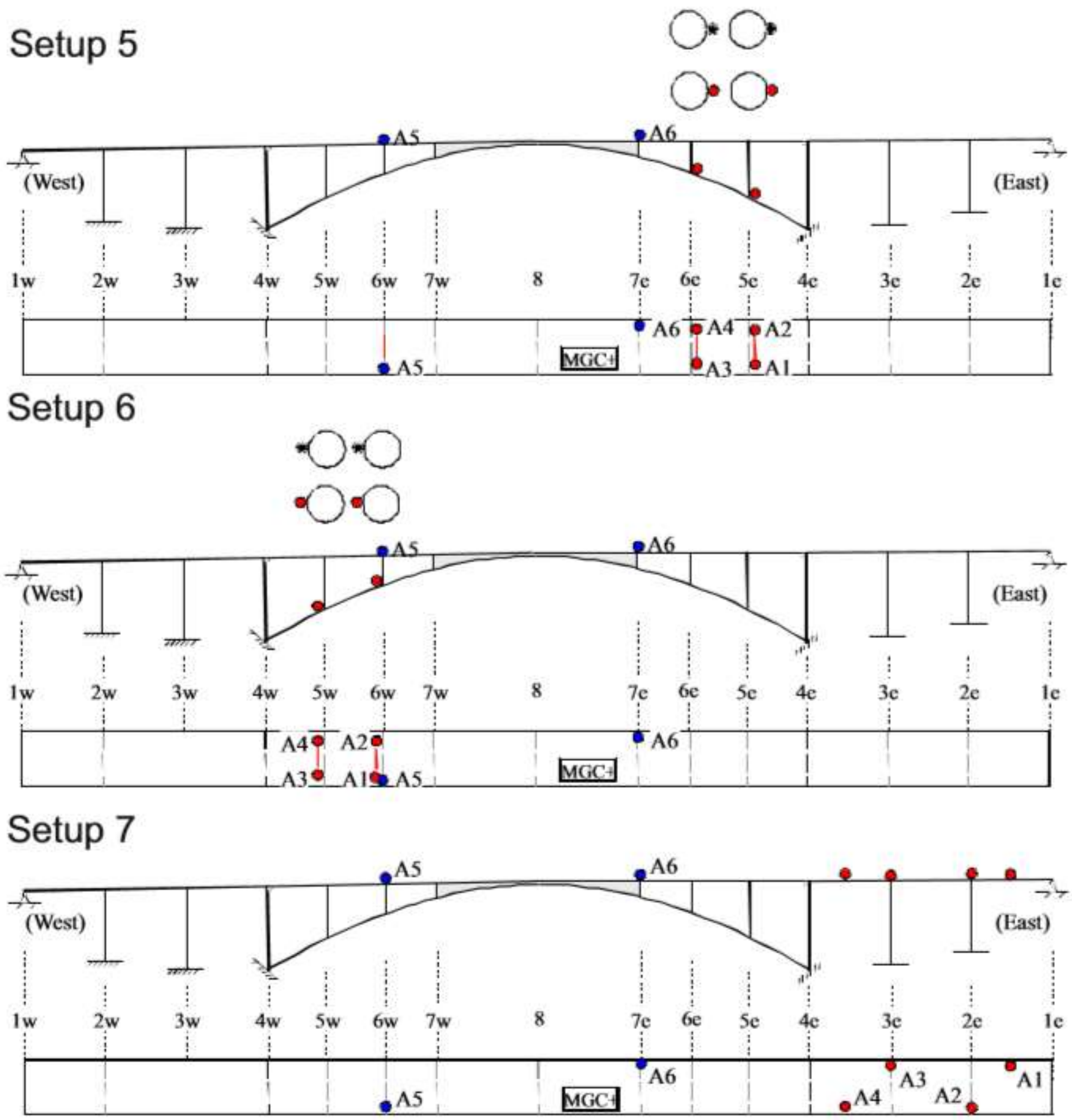


Figure E2.4. Test setups 5-7 used for acceleration measurements

It was not clear if there was enough wind for excitation of ambient vibrations strong for modal analysis. Therefore, a T43 ra 240 railway engine was driven over the bridge at speeds between 35 and 63 km/h before each measurement. The engine weight is 72 tonnes (dynamic weight 79 tonnes), distributed on 8 wheels. The measurements began after the engine passing the bridge to have the same linear system (bridge only) as in our FE models and to exclude nonlinear effects, caused, for instance by noises from the wheels clattering against the rails and bridge endpoints clattering against the foundation. There was a striking decrease of wind during the measurement days, which gradually decreased the signal-to-noise ratio (SNR). The SNR was, however, still good enough for clearly better correspondence between modeled and measured modal data (mode shapes and frequencies than in previous measurements from 2009.

Triaxial Colibrys SF 3000L accelerometers were firmly attached to the bridge with expander bolts and connected with six wire twisted pair cables to an MGCplus data acquisition system with ML801B amplifier module. All measurements were done with 1200 Hz sampling rate. The measurements were calibrated with the six-parameter method described in Grip-Sabourova (2011). Table E2.1.1 summarizes some data for the measurements done. Modal analysis was performed in the software ARTeMIS 4.0 using the principal components Stochastic Subspace Identification (SSI) method.

Table E2.1 Summary of the measurements done on 7-8 September 2011**. Bold red color** indicates the measurements that were used for the modal analysis

Name	**Duration**	**Wind**	**Start**	**Train**	**Notes**
	minutes	m/s	h:m	km/h	
S1M0a	20	7,8-9,1	11:09		Acc 6 not attached to bridge
S1M0b	40	5-8,6	11:09		Acc 6 not attached to bridge
S1M1	**25**		**14:43**	**50-60**	
S1M2a	5		15:11	5	Accidentally interrupted after 5 minutes
S1M2b	2	1-4	15:18	5	
S2M1	25		15:55	54	
S2M1b	25		18:23	8	Light train el/tele, bonus
S3M2	25		18:35	20	
S3M1	**25**	**2-3**	**18:44**	**58**	
S4M2	2	2	19:35	30	
S4M1a	30	2	19:44	49	400 Hz
S4M1b	25	3,1	20:07	29	Engine + 13 carriages
S5M1a	25	2,7	10:45	58	Engine + 2 carriages
S5M1b	**14,8**		**11:11**		**14,8 extra minutes**
S6M1	**25**		**13:18**	**53**	
S6M2	2,7	2,4	13:44	15	400 Hz
S6M3	1,7		13:48	40	400 Hz
S6M4	4,2		13:52	8	400 Hz
S2M3	**25**		**14:48**	**49**	
S1M3	25		15:37	55	
S7M1	25		16:31	52	
S7M2	50		17:08	41	

The results are presented in Appendix F