Bridge over Åby River

Evaluation of full scale testing

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PREFACE

The work presented in this report has been performed on behalf of Trafikverket and LKAB at the department of Structural Engineering at Luleå University of Technology (LTU). The instrumentation was carried out by Complab at LTU. NCC was the contractors responsible for erecting the new bridge over Åby River and assisted in moving the old bridge to the location where it was tested. A special thanks goes to SBUF (Svenska Byggbranchens Utvecklingsfond) which together with NCC has supported the project by co-financing the testing.

The report presents parts of the results gathered from the two Phases of testing the Åby Bridge; both while it was still in service and furthermore after it was taken out of service and subjected to static loading up to failure. The testing of the Åby Bridge took place between the summer of 2012 and the autumn of 2013 when the bridge was finally loaded until failure. Furthermore are the measurements which took place in the autumn of 2015 on the Rautasjokk Bridge briefly presented.
Summary

The former Bridge over Åby River has played a significant role in Sweden’s railway infrastructure. It was situated along the mainline, the only electrified railway line connecting the northern parts of Sweden with the southern parts and has therefore been crucial for both freight trains as well as for passenger trains.

The Åby Bridge consisted of an unballasted open steel truss bridge which was constructed in 1955 and got replaced together with the tracks during the autumn 2012. Before replacement of the bridge, LTU (Luleå University of Technology) performed measurements on the bridge to evaluate its status while it was still in service. This was carried out in 2012 between the 28th of August and the 13th of September. When the Åby Bridge got replaced, the old steel truss bridge was placed on the riverside to undergo further testing where it eventually was statically loaded to failure, which took place in 2013. These measurements were comprehensive, with strain gauges, LVDT’s (Linear Voltage Displacement Transducer), temperature sensors as well as DIC (Digital Image Correlation) and accelerometers.

One reason for the Åby Bridge being of particular interest is that there exists a similar “twin” bridge along the Ore line, the Bridge over Rautasjokk. The Ore line is the main route for transporting Iron ore from northern Sweden and stretches in the south from the steel mill in Luleå to the North, the harbor in Narvik, Norway. Due to the transportation of Iron ore, the Ore line is subdue to heavier traffic then the main line. There was placed a hope that the assessment of the Åby Bridge would provide information that could be used in order to keep the Rautasjokk Bridge in service. If the results from the Åby Bridge could provide information so that the bridge over Rautasjokk doesn’t have to be replaced, cost savings can be made in the range of 15-20 million SEK. In 2015 was measurements carried out on the Rautasjokk Bridge, in order to verify findings provided from the Åby Bridge as well as fatigue assessment of critical details.

Results indicated that the Åby bride was able to carry loads higher than four times the current regulated axle load along the line. Failure actions occurred in a ductile manner with lateral buckling of the top chord as the governing failure mode, which also was expected by numerical simulations. There were no observed failures in any of the riveted or welded connections.

The Åby Bridge was statically tested both with and without the rails in place. It was found that the rails have a significant influence on the behaviour of the stringer beams by distributing the load and possible also to some degree by composite action together with the stringers.

The measured response in the stringers beams indicated a complex behaviour where torsion and out of plane bending are believed to have a major influence on the state of stress. These effects are believed to be displacement induced and therefore of less importance in ULS (Ultimate Limit State). However, for fatigue assessment these effects will give rise to significantly higher stresses at some positions than what is normally accounted for. There have not been any cracks detected in these studied details. Further research is believed necessary in order to assess stringer beams in these types of bridges with an accurate safety level.

The Rautasjokk Bridge has been concluded to have sufficient capacity for ULS, but there are still doubts related to fatigue where the bridge indicates insufficient capacity with regard to code-based controls, however no cracks or damage has been identified.
Extended summary

Introduction
According to UNEP (United Nations Environment Programme), the building sector is responsible for emitting approximately ½ of greenhouse gases and consumes 40% of the global resources (UNEP, 2009). In the same report it is claimed that the building sector has the greatest potential of significantly decrease its environmental footprint.

Sustainable development, or sustainability, is often described in terms of three spheres, dimensions, domains or pillars, i.e. the environment, the economy and society (WCED, 1987). Keeping existing bridges in service for a longer period of time, by means of accurate assessments, maintenance and strengthening actions can often be regarded as sustainable. By postponing replacement of bridges, generally only small amount of resources needs to be used, cost savings can often be made, and it is often approved of by society to keep historical landmarks as well as decreasing downtime which can lead to delays.

Research is needed on both assessments methods of existing structures as well as strengthening methods in order to keep elderly bridges in service for a longer period of time. One way of improving assessment methods for existing bridges is to study their actual behaviour, by measuring structural response.

The former Bridge over Åby River was part of a research project that included monitoring of elderly unballasted open steel truss railway bridges. The measurements on the Åby Bridge were divided in to two Phases and carried out in 2012 and 2013 with additional measurements in 2015 on an identical bridge, the Bridge over Rautasjokk.

The Åby bridge
The Bridge over Åby River has been an important link in the Swedish railway system. Located approximately 45km west of Piteå on the mainline, it connected the northern and southern parts of Sweden. Since the mainline is the only electrified railway connecting the northern parts with the southern parts of Sweden, the Åby Bridge has been crucial for both freight and passenger trains.

The old un-ballasted steel truss railway bridge was built in 1955 and supported one track. The bridge was designed according to the present code F46, corresponding to 12 axles with 25tons loads at 1.6m spacing representing the locomotive and a distributed load of 85 kN/m representing the wagons (Trafikverket, 2010).

The bridge was constructed in 1955 and designed as a single span steel truss bridge, carrying a single track. The span was 33 m and the distance between the upper and lower main chords were 4.7 m. The deck was designed as a steel grillage where the crossties were resting directly onto stringer beams. The connections between members were partially riveted and partially welded. Some of the welded details were already in 1994 found to be overdue with regard to fatigue, however no cracks were detected – of that reason it was kept in service. The bridge was replaced in autumn 2012 together with replacement of the tracks.

One reason that the Åby bridge were of particular interest is that there is a similar “twin” bridge along the Ore line, Bridge over Rautasjokk. The Ore line is the main route for transporting Iron ore in
in northern Sweden and stretches from the steel mill in Luleå in the south to the harbor in Narvik (Norway) in the north. The Ore line is subdued to heavier traffic then the main line. There was therefore hope that the assessment of the Åby Bridge would provide information that could be used in order to keep the Rautasjokk Bridge in service. If the measurements on the Åby Bridge could provide information that prevents replacement of the bridge over Rautasjokk, cost savings can be made in the range of 15-20 million SEK.

**Measurement program**
The project was divided in to three Phases. Phase I consisted of testing the Åby Bridge while it was still in service, in its original location. For Phase II, the Åby Bridge had been replaced and the bridge had been moved beside the track where it was statically tested to failure. The last phase, Phase III consisted of the testing of the Rautasjokk Bridge which is only briefly covered in this report.

Before replacement of the old Åby Bridge, LTU (Luleå University of Technology) performed measurements on the bridge to evaluate its status and find out more about the real state of stresses. This was carried out in 2012 between the 28th of August and the 13th of September while the bridge was subjected to live loading from passing trains. At the time of measurements, there were speed limitations due to the ongoing work of replacing the existing bridge. For Phase I 55 sensors were used, divided between 14 LVDTs (Linear Voltage Displacement Transducer) monitoring displacements of the main truss, 8 strain gauges on the main truss and 30 strain gauges on the load distributing system (stringer beams and crossbeams) and 3 sensors registering temperature. In Phase I was 399 train passages recorded, where one specific passage was chosen to be presented more thoroughly. This train set is referred to as the “reference train” and consists of a train set transporting steel slabs going in the southern direction. The axle loads for this train set is known from the weighting station located in Jörn, 40 km after the passing the bridge.

When the Åby Bridge was replaced, the old steel truss bridge was moved beside the track to undergo further testing (Phase II). The testing took place in 2013, where the bridge was subjected to both dynamic and static loading and finally tested to failure by static loading. The measurements were comprehensive, with strain gauges, LVDT’s, temperature sensors as well as DIC (Digital Image Correlation) measurements.

The loads on the bridge were induced by two hydraulic jacks, which were anchored in to the bed rock. The two jacks pulled the bridge downwards by load distributing beams spreading the load in to four load positions each, representing the loading of one wagon. Phase II consists of 18 test series where the first 15 were carried out with the rail still in place. Another two load series were then carried out with the rail removed, where the loading was applied directly on the stringer beams. The final test was carried out to failure, meaning that the bridge was loaded beyond yielding until a peak-load was reached. In the presented results there are 3 values taken from each of the initial load series and the final loading to failure is displayed in its entity. For Phase II most of the sensor positions used in Phase I was reused, but the instrumentation was also extended. A total of 122 sensors were used which were divided between 16 LVDTs on the main truss, 29 LVDTs on the load distributing system, 24 strain gauges on the main truss and 47 strain gauges on the local load distributing system as well as 6 temperature sensors registering temperature.

Phase III was carried out by measurements on the Rautasjokk Bridge. The Bridge is still in service and measurements took place during the autumn of 2015. Totally 71 strain gauges were used for these
tests carried on 526 passing trains. The sensor configuration was made so that confirmation of the earlier findings from the Åby Bridge could be verified as well as measurements on critical details with regard to fatigue. Phase III is only briefly covered in this report.

**Material testing**

Material testing has been conducted on the steel material for the now demolished Åby Bridge. Three different members were tested where samples were taken from a stringer, a crossbeam and from the bottom chord in the main truss. These specimens were subjected to three different tests; tensile tests, Charpy V-notch tests and fracture mechanical testing.

For the tensile tests, three specimens where tested for each member. The tensile testing proved homogeneous results with yield strength well above design values for all specimens.

The Charpy-V tests consisted of five specimens for each member. The tests indicated a greater variation in results, compared to the tensile tests. It was only the material taken from the crossbeam that proved absorbed fracture energy higher than 27J for all five specimens, which is the regulated material requirement for bridges.

The fracture mechanical testing was carried out according to BVS583.12 (Banverket, 2005b) with three specimens for each member. Measured results indicated that there fracture properties for the main chord and the crossbeam was insufficient, and that the yield strength should be limited for these materials (Eriksson, 2014). The stringer proved sufficient capacity for the fracture mechanical testing.

For both the Charpy-V tests and the Mechanical testing a high variation in results between specimens was achieved.

**Results & conclusions**

The bridge proved a robust ductile behaviour while being loaded to failure, with no failures in connections. The failure mode which prevented the bridge from taking more load was buckling of the top chord. Besides the buckling there was local damage on the stringers due to the position of the loading beams. These damages are of less relevance, since real trains would distribute the loading to a greater extent.

Two questions that were aimed to be answered within this project (which also was the foundation for configuration of sensors):

- Do 3d-effects have to be considered for these types of bridges? Both with regard to stringer behaviour top chord and crossbeams.
- What was the stiffness of the connection between stringers and crossbeams? Should the stringers be considered as simply supported or continuous?

The horizontal displacement of the top chord appears linear for service loads. It was not until 8MN of total loading a non-linear behaviour was reached. The loading position in Phase II is believed to have been unfavorable with regard to buckling of the top chord, since it locally tilted the truss inwards and therefore weaken its buckling resistance. By studying the horizontal displacement of the top chord it can be seen that it moved inward at mid-span and outwards close to the support. This effect will be less likely if a distributed load is applied over the entire length of the bridge, for example a train-set.
Therefore it is not believed necessary to take the 3D effects of the top chord in to consideration for a system analysis, besides the buckling analysis.

Bending of the crossbeams in the bridge’s longitudinal direction could be observed, caused by elongation of the main truss which was restrained by the stringers causing bending around the crossbeam’s weak axis. This effect should be considered when assessing stringers, since these effects causes tension in these members.

Based on the measured response it can be concluded that the rail together with the cross-ties had a significant role on the level of continuity of the stringers in the connection between stringer and crossbeams. The flexibility of the rail and the cross-ties will act as springs, causing a greater distribution of loading and possible some composite action between the stringer and the rail. When studying the rotation in the riveted connection between stringers and crossbeams, major differences can be seen between the tests that were carried out with and without the rail in place. This effect does however not influence the main truss.

Assessment of the connections between stringer beams and crossbeams are often details which are found critical for un-ballasted steel truss railway bridges if the stringers are considered continuous, both with regard to Ultimate Limit State (ULS) and Fatigue Limit State (FLS). The effects of flexibility in the crossbeams together with the already mentioned flexibility of the rail and the cross-ties will reduce the hogging moment-peaks in the connection leading to a reduced load effect in the given detail, significantly reducing bending stresses.

When studying the strain levels on the first stringer beam for live loading it was concluded that the response was not the same for the two sensors mounted on each side of the bottom or the top flange, which would be expected if it was purely bending around its strong axis and normal forces acting on the member. This effect is believed to be caused by prevented torsion causing so called warping and out of plane bending. By splitting the measured response in to components, the magnitude of warping and out of plane bending can be quantified. Based on the response it appeared as bending only represented about half of the measured strain level at the flange edges. It was difficult to evaluate and anticipate the warping and the out of plane bending, since these effects are believed to arise from the constraints given by the connection between cross-ties to and stringers.

In addition to the study of the Åby Bridge, an assessment of the Rautasjokk Bridge (Häggström, 2014) has been made to answer questions regarding the code-based lifespan of this bridge with regard to fatigue. This report also investigates which details that require extra careful inspections for the future. In this report it was concluded that the bridge can withstand loads up to 34.2 tons for TLM3 (iron ore wagons) and 34.4 tons for TLM2 (representing the geometry for the steel slab trains along the main line) in ULS. The limiting structural component was found to be the riveted connection between stringers and crossbeams. The accumulated fatigue damage in the stringers was found to be overdue in the detail where connection plates for the bracing of the stringers are welded to the top flange of the stringers. It should be noted that even though the Rautasjokk Bridge was built using the same drawings and material specifications as the Åby Bridge, it is located along the Iron ore line and has therefore been subjected to historically higher axle loads. Despite the higher loads, no cracks have been identified in the load carrying structural system.
Sammanfattning

Den före detta bron över Åby älv har genom tiderna spelat en stor roll för Sveriges infrastruktur. Eftersom bron var belägen längs stambanan, har den kopplat samman de norra delarna av landet med de södra och är därför viktig för både godstrafik såväl som persontrafik.


En av orsakerna till att Åby bron är av extra stort intresse är att det finns en så kallad ”tvillingbro” längs malmbanan, nämligen bron över Rautasjokk. Malmbanan är huvudtransportleden för järnmalms i norra Sverige och sträcker sig från stålverket i Luleå, i söder, till hamnen i Narvik (Norge) i norr. Malmbanan utsätts för tyngre trafik än stambanan. Det fanns hopp om att bedömnings och provningen av bron över Åby älv skulle ge information som kan användas för att behålla bron över Rautasjokk i fortsatt drift. Om resultaten från dessa mätningar kan ge information som leder till att livslängden på bron över Rautasjokk inte behöver bytas så kan besparingar på 15-20 miljoner SEK göras.

Mätningar på bron över Rautasjokk genomfördes under 2015, med syftet att studera bl.a. utmatning och verifiera att beteendet är desamma som för Åby bron. Mätprogrammet för mätningarna över Rautasjokk bron täcks endast övergripande inom ramen för den här rapporten.

Resultaten av brottprovningen visar på att Åby bron var kapabel att bära en last som uppgår till en storlek nästan fyra gånger så stor som den aktuellt tillåtna axellasten, samt att det brott som uppstod var av duktil karaktär utan att brott i någon knutpunkt, där knäckning av överramen i fackverket begränsade bron från att ta mer last, vilket var förväntat från numeriska simuleringar.

Bron över Åby Älv prövades statiskt både med och utan räl. Resultaten indikerar på att rälen har stor inverkan på fördelnings av laster och möjligtvis viss samverkan med långbalkarna vilket har stor effekt för det lastfördelande systemet som består av långbalkar och tvåarbalkar.

Mätningarna på långbalkarna visar på ett komplext beteende där tredimensionella effekter så som vridning och böjning ur plan verkar ha stort inflytande på de uppstått spänningar. Dessa spänningar tros uppstå genom deformationer och därför av mindre vikt för brottgränstillstånd. För utmatning är det däremot tveklöst att dessa effekter ger upphov till högre spänningar än vad som fås genom normala beräkningsmetoder. Trots detta, har det inte identifierats några utmatningsskador på långbalkarna.

Det har bekräftats att bron över Rautasjokk har tillräcklig kapacitet i brottgränstillstånd för den rådande trafiklasten, men att tveksamheter beträffande utmattningskapaciteten kvarstår där vidare studier anses nödvändigt.
Förlängd Sammanfattning

Introduktion

Enligt UNEP (United Nations Environment Programme) så är byggnadsindustrin ansvarig för en tredjedel av utsläppen av växthushusar och förbrukar 40% av globala resurser (UNEP,2009). I samma rapport så påstår det att byggnadsindustrin har bland alla sektorer störst möjlighet att kraftigt minska sitt ekologiska fotavtryck.

Hållbar utveckling, eller hållbarhet beskrivs ofta som samspelet mellan miljö, ekonomi och samhälle (WCED, 1987). Att behålla befintliga broar i bruk under längre tid genom noggrann tillståndsbildning, underhåll, och förstärkning kan ofta betraktas som främjande av hållbar utveckling. Dessa åtgärder gör generellt mindre anspråk på tillgångar, vilket leder till besparingar med avseende på miljö och kostnader. Att behålla befintliga landmärken i form av äldre broar samt hålla nere förseningarna som uppstår i samband med byggnationer ligger i samhällets intresse.

Forskning på metoder för tillståndsbildning och förstärkning är därför viktigt för att för kunna behålla befintliga broar i bruk med en acceptabel säkerhet. Ett sätt att förbättra tillståndsbildningar för befintliga broar är att studera deras faktiska verkningsmåt, genom mätningar.


Bron över Åby Ålv

Bron över Åby ålv har varit en viktig länk i det svenska järnvägsystemet. Bron var belägen ca 45km väst om Piteå längs med Stambanan. Då Stambanan är den enda elektrifierade banan som ansluter de norra delarna av Sverige med de södra, var bron viktig för både godstrafik såväl som persontrafiik.

Bron bestod av en oballesterad stålackfackverksbro för enkelspårig järnväg som byggdes 1955. Den byggdes enligt den då gällande dimensioneringslasten F46, vilken representerar 12 axlar om 250kN med 1.6m axelavstånd samt en jämnt utbedd last på 85kN/m.

Brons spännvidd uppgick till 33m där avståndet mellan över och underram var 4.7m samt att den var 5.5m bred. Däcket bestod av ett grillage med slipers vilande på långbalkar som i sin tur var anslutna till tvärbalkar som bar lasten till huvudfackverket. Anslutningar mellan konstruktionselement var utfört både med svetsar och nitar. Redan 1994 så identifierades teoretiska utmattningsproblem, men inga sprickor kunde identifieras varav bron behölls i bruk. I samband med ett spårbytesprogram under 2012 så byttes bron ut.

En av orsakerna till att bron över Åby ålv är av extra stort intresse är att “tvillingbron” över Rautasjokk fortfarande är i bruk och belägen längs med malmbanan. Malmbanan sträcker sig från stålverket i Luleå i Söder till hamnen i Narvik i Norr. Malmbanan är utsatt för större laster än stambanan. Förhoppningen med forskningsprojektet var att mätningarna skulle indikera att bron över Rautasjokk skulle kunna hållas kvar i bruk, vilket skulle innebära besparingar mellan 15-20 miljoner kronor.
Utöver Bron över Rautasjokk så finner det ytterligare broar i Sverige som är konstruerade enligt samma ritningar, även om dessa inte utsätts inte för lika höga laster.

**Måtprogram**

Projektet var uppdelt i tre faser med mätningar. Fas I bestod av mätningar på Bron över Åby älv medan den fortfarande var i bruk i dess ursprungliga position och belastades av passerade tåg. Till fas 2 hade Åby bron tagits ut bruk och ställts upp vid sidan av spåret för där den utsattes för statisk belastning som slutligen ledde till brott. Den tredje och sista fasen bestod av mätningar på bron över Rautasjokk, där belastningen utgjordes av förbipasserande tåg.


Efter det att den gamla bron över Åby älv var utbytt, placerades den vid sidan av spåret på tillfälliga stöd för att testas ytterligare (Fas II) under 2013. Under Fas II så återanvändes de flesta givarpositioner från Fas I, samt att måtprogrammet utökades till totalt 122st sensorer, vilka fördelades mellan 16st LVDT på huvudfackverket, 29st LVDT på det lastfördelande systemet, 24st töjningsgivare på huvudfackverket och 47st töjningsgivare på det lastfördelande systemet samt 6st temperaturgivare samt ett system för fotometrisk mätning av töjningar och deformationer (Digital Image Correlation). Åby bron belastades här både dynamiskt och statiskt, där den slutligen statiskt belastades till brott. Den statiska lasten fördes på med hjälp av två stycken domkrafter som var förankrade i bergsgrunden och drog bron neråt. Varje domkraft fördelade lasten till fyra punkter, vilket ledde till att bron totalt belastades i 8st punkter i ett försök att efterlikna lasten från en vagn. Fas II bestod i 18st lastekvenser där de 15 första utfördes med rälen fortfarande på plats. Efter att rälen hade avlägsnats applicerades lasten direkt på långbalkarna, då utfördes ytterligare 2 lastekvenser innan bron i det artonde lastfallet belastades till brott, vilket innebär att lasten succesivt ökades bortom flytning av stålet tills det att ett maxvärde erhölls. De presenterade resultaten visar resultat från både förprovningen såväl som det slutliga försöket till brott.

Materialprovning
Efter försöken i fas II, så genomfördes materialprovning på stål materialet. Tre stycken olika konstruktionselement testades, där provkroppar togs från långbalk, tvärbalk och underramen i huvudfackverket. Dessa provkroppar testades i sin tur för tre olika försök; dragprov, slagseghet samt brottmekanisk provning.

För dragproven så testades tre provkroppar för varje konstruktionselement. Dragproven visade på homogena resultat, där flytgransen var klart över tabulerade karaktärstistiska värden.


Den brottmekaniska provningen genomfördes enligt BVSS83.12 med tre försök för varje konstruktionselement. Resultatet från mätningarna indikerar stor variation och att både underramen såväl som tvärbalken hade tillräcklig kapacitet, vilket föranledder rekommendationer för den maximala tillåtna spänningen. Långbalken visade däremot tillräcklig kapacitet.

För både slagseghet och brottmekanisk provning så erhölls stor spridning av resultat, vilket kan tyda på inhomogen material. För att öka säkerheten kring dessa resultat, så bör fler försök genomföras.

Resultat och slutsatser
Bron visade robusthet och ett duktilt beteende medan den lastades till brott, där inga synliga brott uppstod i någon knutpunkt. Brottmoden som begränsade bron från att ta mer last var knäckning av överramen. Vid sidan av detta, så uppstod lokal buckling på långbalkarna eftersom rälen var avlägsnad och lasten placerad direkt på långbalkarna. Dessa brot anses däremot vara av mindre betydelse, eftersom verkliga tåg är fördelade över hela bronns längd, där räl och slipers fördelar lasten ytterligare.

Två frågor som försöks besvaras inom ramen för det här projektet var:

- Behöver 3d-effekter beaktas för den här typen av broar?
- Bör långbalkarna betraktas som kontinuerliga eller fritt upplagda i knutpunkten mellan långbalk och tvärbalk eller någonstans mitt emellan?

Studeras den laterala förskjutningen av överramen i huvudfackverket så inses att denna är linjär för brukslasten. Det var inte förrän kring 8MN totallast som ett icke-linjärt beteende erhölls.

Lastpositionen i fas två, då lasten var koncentrerad kring mitten av bron tros ogynnsamt med avseende på lateralfackverket av överramen, eftersom böjning av tvåbalkarna ger upphov till en sidokraft på överramen vilket reducerar dess kapacitet mot knäckning. Genom att studera resultaten så går det att se att överramen trycks inåt mot mitten av bron medan den vid stöd åker ut. Det tros att denna effekt skulle vara mindre om hela bronns längd skulle belastas, t.ex. av ett tåg. Därfor anses det inte vara nödvändigt att beakta 3d-effekter för överramen utöver knäckanalyser.

Sekundär böjning av tvåbalkarna i bronns längdriktning kunde ses, detta uppstå av att bron underram förlängs vid belastning vilket till viss del hålls tillbaka att långbalkarna genom sekundär
börjning av tvårbalkarna. Det här är en effekt som bör beaktas vid tillståndssbedömning av långbalkarna eftersom det ger upphov till en dragande normalkraft.


Vid tillståndssbedömning av oballesterade fackverksbroar för järnväg så identifieras ofta knutpunkten mellan långbalkar och tvårbalkar som en kritisk punkt för både brottgränstillstånd och utmattning, om långbalkarna beaktas som kontinuerliga. Effekten av eftergivligheten hos tvårbalkarna tillsammans med den redan nämnda eftergivligheten av tvårbalk och räl kommer att reducera det maximala stödmomentet vilket leder till minskad lasteffekt genom reducerad böjpåning i knutpunkten. Det är därför viktigt att inte betrakta det lastfördelande systemet som en del av hela bron och inte som ett separat system.

Genom att studera töjningsnivåerna i det första spannet av långbalk när bron utsattes för trafiklast kunde det konstateras att mät-resultaten inte var desamma för töjningsgivare monterade på olika sidor av underflänsen eller överflänsen, vilket kan förväntas om balkarna enbart utsätts för börjning runt dess styva axel och normalkraft. Den här effekten tros uppstå av förhindrad vridning vilket ger upphov till välvspännningar samt börjning ur plan. Om spänningskoncentrationer och lokala effekter försummas kan det vara ett sått snittkrafter delas i komponenter, och storleken på dessa bestämmas.

Baserat på mätningarna så verkar det som att den kombinerade effekten av välvning och börjning ur plan för vissa tvärnitt overskrids effekten av börjning runt den styva axeln. Det har konstaterats som svårt att förutsäga och uppskatta storleksordningen av välvning och börjning ur plan, då dessa till stor del tros bero på randvillkoren mellan slipers och långbalkar.

Utöver studierna på bron över Åby ålv, har det gjorts en klassningsberäkning av bron över Rautasjokk. Denna rapport besvarar frågor avseende bronns norm-baserade livslängd med avseende på utmattning. Den här rapporten presenterar även detaljer som bör granskas extra noga vid framtida inspektioner. Det konstateras att bron kan bära 34.2ton axellaster för lastmodell TLM3 (malmvagn) samt 34.4 för lastmodell TLM2 (Motsvarande stärtågen längs stambanan) i brottgränstillstånd. Den begränsande faktorn var då tvärkraft på i nitförbandet mellan långbalkar och tvårbalkar, där nitförbandet något konservativt antogs bära hela lasten. Delskanan med avseende på utmattning i långbalkarna konstaterades att överskrida kapaciteten för detaljen där knutplåtar är svetsade mot överflänsen på långbalkarna för infästning av slingerförbandet mellan dessa. Det bör noteras att trots att bron över Rautasjokk är belägen längs med malmbanan och utsatt för högre laster än bron över Åby ålv. Trots att bron visar på en normmässig delskada på över 500 % så har inga utmattningsskador kunnat identifieras vid dessa knutpunkter.

Genom både mätningar och simuleringar har det kunnat konstateras att rälen har en betydande effekt för konstruktionselement och detaljer som har korta influenslinjer, inte minst med avseende på utmattning. För dessa detaljer så kan en styvare räl både minska maximala spännningar såväl som reducera antalet cyklorer. Det är därför rekommenderat att modellera den här typen av broar i 3-d med hjälp av balk-element där eftergivligheten i räl och slipers beaktas.
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1. INTRODUCTION

Besides safety, the financial aspects are the single most important factor when it comes at replacing bridges. Until today bridges have often been replaced on conservative assumptions that they have reached the end of their technical lifespan. In Figure 1-1 is a distribution of the existing bridge stock in both Sweden and the rest of Europe presented, (Sustainable Bridges, 2007). By study In Figure 1-1 it can be noticed that it is not possible to replace all elderly bridge structures, since a vast majority of the bridge stock are older than 50 years. Replacement should only be considered as a last measure, when repairs and strengthening proves unfavorable from a safety or financial perspective. According to the strategy put out by (Trafikverket, 2012), bridges are to a greater extent going to be assessed with regard to their actual capacity and firstly thereafter necessary measures should be taken, for example repairs, strengthening or replacement.

One way of improving assessment methods for existing bridges is to study their actual state of stress, by measuring structural response. This report present a unique project where a bridge has been tested in two Phases; live service loads and static loading leading to failure. By performing measurements on existing structures it is possible develop models for assessing these and similar structures in order to keep them in service for a longer period of time.

On behalf of Trafikverket, the division of structural engineering has together with Complab at LTU performed and recorded measurements on the bridge over Åby River during the autumn of 2012 and 2013 as well as the Bridge over Rautasjokk in the autumn of 2015.

1.1 Previous work

There have been previous publications related to the Åby bridge. Part of these publications were conducted at LTU but also within the work packages of the MAINLINE project (MAINtenance, renewaL and Improvement of rail transport iNFrastructure to reduce Economic and environmental impacts) project (Mainline, 2014).

- Measurements of the bridge over Åby River during service (Blanksvärd, 2012)
- Structural model for the fatigue stress analysis – Master Thesis (Moreno, 2013)
- Measurements of the bridge over Åby River, Full scale testing (Blanksvärd, 2013)
- Dynamic measurements (Andersson and Grip, 2013)
- Level of constrain in stringers in the joint to crossbeams - Master thesis (Elhag, 2013 )
- Evaluation of robustness (Casas, 2014)
- Damage identification by modal characteristics (Edrees Saaed, 2015)
1.2 Aim and scope
The primary aim of this project was to gain knowledge of the structural behaviour for the Åby Bridge in particular and un-ballasted steel truss bridges in general. The plan was also, that the results from measurements would be used when determining the remaining life for bridge 3500-2118-1 Södra Rautasjokk (hence called Bridge over Rautasjokk). In addition to evaluating the bridge over Rautasjokk, this project will hopefully contribute to the field of Structural Engineering and be of guidance for future assessment and monitoring-systems for steel truss bridges.

1.3 Methodology
Examining a bridge by measuring strains and displacements is nothing new, but rather common in order to find out more about real behaviour of structures. However, the uniqueness of this project lies in the holistic measurement program where the bridges over Åby and Rautasjokk together have been tested in three different Phases as described in Figure 1-2.

- Phase I: Measurements on the bridge over Åby river while still in service (performed during autumn 2012)
- Phase II: Full scale measurements of the bridge over Åby river while exposed to static loading (performed during the Summer/autumn 2013)
- Phase III: Measurements on the bridge over Rautasjokk while in service (performed during the autumn of 2015)

Figure 1-2 Flowchart for the measurement program for the bridges crossing Åby River and Rautasjokk, showing Phases I-III (Blanksvärd 2013)
1.3.1 Pilot study
The Pilot study presented in Blanksvärd (2012), covered the initial measurement program for the Åby Bridge and the Rautasjokk Bridge. The report also summarized the results from the previous code-based capacity controls for both ultimate limit state (ULS) as well as fatigue limit state. The first FEM model of the Åby Bridge were created in this stage, a shell element model developed in the software Ansys that is further presented in 4.1. The aims with this study were stated as:

- Connect the FE models to the real structural response and see how these can be upgraded in a reliable manner.
- Study the constraints, extra capacity for structure is often hidden within conservative assumptions regarding the constraints.
- Investigate if 3D-effects influence the structural behaviour of the bridge.
- Study methods for assessing fatigue capacity. And also clarify the real fatigue loading on the bridge which governs the fatigue damage.
- Investigate the partial coefficients impact on partial damage analysis based on a reliability based analysis.
- Perform measurements on connections in order to capture their real behaviour
- Develop general methods for assessment which can be applied for similar types of bridge structures.

1.3.2 Phase I – Measurements on the Åby Bridge during service
The measurements on the Åby River Bridge (Phase I) were performed during September 2012, and took place when the bridge was still in its original position and in service. The sensors locations were based on the pilot study (Blanksvärd, 2012) and more thoroughly described in chapter 3.4. There was a speed limitation on the line at the time of measurements, limiting the speed to 20 km/h instead of 70 km/h which was the normal speed limit on the line. The cause for these limitations in speed was due to the work at the bridge site, where preparations were ongoing for replacing the Åby Bridge with a new ballasted steel trough bridge as seen in Figure 1-3. During the time of measurements 399 train sets were recorded passing the bridge. Some early results from these measurements can be found in (Blanksvärd, 2013 & Moreno, 2013).
1.3.3 Phase II – Static full scale testing to failure of the Åby Bridge

For the second Phase of testing, Phase II the old bridge had been placed close to the tracks on temporary concrete supports placed directly on the ground. Holes were drilled into the bed-rock in order to attach steel tendons used in to pull the bridge downwards. After stripping the bridge from non-load-carrying components and cleaning the bridge, broken sensors from Phase I was replaced and new ones were added. The bridge was subjected to a total of 18 load scenarios when being loaded to failure.

Figure 1-4 The Åby Bridge placed beside the rail track after being removed from its original position

1.3.4 Phase III - Measurements on the Rautasjokk Bridge during service

The tests performed at the Rautasjokk bridge were performed in order to verify results and conclusions from the Åby Bridge measurements. Besides verification of results, there was a focus on measuring strains related to critical fatigue details and evaluating the dynamic amplification factor. Some results and analysis of the local approach for fatigue assessment can be seen in (Häggström, 2016). For these measurements a total of 60 strain gauges were used. Some results from these measurements are found in (Häggström, 2016)

Figure 1-5 The Rautasjokk bridge during measurements in Phase III
2. THE ÅBY BRIDGE

2.1 History
The superstructure for the bridge over Åby river, “3500-1940-1 Åby älv, km 1049+350” (Figure 2-1) was taken in service in 1955. It was located along the main line in the northern part of Sweden, approximately 45km west of Piteå. The main line is the only electrified railroad that connects southern part of Sweden with the Northern part. This makes the bridge essential for both freight trains as well as passenger trains. The bridge consisted of an un-ballasted steel truss bridge, which was designed according to type F46, corresponding to 12 axles with 25 tons axle loads at 1.6 m spacing representing the locomotive and a distributed load of 85 kN/m representing the wagons (Trafikverket, 2010). Girders and connections in the bridge were partially riveted and partially welded.

Since the Åby bridge was constructed there has been work in verifying the load capacity of the bridge, where the latest assessment was performed in 1994. The code based assessment proved the bridge to be overdue with regard to fatigue. According to the report from 1994, it was the overlapping continuous plate in the connection between longitudinal stringers and crossbeams that had insufficient capacity. There have however not been any visible cracks detected in this location.

In 1997 there was an assessment evaluation performed on the Rautasjokk Bridge, which in principle is identical to the Åby bridge. The difference between the two bridges was that the Rautasjokk Bridge was located along the Ore line, and therefore subdued to higher traffic loads. This assessment reached another conclusion compared to the work performed on the Åby bridge. The report states that the detail where the junction plate for the horizontal bracing is welded toward the bottom chord is the limiting detail. The bridge over Rautasjokk is still in use opposite the Åby bridge which was taken out of service in the autumn of 2012 and replaced by a new steel trough.

A more recent study on the Rautasjokk Bridge (Häggström, 2014) removes the doubts mentioned from the reports produced in 1994 and 1997, but instead identifies the connection between the stringer and the horizontal bracing as a critical detail with regard to fatigue. The way the previously identified doubts were removed was by using rain flow summation for damage accumulation. It was proved that the number of high stress cycles for uniformed loaded trains where much fewer than
what was accounted for in the previous assessment, since the structural members are not unloaded between axles as might be expected.

### 2.2 Geometry
The Åby bridge consisted of a 33m long, 4.9m high and 5.5m wide un-ballasted open steel truss Railway Bridge. An overview from the original drawings can be seen in Figure 2-2 with the corresponding sections and steel grade in Figure 2-3. The rail consisted of SJ50 rail attached to wooden crossties which rested on top of longitudinal stringers. The stringers were attached to the crossbeams in a semi-rigid connection with a riveted endplate and an overlapping welded continuous plate, seen in Figure 2-4. The span length of the stringers was 4.125m and the center distance between the longitudinal stringers was 1.9m.

Live load that acted on the rail was distributed by cross-ties to the stringers which carried the load crossbeam and further to main truss. Besides the vertical load carrying system there were horizontal bracing of both the main truss as well as the stringer beams., the complete set of drawings are found in Annex A – Drawings.
Figure 2-3 Sections used in the for the Åby Bridge

Figure 2-4 Connection between stringers and crossbeams
2.3 Material

The structural steel consisted of quality S1311 and S1411. Stringers of Dimel 55 sections, diagonals and verticals out of 42½ sections where made out of S1311. The top and bottom chords consisting of welded sections with dimensions according to Figure 2-3 and crossbeams consisting of DIP80 were made of S1411. After the Åby Bridge was taken out of service, material tests were carried out on three different non-damaged members, where samples were taken from a stringer, a crossbeam and from the bottom chord of the main truss. From these specimens three different tests were carried out; tensile tests, Charpy V-notch tests and fracture mechanical testing.

For the tensile tests, three specimens where tested for each member. The tensile testing proved homogeneous results with yield strength well above design values for all specimens.

The Charpy-V tests consisted of five specimens for each member. The tests indicated large variation in measured results. For the main truss all 5 specimens resulted in absorbed fracture energy lower than 27J which is the regulated material requirement for bridges, normally graded J2. For the stringer there were 4 out of 5 specimens above 27J. The results from the crossbeam proved fairly small variation with all specimens above the required level.

The fracture mechanical testing was carried out according to BVS583.12 (Banverket, 2005) with three specimens from each member. For the tests on the main truss one specimen did not reach 40kN/m which is the requirement of for S1411 steel while the other two specimens proved a more ductile behaviour. With regard to the varying results where one of the specimens doesn’t meet the requirement it is advised to reduce the allowed yield strength by 30% (Eriksson, 2014).

The Stringer beam consists of S1311 steel where the required Jc is set to 30kN/m. All specimens had a significantly higher capacity.

The crossbeams have the lowest mean value of Jc among the tested members and also the smallest variation. There is one specimen not reaching the requirement of 40kN/m for S1411. Recommendations in (Eriksson, 2014) suggest reducing the allowed stress by 10%.

A summary of the results from the material tests are presented in Table 2-1 which are more thoroughly described in in Appendix B.

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Stringer</th>
<th>Bottom chord</th>
<th>Crossbeam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>Div.</td>
<td>Mean</td>
</tr>
<tr>
<td>Tensile (yielding) [MPa]</td>
<td>304</td>
<td>1.9 %</td>
<td>334</td>
</tr>
<tr>
<td>Charpy V [J]</td>
<td>66.5</td>
<td>56 %</td>
<td>17.8</td>
</tr>
<tr>
<td>Fracture mechanical [kNmm]</td>
<td>291</td>
<td>94 %</td>
<td>327</td>
</tr>
</tbody>
</table>
3. MEASUREMENT PROGRAM

As presented in section 1.3, the research project consisted of three Phases, where the first two were carried out on the Åby Bridge. Phase I for live loading and Phase II for static loading when the bridge had been taken out of service, including loading to failure. Phase III was carried out on the Rautasjokk bridge for live loading.

The Initial measurement program for Phase I was based on the pilot study that is presented in (Blanksvärd, 2012) where the aims are presented in section 1.3.1. The extended measurement program for Phase II was built on the aims but there were better access due to the bridge being taken out of service as well as additional input from the measurements being performed in Phase I.

Some examples in sensor positions to reach the desired goals were.

Measurement of:

- Strains in the stringers, to investigate the real fatigue stresses for passing trains.
- Vertical displacement, to verify numerical simulations and investigate constraints.
- Rotation in connections to investigate the level of continuity in the stringers.
- Lateral displacement of the top chord, to investigate non-linear behaviour when loaded.

In Phase III, some sensor positions remained from Phase I and Phase II in order to verify the same behaviour between the two bridged. The code-based assessment of the bridge (Häggström, 2014) indicated that there were fatigue sensitive details (stringer beams where the gusset plate for the bracing was welded to the top flange) which had code based cumulative fatigue damage beyond 1.0. These details were monitored using a local approach (Hot-spots) in order to investigate the geometric stress-raising effect more thoroughly. Strain measurements were also carried out on the web of the stringers in order to capture the nominal strain. A comparison between nominal strain and hot-spot strain is presented in (Häggström 2016).

3.1 Initial estimation of capacity before testing

An initial estimation of the capacity for the Åby Bridge was performed by FEM-simulations. One model was created before the testing in Phase I and another before Phase II. Since the real material properties were not tested at this time, the material properties were taken from codes. After the material tests were performed, the second model was updated. More information on the modeling can be found in chapter 4.

3.2 Instrumentation

During Phase I totally 55 sensors were used, divided between 14 LVDTs (Linear Voltage Displacement Transducer) monitoring displacements of the main truss, 8 strain gauges on the main truss and 30 strain gauges on the load distributing system (stringer beams and crossbeams) and 3 sensors registering temperature. The measurements in Phase I took place between the 29th of August and the 13th of September 2012. The complete installation of strain gauges was not completed when measurement stared, so there are only complete results for the strain measurements between 1th and 12th of September.

For Phase II the monitoring was extended to 122 sensors, with 16 LVDTs on the main truss, 29 LVDTs on the load distributing system, 24 strain gauges on the main truss and 47 strain gauges on the local
load distributing system as well as 6 temperature sensors registering temperature. All sensors above the track were added for Phase II, since these were easier to access when the bridge was taken out of service. There were also additional LVDT’s (Linear Voltage Displacement Transducer) added, since the ground could be used as a fixed reference for measuring displacements. Besides the added sensors in Phase II, some of the sensors from Phase I was replaced since they had stopped working.

In Phase III totally 71 strain gauges were used to monitor the bridge for 526 passing trains out of which 130 consisted of loaded trains with iron ore. The sensor configuration was made so that confirmation of the earlier findings from the Åby Bridge could be verified as well as measurements on critical details with regard to fatigue. The results from Phase III are not covered in this report.

During all Phases of measurement, the sensors were connected by wire to an MGC (a data acquisition system) where the data was collected and stored on a computer.

3.3 Overview

In Figure 3-1 and Figure 3-2 as well as Table 3-1 to Table 3-5 are all sensors in Phase I and Phase II listed with its individual position. In the tables it is possible to make out in which Phase the sensor was present. The sensor number listed in the tables refers to the raw data files which were logged using hardware from HMB and the software Catman professional, where each column represents one channel/sensor. The type of sensor used is presented more in detail in chapter 3.4.

In Figure 3-3 together with Table 3-6 are the sensor positions presented and described for Phase III
Figure 3.1 Sensors placed on the main truss.
Table 3-1 LVDT’s on the main truss

<table>
<thead>
<tr>
<th>Sensor ID</th>
<th>Units</th>
<th>Description</th>
<th>In place in Phase I</th>
<th>Sensor nr.</th>
<th>In place in Phase II</th>
<th>Sensor nr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>LQ1</td>
<td>mm</td>
<td>LVDT, Quarter point vertical displacement (S)</td>
<td>Yes</td>
<td>24</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td>LQ2</td>
<td>mm</td>
<td>LVDT, Quarter point vertical displacement (N)</td>
<td>Yes</td>
<td>26</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td>LFSR1</td>
<td>mm</td>
<td>LVDT, Rotation at fixed support (N)</td>
<td>Yes</td>
<td>30,31</td>
<td>Yes</td>
<td>82,83 (138)</td>
</tr>
<tr>
<td>LFSR2</td>
<td>mm</td>
<td>LVDT, Rotation at fixed support (S)</td>
<td>Yes</td>
<td>32,33</td>
<td>Yes</td>
<td>86,87 (139)</td>
</tr>
<tr>
<td>LFS1</td>
<td>mm</td>
<td>LVDT, vertical displacement at fixed support (S)</td>
<td>Yes</td>
<td>50</td>
<td>Yes</td>
<td>84</td>
</tr>
<tr>
<td>LFS2</td>
<td>mm</td>
<td>LVDT, vertical displacement at fixed support (N)</td>
<td>Yes</td>
<td>51</td>
<td>Yes</td>
<td>88</td>
</tr>
<tr>
<td>LM1</td>
<td>mm</td>
<td>LVDT, vertical displacement at mid-span (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>116</td>
</tr>
<tr>
<td>LM2</td>
<td>mm</td>
<td>LVDT, vertical displacement at mid-span (N)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>117</td>
</tr>
<tr>
<td>LHTC1</td>
<td>mm</td>
<td>LVDT, horizontal displacement Top chord support (N)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>77</td>
</tr>
<tr>
<td>LHTC2</td>
<td>mm</td>
<td>LVDT, horizontal displacement Top chord mid-span (N)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>82</td>
</tr>
<tr>
<td>LSS1</td>
<td>mm</td>
<td>LVDT, vertical displacement at sliding support (S)</td>
<td>Yes</td>
<td>22</td>
<td>Yes</td>
<td>76</td>
</tr>
<tr>
<td>LSS2</td>
<td>mm</td>
<td>LVDT, vertical displacement at sliding support (N)</td>
<td>Yes</td>
<td>23</td>
<td>Yes</td>
<td>80</td>
</tr>
<tr>
<td>LSSR1</td>
<td>mm</td>
<td>LVDT, Rotation at fixed support (S)</td>
<td>Yes</td>
<td>18,19</td>
<td>Yes</td>
<td>74,75 (140)</td>
</tr>
<tr>
<td>LSSR2</td>
<td>mm</td>
<td>LVDT, Rotation at fixed support (N)</td>
<td>Yes</td>
<td>20,21</td>
<td>Yes</td>
<td>78,79 (141)</td>
</tr>
</tbody>
</table>

Table 3-2 Strain-gauges on the main truss

<table>
<thead>
<tr>
<th>Sensor ID</th>
<th>Units</th>
<th>Description</th>
<th>In place in Phase I</th>
<th>Sensor nr.</th>
<th>In place in Phase II</th>
<th>Sensor nr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>RFD1</td>
<td>µm/m</td>
<td>Rosette SG, first diagonal (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>54,55,56</td>
</tr>
<tr>
<td>RFV1</td>
<td>µm/m</td>
<td>Rosette SG, first vertical (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>45,46,47</td>
</tr>
<tr>
<td>RFV2</td>
<td>µm/m</td>
<td>Rosette SG, first joint top chord (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>42,43,44</td>
</tr>
<tr>
<td>SFVB1</td>
<td>µm/m</td>
<td>SG, First joint bottom chord (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>68</td>
</tr>
<tr>
<td>SQB1</td>
<td>µm/m</td>
<td>SG, quarter point bottom chord (S)</td>
<td>Yes</td>
<td>2,3</td>
<td>Yes</td>
<td>38-39</td>
</tr>
<tr>
<td>SQB2</td>
<td>µm/m</td>
<td>SG, quarter point bottom chord (N)</td>
<td>Yes</td>
<td>4,5</td>
<td>Yes</td>
<td>69,67</td>
</tr>
<tr>
<td>SQV1</td>
<td>µm/m</td>
<td>SG, quarter point vertical(S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>40</td>
</tr>
<tr>
<td>SQT1</td>
<td>µm/m</td>
<td>SG, quarter point, top chord at (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>41</td>
</tr>
<tr>
<td>SMB1</td>
<td>µm/m</td>
<td>SG, mid-span on the lower chord (S)</td>
<td>Yes</td>
<td>34,35</td>
<td>Yes</td>
<td>48,49</td>
</tr>
<tr>
<td>SMB2</td>
<td>µm/m</td>
<td>SG, mid-span on the lower chord (N)</td>
<td>Yes</td>
<td>36,37</td>
<td>Yes</td>
<td>33,66</td>
</tr>
<tr>
<td>SMV1</td>
<td>µm/m</td>
<td>SG, mid-span vertical (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>50</td>
</tr>
<tr>
<td>SMT1</td>
<td>µm/m</td>
<td>SG, west of mid-span on the top chord (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>51</td>
</tr>
<tr>
<td>SMT2</td>
<td>µm/m</td>
<td>SG, mid-span on the top chord (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>52</td>
</tr>
<tr>
<td>SMT3</td>
<td>µm/m</td>
<td>SG, east of mid-span on the top chord (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>53</td>
</tr>
</tbody>
</table>

Table 3-3 Temperature sensors for Phase I and Phase II

<table>
<thead>
<tr>
<th>Sensor ID Phase I</th>
<th>Units</th>
<th>Description</th>
<th>Sensor nr.</th>
<th>Sensor ID Phase II</th>
<th>Units</th>
<th>Description</th>
<th>Sensor nr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1.1</td>
<td>°C</td>
<td>Measuring the temperature in the hut where the data were collected</td>
<td>74</td>
<td>T1.2</td>
<td>°C</td>
<td>Temperature of bottom chord at mid-span (S)</td>
<td>122</td>
</tr>
<tr>
<td>T2.1</td>
<td>°C</td>
<td>Placed on the inside of the first vertical towards the west, on the northern side of the bridge at a height of approximately 1200mm.</td>
<td>75</td>
<td>T2.2</td>
<td>°C</td>
<td>Temperature of bottom chord at mid-span (N)</td>
<td>123</td>
</tr>
<tr>
<td>T3.1</td>
<td>°C</td>
<td>Placed underneath the bridge at the same location as T2</td>
<td>76</td>
<td>T3.2</td>
<td>°C</td>
<td>Temperature of top chord at mid-span (N)</td>
<td>124</td>
</tr>
<tr>
<td>T4.2</td>
<td>°C</td>
<td>Temperature of top chord at mid-span (S)</td>
<td>125</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T5.2</td>
<td>°C</td>
<td>Outside air temperature</td>
<td>126</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T6.2</td>
<td>°C</td>
<td>Air temperature inside the measurement container</td>
<td>129</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 3-2 Sensors placed on the load distributing members
### Table 3-4 Strain gauges on the local load-distributing system

<table>
<thead>
<tr>
<th>Sensor ID</th>
<th>Units</th>
<th>Description</th>
<th>In place in Phase I</th>
<th>Sensor nr.</th>
<th>In place in Phase II</th>
<th>Sensor nr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFC1</td>
<td>µm/m</td>
<td>SG, Crossbeam close to fixed support (S)</td>
<td>Yes</td>
<td>10-13</td>
<td>Yes</td>
<td>2 – 5</td>
</tr>
<tr>
<td>SFC2</td>
<td>µm/m</td>
<td>SG, Crossbeam close to fixed support (mid)</td>
<td>Yes (2/4)</td>
<td>4,5</td>
<td>Yes</td>
<td>6 – 9</td>
</tr>
<tr>
<td>SFC3</td>
<td>µm/m</td>
<td>SG, Crossbeam close to fixed support (N)</td>
<td>Yes</td>
<td>6-9</td>
<td>Yes</td>
<td>14 – 17</td>
</tr>
<tr>
<td>SMS1</td>
<td>µm/m</td>
<td>SG, Stringer close to mid-span (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>62,63</td>
</tr>
<tr>
<td>SMS2</td>
<td>µm/m</td>
<td>SG, Stringer close to mid-span (N)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>64,65</td>
</tr>
<tr>
<td>SMS3</td>
<td>µm/m</td>
<td>SG, Stringer close to mid-span (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>58,59</td>
</tr>
<tr>
<td>SMS4</td>
<td>µm/m</td>
<td>SG, Stringer close to mid-span (N)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>60,61</td>
</tr>
<tr>
<td>SFS1</td>
<td>µm/m</td>
<td>SG, Stringer end-span fixed support (S)</td>
<td>Yes</td>
<td>38,39,59,58</td>
<td>Yes</td>
<td>10 – 13</td>
</tr>
<tr>
<td>SFS2</td>
<td>µm/m</td>
<td>SG, Stringer end-span fixed support (S)</td>
<td>Yes</td>
<td>36,37,60,61</td>
<td>Yes</td>
<td>18 – 21</td>
</tr>
<tr>
<td>SFS3</td>
<td>µm/m</td>
<td>SG, Stringer end-span fixed support (S)</td>
<td>Yes</td>
<td>62,63,49,48</td>
<td>Yes</td>
<td>34 – 37,26</td>
</tr>
<tr>
<td>SFS4</td>
<td>µm/m</td>
<td>SG, Stringer end-span fixed support (S)</td>
<td>Yes</td>
<td>16,17,44,42</td>
<td>Yes</td>
<td>27 – 31</td>
</tr>
<tr>
<td>SFS5</td>
<td>µm/m</td>
<td>SG, Stringer end-span fixed support (S)</td>
<td>Yes</td>
<td>14,15,46,47</td>
<td>Yes</td>
<td>22 – 25</td>
</tr>
<tr>
<td>...</td>
<td>µm/m</td>
<td>Photometric measurements using the ARAMIS system</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>Photometric</td>
</tr>
</tbody>
</table>

### Table 3-5 LVDT’s on the local load-distributing system

<table>
<thead>
<tr>
<th>Sensor ID</th>
<th>Units</th>
<th>Description</th>
<th>In place in Phase I</th>
<th>Sensor nr.</th>
<th>In place in Phase II</th>
<th>Sensor nr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSC1</td>
<td>mm</td>
<td>LVDT, Displacement at second crossbeam (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>90-93</td>
</tr>
<tr>
<td>LSC3</td>
<td>mm</td>
<td>LVDT, Displacement at second crossbeam (N)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>94-97</td>
</tr>
<tr>
<td>LCSC1</td>
<td>mm</td>
<td>LVDT, COD connection second crossbeam (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>110-113</td>
</tr>
<tr>
<td>LSF1</td>
<td>mm</td>
<td>LVDT, Fist stringerbeam (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>106</td>
</tr>
<tr>
<td>LSF2</td>
<td>mm</td>
<td>LVDT, Fist stringerbeam (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>107</td>
</tr>
<tr>
<td>LSF3</td>
<td>mm</td>
<td>LVDT, Fist stringerbeam (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>108</td>
</tr>
<tr>
<td>LSF4</td>
<td>mm</td>
<td>LVDT, Fist stringerbeam (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>109</td>
</tr>
<tr>
<td>LSF5</td>
<td>mm</td>
<td>LVDT, Fist stringerbeam (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>89</td>
</tr>
<tr>
<td>LMS1</td>
<td>mm</td>
<td>LVDT, Mid-span Stringer to the west (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>119</td>
</tr>
<tr>
<td>LMS2</td>
<td>mm</td>
<td>LVDT, Mid-span Stringer to the East (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>135</td>
</tr>
<tr>
<td>LFC1</td>
<td>mm</td>
<td>LVDT, vertical Displacement first crossbeam (S)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>98-101</td>
</tr>
<tr>
<td>LFC3</td>
<td>mm</td>
<td>LVDT, vertical Displacement first crossbeam (N)</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>102-105</td>
</tr>
<tr>
<td>LMC1</td>
<td>mm</td>
<td>LVDT, vertical displacement crossbeam mid-span</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>118</td>
</tr>
<tr>
<td>LMC2</td>
<td>mm</td>
<td>LVDT, vertical displacement crossbeam east of mid-span</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>134</td>
</tr>
</tbody>
</table>
Figure 3-3 Sensor positions on the Rautasjokk bridge

Table 3-6 Description of sensors for measurements in Phase III

<table>
<thead>
<tr>
<th>Sensor ID</th>
<th>Units</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDL1&amp;2</td>
<td>µm/m</td>
<td>Strain measurements under the bottom flange of the stringer brams</td>
</tr>
<tr>
<td>SFS3</td>
<td>µm/m</td>
<td>Same position as for Phase I and Phase II</td>
</tr>
<tr>
<td>SHL</td>
<td>µm/m</td>
<td>Strain measurements for local fatigue analysis of stringers</td>
</tr>
<tr>
<td>SNL</td>
<td>µm/m</td>
<td>Strain measurements for evaluation of nominal strains in stringer</td>
</tr>
<tr>
<td>SQB</td>
<td>µm/m</td>
<td>Same position as for Phase I and Phase II</td>
</tr>
<tr>
<td>SUC</td>
<td>µm/m</td>
<td>Strain gauge on bottom chord between crossbeams</td>
</tr>
<tr>
<td>TMP1</td>
<td>°C</td>
<td>Temperature on steel, bottom chord, sun</td>
</tr>
<tr>
<td>TMP2</td>
<td>°C</td>
<td>Temperature on steel, on stringer, shade</td>
</tr>
<tr>
<td>TMP3</td>
<td>°C</td>
<td>Temperature air, center of bridge</td>
</tr>
</tbody>
</table>
3.4 Sensors

Over the three phases of measurements on the Åby Bridge and the Rautasjokk Bridge, different sensors were used. In Phase I and Phase II, strain gauges were used to monitor the stress state, LVDTs (Linear variable differential transformer) were used to measure displacement, temperature sensors were used to measure temperature and accelerometers were used to provide acceleration data. In Phase II, a Digital Image Correlation system (GOM, 2016) was also used for strain measurements over a surface. Measurements in Phase III were limited to those from strain gauges and temperature sensors.

3.4.1 Displacement Measurement

For displacement measurements, LVDT sensors were used. Different stroke lengths were used at different locations depending on the expected displacement. The bridge consisted of one span, so settlement does not necessarily cause stress. In order to compensate for this, settlement was monitored for Phase II.

3.4.1 Strain measurements

It is not possible to directly measure stress in steel members in a simple non-destructive manner. With known material properties, stresses can be derived by measuring strain at least for loads within the linear elastic range. When performing measurements on existing structures, strain gauges can generally not register absolute levels of strain, but rather the changes in strain as the structure is loaded or unloaded. Strains due to dead-load are therefore difficult to capture. Temperature will also cause strain fluctuations. One way of controlling strains arising from temperature changes is by normalizing the sensors for a load sequence, given that the sequence is short enough for the temperature to not have any major influence. If there are greater changes in temperature during a loading sequence, measured results should be adjusted for this. One way of keeping track of the temperature is the use of temperature sensors. Temperature effects can be canceled out by using a second strain gage is placed in close thermal contact with structure but is not bonded to the structure.

For the strain measurements on the Åby Bridge, welded strain gauges were used. After grinding of the coating and cleaning the surface, the strain gauges were welded to the steel by a pressure activated hand peace. The majority of the strain gauges used during Phase I and Phase II consisted of one directional quarter bridge. During Phase II there were also Rosette strain gauges, which measures the strain in three directions 45 degrees appart, so that the principle strain can be derived.

3.4.1 Temperature sensors

Changes in temperatures causes material to elongate or contract. These effect can typically be seen for strain gauges in long term field measurement. Changes in temperature does not necessarily cause stresses, unless the structure is prevented to move, or due to different temperatures in different parts of a bridge. Temperature sensors have been used during all three phases of measurements. The sensors were bonded to different positions of the bridge, in order to capture variations of temperature.
3.4.2 Digital Image Correlation
During Phase II a photometric strain measurement system was used to measure strains over a specified area, see Figure 3-4. Measurements was performed on a connection between longitudinal stringer beams and crossbeams with the same purpose as the LCSC-sensors, to determine to what degree the stringer beams acts as pinned or continuous beams in the connection with the crossbeams. The photometric strain measurements are further described in (Elhag, 2014).

Figure 3-4 Upper left: The surface is being prepared. Upper Right: The Aramis system Lower Left: Position for measurement. Lower Right: The position for measurement after completing the final test.

3.4.3 Accelerometers
Both uniaxial and tri-axial accelerometers where used for the acceleration measurements. In addition to the sensors was a load exciter consisting of a rotating mass was used to induce vibrations, for Phase II, (Nuno, 2013).

3.5 Loading
3.5.1 Reference train for live measurements (Phase I)
For Phase I, when the bridge was still in service, the loading consisted of real live trains. During the time of measurements a total of 399 trains was recorded over a period of approximately two weeks. The loading from passing trains could be extracted from the monitoring station located in Jörn. Since the work with replacing the old bridge took place at the same time the speed of the trains were limited. The allowed speed on the line was limited to 20km/h instead of 70km/h which is the normal speed limit on the route, of that reason the full dynamic response on the bridge is not possible to evaluate.
Bridge over Åby River - Evaluation of full scale testing

For the presented results in this report, one specific train set has been chosen to be displayed. The ID of this specific train is Re 1423 with the axle configuration displayed in Figure 3-5. The reference train was going south along the main line at the time it was passing the bridge (passing the bridge from east to west).

Figure 3-5 The locomotive pulling steel slabs and it’s axle distribution (Green Cargo).

The vehicle was identified with its specific number which was verified with the help of a video camera filming the passing trains. In Table 3-7 is the weight of each bogie presented. The skewness in loading within the bogie both in the longitudinal and transverse direction is measured. In Figure 3.6 is the axle loads plotted with its corresponding position. Based on the data-log the train passed Jörn at 15:19 2012-09-03 and according to the video-log passed the bridge at 14.28. The train can be seen in Figure 3-5 as it passed the Åby Bridge. The cargo of the train set consisted of steel-slabs where these particular train sets are referred to as “stål-pendeln (The steel commute)” in Swedish. The number of slabs varies from 2 to 4 steel slabs (as seen in Figure 3-6) and therefore also the weight, as seen by Figure 3-7.

The wagons assumed for the reference train consists of “Smmnps 13” which is used by Green cargo for transporting steel slabs. The wagon is 13.9 m long with a bogie-distance of 1.8 m and a centrum-distance between bogies of 8.6 m as shown in Figure 3-6.

Figure 3-6 Smmnps 13, used by the reference train for transporting steel slabs (Green Cargo). To the right are frames from the video recording indicating that the wagons were loaded with 2-4 slabs per wagon.

The speed of the train was not directly measured, but knowing the length of the train it is possible to estimate the speed.

The passage took 46 seconds and the length of the train set was estimated to 524 meters, which gives a mean velocity of about 41 km/h. The passage is the 110th out of 399 trains passing the bridge during the time for the measurements.
### Table 3-7 Axle loads for the reference train at the weight station in Jörn

<table>
<thead>
<tr>
<th>Vehicle number</th>
<th>Axles</th>
<th>Total load</th>
<th>skewed loading</th>
<th>skewed loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Front/Back</td>
<td>Left/Right</td>
</tr>
<tr>
<td>1 RE1423 (Train)</td>
<td>1-4</td>
<td>66.2</td>
<td>0.988</td>
<td>1.293</td>
</tr>
<tr>
<td>2 RE1428 (Train)</td>
<td>5-8</td>
<td>66.9</td>
<td>0.991</td>
<td>1.32</td>
</tr>
<tr>
<td>3 837447215224</td>
<td>9-12</td>
<td>61.2</td>
<td>1.013</td>
<td>1.32</td>
</tr>
<tr>
<td>4 837447215133</td>
<td>13-16</td>
<td>78.3</td>
<td>1.034</td>
<td>1.128</td>
</tr>
<tr>
<td>5 837447216388</td>
<td>17-20</td>
<td>70.5</td>
<td>0.958</td>
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Mean: 71.9 | 1.0 | 1.3 [tons]

Figure 3-7: The load geometry of the reference train (locomotive to the left)
3.5.2 Static loading in Phase II

In Phase II, the bridge was taken out of service and put beside the track on temporary supports. Here the bridge was loaded using hydraulic jacks, where the load was applied through distribution beams simulating a train wagon. Steel tendons were anchored in the bedrock and jacks were used to pull the bridge downwards. In comparison to Phase I the load is purely static and manually controlled and in a fixed position as displayed in Figure 3-8 & Figure 3-9. The registered applied force from the jacks was calculated as the oil-pressure in the cylinder multiplied by the area of the cylinder. The loading position was initially planned to be symmetric around the center of the bridge, but drilled holes ended up slightly displaced which led to the final loading position.

Figure 3-8 Loading during Phase II from a 2D perspective. The western side is to the left and the eastern side to the right.

Figure 3-9 Left: The steel cables attached to the hydraulic jack. Middle: The load distributing beams Right: Compressor supplying pressure to the jacks.

The load distributing beams were specially made for the purpose of testing the Åby bridge. The beams consisted of steel plates assembled to a welded girder. The design was made by Clas Fahleson at Fahleson & Co AB and designed for 1000 tones (10MN) each. The manufacturing drawings for the beams are presented in Figure 3-10.

The testing in Phase II was carried out in 2 different stages with a total of 18 load scenarios. For the first 15 load scenarios it was tested with the rail still in place, then the rail was removed and it was further tested without the rail for the last 3 load scenarios. In the last (18th) load scenario, the bridge was loaded to failure. Figure 3-11 shows the load-series for all these stages where the red markers indicate the instances when results have been extracted in order to be presented in section 5.

The first eight load scenarios were carried out August 28, 2013. The sequence consisted of 4 similar scenarios which were performed two times in order to identify deviations in results and have the initial settlements acted out. The load was applied and then remained constant for 10 minutes before unloading; the sequence was repeated three times for each load scenario. This procedure was carried out for the 8 load scenarios with loads of: 1000kN, 1320kN, 1600kN and 1800kN as seen
in Figure 3-11. Load scenarios 9-15 were tested the following day, August 29 2013. The last three load scenarios were tested on the 12th of September where the final test to failure ended around 13.00.

During the final test to failure it appears as there is an over pressure in the jacks causing the calculated force to start at 600-800kN. This effect is believed to be caused by the resistance of moving the hydraulic cylinder, which is not present for the other tests since these are taken at a steady state of loading. This is an effect which becomes visible if trying to fit the data to a linear response within the service loading-range. The measured response representing the total force has therefore been decreased by 600kN. The distribution in loading between the two jacks for the final test can be seen in Figure 3-12. As seen are the pressure kept at an equal level during the testing to failure as well as for the non-destructive pretesting. Some deviations occur due to manual control of the hydraulic equipment, as can be seen by the lines not being entirely on top of each other.

For both the pre-loading sequences and the final test to failure, the raw data was normalized to the mean value of the first 20 readings. This applies to strain gauges and LVDTs, meaning that the
temperature and pressure-sensors has not been normalized. The reason for using the first 20 measurements is to reduce errors which might be caused by wind, vibrations or similar.

The results for Phase II are presented as a function of the total load rather than time as in Phase I. During the final test to failure was the stroke of the jacks insufficient to bring the bridge to failure, therefore the stroke had to be reset several times in order to bring the bridge to failure. Since the load was calculated based on the oil-pressure in the jacks, a dip will appears in the measured response while resetting the stroke of the jacks – despite the force remained constant during this time. In order to create readable figures, the time for resetting, has been cut out from the results - as seen in Figure 3-12.

![Figure 3-12](image)

Figure 3-12 Left: Distribution of force between the two hydraulic jacks. The total load is displayed on the y-axis and the load of from the individual jack on the x-axis. Right: Cutouts due to resetting of hydraulic jacks, the red markings indicate the used results.

### 3.5.3 Live Loading in Phase III

The Rautasjokk Bridge was monitored as normal traffic was acting on the bridge. During the measurements a total of 526 train-sets were monitored out of which 130 were loaded iron ore trains going north. A comparison of load effects between Åby Bridge and Rautasjokk Bridge is presented in section 5.2. Compared to the Åby Bridge, the speed on the line was not limited, Figure 3-13 describes the speed for the monitored train-sets. The first peak around 60km/h represents the iron ore-trains.

![Figure 3-13](image)

Figure 3-13 Speed of train-sets during measurements in Phase III
4. FEM (Finite element modeling)
The work with the Finite Element Modeling has been an ongoing process during the project. During the pilot study Phase I, a model was developed by using the Ansys software together with adjunct Professor Mikael Möller. For Phase II another model was developed using the Abaqus software together with Professor Yongming Tu.

4.1 Ansys Model (Pre Phase I)
For Phase I a shell-element model was created using the Ansys software (Figure 4-1). The bridge was subjected to tree stages of loading. The first stage included dead load and live-loading corresponding to BV-3. The second stage consisted only of the dead load and the third stage was performed as a limit stage analysis where the objective was to find the load which would cause parts of the bridge to collapse.

The model was created using four nodal shell elements, consisting of a total of 43,000 nodes causing 260,000 degrees of freedom. Young modulus of elasticity is set to 210GPa and Poisson ratio is set to 0.3. The model is based on the drawings (Annex A – Drawings) of the bridge with minor simplifications. For example are small eccentricities disregarded in the connection in the bottom flange, Welds and rivets are not modeled but connected as if they were butt welded. The horizontal wind truss has not been included in the model with the argument that it does not contribute to the global load carrying capacity for trainloads neither is the cross ties or rail which might assist in distributing the load further. The model is made for ¼ of the bridge where double symmetry is applied (Blanksvärd, 2014).

Evaluation of the simulated results are presented in 6.1.

Figure 4-1 FE Model created prior to the Phase I
4.2 Abaqus Model

For Phase II a shell-element model was developed using the Abaqus software. Design models are often simplified in a conservative manner in order to deliver fast but still reliable results. In this case the model has been thoroughly modeled with details taken into consideration which normally might be disregarded. All connections (riveted and welded) have been assumed as fixed, this also includes the stringer to crossbeam connection. Therefore, all connections between shells have been considered as fixed. The cross-ties and the rail were not implemented in the model, meaning that the loads are applied directly upon the stringer beams. However, their mass was added by using non-structural-mass elements. The supports were modeled according to the configuration for the bearings, with one side fixed in the longitudinal direction and the other sliding disregarding friction, with both sides hinged. No springs were used for vertical stiffness.

The model consisted of 30474 elements and 21225 nodes creating 128 000 degrees of freedom. Initially, the material properties applied were taken from codes but were later updated based on material tests from the Åby bridge described in Annex B – Material testing.

The bridge has been subjected to static loading according to the test setup in Phase II, and has also been tested for moving loads in order to create influence lines which are further described in 0.

Figure 4-2 Abaqus shell model of Åby bridge. Upper left: Top-view. Upper Right: Side view. Lower left: 3D Section view. Lower Right: Overview

Figure 4-3 Meshing of the model
4.2.1 Updating of FE-model (Post Phase II)

For the update of the FE-model were only the material properties adjusted. The mean stress/strain curve was simply applied as a material that includes strain hardening which was neglected in the earlier model.

Since the material tests proved higher yield strength than the initial model, a higher load carrying capacity was achieved. With revised material properties a non-linear behaviour of the displacement of the main truss occurred about 9.0MN and 68mm of deflection, before having small non-linearity where the peak load ended up 11.5MN and 90mm of deflection as seen in Figure 5-5.
5. RESULTS FROM MEASUREMENTS
In this chapter the results from measurements in Phase I and Phase II are presented. The results are divided on strain measurements and displacement measurements on the main truss as well as the load distributing system respectively.

For Phase I focus has been placed on the reference train, described in 3.5.1. Measured results are normalized for the specific train passage and presented as a function of time.

For Phase II the results are presented as a function of the total load instead of as a function of time. The results cover both pre-testing with and without the rail in place as well as the result from the final test to failure.

5.1 Introduction
The results from the reference train were sampled at 50Hz, meaning that the train moves approximately ¼ of a meter between every reading. The different train-sets passing the bridge during the time of measurement besides the reference train are presented in (Moreno, 2013).

For a few sensors during Phase II are also the results from the numerical simulation are presented, where the model is described in chapter 4.2.

Sensor positions are described for each section, but an overview of all sensors can be seen in Figure 3-1 and Figure 3-2.

During the time of a train passage, the change in temperature has a negligible effect unless there are built in stresses which are released as a train pass, so called slips, for example friction in a bearing which is exceeded for a train passage. This effect can be seen for some sensors for the reference train, since the response is not returning to zero after passage of the train set.

5.2 Classification of traffic
By using a reference train with known axle loads it was possible analyze the structural response related to that specific load. In order to assess bridges, it is often important to consider the variation in loading both with regard to ultimate limit state and fatigue limit state. This is generally done in a deterministic way for new bridges, but when assessing existing structures it can be based on field measurements.

In the Swedish assessment code for railway bridges this is taken care of by collective parameters which describe the intensity of fatigue related train loads relative to the heaviest train.

The variation can be visualized based on measured values. One way of doing this is by plotting a histogram of the load effects. In Figure 5-1 is the strain width for sensor SQ81_2 (strain at ¼ point) illustrated for all the 399 train sets for the Åby bridge as well as the 526 monitored train passages for the Rautasjökk bridge. The strain width was defined as the maximum strain minus the minimum strain for a single passage. For the reference train the strain width was 108μm/m. It can be noted that there are several passages that indicates a significantly higher response for the Åby bridge. The measurements on the Rautasjökk bridge indicate several passages with strain-widths around 150-190μm/m representing the Iron ore train-sets.
5.3 Displacement measurements

5.3.1 Main truss
LQ-sensors

The LQ – Sensors (LVDTs at Quarterpoint) were located on the main truss 8.25m from the support measuring vertical displacement as seen in Figure 5-2. This sensors were only located on the western side of the bridge (towards Bastuträsk). The LQ-sensors were only present during Phase I.

The LQ-sensors were the only sensors detecting deflection of the main truss apart from the sensors located at the abutment during Phase I. Since they monitor the deflection of the main truss, the influence-length was about the same length as the bridge. A constant response is therefore received with smaller variations based on the positioning of the train set and variations of loads. From the measured displacement presented in Figure 5-3, it is seen that both sides of the bridge deflected...
equally, with a constant deflection of 7mm during the passage and amplitude of about 1-1.5mm dependent on the load position.

![Graph showing vertical displacement of the main truss at quarter point for the reference train.](image)

**Figure 5-3** Vertical displacement of the main truss at quarter point for the reference train.

**LM-Sensors**

The LM – sensors (LVDTs at Mid span) were located at the center of the bridge on the main truss measuring vertical displacement as seen in Figure 5-4. The LM-sensors were only present during Phase II.

![Image showing LM sensors and their positions.](image)

**Figure 5-4** Left: LM 1 (LVDT on main truss at Mid-span) in Phase II, Middle: LM 2 in Phase II. Right: The position of LM1 & LM2

In Figure 5-5 is the deflection at mid span displayed. The readings displayed have been adjusted for settlements at supports. From the diagram it can be seen that a linear behaviour is achieved up to about 7MN of total loading where the deflection-curve deviates, but the loading continues to increase up to about 10-11MN. The FEM-results are also displayed in the figure where the results prior testing as well as the results after upgrading the model with material properties based on testing. It can be seen that the predicted of capacity is increased when the material properties is increased. The results from pre-testing are also presented for the instances described in Figure 3-11, these results correspons well with the behaviour seen for the final test to failure.
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**Figure 5-5** Deflection at mid-span. Results have been adjusted for settlements of supports. The results from the numerical simulation are also presented, both prior and post the material was tested.

**LFSR & LSSR – Sensors**

The LFSR (LVDT at Fixed Support measuring Rotation) and the LSSR (LVDT at Sliding Support measuring Rotation) were sensors positioned at each corner of the bridge measuring longitudinal displacement (Figure 5-6). There were a total of 8 sensors with two sensors at each position.

**Figure 5-6** Position of LFSR & LSSR

In Phase I the L(F/S)SR(1/2)_1 sensor was placed 150 from the top and L(F/S)SR(1/2)_2 100 mm from the bottom which makes them 550mm apart. During Phase II the sensors were 605mm apart except for the LFSR2 sensors which ended up 607mm apart. As can be seen from Figure 5-7 and Figure 5-8 sensors were in Phase I located on the side of the web while as for Phase two they were located at the center of the back plate.

**Figure 5-7** 1st: LFSR 1 (LVDT at Fixed support measuring Rotation) during Phase I. 2nd: LFSR 1 during Phase II. 3rd: LFSR 2 during Phase II. 4th: Sensor positions
In in Figure 5-9 and Figure 5-10 are the readings from the LSSR and LFSR sensors presented for the passage of the reference train. The displacements for the sliding side are as expected higher than the fixed, since the fixed bearings are locked for movement in the longitudinal direction. The rotation at the supports is calculated and presented in Figure 6-43 to Figure 6-46.
In Figure 5-11 and Figure 5-12 are the data from the LVDT’s measuring longitudinal displacement of the bridge. At about 9MN of total loading there is a jump, possible due to cracking of the temporary support.

When the peak load was reached (around 10-11MN) rotation appears to continue to increase despite the load remains constant. It can also be observed that the LSSR2-sensors registered significantly higher displacements compared to LSSR1, for both pre-testing and the final test to failure which was not the case in Phase I.

![Figure 5-11 Horizontal displacement at fixed supports](image1)

![Figure 5-12 Horizontal displacement at sliding supports](image2)
LHTC - sensors

The LHTC-sensors (LVDT measuring Horizontal displacement of the Top Chord) were only present in Phase II and consists of two LVDTs which are attached to columns fixed in ground as seen in Figure 5-13. The sensors were positioned at the first vertical from the eastern support on the northern side and at mid span on the northern side of the bridge at the top chord measuring the horizontal displacement.

Since out of plane buckling of the top chord was expected to cause the structural failure, the transversal displacement was monitored in two locations of the top chord. The measured results as well as the FEM-simulation for these sensors are displayed in Figure 5-14. As can be seen are the behaviour of the chord similar to the simulation, even though the nonlinear behaviour starts earlier for the measured response. It can be observed that as LHTC2 moves inwards to the center of the bridge LHTC1 is pushed outwards. This is further analyzed in the chapter 6.5.1.

Figure 5-14 Horizontal displacement of the top chord together with results from the numerical simulation performed in ABAQUS.
**LFS & LSS – Sensors**

The LFS (LVDT at Fixed support measuring Settlements) and LSS (LVDT at Sliding Support measuring Settlements) were located at each of the supports to measuring settlements. In Phase I (Figure 5-15) were the sensors measuring the settlement of the bridge relative to the abutment and in Phase II (Figure 5-16 and Figure 5-17) relative to the ground. The settlements of the abutments are therefore not registered in Phase I. During Phase I it was not possible to position the sensors at the bearings; therefore the sensors were moved 1000mm towards mid span.
The result from LFS (Lvdt, Fixed Support in vertical direction) and LSS (Lvdt, Sliding Support in vertical direction) sensors are presented in Figure 5-18. These sensors capture the compression of the bearing as well as the deflection due to the sensors was positioned 1 m from the bearing. From the plot below, it can be seen that the sensors capture the global behaviour, based on the response consisting of one long cycle. LFS_2 indicated slightly bigger displacements and fluctuations in comparison with the other symmetrically positioned sensors. From the results, it can be verified that the train first entered the bridge on the side with the sliding bearings since these sensors have an earlier response compared to the fixed ones.

**Figure 5-18 Vertical displacement close to supports for the reference train.**

Due to the bearings being positioned on a temporary foundation, displacements were expected to occur. In Figure 5-19 are the settlements of the bearings during Phase II presented. At 8.5-9MN of total loading it is seen that the vertical displacement is significantly increased, which is believed to be the effect of cracking/crushing of the temporary supports.

**Figure 5-19 Vertical settlements at the fixed supports**
5.3.1 Load distributing system

**LSF-sensors**

The LSF-sensors (LVDT on Stringer (First)) consist of five LVDTs which measured the relative displacement of the stringer beam close to support, where LFS1 were the sensor closest to the support and LFS5 is the furthers from the support (Figure 5-20). The LVDTs were attached to a steel-frame that were anchored to the main truss, meaning that it is the local displacement of the stringer and the crossbeam that is captured and that it is isolated from global deformations of the main truss and settlements. The deformation of the second crossbeam does however influence the results, but can be accounted for since the LSC-sensors were measuring the movement of the member. The LSF-sensors were only present during Phase II.

Figure 5-20 Upper Left: From the right to the left LSF 1-3 (LVDT on the first Stringer close to Fixed support). Upper Right: From the right to the left LSF 3-5 during Phase II. Lower: Position of LSF-sensors.

The sensors are located far from the loading and are not registering the global deflection, therefore were the readings from the sensors very small as seen in Figure 5-21. At around 9.0MN and 10.3MN of total loading for the final test there is flutter in the readings which was likely to be a consequence of the failure of the temporary support.

Figure 5-21 Vertical deflection of the first stringer towards the fixed support
LFC – Sensors

The LFC-sensors (LVDT on Fist Crossbeam) measure displacement of the first crossbeam. There were two sections that were measured; LFC1 and LFC3 as seen in Figure 5-22. The reasons for the litter of the sensors were that they correspond to the position of the SFC-sensors (Strain measurements on First Crossbeam). The LFC-sensors were only present in Phase II whereas the SFS-sensors were present during both Phase I and Phase II. In each section there were four LVDTs, so that both vertical and lateral displacement as well as rotation around its own axle could be evaluated. The LVDTs are mounted on a frame which is attached to the main truss, meaning that the measured response was relative to the main truss. It should be noted that the supports which were underneath the crossbeam were not present during Phase II.

Figure 5-22 Upper 1st: LFC 1 (four sensors) (LVDT at First Crossbeam close to fixed support) during Phase II. Upper 2nd: LFC3 (four sensors) during Phase II Upper 3rd: Supports underneath the first crossbeam during service (Phase I). Upper 4th: Position of the sensor in the cross-section. Middle: overview of the LFC sensors, it can also be seen that the supports underneath the crossbeam are absent. Lower: Position of the sensors
Since the frame was welded to the bridge, LFC1&3_3&4 was supposed to register the actual rotation of the crossbeam isolated from global effect. Based on the results registered by the LFC-sensors (Figure 5-23 and Figure 5-24) and the LSC-sensors (described in the following chapter) it can be seen that the first and the second crossbeam moves towards away from the support. This was believed to be a consequence of the stringers restraining the elongation of the main truss, causing bending in the weak direction of the crossbeams. It was not clear what happened with LFC3_4 for the final test to failure. No rotation of the first crossbeam could be identified, since the load was applied so far from the measured section.
**LSC – Sensors**

The LSC-sensors (LVDT on Second Crossbeam) are similar to the LFC-sensors. The setup consists of four sensors in each point so that both vertical and longitudinal displacement as well as rotation could be determined (Figure 5-25). These sensors were attached to the same frame as the LFC-sensors meaning that it is only the deformation of the crossbeam relative to the main truss that was monitored. The LSC-sensors were only present during Phase II.

![Figure 5-25](image)

*Figure 5-25 Upper left: LSC1 (LVDT on the Second Crossbeam from fixed support) during Phase II. Upper middle: LSC2 during Phase II. Upper right: sensor position at cross-section, Lower Left: The frame underneath the bridge, the attachment to the main truss can be seen to the right in the figure and the vertical member is the same as can be seen in the upper left picture from the other direction. Lower Right: Position of the sensors*

Similar to the LFC-sensors a longitudinal displacement is registered where the bending of the crossbeams in the weak direction could be registered as seen in Figure 5-26 and Figure 5-27. The sensors were mounted from the other direction, which was why there reading had the opposite sign compared to the LFC-sensors. It can be seen that the longitudinal displacement of the second crossbeam is smaller than for the first.
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Figure 5-26 Displacement of the second crossbeam

Figure 5-27 Displacement of the second crossbeam

**LCSC – Sensors**

The LCSC sensors (LVDT Crack opening displacement for connection at Second Crossbeam) consisted of four sensors located in the joint where the stringer beams connected to the second crossbeam, as seen in Figure 5-28. The LCSC sensors measured the opening in the riveted connection so that the rotation in the joint could be determined. The opening of the connection serves as a measurement of the continuity of the beam. By using four sensors it was possible to determine the rotation in the joint, both around its own axle and as well as rotation out of plane. The LCSC-sensors were only present during Phase II.
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Figure 5-28 Upper left: The backside of LCSC1_1&2 (LVDT in the Connection between the second Stringer from fixed support and Crossbeam), Upper middle: sensor LCSC1_1&2 during Phase II, Upper right: Sensor LCSC1_1&2 Lower left: the backside of LCSC1_3&4, Lower middle: sensor LCSC1_3&4 during Phase II. Lower Right: Position of the sensors.

By observing the magnified data in Figure 5-29 it was possible to conclude that the crack-opening was far greater when the rail and cross-ties was removed which indicates that the rail aids the continuality of the stringer beams.

Figure 5-29 Crack opening at second crossbeam
**LMS-sensors**

The LMS (LVDT close to Mid-span on Stringer) were positioned under the stringer beams close to mid-span and consisted of two sensors, one on each side of mid-span as seen in Figure 5-30. The LMS-sensors were only present during Phase II.

![Figure 5-30 Four LVDT’s are seen in the figure, from the left it is LMS2, LM1, LMC1, LMS1. Wires from the anchoring can also be seen to the right as well as the tripod for the photometric strain measurements.](image)

The LMS-sensors were positioned close to mid-span and measures the vertical deflection. In comparison to LSF-sensors, these had their reference point on the ground, meaning that also the global deflection of the main truss is included. From Figure 5-31 it can be seen that LMS2 indicated a stiffer behaviour when the rail was still in place, however – the same difference was not equally clear for LMS1. It appears as the measured deflection was mostly from the global deflection of the main truss when comparing with LM-sensors in Figure 5-5 and bending of the crossbeams, seen by the LMC-sensors in Figure 5-33.

![Figure 5-31 Displacement of the stringers close to mid-span.](image)
**LMC-sensors**

The LMC-sensors (LVDT close to Mid-span on Crossbeams) consisted of two sensors measuring displacement in the middle of two crossbeams close to mid span as seen by Figure 5-32. The LMC-sensors were only present during Phase II.

![Figure 5-32 Upper left: LMC1 is seen in the middle of the figure. Upper middle: LMC2. Upper right: The position of the two LVDTs. Lower: the sensor positions.](image)

The response in the two crossbeams was almost identical. Deflections appear linear up to about 8MN and 75mm of deflection. Most of the measured deflection arose from global deflection, which can be seen if compared with the LM-sensors in Figure 5-5.

![Figure 5-33 LMC (Displacement of crossbeam close to mid span).](image)
5.4 Strain measurements

5.4.1 Main truss
RFD and RFV -sensors

The RFD-sensors (Rosette strain gauge on First Diagonal) and the RFV (Rosette strain gauge on First Vertical) consisted of a total of three rosette strain gauges. The Rosette strain gauges measured strain in three direction 45 degrees apart. The sensors were located in the middle of each member at the center of the section, besides for RFV2 which was located in the intersection between top chord, diagonals and the first vertical as seen in Figure 5-34. The sensors are orientated as presented in the figures below.

The red cable is orientated orthogonal to the member, the white is orientated perpendicular and the green is at a 45 degree angle between the two.

The RFD & RFV – sensors were only present during Phase II.

In the RFD sensor, the highest response was received by RFD1_3 which is orientated in the direction of the member. The principle strain is however not orientated in the direction of RFD1_3 since there is also a response in RFD1_1. The response in is linear during the entire test to failure, despite buckling of the top chord.
Figure 5-35 RFD (Rosette strain gauge at first diagonal)

The measured response from RFV1 is presented in Figure 5-36. The sensors were orientated so that RFV1_3 is perpendicular to the member, RFV1_1 orthogonal to the member and RFV1_2 is between the other two sensors. If studying RFV1_1, it can be seen that there is a slightly different response during pre-testing compared with the final destructive testing to failure. For the final testing, RFV1_3 was the direction of the principle strains since RFV1_1 did not indicate any variance in strain. RFV1_2 is therefore by definition cosine (45°) of RFV1_3. At approximately 8MN of total loading when the non-linear behaviour starts, the vertical appears to be unloaded.

Figure 5-36 RFV1 (Rosette strain gauge at first vertical)

RFV2_1 was oriented in the direction of the top chord, RFV2_3 in the direction of the vertical and RFV2_2 measured the strain in between with a 45 degree angle to the other two sensors, in the direction of the first diagonal. The response was linear up to about 8MN. Based on Figure 5-37 it can be seen that the RFV2_1 and RFV2_2 sensors are subjected to compression and RFV2_3 to tensions as the bridge was loaded.

Figure 5-37 RFV2 (Rosette strain gauge at the top of the first vertical)
Strain gauges mounted on the bottom chord (SFVB, SQB and SMB)

The SFVB-sensor (Strain gauge at First Vertical on Bottom chord) consists of only one strain gauge located on the bottom chord 4125mm from the center of the bearing on the South Western side of the bridge as seen in Figure 5-38. The SFVB-sensor was only present during Phase II.

The SQB-sensors (Strain gauge at Quarter point on Bottom chord) consisted of a total of 4 SG’s located at the quarter point of the main truss with two strain gauges at each side of the bridge (Figure 5-39). The sensors were positioned 8250mm from the center of the bearing on the western side of the bridge. SQB1&2_2 were located on top of the bottom flange and SQB1&2_1 425mm above. The sensors measured strains in the longitudinal direction of the bridge. The SQB-sensors were present during both Phases of measurement.

The SMB-sensors (Strain gauges at Mid-span on Bottom chord) were similar to the SQB sensors but located at the center of the bridge. The sensors consist of a total of 4 sensors located in accordance to Figure 5-40. The sensors measured strain in the longitudinal direction of the bridge and were present during both Phases of measurements on the Åby Bridge.
In Figure 5-41 are the strains for the quarter point of the main truss presented. It can be seen that the SQB_2 sensors received a higher response compared to the SQB_1 indicating bending as well as normal force in the lower chord. The results are compared with the static testing in Phase II in Figure 6-41.

![Strain gauges at quarter point on the main truss](image)

**Figure 5-41** Strain gauges at quarter point on the main truss for the reference train

The strains in the bottom chord at mid-span are presented for the passage of the reference train in Figure 5-42. The same behaviour as for the ¼ - point is seen but with higher strain levels, when
compared to Figure 5-41. The response for the both sides are fairly similar, however for the SMB2-sensors the starting value and ending reading for the passage is not the same which could be explained by temperature elongation causing stresses which is released as the train passes through slip in the bearings. This is an effect which can be seen for several of the strain gauges during Phase I.

Figure 5-42 Strain gauges at mid span on the main truss for the reference train

For Phase II, all strain gauges on the bottom chord (SFVB, SQB and SMB - sensors) registered a linear behaviour in tension. The measured strain level is however significantly different depending on their position.

SFVB1 consists of only one sensor and is located on the bottom flange at the first vertical. The results presented in Figure 5-43 was linear up to about 10MN.

Figure 5-43 SFVB1 (Strain measurement at the first vertical on the bottom chord)

The results from the SQB-sensors are presented in Figure 5-44. It can be seen that SQB1_1 registered a lower strain than SQB2_1 but and SQB1_2 registered a higher response than SQB2_2. Given that the sensors were positioned symmetrically on the bridge, it seems as both sides were exposed to about the same level of normal force but that the side where SQB2 is mounted was exposed to a higher amount of bending.
The SMB-sensors were mounted with the same configuration as the previously described SQB-sensors with the measurements presented in Figure 5-45. In comparison to SQB-sensors, the response is more evenly distributed between the two sides. At about 8MN there was a change of inclination for SMB1 & 2_2 and at about 10MN of total loading SMB1_2 appears to have reached yielding.
Strain gauges mounted on the verticals (SQV & SMV)

The SMV (Strain gauge at Mid-span on Vertical) and SQV (Strain gauge at Quarter point on Vertical) consisted of two sensors in total which was located on the verticals at quarter point and at mid span on the southern side of the bridge (Figure 5-46). The sensors are orientated along the direction of the member. RFV1 presented in Figure 5-34 has one component in the vertical direction with is presented in the comparison. The SQV, SMV and RFV-sensors were only present during Phase II.

Figure 5-46 Left & middle: Position of the SQV and SMV sensors Right: SMV1

Strains were measured on the first, second (quarter point) and fourth (mid-span) vertical with results presented in Figure 5-47. SMV1 and RFV1_3 did not take any significant load until yielding at about 10 MN, where SMV1 was loaded in tension. SQV1 had during the entire test a linear response in tension. The final test to failure correlates well with the pre-testing.

Figure 5-47 Strain in verticals at quarter point and mid-span as well as the vertical component from RFV1 which is presented in it entity in Figure 5-36
**Strain gauges on top chord (SQT & SMT)**

The SQT (Strain gauge at Quarter point on Top chord) & SMT-sensors (Strain Gauge at Mid span on Top chord) consisted of 1 strain gauge at quarter point and 3 strain gauges located along the top chord close to mid span on the southern side of the bridge (Figure 5-48). The sensors were orientated in the direction of the top chord, and only present during Phase II.

The sensors on the top chord were exposed to compressive forces. SMT 1-3 was located close to each other, but the response between the three sensors was different. SMT1 indicates a linear behaviour until the testing was concluded, SMT2 shows a non-linear behaviour at about 9.5MN while SMT3 show a non-linear behaviour already 4MN of total loading. Despite SMT3 appears to have reached strain levels corresponding to yielding early on, the total loading could still be increased. The SQT-sensor registered a lower response compared to the SMT-sensors and was unloaded at about 9MN of total loading.

![Figure 5-48 Upper: Position of the SQT and SMT-sensors. Lower 1st: SQT1. Lower 2nd & 3rd: SMT2. Lower 4th Position of the sensors](image)

![Figure 5-49 Strains in the top chord around mid-span](image)
5.4.2 Load distributing system

Strain gauges on First Stringer (SFS)

The SFS-sensors (Strain gauge on First Stringer from fixed support) consist of 5 sections with 4 to 5 sensors for each section making it a total of 27 sensors (Figure 5-50-Figure 5-56). There was one strain gauge on each flange edge and an additional sensor on the continuation plate on top of the beam for SFS4 and SFS5. In Phase I the sensor on top of SFS5 (SFS5_5) was not accessible and was relocated to the location SFS3_5 as seen in Figure 5-50. The SFS-sensors were present during both Phase I and Phase II.
Figure 5-53 SFS3 during Phase I. Upper left: SFS3_1, Upper right: SFS3_2, Lower left: SFS3_3, Lower right: SFS3_4

Figure 5-54 SFS4 during Phase I. Upper left: SFS4_1, Upper right: SFS4_2, Lower: SFS4_4

Figure 5-55 SFS5. Upper left: SFS5_2, Upper right: SFS4_3, Lower left: SFS5_4, Lower right: SFS5_5 (on continuous plate)

Figure 5-56 Left: Strain gauges on top on the continuous plate during Phase II (SFS4_5 and SFS5_5). Right: SFS2_4 and SFS1_4 seen from underneath the bridge
For the SFS1-sensors, SFS1_4 receives the highest variations in strain as seen by Figure 5-57. For the passage of the reference train this sensor was subjected to compression while SFS1_1 was subjected to tension even though both sensors were located on the top flange. Both SFS1_2 and SFS1_3 are located on the lower flange where SFS1_2 was subjected to significantly higher tensional strain compared to SFS1_3 which is subjected to both tension and compression.

Figure 5-57 Sensors SFS 1 plotted for the reference train

SFS2 displayed the same behaviour as SFS1, as seen by Figure 5-58. SFS2_4 was by subjected to the highest strains.

Figure 5-58 Sensors SFS 2 plotted for the reference train
Since SFS3 was located close to the middle of the first stringer beam, higher stress due to bending was expected. SFS3_3 registered high tensile strains while the other sensors registered compressive strains as seen in by Figure 5-59. SFS3_4 indicated the highest compressive strains, which was lower than the readings in SFS1 and SFS2. It appears as if SFS3_3 was not unloaded between bogies or subjected to global tension in comparison to the other strain gauges within the cross-section.

The results from the SFS4 sensors are presented in Figure 5-60. Compared to the SFS1 and SFS2 sensors, the readings have changed signs, indicating that the cross section is exposed to negative bending moment. The readings are lower than for the other SFS-sensors. SFS4_1 and SFS4_4 indicated the opposite sign even though both these sensors were located on the top flange. SFS4_5 which is located on top of the continuous plate is also represented in the graph, with results between the strain gauges on the edges of the top flange indicating transverse bending or warping.
**SFS5**

The measured response from the SFS5-sensors is presented in Figure 5-61. The measured response indicated a similar behaviour to SFS1-3 rather than SFS4, where SFS5_3, located on the bottom flange was exposed to tension and SFS5_4 located in the top flange was exposed to compression.

![Graph](image)

*Figure 5-61 Sensors SFS 5 plotted for the reference train and histogram of the strain width for the rest of the trains*

For all SFS-sensors there was a drift in SFSx_3&4, indicating that compressive strain likely were released as the train set passed the bridge.

The stringer beams were expected to be subjected to mainly bending and some degree of normal forces. Due to the unexpected variance of strains within the cross-sections the max/min strain was plotted for all 399 train sets in order verify that these effect are not only present for the reference train. In Figure 5-62 is a histogram presented for all train sets passing the Åby Bridge during Phase I. From the histogram it can be seen that the effects from the reference trains were similar to the other passages.
Figure 5-62 Histogram for max/min strain levels in the SFS-sensors during Phase I
**SFS-sensors along the stringer**

The response in the strain gauges are also plotted along the length of the stringer beams, with all five sensors located on the same flange plotted in the same diagram. This is done for all four edges, seen in Figure 5-63 to Figure 5-66. The position of the sensors can be seen in Figure 5-50.

![Figure 5-63 Response in SFS1-5_1 for passage of reference train (located on the outside of the top flange)](image1)

![Figure 5-64 Response in SFS1-5_2 for passage of reference train (located on the outside of the bottom flange)](image2)

![Figure 5-65 Response in SFS1-5_3 for passage of reference train (located on the inside of the bottom flange)](image3)

![Figure 5-66 Response in SFS1-5_4 for passage of reference train (located on the inside of top flange)](image4)
It can be seen that the SFS sensors located on the inside of the bridge (SFSx_3 & SFSx_4) receive the highest extreme values with regard to strain. Based on only the strain measurements alone it is difficult to draw any conclusions regarding the section forces in the stringer members. The results are further investigated in section 6.3.3.

The strain level in the stringer beams were measured during both Phases of measurements performed on the Åby Bridge. The loading in Phase II was not applied directly on the stringers, but second order effects were captured. The position of the strain gauges can be seen in Figure 5-50 with photos of the sensors in Figure 5-51 to Figure 5-56.

The results for SFS1 during Phase II can be seen in Figure 5-67. SFS1_1 and SFS1_2 registered tensional strain about the same level as the compressive strains in SFS1_3 and SFS1_4, indicating bending around the stringers weak axis. A difference was identified between when the rail was in place compared to when it was removed, where the strain gauges on the top flange (SFS1_1&4) received higher strain with the rail removed.

SFS2 was located close to SFS1 and displays a similar behaviour, which can be seen in Figure 5-68.

SFS3 was located close to mid-span of the first span of the stringer beams. By studying the signs of the measured strain in Figure 5-69 it can be seen that SFS3_1 was in compression while SFS3_4 was in tension. These two sensors are both located on the top flange but on opposing sides. The same effect can be seen for SFS3_2 and SFS3_3 but with lesser values. The strain level for SFS3 was by far the highest among the SFS-sensors during Phase II.
In Figure 5-70 are the strains for SFS4 presented. The measured strain level was small in comparison to the other SFS-sensors. SFS4 and SFS5 (Figure 5-71) indicate a non-linear behaviour in comparison to SFS1-3. By studying the top flange it can be seen that SFS4_1 and SFS4_4 have opposite signs and that SFS4_5 indicate strain levels between, indicating that the top flange is bent out of plane. For the lower flange, was SFS4_3 subjected to tension and SFS4_2 to compression.

The measured results from SFS5 is presented in Figure 5-71. SFS5_1 was in compression and SFS5_4 is about the same level of tension. The same effect was observed for the lower flange, but with the opposite signs where SFS5_2 was initially in tension and SFS5_3 in compression indicating that the flanges are bent in different direction, likely due to torsion. A non-linear behaviour is seen for the bottom flange at an early stage.
SFC-sensors

The SFC-sensors (Strain gauge on First Crossbeam) were located on the ending crossbeam on the western side of the bridge. The sensors consist of totally 10 sensors in Phase I and were extended with SFC 2_3&4 to 12 during Phase II, since these were not accessible during Phase I. The sensors cover three sections, with strain gauges mounted on each flange edge orientated in the direction of the beam as seen by Figure 5-72. The constraints for the ending crossbeam differ between Phase I and Phase II. During Phase I the ending crossbeam had additional supports underneath the position where the longitudinal stringers were attached to the crossbeam. These supports consisted of concrete cubes with steel plates used to shim the distance between the steel and the abutment; these were not transferred to Phase II as can be seen in the figure. The sensors are seen in Figure 5-73 and Figure 5-74.

![Figure 5-72](image)

Figure 5-72 Upper left: Position of the SFC-sensors on the ending crossbeam. Upper right: Position of each sensor over the section. Middle left: The support during Phase I, the concrete cubes underneath the stringer/crossbeam can be seen between the bearings. Middle right: The support during Phase II. Lower left TFT2. Lower right: TFT1.
For the SFC1 sensors described in Figure 5-72 and Figure 5-73 it appears as if the upper flange received higher strains compared to the bottom flange as seen by Figure 5-75. The response in the sensors on the upper flange (SFC1_1 and SFC1_4) was not unloaded during the passage. SFC1_4 has compared to the other sensors a smaller variation in strain as the train passes the bridge.
During the first Phase of measurements was SFC2_3&4 not accessible and therefore no sensors mounted. From Figure 5-76 it can be seen that SFC1_1 and SFC2_2 display a similar result but with opposite sign. A drift in the measured response was noticed for the SFC2-sensors which was not found in the other SFS-sensors.

The SFC3 sensor was mounted symmetrically to SFC1 (Figure 5-75) but indicates a different behaviour as seen in Figure 5-77. SFC3_2 displays strains significantly higher and with the opposite sign compared to SFC1_2. One factor for these variations might occur due to a gap between the crossbeam and the support underneath them for one or both stringers.

In Figure 5-78 are the results from the SFC1 sensors presented. It appears as the sensors located on the bottom flange were exposed to tension and SFC1_4 were exposed to compression while SFSC1_3 remained unloaded.
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Figure 5-78 SFC1 (Strains in the first crossbeam)

SFC2 register the smallest strains of the SFC-sensors, seen in Figure 5-79. SFC2_3 and SFC2_4 are during the entire test close to zero while SFC2_1 and SFC2_2 are subjected to tension.

Figure 5-79 SFC2 (Strains in the first crossbeam)

The results from SFC3, presented in Figure 5-80 register a different behaviour compared to SFC1. SFS3_2 was in compression while the rest of the sensors within the cross-section were in tension. SFC3_1 and SFC3_3 have a similar response as SFC3_2 but with the opposite sign while SFC3_4 were subjected to less loading. The response appears linear up to the peak load.

Figure 5-80 SFC3 (Strains in the first crossbeam)
SMS-sensors

The SMS-sensors (Strain gauges close to Mid span on Stringer beams) consisted of 4 positions located on the stringer beams close to mid-span with 2 strain gauges at each position (Figure 5-81). The strain gauges were orientated in the direction of the bridge and were only present during Phase II.

![Figure 5-81 Position of the SMS-sensors](image)

SMS1 and SMS2 were symmetrically located to each other as well as SMS3 and SMS4. Since these positions consisted of only two sensors in the middle of the top flange and at the middle of the bottom flange, effects like warping and bending around the weak axis could not be captured. A linear response was achieved up about 8MN, where both SMS1 and SMS 2 were unloaded as seen by Figure 5-82. SMS1_1 and SMS2_1 located on the bottom flange are both subjected to tension. SMS1_2 and SMS2_2 located on the top flange have a similar response but in compression. SMS1 and SMS2 has an almost identical response. The response in tension for both SMS1 and SMS2 are higher than the response in compression. It can also be seen that a lower strain level was measured in the top flange when the rail was when the rail was still in place.

![Figure 5-82 SMS1&2 (Strain measurements on stringers close to mid-span)](image)

SMS3 and SMS4 were mounted in the same way as SMS1 and SMS2, with the results presented in Figure 5-83. The measured response was the opposite of SMS 1 & 2, with the top flange in tension and the bottom flange, first unloaded then in compression. The loading was linear up to about 8MN but instead of being unloaded as the SMS1 & SMS2 the strain was accelerated.
5.5 Temperature

The temperature in the steel does not have time to change significantly during the period of one train passage. From long term effects, it could be seen on both strain gauges and LVDTs that the elongation of the bridge correlated with the measured temperature. Despite strain gauges registering changes in strain, it does not necessarily lead to affect stresses, in not restrained. As seen for some strain gauges; strains are built up by temperature changes, which were released when a train passed. This is believed to be a consequence of forces being built up in for example bearings where a slip causes a release of stresses which could be the reason for readings on strains not being the same in the start as in the end of a train passage. For strain and displacement measurements the reading were normalized, so that temperature effects would not have time to develop. In Figure 5-84 are the temperature presented for the passage of the reference train.

During the final test to failure was the temperature registered at six different locations. The location of the different sensors can be seen in Table 3-3. The temperature coefficient for steel is \(1.2 \times 10^{-5}\), meaning that a change in temperature of 1 degree Celsius will cause 12 \(\mu\text{m/m}\). The temperature for the final test to failure can be seen in Figure 5-84. Since the temperature doesn’t deviate more then 1-2 degrees over the test, these effects from temperature are considered negligible.
6. ANALYSIS

6.1 Results from FEM (Ansys-model)

6.1.1 Dead load
In this setup, dead load was the only load acting on the bridge. The stresses in the contour plot presented in Figure 6-1 were scaled to 15MPa. In general the stresses are low in the load distributing system and slightly higher for the main truss. This was expected since the load distributing system consisted of smaller members with short spans that only have to carry the weight of member itself, in comparison to the main truss, which carries the weight of the whole structure. The stress-levels were less than 5% of the materials yield strength.

![Figure 6-1 Stresses from deadload](image)

6.1.2 BV-3 together with dead load
The BV-3 was a load model in the former Swedish codes identical to the European E4. The load model consisted of a 12.5m long symmetric wagon with 4 axles of 250kN 1.8m apart and 7.7m between the bogies as well as 80kN/m uniformed distributed load along the rest of the bridge. For the presented load case, the wagon was located at the center of the bridge.

Figure 6-2 shows the deformations (scaled 100 times) and stresses (Von mises) for the main truss. The deformation/elongation of the main truss be seen in Figure 6-2 as well at tilting of the top chord bending of the crossbeams in the weak direction, which could be seen on the measurements by the LFC-sensors in Phase II.

![Figure 6-2 Global deformations and stresses for dead load and BV-3. Deformations are scaled 1:100 and stresses are scaled 1:100.](image)
In Figure 6-3 are the stresses displayed for the connections. Stress-concentrations were identified for connection and geometric discontinuities as well as second order effects such as bending of the crossbeams in the weak direction. The full results from the analysis are presented in (Blanksvärd, 2012).

Figure 6-3 Local deformations and stresses for dead load and BV-3. Deformations were scaled 100 times and stresses were scaled to 100MPa

6.1.3 Loading in limit state
The purpose with the limit state analysis was to determine the required load that would cause a collapse of components or the entire bridge. The analysis has been carried out for several different locations where the most sensitive ones are presented.

For this analysis was the material strength of importance. The bridge itself consists of two different materials S1311 and S1411 which corresponds to different yield strengths, in the model the yield
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Strength of 220MPa was applied to the entire structure of this reason a modification factor was used for the members of the stronger material. For this analysis the modification was 260/220=1.18 for S1411.

Stringer beam

The stringer beam was loaded in the third span. Since the rail and cross-ties was not incorporated in the model, loading was applied as a distributed load over the length of 0.8m in the center of the span in order to avoid singularities.

In Figure 6-4a) the top flange was braced for buckling giving an ultimate load of 1715kN and for the un-braced flange a load capacity of 1700kN was reached, displayed in Figure 6-3b). The failure was considered mainly being a shear failure.

Crossbeams

Two load scenarios were carried out to control the load carrying capacity for the crossbeams. One scenario where the crossbeams are loaded centrically with a load distributed over 0.8m as seen in Figure 6-5. For this case, failure was reached at 1578kN, but since the crossbeams consisted of S1411 and there would be two loads acting on the member, the limit load reached was 1578*1.18*2=3724kN. For the other load scenario, the load was distributed over a length of 1.6 m, the capacity was increased to 4585kN, for both these cases the crossbeam were considered as un-braced.
Main truss

If loading three crossbeams close to mid-span over the length of 1.6m each, the bridge was expected to fail through buckling of the top chord instead of the crossbeams failing. The buckling was expected to occur at 9891kN of total loading or 3297kN per crossbeam as seen in Figure 6-6.

Figure 6-6 Failure of the main truss by loading three crossbeams in the center.
6.2 Results from FEM (Abaqus-model)

6.2.1 Loading in limit state

Prior to Phase II, the model were mainly used to estimate the failure of the Åby Bridge and planning the sensor positions for Phase II. Figure 6-7 show von Mises stresses displayed right before initial yielding. In Figure 6-8 can displacements and stresses be seen after buckling and redistribution of forces. Presented in Figure 5-5 is the load displacement curve displayed for the FE-model with with both code based material properties and with measured values together with the measured response. From the diagram it can be seen that the measured peak load lies between the two FE-results. With tabulated values for material properties the non-linear behaviour was estimated to 8.0MN with a vertical displacement of about 0.06m and the peak load is achieved at about 9.5MN with a vertical displacement of 0.1m. The deformation behaviour in Figure 6-8 can be compared with Figure 6-52 where the plastic deformation of the top chord is seen.

![Figure 6-7 Von Mises stresses the instance before yielding](image1)

![Figure 6-8 Von Mises stressed together with deformation behaviour after initial yielding](image2)
6.2.2 Influence lines for sensor response

The FEM-model using Abaqus, developed for Phase II were after testing used for making influence diagrams for different sensors, where three different sensors are presented in Figure 6-9 to Figure 6-11.

In Figure 6-9 is the influence diagram for the LQ-sensors (deflection at quarter-point) for loads acting on the rails individually. It was seen that loads acting on rail 2, which is the rail closest to the studied sensor position will give rise to deflections about 70% higher compared to loads acting on rail 2.

![Influence lines for LQ-sensors](image)

**Figure 6-9 Influence-chart for the LQ-sensors based on FE-results**

In Figure 6-10 are influence diagrams for strains at mid-span in the bottom chord presented. Rail 2 represents the rail closest to the studied position and can be seen to have a higher influence on strain, similar to Figure 6-9. Since the studied position is located at mid-span, a symmetric response is achieved.

![Influence lines for SMB-sensors](image)

**Figure 6-10 Influence-chart for the SMB-sensors based on FE-results**
Since the rail and cross-ties were not considered in the model, the load was applied directly on to the stringers. For local effects on the stringers, this resulted in somewhat misleading results. In Figure 6-11 are the influence diagrams presented for the four strain gauges located in SFS3, the position of the sensors are described in Figure 5-50. Since the load was applied directly on the longitudinal stringers, singularities in the model were received, as seen in for the top flange in Figure 6-11. It can also be seen that the response in both sides of the flanges are almost identical, meaning that effects such as torsion and out of plane bending are not captured, which is further analyzed in section 6.3.3.

A new model was developed, where the cross-ties and the rail was considered which is presented in section 6.3.3.

![Top flange - SFS3_1 & SFS3_4](image1)

![Bottom flange - SFS3_2 & SFS3_3](image2)

*Figure 6-11 Influence diagram for the SFS3-sensors based on FE-results. Upper: SFS3_1 and SFS3_4 Lower: SFS1_2 and SFS1_3.*
6.3 **Simulation of response in Phase I**

In this section are the influence diagrams presented in section 6.2.2 used to simulate the response from the reference train which were recorded during Phase I. Axle loads were known for the reference train, and by multiplying the load of each individual wheel with the influence value from the diagrams it is possible to validate the model against measured results from Phase I.

Since the reference train consisted of varying axle loads unevenly distributed in the transverse direction a routine was created making it possible to simulate the response in sensors by using influence lines for each of the rails together with known train loads. The routine consisted of an excel script that multiplies the load of each wheel with the corresponding influence value. Interpolation is done for intermediate values. The routine then summarized the results for the given load-position and moves the loads to the next load-step. This is repeated until the entire train-set has passed the bridge, with the desired length of load-steps. By this simulation, the load effect is reduced to a linear response, but was considered as an accurate estimation for service loads.

The comparison was carried out for the reference train for three different sensors listed below, where the train is moved 1.15 meter per load step making it a total of 500 load steps for the sensors on the main truss and 0.575m making it 1000 load steps for the stringer beam. A sensitivity analysis was carried out where smaller step-length proved no significant improvement of results. The train-speed in the simulation was adjusted to fit the measured values, which resulted in a velocity of 41 km/h. The measured response was sampled at 50 Hz which gives a load step of 0.23m.

- The deflection at quarter-point (LQ-sensors)
- The strain at mid-span (SMB-sensors)
- The strain close to the middle of the first stringer beam (SFS-sensors)

6.3.1 **LQ1 - sensors**

From Figure 6-12 it can be seen that the response between measures and simulated results are similar, with the measured results indicating slightly higher displacements as well as fluctuation. The fluctuations are likely due to vibrations in either the bridge or the LVDT which were measuring the deflection. When considering the deflection, the measured response also includes the settlements of the abutment itself, while the numerical model assumes it to be fixed in the vertical direction.

The settlements of the abutment itself were not measured during this Phase, making it difficult to adjust the measured response in a reliable manner. This effect does not have any major effect on the state of stress in a single span bridge but causes the results to not match. If the simulated response was amplified about 21% it matched the measured response very well.
6.3.2 SMB-sensors

The SMB-sensors captured the stress/strain behaviour at mid-span in the bottom chord of the main truss. The simulated response was in very good agreement with the simulated response for the SMB_1 sensors located on the top flange, as seen by Figure 6-13. The same analysis was made for the bottom flange, where the measured strain was 10-20% higher than the simulated response as seen in Figure 6-14. By studying the response in SMB2 from the two figures, it can be observed that there was a release of compressive strains, as the train passed by.
6.3.3 SFS3

Based on similarities between SFS3_1 and SFS3_4 and SFS3_2 and SFS3_3 in the Influence-diagram presented in Figure 6-11 the simulated response would also be similar. The response based on measurements did however not indicate this behaviour as can be seen in for example Figure 5-59. By studying the top flange, it was seen that SFS3_4 indicated strain levels which was more than twice the size of SFS3_1 as seen in Figure 6-15. For the lower flange it was concluded that SFS3_3 was subjected to high tensile strains while to SFS3_2 was exposed to compressive strains.

This effect was believed to originate from the cross-ties being located on top of the stringer beams transferring an eccentric load which gives rise prevented torsion and so called warping strains and bending out of plane.

![Figure 6-15 Measured responses in SFS3 for passage of the reference train. Upper: Top Flange. Lower: Bottom Flange](image)

In Figure 6-16 are the simulated response from the reference train based on the influence diagrams presented in Figure 6-11 compared with the measured response seen in Figure 6-15.
By analyzing the influence charts in Figure 6-11 together with the responses received from the measurements it was clear that the simulation is not accurate and that there are out of plane effects taking place.
**Refining of model**

In the influence diagrams used for the simulations presented in Figure 6-16 were the cross-ties and rail disregarded. Since the measured response indicated that there are actions which cause torsion and/or bending around the weak axis of the stringers, a new model was created where the rail and the cross-ties are taken into account. The model was developed using the conventional available FE-software Robot Structures where a combination of beam elements and shell elements were used. For this model the load was applied to the rail, which distributed the load through the cross-ties to the stringer beams. The new model is displayed in Figure 6-17.

![Figure 6-17 Model created for analysis of the stringer beams](image)

The new model developed in Robot Structures has the same constraints as the as the previously developed Abaqus model, meaning that all connections are considered as fixed. The model was developed with the purpose to evaluate strains in the stringer beams, therefore was the mesh-size refined within the studied area. An Initial study was made to see if there was any difference in placing the load directly on the stringers compared to placing it on the rail. The static load setup from Phase II was used with a load of 1500kN acting in eight positions. The load setup can be seen in Figure 6-18.
If looking at the deformations in Figure 6-19 it can be seen that the stringer beams deforms differently dependent if loaded on the rail or directly on the stringers. When the bridge was loaded on the rail, the rail and cross-ties will distribute the load but also initiate a torsional bending moment on the stringer beams.

When concluded that the position of the applied load as well as the distribution of the stringers had influence for the strains in the stringer beams new influence charts were extracted, using the newly developed model. A point-load was moved along the bridge where the response in SFS3 was recorded for each position to create the influence diagram. The influence diagram for the new model can be seen in Figure 6-21 and Figure 6-23 together with the results from the previous Abaqus model for comparison, presented in Figure 6-20 and Figure 6-22.

It can be seen from Figure 6-20 and Figure 6-21 that the local strain/stress-effects were significantly reduced when taking the cross-ties and stringers into account, due to higher distribution of forces by the rail and cross-ties. These effects were greater for the top flange than for the bottom flange, as seen by in Figure 6-22 & Figure 6-23. Based on the influence diagrams when the load was placed on the rail it can also be noted that there is a different response in sensors located on the same flange, which also is seen in the measured data. Indicating that taking the cross-ties and the rail in to account is crucial for capturing the structural behaviour in the load distributing system.
Figure 6-20 Influence diagram for strain in top flange loaded directly upon the stringer (same as Figure 6-11)

Figure 6-21 Influence diagram for strain in top flange loaded on the rail

Figure 6-22 Influence diagram for strain in bottom flange loaded directly upon the stringer (same as Figure 6-11)
Based on the influence diagrams presented in Figure 6-21 and Figure 6-23 the reference train was simulated again with the results presented in Figure 6-22 Figure 6-27 together with the measured response.

In Figure 6-24 is the simulated response in SFS3_1 (outside of top flange) plotted together with the measured response for the reference train. The simulated strain is almost identical to the measured response, if disregarding the drift in the sensor.
Figure 6-25 shows the comparison between measured and simulated response in SFS3_2 (outside of bottom flange). Since the strain gauge was located on the bottom flange close to the middle of the first stringer beam a response in tension was expected, as the simulated response illustrates. The measured strain does however indicate a different behaviour, where the flange is in compression.

![Graph](image1)

**Figure 6-25** Simulated response in SFS3_2 compared with measured response for the reference train.

Figure 6-26 shows the comparison for SFS3_3 (located on the inside of the bottom flange) between the simulated strain as well as the measured strain for the passage of the reference train. In contrast to the other sensor located within the same cross-section SFS3_3 is not unloaded between cycles, an effect which was not captured by the simulation. The measured strain was much higher than what was simulated.

![Graph](image2)

**Figure 6-26** Simulated response in SFS3_3 compared with measured response for the reference train.

The simulated response for Figure 6-27 does not capture the behaviour of the SFS3_4 sensor. The measured strain was much higher than what was simulated.

![Graph](image3)

**Figure 6-27** Simulated response in SFS3_4 compared with measured response for the reference train.
Splitting measured response into section forces

If disregarding local effects and out of plane actions the response in the measured strain would have been the same for the sensors located on the same flange. The measured response indicated that this however was not the case for the Åby Bridge. From the FEM-analysis in Figure 6-20 to Figure 6-23 it can be seen that eccentric loading gave rise to out of plane actions leading to variations within the same flange. When these are compared with the measured response in Figure 6-24 to Figure 6-27 they do however not match very well.

If assuming plane sections for the SFS - sensors and that the measured strain is the combined effect of (as illustrated in Figure 6-28):

- Strain by normal forces
- Strain by bending around the strong axis
- Strain by bending around the weak axis
- Strain by warping

By measuring the response in each of the flange edges it possible to form an equation system and derive the magnitude of each effect. The stringer beam consisted of a symmetric cross-section, each effect was assumed affect each sensor equally in the sense of absolute values. Four equations can be formulated with four unknowns, leading to a closed form solution.

![Assumed actions on stringers](image)

<table>
<thead>
<tr>
<th>Sensor</th>
<th>Normal strain</th>
<th>Bending strain (strong axis)</th>
<th>Bending strain (weak axis)</th>
<th>Warping strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>+</td>
<td>-</td>
<td>-</td>
<td>+</td>
</tr>
<tr>
<td>2</td>
<td>+</td>
<td>+</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>4</td>
<td>+</td>
<td>-</td>
<td>+</td>
<td>-</td>
</tr>
</tbody>
</table>
The equations can be then be written as a system:

\[ +N - B_s - B_w + W = \varepsilon_{FS,1} \]

\[ +N + B_s - B_w - W = \varepsilon_{FS,2} \]

\[ +N + B_s + B_w + W = \varepsilon_{FS,3} \]

\[ +N - B_s + B_w - W = \varepsilon_{FS,4} \]  

Eq. 6.1

Which can be solved as:

\[ W = \frac{\varepsilon_{FS,1} + \varepsilon_{FS,2} - \varepsilon_{FS,3} - \varepsilon_{FS,4}}{4} \]

\[ B_s = \frac{\varepsilon_{FS,2} + \varepsilon_{FS,3} - \varepsilon_{FS,1} - \varepsilon_{FS,4}}{4} \]

\[ B_w = \frac{\varepsilon_{FS,3} + \varepsilon_{FS,4} - \varepsilon_{FS,1} - \varepsilon_{FS,2}}{4} \]

\[ N = \frac{\varepsilon_{FS,1} + \varepsilon_{FS,2} + \varepsilon_{FS,3} + \varepsilon_{FS,4}}{4} \]  

Eq. 6.2

Instead of plotting the response for individual sensors, the components N, B_s, B_w and N can be displayed as individuals as shown in Figure 6-29. From the plot of the components it can be seen that the effect of the normal force seem negligible. The steady increase in normal strain was a consequence of temperature strains being released. The warping and bending out of plane stresses appear to have a major effect on the response, where the combined effect of warping and bending around the weak axis exceeds the effect of bending around the strong axis. These effects were believed to be highly dependent on the position relative to the cross ties.

Figure 6-29 Results from the SFS3 sensors displayed as individual components

The strain level isolated as the bending component around the strong axis could then be compared with the simulated bending strains. Presented in Figure 6-30 is the comparison, a good agreement between measured and simulated strain was be observed.
The other SFS-sensors were filtered in to components, as seen in Figure 6-31 to Figure 6-34. From these figures it can be noted that there was a varying amount of warping and out of plane bending for the different sensors, effects which prove difficult to predict by hand calculations and even FEM-simulations.
Figure 6-33 Results from the SFS4 sensors displayed as individual components

Figure 6-34 Results from the SFS5 sensors displayed as individual components

The method was also applicable to the static testing in Phase II. Seen in Figure 6-35 is the component as a function of the total load. A linear response can be seen for the static pre-loading when the rail was still in place.

Figure 6-35 Results from the SFS3 sensors during the static pre-testing in Phase II
6.4 Load carrying capacity according to code

Upon the request from Trafikverket a code based assessment of the Rautasjokk bridge was performed (Hägström, 2014) using the Swedish assessment code BVS 583.11 (Banverket, 2005a). Since the geometry and the material was identical to the Åby bridge the results with regard to capacity would expected to be the same range for the Rautasjokk bridge. According to BVS 583.11 were the highest allowed axle load supposed to be calculated for three different train sets with the axle configuration illustrated in Figure 6-36. TLM 2 was the train set which represents the freight trains running along the main line (Stambanan) whereas the TLM3 represented the iron ore train sets along the ore lane (Malmbanan). Since the Åby Bridge was located along the main line was the TLM2 which were of greatest interest. For ULS the axle load was limited by the shear forces in the riveted connection between longitudinal stringers and the crossbeam. The maximum code based axle load which this detail could be subjected to was 34.4 tons, based on the conservative assumption that the rivets transferred the entire shear force and that neither the rail nor the welded continuation plate on top of the stinger contributed.

![Figure 6-36 Train geometry (BVS 583.11)](image)

With regard to fatigue, the two bridges were different. The load history for the bridge over Rautasjokk has been subjected to higher loads due to the transportation of iron ore. The tendencies between the two bridges was however comparable. With refined assessment methods using the Hypothesis of Palmgren-Miner (Miner, 1945) of partial damage accumulation together with rain flow summation based on a 3D-frame analysis all details except one were found to pass the fatigue checks. The detail which caused the problem was the joint where a connection plate was welded to the upper flange of the longitudinal stringer seen in Figure 6-37. Since the influence length for this detail was short, refined methods did not prove significantly more favorable results since one load cycle was received for each bogie. The rail and cross-ties were not modeled for this analysis. If considering the differences between Figure 6-20 and Figure 6-21, it is clear that taking the rail and cross-ties in to consideration will likely provide different results.

By decreasing fatigue-stresses an exponential effect is achieved, where decreasing the stresses with about 20% will reduce the accumulated damage with regard to fatigue to about 50%. The effects based on the findings in this report, that stresses are also caused by warping and bending around the weak axis were likely to have a significant effect.
Even though the theoretical lifespan of the Rautasjokk Bridge with regard to fatigue were found depleted, no fatigue cracks have been spotted. The C-classes used for fatigue evaluation are often rough, where the assessed detail is not always identical to the one used taken from codes. Fatigue tests often indicate a big variation in results, where the 2% fraction governs the check. The critical detail located on the top flange, close to the middle of the span for the stringers should mainly be exposed compressive stresses. This was verified by the measurements of SFS3_4. Since the detail was welded, residual stresses might have shifted the stress-variations in to tension. If analyzed from a fracture mechanical perspective, it could be concluded that if a crack were to occur, the propagation of the crack were expected to be slow.

6.5 Main truss

6.5.1 Out of plane bending of upper chord
Traditionally have bridges similar to the Åby bridge been regarded as a two dimensional structures, especially in the design of elderly structures where numerical modeling where not available. When loading the bridge, the cross-beams will be subjected to bending and cause the upper part of the main truss to tilt slightly inwards. Loading of one singular cross-beam will cause a horizontal force on the top chord pointing inwards, to what degree is dependent on the bending and torsional stiffness relationship between the structural members. This will also to some degree affect the buckling resistance of the top chord. If instead subjected to a distributed load, there will instead be a small change in lever between the upper and bottom chord – since both top chords will tilt more evenly inwards, but there would not be any major horizontal forces acting on the upper chord.

During Phase II, was the horizontal displacement of the top chord was monitored. In comparison with a real train passage, the bridge was only loaded in eight points close to the center of the bridge which caused transversal bending of the top chord as well normal force caused by global bending. From the monitored data it can be seen that LHTC2 (located at the center) is moving inwards whereas LHTC1 is in the opposite direction which correlates well with the numerical simulation. This effect is believed to be enhanced by the bridge only was loaded close to the center. For loads within the service-range this did however not appear to cause any significant effect since there was a linear response, second order effects of buckling appears first after significantly higher loads. Since loading caused normal forces in the top chord, and the center was pushed inwards from the bending of crossbeams close to center the top chord close to supports will be pushed in the other direction due
to second order effects. In Figure 6-38 are the measurements illustrated for one load scenario going up to 1800kN during pre-testing in Phase II.

The horizontal displacement of the top chord could unfortunately not be measured during Phase I. The response for the preloading indicated a significant scatter, since the sensors were positioned on top of a steel column with readings of only a few mm, as seen by Figure 6-38. The trend can be observed for LHTC2 at mid span but the displacement is too small for the LHTC1-sensor.

If studying the SMT & SQT-sensors located on the top chord close to mid-span it is possible to identify a deviating behaviour in the SMT3 sensor, where a change of slope starts already at approximately 3.0MN of total loading. The other strain gauges remain linear up to 8MN of total loading, as can be seen in Figure 5-49. The response in SMT3 might to occur due to redistribution of forces or a slip in a connection. This effect can also be seen by creating a fit for the pre-loading tests, where the inclination of SMT3 deviates from SMT1 and SMT2 which should be expected to give the same response as seen in Figure 6-40.
6.5.2 Bottom chord in main truss & comparison of load level

By comparing the strain level in the bottom chord it is possible to compare global load effects Phase I and Phase II, despite the bridge being loading differently.

For the passage of the reference train, the strain level at the quarter point at the top of the bottom flange keeps fairly constant around 100\(\mu\)m/m during the entire passage and around 60\(\mu\)m/m for the sensor located at the same level as the center of the top flange seen by Figure 5-41. At mid-span the response was about 150\(\mu\)m/m for the sensor on the bottom flange and around 75\(\mu\)m/m for the sensor at the level of the center of the top flange as can be seen in Figure 5-42. For the comparison focus has been on the preloading series for a higher certainty due to the more results and readings taken at a steady state loading. Figure 6-41 and Figure 6-42 shows the pre-loading with the rail in place where a curve-fit has been added. The train passage of the reference-train has been plotted against the secondary y-axis based on time rather than applied force. This means that the crossing of the trend-line indicates corresponding load level with the static test for the specific sensor. For the quarter-point a static total load of 1800kN corresponds to the response achieved by the reference train and for the mid-span 1450kN of static loading was sufficient to achieve the same response as the reference train. The difference in response between the quarter-point and mid-span was likely because the load was concentrated closer to mid-span during Phase II which will give a relatively higher response in these sensors and therefore a lower force needed to achieve the same response.

The mean axle load of the reference train was 18 tones which gave raise to fairly small strain levels (about 21MPa) and could be compared with a static load in the range of 1450kN to 1800kN, considering only the bottom chord. If a linear response was assumed, this load represent less than one fourth of the load required to achieve a non-linear behaviour for the load-deflection curve of the main truss.
During both Phases has the rotation at the supports been monitored by two LVDT’s with known distance between the sensors. By dividing the difference in response with the distance between the sensors it was possible calculate the rotation. The absolute values for the sliding support were higher, but for a symmetrically distributed load, the rotation should be about the same for all positions. The calculated rotation for each of the corners of the bridge does not give such a clean response as might be expected. Seen for the measurements of Phase II in Figure 6-43 the rotation in the sliding supports (LFSR) deviates at approximately 4MN of total loading, but then continues with the same slope. This might be caused by small settlements, slip in friction or cracking of the temporary support. After 8MN of loading, LSSR2 deviates by going back to about zero. This might
have been caused by crushing of the temporary support. At 10MN of total loading the rotation kept increasing without an increase of the total load, indicating that peak load was reached.

![Rotation at supports](image1)

**Figure 6-43 Rotation at supports during Phase II.**

For loads within the service-range a linear behaviour was expected. The rotation in each of the corners is plotted in Figure 6-44. The curve-fit is a linear fit that are set to go through the origin even though a custom-fit on the basis of \(Y = B + Ax\) would provide a better fit to the measured data.

![Rotation at supports](image2)

**Figure 6-44 Rotation for the pretesting in Phase II with curve-fits adjusted to go through the origin.**

In Figure 6-45 was the rotation calculated for the passage of the reference train. The response appears reliable for the fixed side, but for the sliding side there are significant disturbances. This might be explained by the sensors on the sliding support register significantly higher absolute values since the elongation of the bridge was also captured. The measured data from the sensors are presented in Figure 5-9 and Figure 5-10.
Because of the disturbances for the sliding side of the bridge in Phase I, a comparison is made between the reference train in Phase I and the static testing in Phase II. In Figure 6-46 are both Phase I and the pre-loading in Phase II plotted. The x-axis are displaying the rotation and the y-axis represent the total load in Phase II and the secondary y-axis represent time for the live-measurements. In Phase I it can be seen that LFSR2 give rise to a higher rotation, while it is the reversed effect in Phase II. By comparing the results it can be seen that the reference train represents a static load somewhere between 1750kN and 2750kN with regard to rotation at the fixed support.
6.5.4 Deflection of reference train

During Phase I was the deflection of the bridge monitored at the quarter point. A deflection of 8mm was registered as seen by Figure 5-3. If considering the bridge as a simply supported beam with a distributed load the deflection can be described as:

\[ u = \frac{q \cdot a \cdot L^3}{24EI} \left(1 - \frac{2a^2}{L^2} + \frac{a^3}{L^3}\right) \]  

eq.6.3

Where “a” is the checked distance from the support. If q, a and EI is remains constant, the deflection curve can be plotted as in Figure 6-47. With a maximum value of 11.2mm which corresponds to about L/2960 and about 27% of the L/800 requirement stated in code limitation. If a portion of the measured deflection came from settlements, the total deflection at mid-span would have been smaller. Bending of the crossbeam and stringers will however lead to a slightly increased deflection.

![Figure 6-47 Deflection curve for the passage of the reference train, based on readings in the LQ-sensors](image)

6.5.5 Temperature

Since strains and displacement vary with temperature, all sensors have for the evaluation been normalized to the starting value for the passage / testing series. For some sensors it can be seen that the sensor does not return to its starting value which was believed to be caused by release of friction where stresses has been built up due to temperature. Figure 6-48 displays the strains in at mid span for the SMB-sensors as well as temperature. The measured data is sampled over the period of one day, where every spike in the curves represents the passage of one train. From the plot it can be seen that the strain-level follows the temperature-curve.

![Figure 6-48 Strain in SMB-sensors together with the varying temperature.](image)
When studying the long term elongation of the bridge it could be seen that the sliding bearing follows the shape of the temperature curve. The curve in Figure 6-49 presents the elongation is not completely smooth, when looking closely it is possible to identify a slip as a train set passes by, possibly from the sliding bearings. By knowing the elongation, temperature and the length between the measured positions it is possible to derive that the thermal coefficient to about $1.2 \times 10^{-5}$.

![Figure 6-49 Long term elongation of the bridge due to temperature](image)

### 6.6 Local load distributing system

#### 6.6.1 Stringer beams

The stringer beams were monitored in the first span with strain gauges during both Phases of measurements as shown in Figure 5-50, where additional LVDTs were added during Phase II as seen in Figure 5-20. During Phase II was the stringers also monitored in the span next to mid-span as can be seen in Figure 5-81 with the results presented in Figure 5-82.

The stringers purpose was to distribute loads to the crossbeams which are carried by the main truss. During Phase II the load was not positioned in the monitored span of the stringers, causing the sensors to monitor secondary effects. Phase I is therefore of more interest since the load from the trains acted over the entire length of the bridge, including the measured span.

From the analysis performed for SFS3 it appears as the stringer beams are not only subjected to bending and normal force since the strain response at the flange edges vary a great deal between the two sensors located on the top flange as well as the two located on the bottom flange. In chapter 0 was it proved that warping strains and out of plane bending had a major influence on the response.
6.6.2 Continuation of longitudinal stringers
In (Elhag, 2014) was the continuation of the riveted joint between longitudinal stringers and the crossbeams investigated. The degree of constraint in these joints were at the time the bridge was built considered as simply supported, in the thesis it was however estimated to be about 60% constrained. Besides the LCSC sensors which measured the crack opening and strain gauges that measured the strains in the stinger the Aramis system was used for Digital Image Correlation in order to evaluate the strains in the connection itself. The results regarding deformations were ok, but the measurements of local strain proved difficult to evaluate due to the changing light conditions over the day and the fact that the cameras was not perpendicular to the surface.

During Phase II the crack opening was measured by the LCSC-sensors, where rotation in the connection can be considered as an indication of its constraint. A non-existing rotation would indicate a continuous effect. As seen by Figure 6-51, the rotation in the joint seems to be almost non-existing when the rail still in place. The blue circles represent the rotation when the rail was removed and the green dashed line is representing the final test to failure. By this figure it appears that the rail has a significant effect for the continuation of the stringers when exposed to static loading as in Phase II.
6.7 Failure actions

Despite the high load level, the failure occurred as a ductile non-dramatic failure with no joint failing. There was however severe yielding at several positions when the bridge finally reached its peak load.

It was as expected buckling of the top chord which prevented the bridge from taking more load as seen in Figure 6-52. There were severe cracking in the concrete acting as temporary support which can be seen in the plots for the LSS and the LFS – sensors.

Since the rail and the cross-ties distributes the load, and were removed during the test to failure the loading on the stringer beams became concentrated. Due to this, local buckling occurred as shear buckling and patch loading as seen by Figure 6-53. It could be argued that this type of failure is not relevant for the bridge since the load would have been distributed to a far greater extent if exposed to real trains. It can be seen by the cracking of the paint that the joint tested by the DIC-system was subjected to high strains during the final test to failure.

Figure 6-52 Global failure - buckling of the top chord

Figure 6-53 Local failure – local buckling underneath the load
7. CONCLUSIONS
The Åby Bridge proved a robust ductile behaviour while being loaded to failure, with no failure in welded or riveted connections. The failure mode which prevented the bridge from taking more load was buckling of the top chord. Besides buckling there was local damage on the stringers due to the position of the loading beams. These damages are in reality of less relevance, since real trains will have a more distributed load.

The two questions that are aimed to be answered within this project are (which also was the foundation for the configuration of sensors):

- Do 3d-effects have to be considered for these types of bridges? Both with regard to stringer behaviour top chord and crossbeams. How can this be taken in to consideration for future capacity checks?
- What is the stiffness of the connection between stringers and crossbeams? Are these to be considered as simply supported or continuous?

When studying the strain levels on the first stringer beam for live loading it can be concluded that the response was not the same for the two sensors mounted on each side of the bottom or the top flange, which would be expected if it was purely bending around its strong axis and normal forces acting on the beam. This effect is assumed to be caused by prevented torsion causing so called warping strains as well as strain caused by out of plane bending. By splitting the measured response in to components, the magnitude of warping and out of plane bending can be quantified. Based on the response it appears as bending is only representing about half of the measured strain level at the flange edges. It is very difficult to evaluate and anticipate the warping and the out of plane bending, since it is believed to arise from the constraints given by the cross-ties to the stringers.

The horizontal displacement of the top chord appears linear for service loads. It is not until 8 MN of total loading a non-linear behaviour is achieved. The loading position in Phase II is assumed to be unfavorable with regard to buckling of the top chord, since it will locally tilt the truss inwards and weakening it’s buckling resistance. By studying the horizontal displacement of the top chord it can be seen that it moves inward at mid-span and outwards close to the support. This effect will be less likely if a distributed load is applied over the entire length of the bridge, for example a train-set. Therefore it is not necessary to take the 3D effects of the top chord in to consideration for a system analysis, besides the buckling analysis.

Bending of the crossbeams in the bridge’s longitudinal direction could be observed, caused by elongation of the main truss which is restrained by the stringers causing bending around the crossbeam’s weak axis. This effect should be considered when assessing the stringers, since it caused tension in these members.

Based on the measured response it can be seen that the rail together with the cross-ties plays a significant role with regard to the stringers. The flexibility of the rail and the crossties will act as springs, causing a greater distribution of loading. When studying the rotation in the riveted connection between stringers and crossbeams, major differences can be seen between the tests that were carried out with and without the rail in place. This effect is however negligible for the main truss.
Assessment of the connections between stringer beams and crossbeams are often details which are found critical for un-ballasted steel truss railway bridges if the stringers are considered as continuous. The effects of flexibility in the crossbeams together with the already mentioned flexibility of the rail and the cross-ties will reduce the hogging moment-peaks in the connection leading to a reduced load effect in the given detail significantly reducing the bending stresses in this detail.

In addition to the study of the Åby Bridge an assessment of the Rautasjokk Bridge (Häggström, 2014) has been made to answer questions regarding the code-based lifespan of this bridge with regard to fatigue. And also what details that needs to be investigated more careful for future inspections. In this report it is concluded that the bridge can withstand loads up to 34.2tons for TLM3 (iron ore wagons) and 34.4tons for TLM2 (representing the geometry for the steel slab trains along the main line) in ultimate limit state. The accumulated fatigue damage in the stringers w found to be overdue in the detail where connection plates for the bracing of the stringers are welded to the top flange of the stringers. It should be noted that even though the Rautasjokk Bridge is built based on the same drawings and material specifications as the Åby Bridge, it is located along the Iron ore line and has therefore been subjected to historically higher axle loads.

### 7.1 Future Research
The measurements indicate that the rail has a major influence on the load-distributing system. Both strain and displacement increased when the track was removed for the same amount of loading. Measurements of the Rautasjokk Bridge were carried out with the purpose of evaluating this, as well as unintentional composite action between the rails and the stringers which is still left to investigate. By increasing the stiffness of the rail, the train loads will act more as distributed load, which is favorable in ULS by decreasing stress-peaks and for fatigue by both reducing stress and the number of cycles.

Inspections and theoretical assessments of bridges are often regarded as two separate activities carried out by different operators. The annual inspections are often focused on maintenance while the theoretical assessment is focused on structural capacity. It believed that greater safety can be achieved by sharing knowledge between the two activities. Just as the theoretical assessment forms guidelines and intervals for inspections and structural health monitoring, the output from inspections could provide input to that assessment. Theoretical work has been conducted in the field of probabilistic fatigue assessments, the results of which might be possible to implement.

In some cases it is not be possible to theoretically upgrade the bridge with regard to fatigue or that fatigue cracks has already occurred. Retrofitting parts of the bridge in order to improve its fatigue resistance or structural behaviour can in these cases be a valid alternative. Much research has been done in this area, and it is believed that collecting this information can help to form guidelines for which actions to take.
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A.2
Bridge over Åby River - Evaluation of full scale testing
Bridge over Åby River - Evaluation of full scale testing

Ver även broar:

huvudriktning B 1567-6
B 3772-3
B 3401-10
B 3626-3
B 1734-2

Dimel 42

Förzinkad gallerduk
Bärjärns höjd = 30 mm
Minnsta godtsjöckel = 3 mm
Största avstånd mellan bärjärnen = 30 cm
Varje element fästes vid vardera balken med 3 genom halkfläns
gående ¾" förzinkade skruvar som förses med läsmathrar

Sektion J-J

Dimel 42

Beträffande tågbelastning och normer
se ritning B 1546-7
10. Annex B – Material testing
The material consisted of steel quality S1311 and S1411 which according to the Swedish assessment code for railway bridges (BVS 583.11) corresponds to design values listed in Table 10-1. The listed strength of the material is the characteristic material properties, for the FEM-simulation higher yield strength was used trying to anticipate the structural capacity rather than doing the design for the structure. For the FEM-simulation performed after the material tests were performed, the measured material properties were used.

Table 10-1 Material properties prior to testing

<table>
<thead>
<tr>
<th>Part</th>
<th>Material</th>
<th>( F_{yk} )</th>
<th>( F_{uk} )</th>
<th>( F_Y ) Used for modeling for Phase 1</th>
<th>( F_Y ) Used for modeling for Phase 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stringer beams, verticals, diagonals</td>
<td>S1311</td>
<td>240 MPa</td>
<td>360 MPa</td>
<td>220 MPa</td>
<td>245 MPa</td>
</tr>
<tr>
<td>Main truss, Cross girders</td>
<td>S1411</td>
<td>270 MPa</td>
<td>430 MPa</td>
<td>220 MPa</td>
<td>245 MPa</td>
</tr>
</tbody>
</table>

Table 10-2 shows some of the Equivalence construction steel nomenclature in Europe and other countries according the codes used. It is a widely accepted general purpose structural quality steel offering a constant min 250MPa yield point for all thicknesses of material and has been widely used in the construction of buildings, bridges and other structures.

This material can be regarded as a carbon structural steel, with the chemical composition in percentage [%] of grade displayed in Table 10-3.

Table 10-2 Different nomenclature for the SS1311 around the world

<table>
<thead>
<tr>
<th>Country</th>
<th>SS1311</th>
<th>Fe E</th>
<th>RSt 37-2</th>
<th>E24-2</th>
<th>Fe 360B</th>
<th>AE 235B</th>
<th>SS 1311 00</th>
<th>40 (A) B</th>
<th>SS400A</th>
<th>Fe 360B</th>
<th>A 284 gr.C.D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Europe</td>
<td>S 235 JR (G2)</td>
<td>Rst 37-2</td>
<td>E24-2</td>
<td>Fe 360B</td>
<td>AE 235B</td>
<td>SS 1311 00</td>
<td>40 (A) B</td>
<td>SS400A</td>
<td>Fe 360B</td>
<td>A 284 gr.C.D</td>
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</tr>
</tbody>
</table>

Table 10-3 Amount of each content in SS1311 (MEsteel 2000)

<table>
<thead>
<tr>
<th>C</th>
<th>Si</th>
<th>Mn</th>
<th>Ni</th>
<th>P</th>
<th>S</th>
<th>Cr</th>
<th>Mo</th>
<th>Al</th>
<th>Cr+Mo+Ni</th>
</tr>
</thead>
<tbody>
<tr>
<td>max 0.2</td>
<td>max 0.55</td>
<td>max 1.4</td>
<td>max 0.3</td>
<td>max 0.045</td>
<td>max 0.045</td>
<td>max 0.3</td>
<td>max 0.08</td>
<td>min 0.02</td>
<td>&lt; 0.48</td>
</tr>
</tbody>
</table>
The process of producing steel was still during strong development during the fifties and therefore steel produced during this time often has varying quality regarding toughness and homogeneity (Cotterell 2002). Toughness testing of the Åby bridge was of this reason important for calibration and verification. Material testing was performed on bridge parts after destructive testing had been carried out. This testing was carried out in collaboration with Kjell Eriksson and Complab at Luleå University of Technology in September 2014 (Eriksson 2014). Three different types of testing were carried out for steel taken from three different locations: The bottom chord in the main truss (S1411), one of the crossbeams (S1411) and one of the longitudinal stringers (S1311) were all tested for tensile strength, Charpy V testing and fractural mechanical testing. The size of the specimens is displayed in Figure 10-1 and the retrieving of the specimens is shown in Figure 10-2.

Three specimens for each member were taken for the tensile tests and the fracture mechanical tests whereas five were taken for the Charpy V tests. The number of tests was taken so that a variance could be proven but with limitations of founds. Based on the large variations in the results, it can be concluded that the number of specimens must be higher in order to get reliable results.

### Figure 10-1 Specimens for fracture mechanical testing, tensile testing and charpy V testing

### Figure 10-2 Cutting out specimens for material testing, Upper left: longitudinal stringer, Upper middle: Bottom chord in main truss (16mm bottom flange), Upper right: Crossbeam, Bottom left: Bottom chord in main truss (12mm upper flange), Lower middle: Diagonal, Lower right: Cutting the steel.
10.1.1 Tensile test

Tensile testing was carried out according to SS-EN 10 002 with flat samples with a length of 300 mm for all steel grades. The width of the tested samples was 30 mm towards the ends and 10 mm for the measured part. The transition between ends and measured zone are rounded in order to avoid stress concentrations. The thickness of the specimen was 10 mm and taken symmetrical around half the thickness of the plate. The longitudinal direction of the specimen was oriented in the rolled direction of the material. Tensile testing was carried out at room-temperature. Three specimens were tested for each bridge part. Results from the measured specimens are presented in Table 10-4 to Table 10-6.

Table 10-4 tensile testing of stringer (S1311)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Yield strength [MPa]</th>
<th>Ultimate strength [MPa]</th>
<th>Elongation at failure [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>297</td>
<td>448</td>
<td>29</td>
</tr>
<tr>
<td>2</td>
<td>308</td>
<td>457</td>
<td>29</td>
</tr>
<tr>
<td>3</td>
<td>306</td>
<td>457</td>
<td>40</td>
</tr>
<tr>
<td>Mean</td>
<td>304</td>
<td>454</td>
<td>33</td>
</tr>
<tr>
<td>Deviation</td>
<td>5.9</td>
<td>5.2</td>
<td>6.3</td>
</tr>
</tbody>
</table>

Table 10-5 tensile testing of main chord (S1411)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Yield strength [MPa]</th>
<th>Ultimate strength [MPa]</th>
<th>Elongation at failure [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>316</td>
<td>462</td>
<td>30</td>
</tr>
<tr>
<td>2</td>
<td>337</td>
<td>479</td>
<td>24</td>
</tr>
<tr>
<td>3</td>
<td>348</td>
<td>484</td>
<td>29</td>
</tr>
<tr>
<td>Mean</td>
<td>334</td>
<td>475</td>
<td>28</td>
</tr>
<tr>
<td>Deviation</td>
<td>16.3</td>
<td>11.5</td>
<td>3.2</td>
</tr>
</tbody>
</table>
Table 10-6 tensile testing of crossbeam (S1411)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Yield strength [MPa]</th>
<th>Ultimate strength [MPa]</th>
<th>Elongation at failure [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>319</td>
<td>471</td>
<td>29</td>
</tr>
<tr>
<td>2</td>
<td>297</td>
<td>462</td>
<td>29</td>
</tr>
<tr>
<td>3</td>
<td>322</td>
<td>467</td>
<td>29</td>
</tr>
<tr>
<td>Mean</td>
<td>313</td>
<td>467</td>
<td>29</td>
</tr>
<tr>
<td>Deviation</td>
<td>13.7</td>
<td>4.5</td>
<td>0</td>
</tr>
</tbody>
</table>

According to (Eriksson, 2014) are the results from the tensile tests are normal for the corresponding steel materials and are well within the limitations of codes.

**Toughness testing**

The toughness test has been carried out according to the SS-EN 10 045 standard as Charpy-V tests. The specimens are shaped to the size of 10x10x55 mm, with a 2 mm notch with an angle of 45° and a radius at the tip of 0.2 mm. The length of the specimen is orientated in the rolling direction for the material. The notch-plane is orientated perpendicular to the rolled direction of the material and the growth of the failure is orientated in the specimens’ plane (not in the thickness direction). The specimens temperature at testing was -20°C. Five tests were performed on each structural part, making it fifteen in total. The setup for the testing is displayed in Figure 10-3 and the measured results in Table 10-7.

![Figure 10-3 Test-setup for Charpy-V tests (Esabna 2000)](image-url)
Table 10-7 Toughness tests

<table>
<thead>
<tr>
<th>Part</th>
<th>Thickness [mm]</th>
<th>Toughness KV J</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Sample</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Main chord</td>
<td>16</td>
<td>16.3</td>
</tr>
<tr>
<td>Stringer</td>
<td>24.5</td>
<td>101</td>
</tr>
<tr>
<td>Crossbeam</td>
<td>34</td>
<td>40.7</td>
</tr>
</tbody>
</table>

The steel in the main chord did not fulfill the requirements for class J2 according to Eurocode 3 (previously quality grade D) with a minimum of 27 J at -20°C. The tests from the main chord has a fairly small spread and the mean value is only around 65% of the minimum requirements.

The results from the stringer have some spread. The mean value was substantially higher than the required 24J even though one out of five samples did not exceed the minimum requirement. According to (Eriksson, 2014) is the spread an indication that the steel is inhomogeneous.

The result from the crossbeam has smaller spread that the stringers and all samples exceed 27J by far with a mean result of 53.8 J.

10.1.3 Fracture mechanical testing

The fractural mechanical testing has been carried out according to the Swedish traffic administrations standard BVS 583.12 (Banverket, 2005b). Three samples were made from each element, making it a total of 9, prepared as illustrated in Figure 10-4. The dimensions of each specimen are presented Table 10-8 together with its results in Figure 10-5-Figure 10-7. The specimens were orientated so that the plane of the crack is perpendicular to the rolling direction of the steel material when exposed for three point bending. The thickness of the material was not treated, using the full thickness of the flange. According to (Cotterell, 2002), elderly steel grades are often inhomogeneous meaning that the fractural mechanical properties might vary over the thickness. If only using material from the middle of for example a flange, it is likely to a lower result compared to using the material close to the surface. By using the entire thickness a representative result for the particular member is achieved.

The mechanical reference in the specimens was of Chevron-type. The specimens were exposed to fatigue at room temperature until crack initiation. The initiation and propagation of a crack was electronically monitored and automatically stopped after a certain crack width was reached which also was verified by a visual inspection.

The fractural mechanical testing was carried out at a temperature of -30°C. Before testing, the specimens were cold down to a temperature slightly lower than testing temperature and where kept at this temperature for approximately 30 minutes. During the fractural mechanical testing both loading and displacement of the applied load were monitored. The loading time was normal for slow static testing. After testing was the fractural surface conserved by protective oil.
Bridge over Åby River - Evaluation of full scale testing

The fractural mechanics has been evaluated according to theory of non-linear fractural mechanics. The J-integrals critical value has been decided with the expression seen in Eq1.

\[ J = \frac{2U}{B(W-a)} \]  \quad \text{In which} \quad U = \int_0^{\delta_{cr}} Pd\delta \quad \text{Eq.1}

Where \( P \) is the force on the specimen, \( \delta \) the displacement of the load and \( \delta_{cr} \) the critical value for the first maximum load. \( B \) is the width of the test rod, \( W \) is the height and \( a \) the length of the crack. \( U \) is the deformation energy in the instant crack growth is occurring. \( U \) has been calculated numerically based on the specimens' load-displacement curve where the other entities have been measured directly from the specimen.

![Figure 10-4 Illustration of how samples are taken for mechanical testing (Banverket, 2005b)](image-url)

| Table 10-8 Fractural mechanical properties for the main chord, stringers and crossbeams |
|---------------------------------|----------------|---------------|----------------|---------------|
| **Main Chord**                  | Sample | W [mm] | B [mm] | a [mm] | \( J_c \) [kN/m] |
| 1                               | 32.0   | 16.1   | 15.8   | 427     |
| 2                               | 31.3   | 16.4   | 17.2   | 527     |
| 3                               | 31.5   | 16.1   | 16.3   | 26      |
| **Mean**                        |        |        |        |         | 326.7     |
| **Deviation**                   |        |        |        |         | 265.1     |
| **Stringer**                    | Sample | W [mm] | B [mm] | a [mm] | \( J_c \) [kN/m] |
| 1                               | 49.0   | 25.0   | 25.4   | 123     |
| 2                               | 49.0   | 24.4   | 24.4   | 605     |
| 3                               | 49.4   | 25.0   | 25.5   | 144     |
| **Mean**                        |        |        |        |         | 290.6     |
| **Deviation**                   |        |        |        |         | 272.4     |
| **Crossbeam**                   | Sample | W [mm] | B [mm] | a [mm] | \( J_c \) [kN/m] |
| 1                               | 66.0   | 34.6   | 34.0   | 57      |
| 2                               | 66.0   | 34.4   | 34.0   | 32      |
| 3                               | 66.0   | 34.4   | 34.0   | 46      |
| **Mean**                        |        |        |        |         | 45        |
| **Deviation**                   |        |        |        |         | 12        |
For the main chord the load-displacement curves are for sample 1 and 2 non-linear before failure. The fracture surfaces were mainly plane and orientated perpendicular to the loaded direction. A great amount plastic deformation before failure verified ductility. The load-displacement curve for sample 3 did not show the same non-linear behaviour as the first two before failure. The fact that one specimen is substantially different from the others two, is an indication of material inhomogeneity.

Two out of three samples for the stringer clearly fulfill the requirements for fractural toughness for steel grade 1411 which is $J_c = 40 \text{ kN/m}$. With regard to the lowest fractural toughness (sample 3) should the maximum allowed stress be decreased with 29% meaning that $f_{yd}$ goes from 197 MPa to 140 MPa according to the Swedish assessment standard for railway bridges BVS 583.11 (Trafikverket, 2005).

Fracture mechanical testing of the stringer indicated non-linear behaviour before fracture. Where all samples exceeds with good margin the limiting value for fracture toughness for steel grade 1311
Bridge over Åby River - Evaluation of full scale testing

which is $J_c = 30\, \text{kN/m}$. No decrease in allowed stress levels is necessary for these members (Eriksson, 2004).

For the crossbeams, all measured load-displacement curves for all samples were non-linear before fracture. The variances in dissipated energy between samples were relatively small, compared to the other tests. Some plastic deformation occurs before fractural failure and the mechanical properties prove some ductility.

Two out of three samples exceeds the limiting value for fracture toughness for steel grade 1411 which is $J_c = 40\, \text{kN/m}$. But with regard to the lowest fractural toughness (sample 2) should the maximum allowed stress be decreased with 11% meaning that $f_{yd}$ goes from 197 MPa to 175 MPa according to the Swedish assessment standard for railway bridges BVS 583.11 (Eriksson, 2014).