Assessment and condition monitoring of a concrete railway bridge in Kiruna, Sweden

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ABSTRACT: A two-span railway concrete trough bridge over Luossajokk in Kiruna in northern Sweden has been studied. The owner wanted to increase the axle loads from 250 to 300 kN in order to reduce freight costs for iron ore. Examples are given of methods used and results obtained from the assessment where bending, shear and fatigue were studied. Material properties, loads and load carrying capacity were evaluated using deterministic and probabilistic methods. It was shown that the bridge could carry the higher loads with a safety index $\beta > 4.7$ for reasonable assumptions of the load distributions. A measurement system was installed to check the actual level of critical strains and the worst positions of the train. Results are also given from a condition monitoring program 2001–2006, launched to periodically check the development of strains with time.

1 INTRODUCTION

Assessment of a bridge usually starts with a calculation according to a code especially adopted for assessment, see e.g. BV Bärighet (2005). If needed, the calculation is updated with tests and evaluation of the material properties (e.g. concrete and steel), see e.g. Enochsson et al (2004). A stepwise refined calculation model can be applied, often in accordance with results from condition monitoring of the structure. In some cases a probabilistic method is used to evaluate the safety index of the structure. Finally one reaches an allowable load carrying capacity. If the results are not good enough, the bridge can be strengthened, replaced with a new bridge or allowed to be used for a reduced load. In special cases, an inspection or a monitoring program can be launched to continuously or periodically check certain critical sections and parameters.

In this paper, examples are given of methods used and results obtained from a bridge assessment in Sweden. Bending, shear and fatigue are studied for a two-span railway concrete trough bridge. Material properties, loads and load carrying capacity are evaluated using deterministic and probabilistic methods, Enochsson et al (2002). Results are also presented from a condition monitoring program used to check the actual level of critical strain, the worst positions of the train and the development of strains with time, Enochsson et al (2003, 2006).

The bridge is situated on the Iron Ore Line “Malmbanan” in northern Sweden and passes over Luossajokk in Kiruna. The railway line is mainly used for transportation of iron ore from northern Sweden to Narvik and Luleå on the costs of Norway and Sweden, respectively. Here the owner wanted to increase the axle loads from 25 to 30 tons to reduce the iron ore transportation costs, Paulson & Töyrä (1996).

A recalculation according to the design code, BV Bärighet (2000) showed that the increased axle load would exceed the yield limit in the reinforcement. Before any decision was taken regarding strengthening or replacing of the bridge an assessment with probabilistic methods was carried out.

2 GEOMETRY, MATERIAL PROPERTIES AND LOADS

An outline of the bridge is shown in Figure 1. The bridge is a two-span reinforced trough bridge that was built in 1965. The mid-foundation is a concrete wall from the same period, whereas the end-foundations are stone walls that were constructed when the line was built in around 1890. A first assessment of the capacity showed that there were three sections where the capacity was too small: (1) in the top of the short span the bending capacity was too low in the longitudinal direction, (2) in the bottom of the short span the bending capacity was too low in the transverse direction, and (3) close to the mid support the shear transfer was insufficient between the beams and slab. It was decided to carry out strain measurements in these sections to check the real influence of the loads, see Figure 1.

The traffic load that causes the largest moment in the top of the short span is shown in Figures 2 and 6. The effect of dynamic load was calculated according to the Swedish Code to give a 24% increase. Contributions to the moment were also obtained from uneven temperature, break and acceleration forces. Three structural models were used, A, B and C, having increasing accuracy, see Figure 3. For one structural model (2:C3g) the following contributions were given to the top bending moment in the short span: *S* = –153.4 (Dead load) –115.9 (Ballast) –795.6 (Traffic load) –190.9 (Dynamic increase of traffic load) –419.8 (Breaking force) +17.7 (Earth pressure) = –1657.5 kNm.

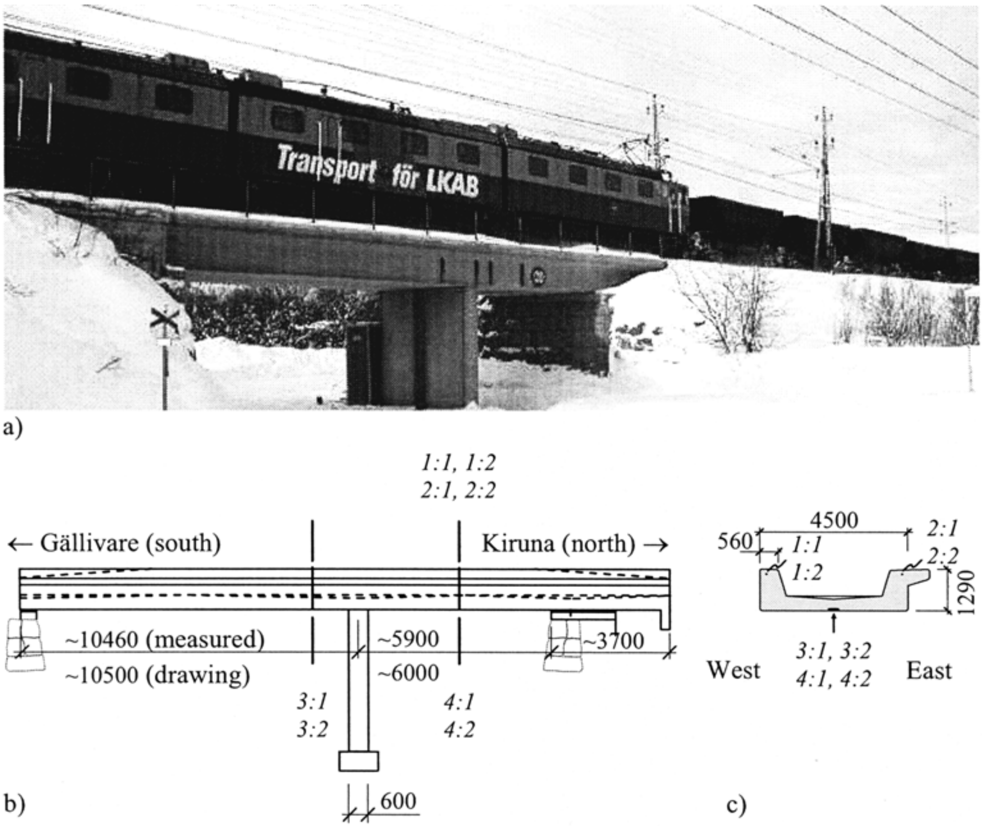


Figure 1. Bridge over Luossajokk in Kiruna in northern Sweden; a) Older type of iron ore train (DM3 25 tons) heading south towards Gällivare, b) elevation and c) cross section with location of strain gauges.

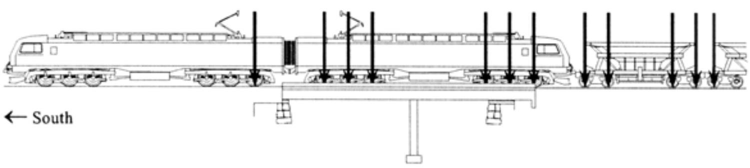


Figure 2. Position of train giving the maximum load effect in top of the short span. Arrows indicate the contributing wheel sets.

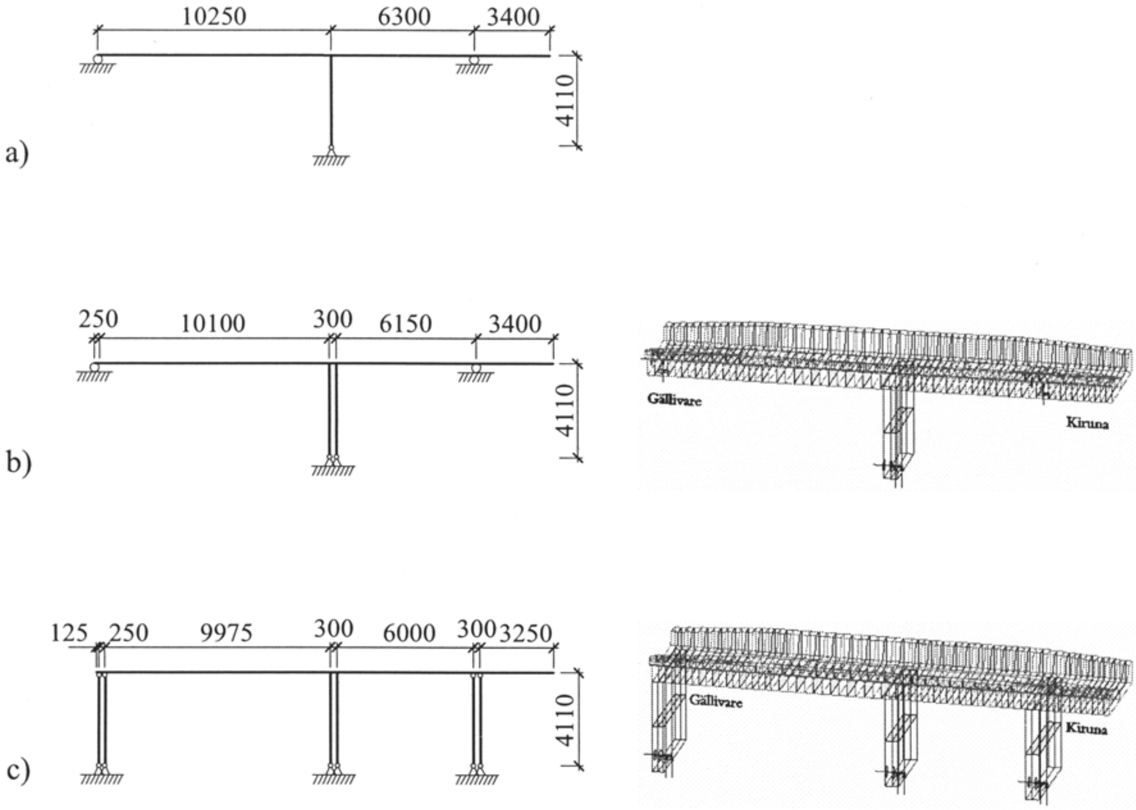


Figure 3. Principal structural system and beam models used in the FE analysis A – C.

3 CAPACITY

3.1 *General*

The bridge was built with a required concrete strength of 40 MPa. Tests of drilled out concrete cores gave a characteristic strength of f_c = 49.5 MPa. The concrete strength in Swedish Bridges is discussed by Thun (2001) and Nilsson (1999). The reinforcement in the top consisted of 11 Ø25 with an area of A_s = 5400 mm^2 and with a yield strength of f_s = 460 MPa. The critical bending resistance R can be calculated from these values, using a calculated internal lever arm z = 1163 mm, to

$$R = A_s f_s z = 2889 \text{ kNm}$$

This can be compared to the original capacity calculated according to the code which gave R_0 = 1988 kNm.

3.2 *Safety*

The safety can be expressed with a function $G = R - S$ which shall be > 0. Using the values in section 2 we obtain the mean value of G as m_G = 2.89 – 1.66 = 1.23 MNm. If all variables are normally distributed the safety index β can be calculated as

$$\beta = m_G / s_G$$

Here m_G and s_G are the mean value and the standard deviation respectively of the function G. For railway bridges in Sweden the factor β shall be larger than 4.75. This corresponds to one failure in one out of one million bridges per year. To obtain β = 4.75 the standard deviation s_G should then be less than m_G/β = 1.23/4.75 = 0.26 MNm.

3.3 *Probabilistic evaluation*

A probabilistic evaluation of the safety was made according to a First Order Reliability Method, FORM, using the program VaP, see Schneider (1997) and VaP (1999). The assumptions regarding types of distributions, mean values, *m*, and coefficients of variations, $v = s/m$, were chosen using Diamantides (2001) and JCSS PMC (2001). The chosen assumptions are given in Table 1. The geometric dimensions are measured values, so they are deterministic. The load of each ore wagon is weighted and adjusted before the train leaves the station so this load can also be regarded to be deterministic. However, the moment from breaking and acceleration forces is complicated. Here, in one calculation, a beta distribution was chosen with a mean value of zero and a standard deviation s = 209.4 kNm, which is half of the nominal design value. The end values of the function were chosen as double the nominal design value. Also other choices were tested, see Figure 4, Enochsson et al (2002).

Table 1. Values used in the probabilistic evaluation of safety.

Variable		Distribution	Mean value, *m*	Coefficient of variation, *v*
Steel area, A_s	[mm]	N	5400	0.002
Steel strength, f_s	[MPa]	N	460	0.065
Concrete strength, f_c	[MPa]	LN	62.4	0.085
Width, *b*	[m]	D	1.400	–
Depth, *d*	[m]	D	1.178	–
M of dead load	[kNm]	N	153.4	0.04
M of ballast	[kNm]	N	115.9	0.05
M of traffic	[kNm]	D	795.6	–
M of dynamics	[kNm]	N	190.9	0.25
M of horizontal force	[kNm]	B	0	0.50
M of earth pressure	[kNm]	N	17.7	0.50

D = Deterministic value, N = Normal distribution,
LN = Lognormal distribution and B = Beta distribution

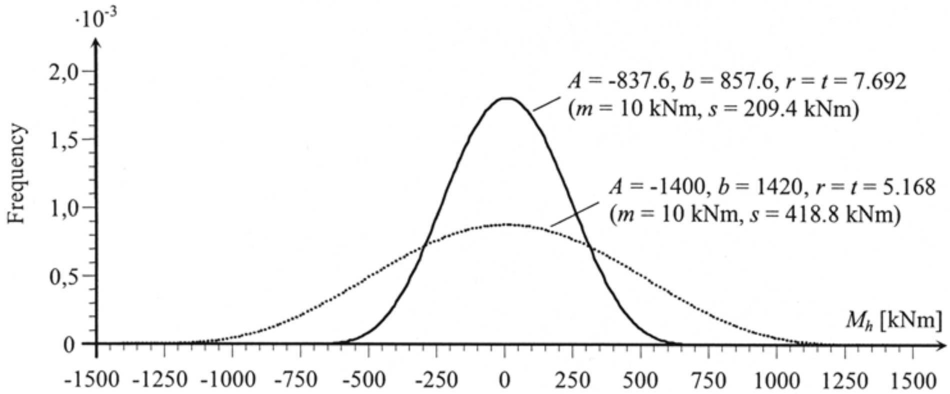


Figure 4. Assumed distribution function for bending moment due to horizontal forces by breaks or acceleration. The flat distribution has a standard deviation s = 418.8 kNm and the pointed distribution has s = 209.4 kNm. The values correspond to the nominal design value and the half of it, respectively. The frequency $f(x) = \Gamma(r + t)(x - a)^{r-1}(b - x)^{t-1}/\Gamma(r)\,\Gamma(t)(b - a)^{r+t-1}$ for $a \le x \le b$, $a \ne b$ and $r, t > 1$.

Using the values in Table 1 the probabilistic analysis with VaP gave a mean value of the safety function m_G = 1.61 MNm and a safety index β = 6.03. The variables that have the greatest influence of the level of safety are the steel strength f_s, the horizontal forces and the dynamic amplification factor. This can be seen from sensitivity coefficients α ($-1 < \alpha < 1$). The variable is more sensitive the bigger the absolute value is for α. The three variables above had α-values of 0.79, 0.55 and 0.21, respectively.

A few other load combinations and load distributions were also tested. The results are summarized in Figure 5.

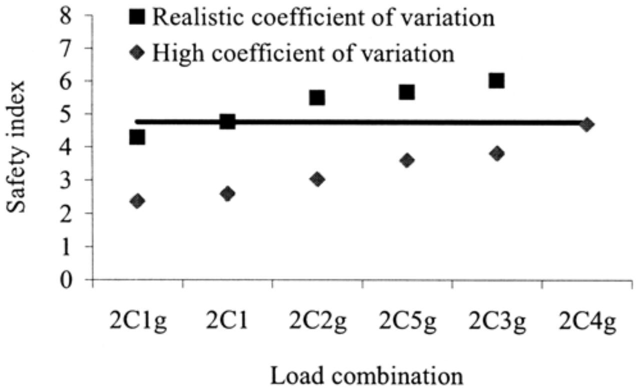


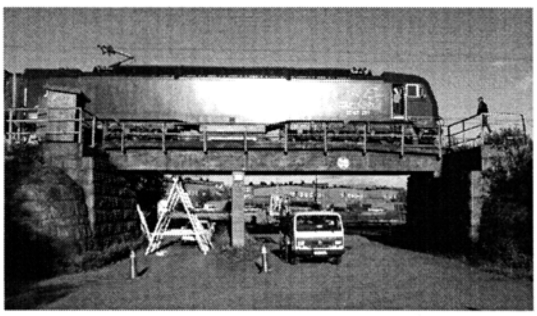
Figure 5. Safety factor β for different load combinations. In the lower curve, large standard deviations have been assumed for the dynamic load factor, the horizontal load and the temperature load, whereas in the upper curve, more realistic values have been assumed. The horizontal line indicates the demanded value β = 4.75 in the Swedish code.

4 MEASUREMENTS

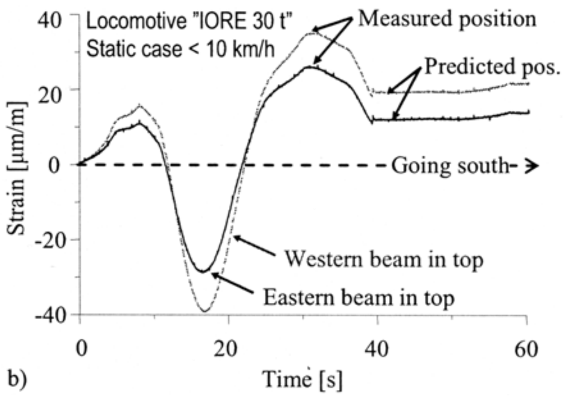
4.1 *Short term monitoring*

The strains have been measured in the bridge since 2001. The maximally recorded peak-to-peak strains are of the order of 75 microstrain that corresponds to about 15 MPa, which is quite a small value. The highest recorded tension in top of the short mid span in 42 microstrain that corresponds to about 8.4 MPa of the steel reinforcement (dynamic part). This is much lower than the design value of the yield strength with 283 MPa. However, the corresponding measured tensile stress in the concrete is 1.4 MPa. This is lower than the characteristic tensile strength of 2.1 MPa, but higher than the corresponding design value of 1.17 MPa. The stress level indicates that some cracks could have been initiated in the section, which has also been observed in top of the western edge beam.

When the response and the worst position of the traffic load were checked, see Figure 6a, the measured position was found to differ from the evaluated position with only 1 dm, see Figure 6b. The worst position here refers to upwards bending in the short mid span. Examples of typical results from the measurements are given in Figure 7.



a)



b)

Figure 6. a) Locomotive "IORE" 30 tons, heading southward, placed in the worst position evaluated from the static calculation, and b) strain in the upper edge of the western and eastern edge beam when a locomotive is slowly driving towards the worst position found from the calculation. When the train first enters the console there is tension in top of the edge beam. When the train continues out on the short span there is compression in the top, followed by tension again when the first bogie enters the long span, as depicted in figure a). The model had a slightly too long span length which gives a slight deviation of measured and predicted position for the worst load case.

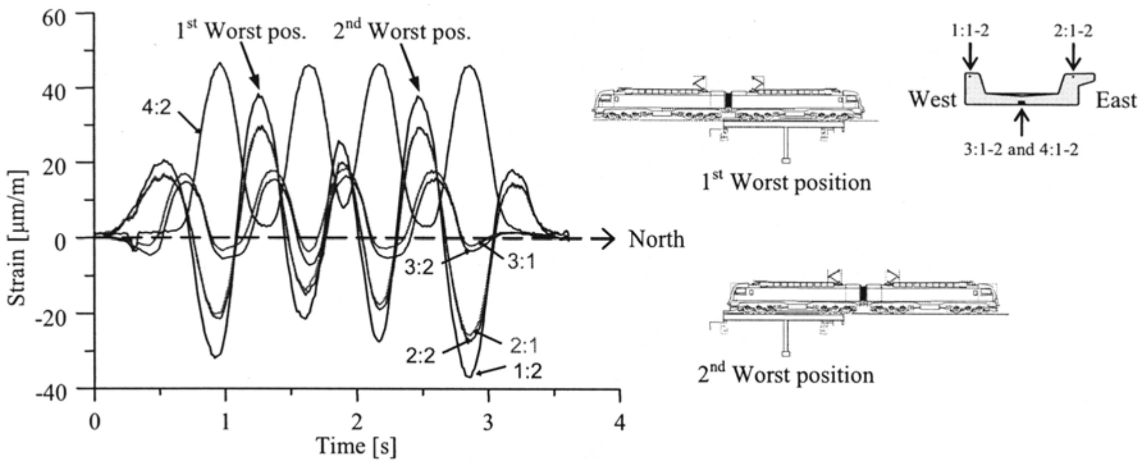


Figure 7. Typical results of strain measurements when locomotive "IORE" 30 tons is driving towards Kiruna town (north). The worst positions refer to upwards bending in the middle of the short span. High tensile strains are also observed in gage 4:2 (in the transverse direction in the slab) when any group of axles has its position in middle of the short span.

4.2 *Long term monitoring*

The strain development is periodically controlled since 2001 and onwards. Figure 8a shows the effect on the level of strain when the ballast is frozen and unfrozen. The ratio of the phenomena is calculated in Table 2.

The effect of the axle configuration for the old type and the new type of locomotive (DM3 and IORE) is demonstrated in Figure 8b, and the ratio is calculated in Table 3.

Figure 9a shows that the bridge behaves linearly and continues to do so in Figure 9b. The ratio of the strain levels shown in the two figures can be compared in Table 3.

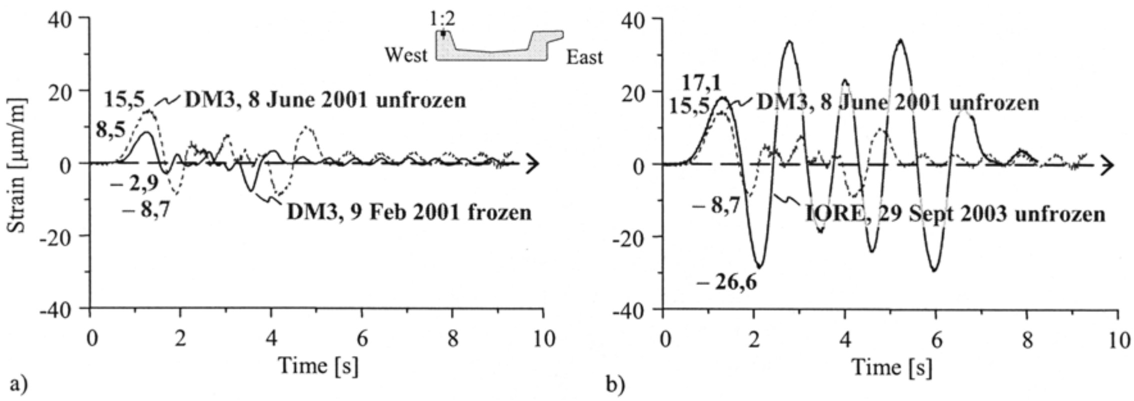


Figure 8. a) Strain levels when the ballast is frozen and unfrozen, and b) strain levels for old type of locomotive (DM3 25 tons) and the new type (IORE 25 tons).

Table 2. Comparison of strain levels between frozen and unfrozen ballast.

Action	Frozen μm/m	Unfrozen μm/m	Ratio of frozen-to-unfrozen
Tension	8.5	15.5	0.55
Compression	–2.9	–8.7	0.33

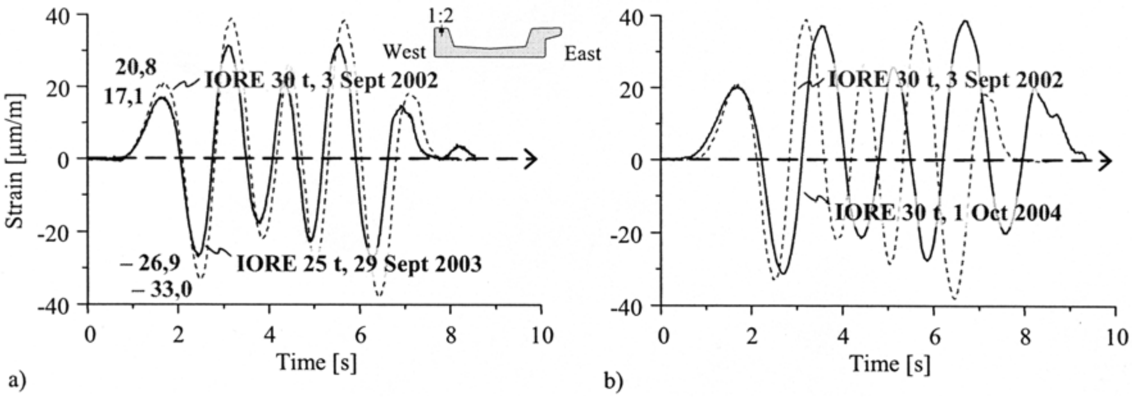


Figure 9. a) Strain levels for IORE 25 tons and 30 tons, and b) strain development with time for IORE 30 tons.

Table 3. Comparison of strain levels between DM3 25 t, IORE 25 t and IORE 30 t.

Action	DM3 25 t µm/m	IORE 25 t µm/m	IORE 30 t µm/m	Ratio of IORE (25 t)-to-DM3 (25 t)	Ratio of IORE (30 t)-to-(25 t)
Tension	15.5	17.1	20.8	1.10	1.22

5 CONCLUSIONS

The assessment of the load-carrying capacity of the bridge has shown that the bridge can carry the increased loads without strengthening. There are many assumptions that could be worth to investigate further e.g. the influence of train velocity (dynamic load factor), break and acceleration loads and uneven temperature, see Enochsson (2002). A structural health monitoring system has been installed and the bridge has been successfully monitored from Luleå University of Technology during the period 2001–2006, see e.g. Hejll (2004), Enochsson et al (2003, 2006).

The result shows that the bridge behaves linearly for an increase of the axle load from 25 to 30 tons and that there is a very small time dependent influence. However, there is quite a big variation during the year depending on the influence of increased stiffness of frozen ballast during the winter period.

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