

FULLSCALE TESTING AND ASSESSMENT OF A STEEL STRUSS BRIDGE

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ABSTRACT

A large amount of resources has been invested in building and maintaining existing infrastructure. Several of these structures are now becoming old and do not meet the requirements of today and/or are reaching the end of their technical lifespan. It is not possible to replace all of these structures, that are deemed or are about to be deemed obsolete, due to high cost and environmental impacts. An approach to keep these structures in use for a longer time is through an innovative and intelligent assessment of the actual state of stress and behavior. In such cases, using structural health monitoring to assess the structure might be an efficient way to extend the life of the structure.

This paper describes a unique monitoring program for two similar 33 m long steel truss bridges situated in Sweden. One of these bridges, Aby River Bridge, had a regulated axle load of 25 tons and was tested to failure in 2013. The other bridge, Rautasjokk Bridge, has a regulated axle load of 30 tons which will be upgraded to 32.5 tons and will be in use for the coming years. The monitoring program was performed as; monitoring of the bridge over Aby River when it was still in service. After replacement the old bridge was moved and tested under static loads to assess boundary conditions and state of stress. Structural parts from this bridge were then disassembled and tested for material properties and fatigue capacity. A theoretical assessment of the Rautasjokk Bridge was performed based on the conclusions from the measurements on the Aby Bridge. Finally, the plan is to verify findings by performing measurements on live loading for the Rautasjokk Bridge in service limit state, to be performed during autumn 2015. The aim of this project is to verify the continuous safety for the Rautasjokk Bridge by using input from tests performed at both bridges.

KEYWORDS

Structural Health Monitoring, Steel truss bridge, Fatigue, Model updating, Assessment.

INTRODUCTION

Bridges have often been replaced on theoretical assumptions that they have reached the end of their lifespan. Beside the safety aspects, the economy is the single most important factor when it comes to exchanging bridges. In later years the environmental burden has also gained influence in becoming a concern in decision-making. The Swedish Traffic Administration has declared intentions to increase their work with Life Cycle Analysis (LCA), (Trafikverket 2012). For bridges, this will lead to that a greater amount will be assessed for their actual capacity before necessary actions are taken, whether it is repairs, upgrading or replacing the entire structure.

The assessment of an existing bridge can be performed with different levels of accuracy and effort. Generalized load-models might often be sufficient in order to verify if the load capacity is good enough or at least serve as an initial estimation. Conservative assumptions may, however, lead to an exaggerated safety level. Together with increased loads, the simple solution is often to replace the structure. The opposite, overestimating the capacity can however be catastrophic. Failure of a bridge may result in major delays and possible human casualties.

Up to 2012, the Swedish authorities own 3842 railway bridges and 145 tunnels and over 13,642 km railway tracks, (Du and Karoum 2014). A substantial part of the bridge stock in Sweden and Europe is older than 50 years, as shown in Figure 1, at the same time; loading and traffic intensity on our existing bridges are increasing as time goes by.

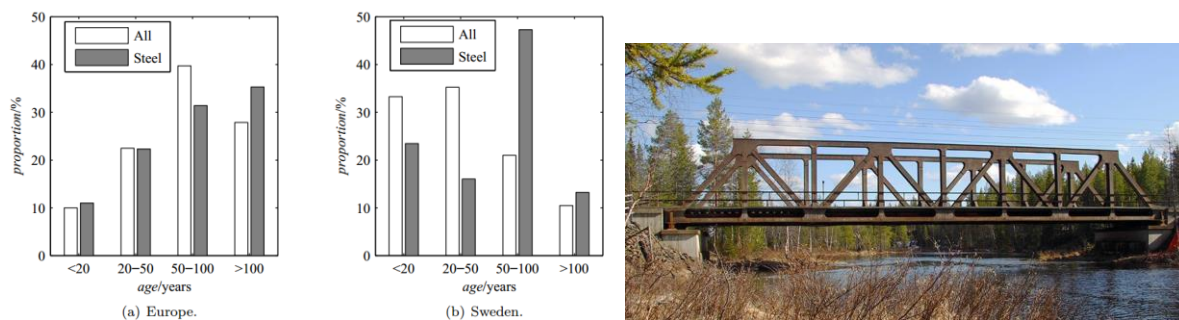


Figure 1 Left, age distribution for bridges in Europe/Sweden (Sustainable Briges 2007). Right, the Åby River Bridge in its original position.

An increase in traffic, both in regard of weight and intensity significantly reduces the lifespan of steel bridges due to Fatigue. The load models and estimation of the fatigue capacity is not as straight forward as for the ultimate limit state, which makes it more difficult verify the proper safety levels due to uncertainties.

Even if several old bridges theoretically have served their lifespan concerning fatigue or insufficient load capacity due to increased loads and increased traffic they are not necessarily in need of being exchanged. With the help of new knowledge together with refined calculations and inspections it might be possible to prolong the lifespan of these bridges. In order to ensure continued safety of the bridge it is often required to monitor the structure by performing measurements.

The bridge over Aby River is one of these bridges that theoretically had reached the end of its lifespan, when it was replaced in 2012. In order to gain knowledge of its structural behavior, the old bridge was moved to temporary supports close to its original position in order to be tested for both static and dynamic loading

The overall aim of this study was to identify critical hotspots for the Aby River-bridge and to develop a method for assessing these. Another objective is to identify measurements that characterize the structural behavior, in order to create a method for non-destructive assessment of similar bridges. The reason for the particular interest of this bridge is that there is an identical bridge over the Rautasjokk River, located on the iron ore line in the northern parts of Sweden. If the measurements from the Aby River-bridge can provide information that the bridge over Rautasjokk doesn't need to be replaced, great savings can be made.

PREVIOUS WORK

Before the Aby-River Bridge was taken out of service in the autumn of 2012, measurements were performed while it was still in use. Since the live measurements was less comprehensive than the final tests, it served as a step towards planning the full scale tests. Train loads were known and therefore it was possible to calibrate models to measured data (Blanksvärd 2012a; Blanksvärd 2012b; Moreno 2013).

Simulations of the intended load case were performed prior to the test by a Finite Element Model created in ABAQUS. The model was made as a shell model with the limitations of not assigning any constraints at the joints; therefore all connections are fully rigid. Strain hardening is not considered either.

Digital image correlation (DIC) measurements were performed at the joint between the longitudinal stringer beams and the crossbeams, in order to evaluate the degree of constraint in the connection between the stringers and the crossbeams which according to calculations were the critical detail which led to the exchange of the structure. The evaluation of these results (Elhag 2012) can be found in the master thesis by Elhag.

GEOMETRY AND MATERIAL

The Bridge consisted of a 33 meter long steel truss railway bridge that was located along the Swedish mainline. Since it was built in 1951, it was designed according to the present trainloads type F46 which corresponds to 25 tons axle load which also is the present load on the railway. The location for the bridge is in a rural environment and approximately 50 km from the coast. Girders and connections in the bridge are partially riveted and partially welded. The steel used in the superstructure is described in Table 1 with material properties according to (Trafikverket 2005). Compared with the steel materials used today the variation of material properties from the time of construction are far greater (Larsson and Lagerqvist 2009). The yield strength used in the modelling of the bridge are higher than the measured values. The modelling in this paper were done based on a priori knowledge, data from similar structures, the actual properties were tested after destruction of the bridge.

Table 1 Material properties used for analysis

Part	Material	F_{yk}	F_{uk}	F_y Used for modelling	F_y Measured	F_u Measured
Stringer beams, verticals, diagonals	S1311	240 MPa	360 MPa	345MPa	308MPa	460MPa
Main truss, Cross girders,	S1411	270 MPa	430 MPa	345MPa	333MPa	475MPa

EXPERIMENTAL WORK

Test Set Up

After the bridge was taken out of service it was transported and mounted on temporary supports close to the tracks. When in place, a vast number of sensors were mounted. The load on the bridge is induced by two jacks where the jack is attached to a girder that distributes the load to four equally distributed point loads. In order to be able to archive the force needed to load the bridge to failure the jacks were anchored to the bed rock. During the phase of drilling the intended precision was not possible which is the reason to why the loads aren't symmetrical around the center of the bridge.

Monitoring Program

This project is a unique opportunity to gain knowledge of the structural behavior of this kind of bridges. Since the full scale testing was performed under a limited amount of time which will finally resulted in failure of the bridge, the program for measurement was made as comprehensive as possible. The limitations for the test equipment were 145 channels and 141 were used during the tests. In addition to this, DIC measurements were made. The different sensors were divided between 72 strain gauges, 46 LVDT's, 8 temperature gauges and the DIC measurements (Aramis-system, from GOM). This paper will be limited to the measurements for the global analysis of the main truss. The measurements consist of 18 different predefined load series where three last ones are done with the rail removed and the last one to failure. Figure 3 show how the load was varied over the different series of measurement. The sensors placed on the main truss for the global analysis are illustrated in Figure 2 and Figure 4 and further described in Table 2.

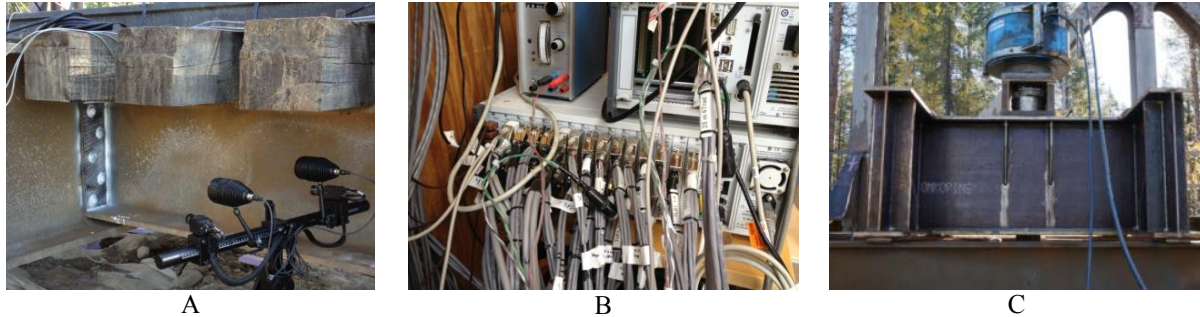
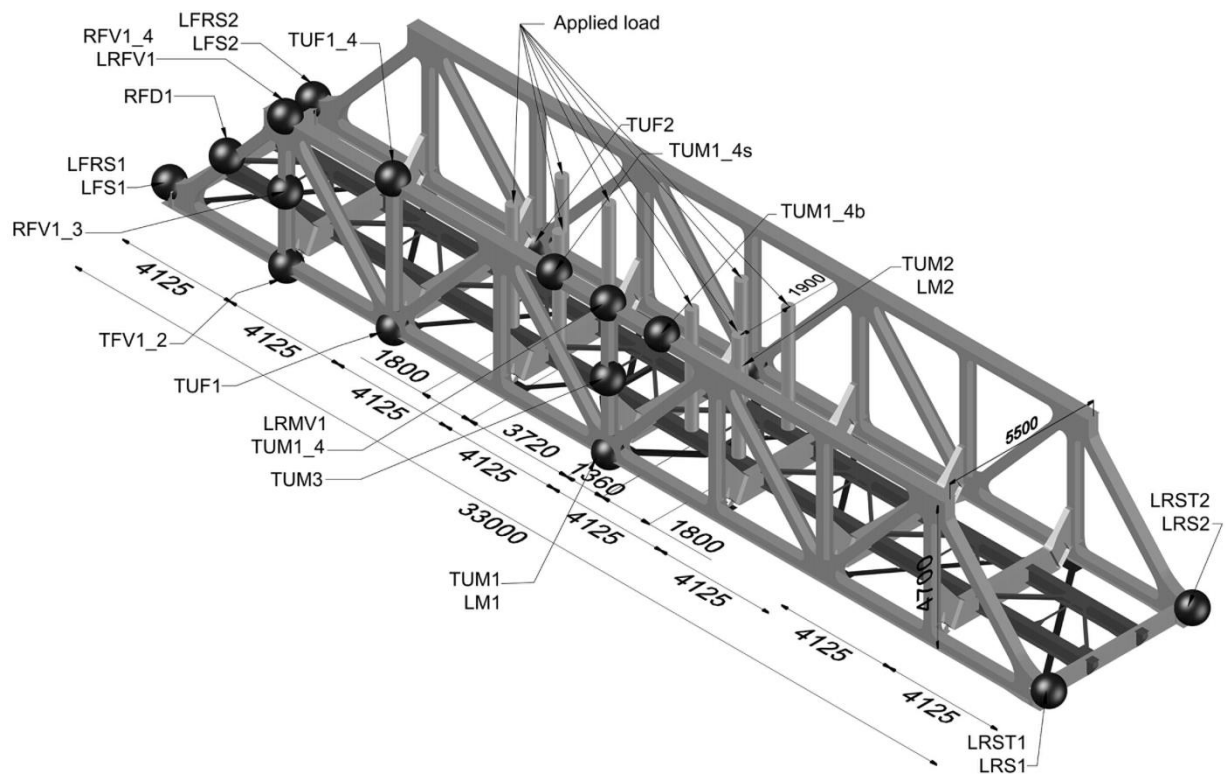
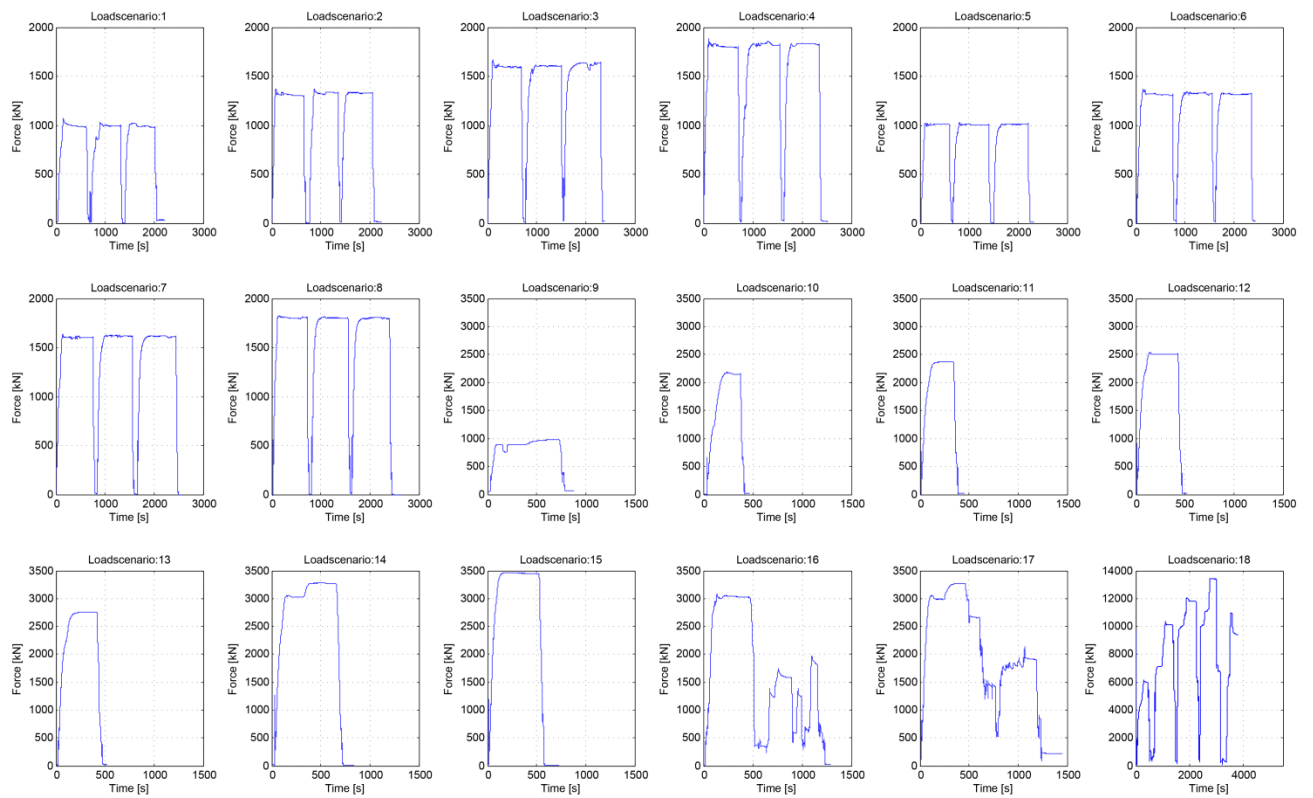


Figure 2 A) The Aramis setup for the DIC measurements B) Cables form sensors for measurements C) Hydraulic jack and one of the load distributing beams

Table 2 Sensors for measuring global effects

Sensor	Units	Description	Sensor	Units	Description
LFRS1	mm	Rotation at support	RFD1	$\mu\text{m/m}$	Three directional strain gauge
LFRS2	mm	Rotation at support	RFV1_3	$\mu\text{m/m}$	Three directional strain gauge
LFS1	mm	Settlements at support	RFV1_4	$\mu\text{m/m}$	Three directional strain gauge
LFS2	mm	Settlements at support	TFV1_2	$\mu\text{m/m}$	Strain gauge in main direction
LM1	mm	Deflection at mid span	TUF1	$\mu\text{m/m}$	Strain gauge in main direction
LM2	mm	Deflection at mid span	TUF1_4	$\mu\text{m/m}$	Strain gauge in main direction
LRFV1	mm	Horizontal deflection	TUF2	$\mu\text{m/m}$	Opposite side of TUF1
LRMV1	mm	Horizontal deflection	TUM1	$\mu\text{m/m}$	Strain gauge at both top and lower flange
LRS1	mm	Settlements at support	TUM1_4	$\mu\text{m/m}$	Strain gauge at mid span
LRS2	mm	Settlements at support	TUM1_4s	$\mu\text{m/m}$	Strain gauge in main direction
LRST1	mm	Rotation at support	TUM2	$\mu\text{m/m}$	Strain gauge at both top and lower flange
LRST2	mm	Rotation at support	TUM3	$\mu\text{m/m}$	Strain gauge on the vertical at mid span



Since the FEM-simulation indicated that buckling of the top frame would be the limiting failure mode, the horizontal displacement was monitored; the sensor is shown in Figure 6b. The positions of the both sensors for measuring the horizontal displacement are shown in Figure 4. Figure 6b illustrates the sensor measuring the global deflection at mid span. At the same point the strains were measured for the lower frame for both the upper and lower flange. By measuring the strain of two or more points on a cross section it is possible to calculate the section forces that are caused by a specific load. Since the bridge was placed on temporary supports and loaded to failure, settlements were likely to occur. In order to be able to adjust and get correct results for deflection it was necessary to measure the settlements at the supporting points which are shown in Figure 4 together with the sensors for measuring rotation at the support.

RESULTS AND DISCUSSION

Estimations made before testing by the Finite Element Software Abaqus indicated that the global failure would occur at approximately at 9MN and that it would be buckling of the top frame in the main truss. Before buckling of the frame there would be some yielding and redistribution of forces in the structure. The results in Figure 6 show both the static non-destructive testing and the final test to failure. For the static load scenarios one point is taken for each terrace point, which means that for load scenario 1-8 three points of measurement were taken and for 9-17 it is just one whereas the failure test are shown in its whole. Values from the sensors are given as a function of the total force induced by both of the hydraulic jacks. The jacks are manually controlled, but kept at an equal load level. The force is calculated as a function of the oil pressure and the area of the cylinder. In Figure 5 the expected results from the simulation are displayed together with the measured results. Looking at the diagram, one can conclude that the simulation corresponds well to the measured results regarding global deformation within the linear elastic range. However, once it starts yield the results differs to some extent. The reason for the difference in FEA results compared to the measured response beyond the elastic range is that redistribution of stresses close to the load (patch loading failure) was conservative in the FEA. Figure 5 also show the magnitude of what live load that the tested load corresponds to.

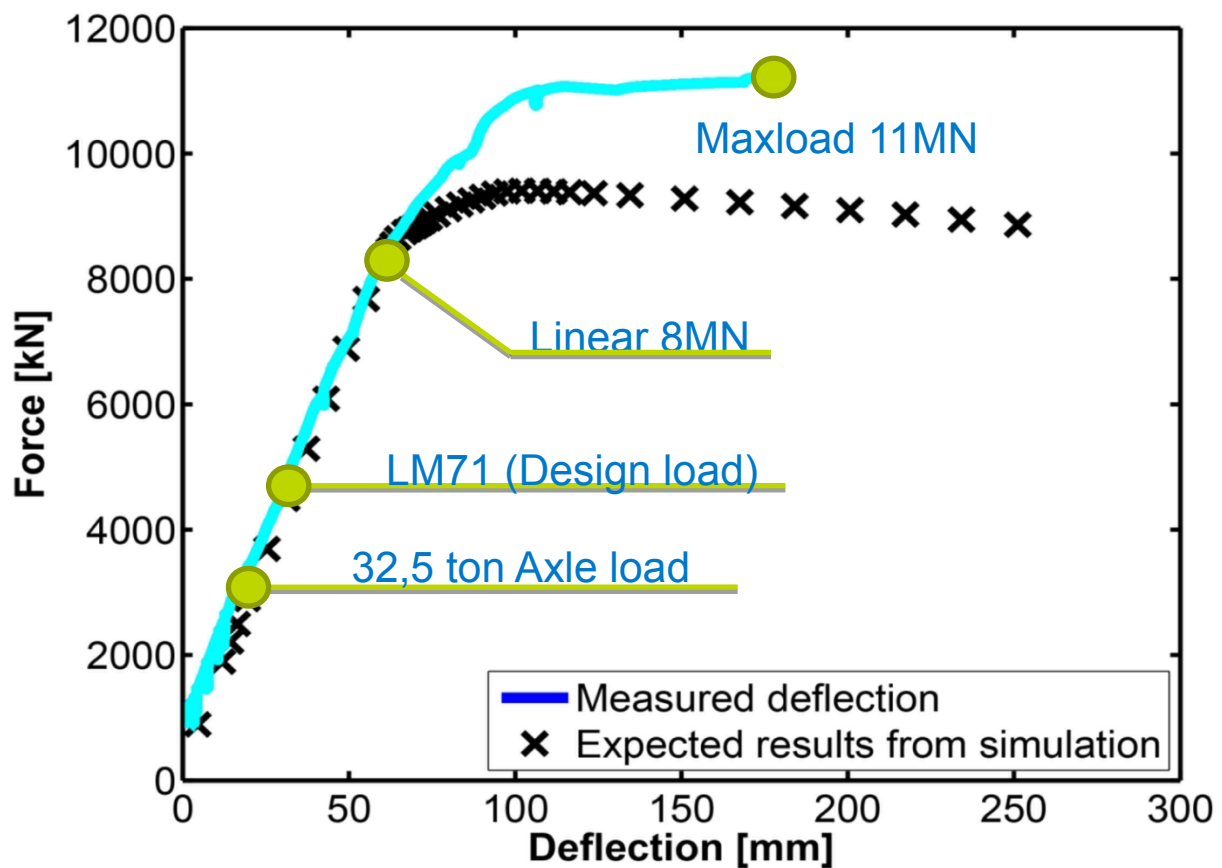


Figure 5 Deflection at mid span (sensor LM2). The figure also show the expected outcome based on the simulation and the parts that are cut out because of repositioning of the jacks.

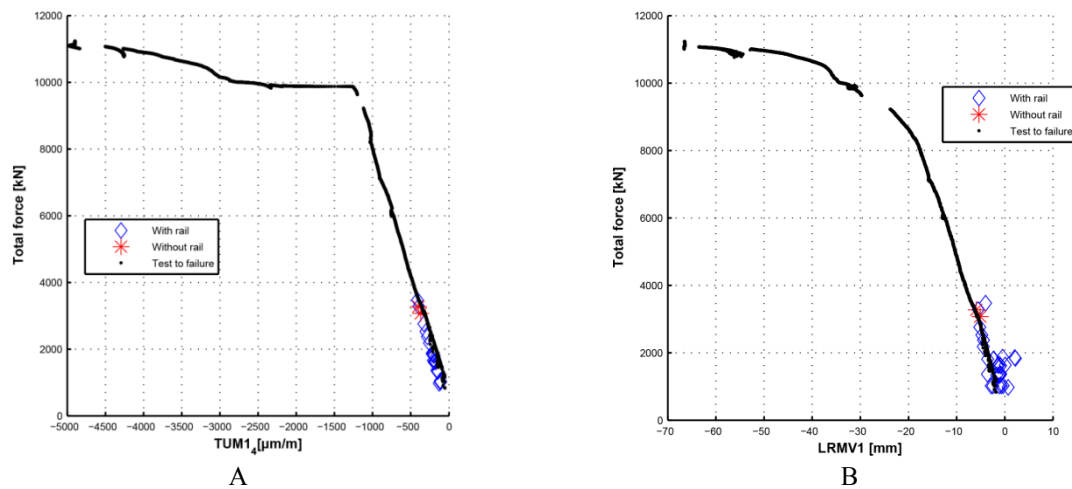


Figure 6 A) Strain as a function of total force, B) Horizontal deflection for the top frame in the upper frame at mid span

As the load increases, the top frame eventually starts to yield. By observing Figure 6a it appears that the yielding starts at approximately 10MN, since the member is compressed without constraints in the horizontal direction buckling will occur as a consequence, as seen in Figure 6b and 7b.

Besides the global buckling mode of the top frame there were local failures underneath one of the load distributing beams Figure 7a. The web could not withstand the high concentrated force which resulted in local shear buckling as well as local buckling due to patch loading.

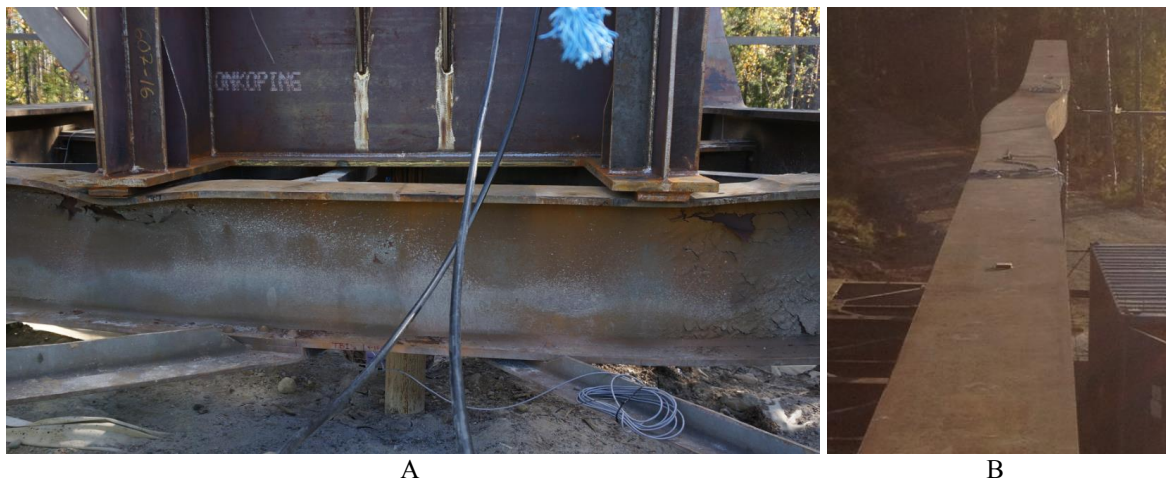


Figure 7 Left, the local failure of the longitudinal stringer beams. Right, the global failure mode, buckling of the top frame in the truss

An often discussed matter regarding these kinds of bridges are the consideration if the longitudinal stringers should be considered as simply supported or continuous. This is an issue that is of great interest, especially with regard to fatigue. Due to this; the curvature in the joint between the longitudinal stringers and the crossbeam was measured, as can be seen in Figure 8. From the measurements showed in Figure 8c, it is clear that the rail has a significant effect for the continuity of the longitudinal stringers, which could be used when assessing the Rautasjokk Bridge.

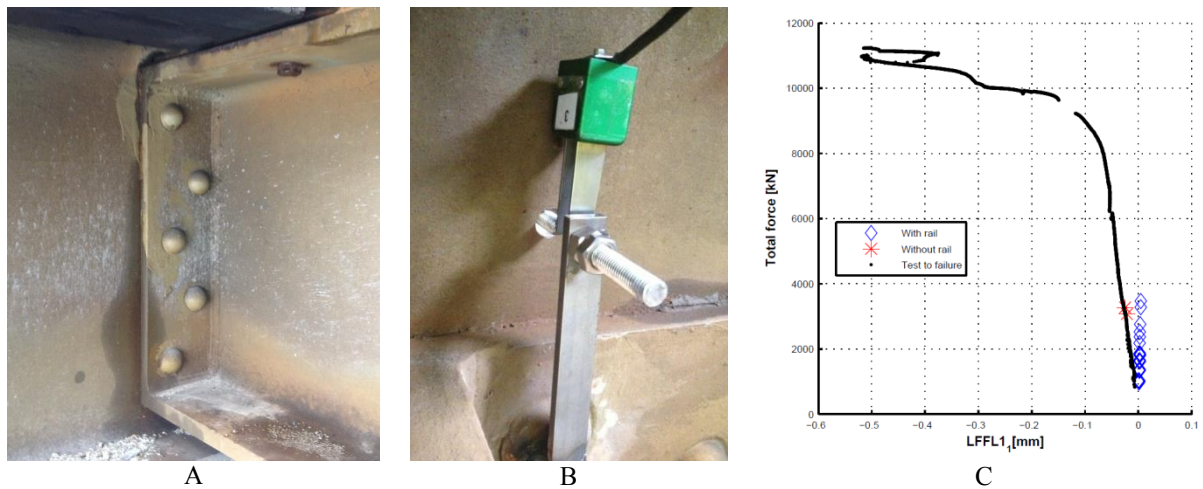


Figure 8 a) The joint between stringers and crossbeams, b) The sensor in place, c) measured gap in the join

CONCLUSION

The failure that eventually prevented the bridge from taking more load was buckling of the top frame, which was an estimated outcome according to the simulation performed on the bridge. Besides buckling of the top frame, there were local failures in the web in the area where the load was applied. The local failure might be interesting to study from a scientific point of view, but is not relevant with respect to the capacity of the bridge since the trainloads will be more distributed. The assessment calculations performed according to the Swedish assessment code for railway bridges proves that the bridge can carry the higher load with regard to ultimate limit state, however, fatigue proved to be the governing problem for the technical lifespan. For the assessment of fatigue, reduction of real stresses through measurement is likely to prove even more fruitful than for ultimate limit state since measurements can be performed on hotspots and a small reduction of the stresses influence fatigue more due to the slope of the Wöhler curve.

FUTURE WORK

The measurements for the Aby bridge has, to date, only been roughly investigated, and will be evaluated further in combination with future work. The steel material has been subjected to tensile testing, toughness tests and fracture mechanical testing, which remains to be evaluated. This involves updating both analytical calculations and numerical modelling.

During the autumn of 2015 a monitoring program for the Bridge over Rautasjokk is going to be performed. The purpose of this monitoring program is to verify assumptions and conclusions made on the Aby Bridge, evaluation of dynamic response as well as measuring hot spots for fatigue under live loading.

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