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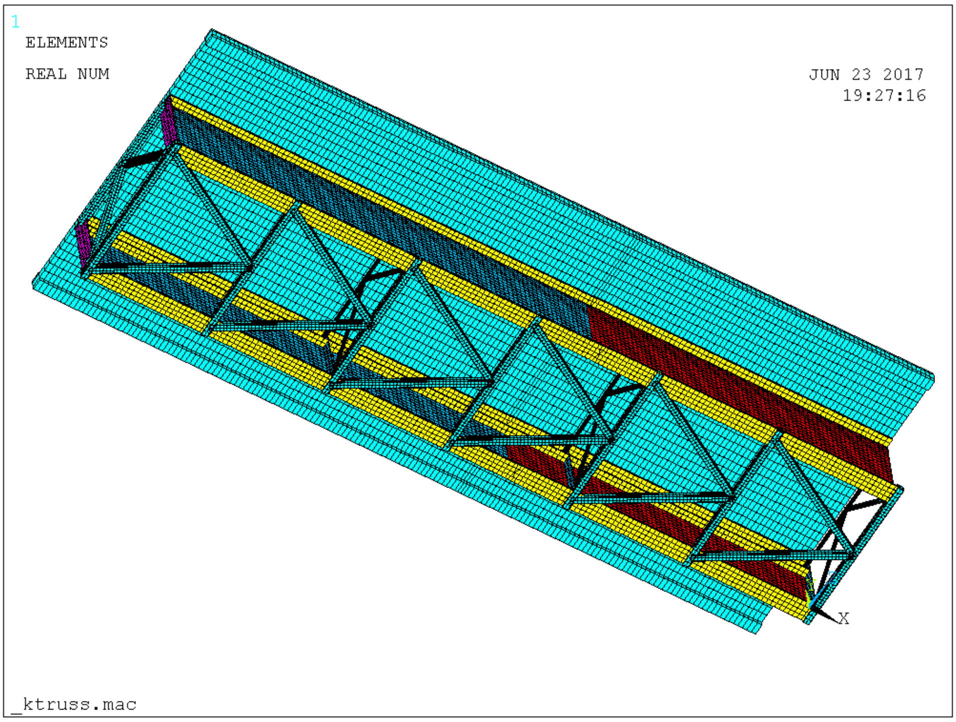
Förlängning av gamla stålbroars livslängd genom lådverkan



Victor Vestman, Peter Collin, Ramboll

2019-07-09

SBUF stödjer
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FÖRORD

Föreliggande rapport beskriver en studie av en innovativ metod för att minska spänningar av främst excentrisk utmattningslast i äldre samverkansbroar av I-balkstyp. Projektet har genomförts av Victor Vestman och Peter Collin (Rambolls kontor i Luleå) samt Mikael Möller, Senior Structural Engineers AB.

Projektet har följts av en referensgrupp bestående av

- Sven-Erik Granli, PEAB
- Tobias Larsson, NCC
- Thomas Blanksvärd, Skanska.
- Hans Petursson, Trafikverket
- Ibrahim Coric, Trafikverket
- Robert Hällmark, Trafikverket

Projektet är en del av det europeiska FoU-projektet Prolonging Life Time of Old Steel and Steel-Concrete Bridges, som har stöttats av

- Svenska Byggbranschens UtvecklingsFond, SBUF
- Research Programme of the Research Fund for Coal and Steel, RFCS
- Trafikverkets Branschprogram BBT
- Liikennevirasto, Trafikverket i Finland
- Statens Vegvesen, Norge
- Rambollfonden
- Ramboll Sverige AB

Luleå, juli 2019

SAMMANFATTNING

Föreliggande rapport beskriver en studie av en innovativ metod för att minska spänningar av främst excentrisk utmattningslast i äldre samverkansbroar av I-balkstyp. Idéen är att skapa vridstyvhet genom att fästa fackverk i underflänsarna, och därigenom få ett vridstyvt tvärsnitt vilket gör I-balkarnas deformationer och därigenom även böjspänningar följs åt.

För nya broar används konceptet i Finland för nya broar med större spännvidder, så även i Spanien. I Spanien har man vid större spännvidder på vissa broar gjutit betong mellan I-balkarnas underflänsar för att dels få samverkan mellan den tryckta betongen i underkant över stöd, dels vridstyvhet. I fältpartierna har denna lådbotten i betong ibland ersatts av fackverk.

Två viktiga fördelar med förstärkningsmetoden är att den torde kunna ske under bron utan att arbetsmiljön för entreprenörens personal försämras av förbipasserande fordon, samt att bron inte behöver stängas av.

De tre broar som studerats i projektet: Bergeforsen, Smedjebacken och Pitsundsbron, visar alla att den ökade vridstyvheten i det av fackverken slutna tvärsnittet spelar stor roll för fördelningen av laster, moment och spänningar. Då effekterna av egenvikten i brottgränstillstånd, redan är jämnt fördelade, får metoden anses ha störst potential för utmattning. Om balkarna kan dela lasten mer broderligt, genom att fördela lasteffekterna från excentriska laster, kan spänningsvariationen reduceras. Exempelvis skulle en spänningsreduktion ~30% i undre flänsen, (d.v.s. excentrisk last fördelas mer lika mellan balkarna) och en lutning, m=5 i Wöhler-diagrammet betyda en förlängning av kvarvarande livstid med (1/0,7)^5 ≈ 6 ggr.

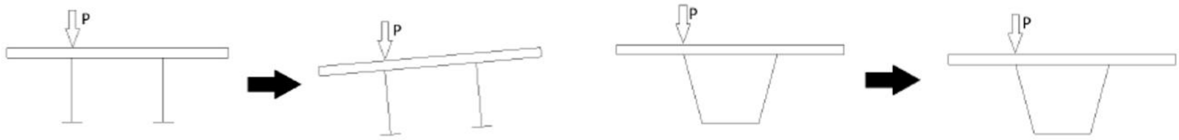
Vidare har vi funnit att en andra funktion för fackverket, att bidra till underflänsen i global böjning, måste anses ha underordnad betydelse. Vår förhoppning är att framgent få förstärka en bro i Sverige eller Finland med detta koncept.

INNEHÅLL

1 Inledning

Broar är av avgörande betydelse för infrastrukturen runt om i världen. Med tusentals gamla stål- och samverkansbroar runt om i Europa finns det behov av rationella metoder för förstärkning av dem. Detta inte bara för att kompensera för deras åldrande, utan även för att kunna tillåta högre belastningar i framtiden. Att kunna förstärka och spara äldre broar på ett för entreprenören effektivt sätt bidrar även till en minskad miljöpåverkan, samtidigt som trafikstörningar kan minimeras genom innovativa förstärkningsmetoder. Underhåll av befintliga broar kommer att vara en växande utgift för broförvaltare, vilket innebär att sådana här projekt ligger till stor vikt för fortsatt effektivisering och utveckling inom området.

Det här rapporterade projektet handlar om att skapa lådverkan för I-balksbroar med samverkande betongplatta genom att fästa ett fackverk, bestående av VKR-balkar, mellan de nedre flänsarna. Lådverkan för tvärsektionen innebär att balkarna delar broderligt på excentriska laster vilket reducerar utmattningsspänningar vid excentrisk belastning, se figur 1. Vidare behandlas också utveckling av friktionsförband där tester av friktionsförband med förspända skruvar utförs. Förstärkningen av dessa förband utförs genom att uppnå en högre friktionskoefficient med små stål- och/eller keramik-kulor som placeras mellan ytorna som de förspända skruvarna kopplas till. På detta sätt kan en högre friktion mellan de förspända ytorna erhållas än när inga kulor används. Detta projekt har initierats för att skapa en djupare kunskap kring dessa förstärkningsmetoder.



Figur 1. Schematisk skillnad mellan hur låd- och I-balktvärsnitt tar excentrisk belastning.

Projektet är en del (ett av 6 arbetspaket) av det europeiska FoU-projektet (RFCS[1]) PROLIFE (Prolonging Lifetime of Old Steel and Steel-Concrete Bridges). Det är ett samarbetsprojekt mellan sju forskningspartners från sex olika länder, och koordineras av Luleå Tekniska Universitet, (LTU). Andra delar av projektet inkluderar förstärkning genom att i efterhand skapa samverkan mellan stål och betong, genom att installera skjuvförbindare i form av spiralbultar underifrån och binda samman en stålbalks överfläns med ovanliggande betongplatta, samt förstärkning av gamla fackverksbroar. Inom projektet finns forskare, brokonstruktörer och materialleverantörer representerade. Som avstamp för forskningsprojektet anordnades i september 2015 i Stockholm en Workshop där europeiska konstruktörer, forskare och broägare (trafikverk) från 12 länder närvarade, se figur 2.

[1] Research Fund for Coal and Steel. Finansierad via industriella aktörer inom den tidigare Kol och Stål Unionen.
För information om RFCS, se www.metalliskamaterial.se.

Figur 2. Deltagare från 12 europeiska länder vid inledande workshop i Stockholm hösten 2015.

2 Bakgrund

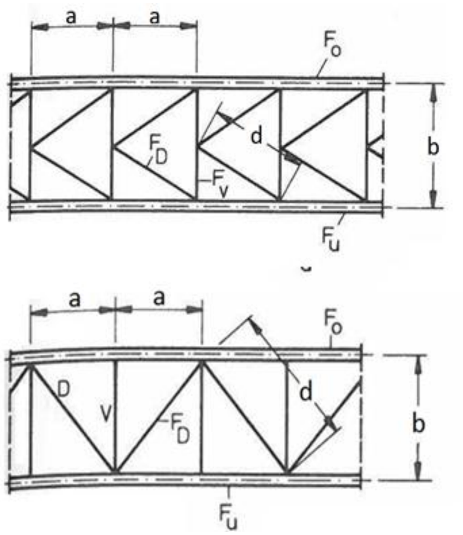
Under de senaste decennierna har trafikvolymen på de europeiska väg- och järnvägsnäten ökat markant, vilket lett till att nya väg- och järnvägsprojekt utförts i en allt högre takt för att möta behovet. Samtidigt som det byggs nytt så finns dock en stor mängd befintliga broar som måste kunna bära den allt högre trafikbelastningen. Många av dessa konstruktioner har dimensionerats för helt andra trafikmängder och fordonsvikter än de som idag tillåts. Ur en samhällsekonomisk synvinkel är det väldigt fördelaktigt att via tämligen små åtgärder förlänga livslängden på de befintliga konstruktionerna och skjuta investeringskostnaderna på framtiden. En av flertalet åtgärder som studeras i RFCS-projektet PROLIFE är att på ett för entreprenören och samhället tids- och kostnadseffektivt sätt skapa lösningar för förstärkningsarbeten på äldre stål- och kompositbroar.

Vid förstärkning av existerande I-balksbroar med samverkan är ett koncept att införa lådverkan i tvärsnittet. Det kan göras genom att införa ett horisontellt fackverk mellan de nedre flänsarna. Detta i sin tur kommer att innebära att excentriska laster ger upphov till ett vridande moment som kommer transporteras i form av tvärkrafter runt i tvärsnittet. K-fackverk, se figur 2, kan vara att föredra framför andra typer av fackverk då andra typer gör så att diagonalerna medverkar i den globala böjningen, vilket gör de känsliga för knäckning mellan knutpunkterna. Detta betyder mycket i brottgränstillståndet, men mer i utmattningsgränstillståndet. I utmattningsgränstillståndet så bestäms utmattningen från de spänningsvariationer som uppkommer i specifika detaljer i bron, till exempel svetsade detaljer i en I-balk. Om balkarna kan dela lasten broderligt eller åtminstone som styvbröder, genom att fördela lasteffekterna från excentriska laster, kan spänningsvariationen reduceras. Exempelvis skulle en reducering med ~30% i undre flänsen, (d.v.s. excentrisk last fördelas mer lika mellan balkarna) och en lutning, m=5 i Wöhler-diagrammet betyda en förlängning av kvarvarande livstid med (1/0,7)^5 ≈ 6 ggr. Två viktiga fördelar med förstärkningsmetoden är att den kan ske under bron utan att arbetsmiljön för entreprenörens personal försämras av förbipasserande fordon, samt att bron inte behöver stängas av.

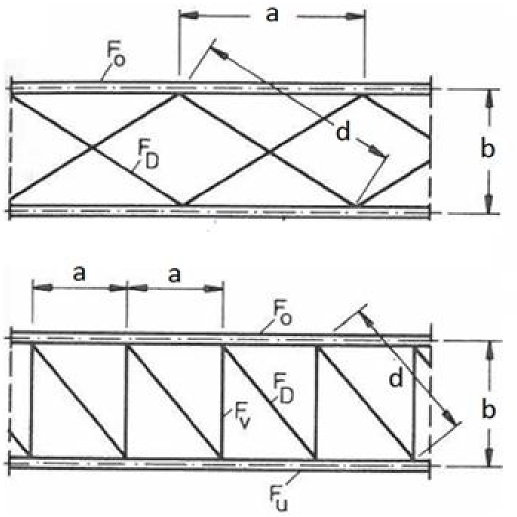
Sedan införandet av Eurokoderna har dimensioneringskraven för nya stål- och samverkansbroar med avseende på utmattning blivit hårdare, och många länder har skruvat ned dessa genom att minska säkerhetsfaktorn γ_{Mf} eller minska det dimensionerande axeltrycket Qm1. Eurokoderna gäller inte för befintliga konstruktioner men det finns planer att inkludera även dessa i Eurokodsystemet. finns flera vägar att öka återstående livslängd med avseende på utmattning för gamla samverkansbroar. Bland dessa kan nämnas:

- att höja detaljkategorin i brons svetsförband
- att ändra det statiska systemet för att minskar snittkrafterna
- att skapa samverkan med en eventuell betongfarbana utan samverkan
- att helt enkelt minska spänningarna genom att addera mer material genom skruvning, limning eller svetsning.

Detta projekt har fokuserat på idén att väsentligt öka torsionsstyvheten hos bron, genom att använda ett horisontellt fackverk för att skapa en låda där toppen och sidorna bildas av betongplattan och stålplattorna. Detta kräver att stålbalkarna är anslutna till betongdäcket, vilket medger ett skjuvflöde i det lådformade tvärsnittet. Effekten av fackverken har beskrivits av Roik [1] genom att lägga en fiktiv extra bottenfläns till I-balkarna, med en tjocklek som ger tvärsnittet samma styvhet för vridning som fackverket. Se även Figur 3 och 4 nedan.



Figur 3 K- och D-fackverk.



Figur 4. X- and N-fackverk.

Den fiktiva tjockleken ges av Roik samt i Handboken Bygg [2] som.

K-fackverk:

$$t^* = \frac{E}{G} * \frac{ab}{\frac{2d^3}{F_D} + \frac{b^3}{4F_V} + \frac{a^3}{12}\left(\frac{1}{F_O} + \frac{1}{F_u}\right)} \tag{2.1}$$

D-fackverk:

$$t^* = \frac{E}{G} * \frac{ab}{\frac{d^3}{F_D} + \frac{a^3}{3}\left(\frac{1}{F_O} + \frac{1}{F_u}\right)} \tag{2.2}$$

X-fackverk:

$$t^* = \frac{E}{G} * \frac{ab}{\frac{d^3}{2F_D} + \frac{a^3}{12}\left(\frac{1}{F_O} + \frac{1}{F_u}\right)} \tag{2.3}$$

N-fackverk:

$$t^* = \frac{E}{G} * \frac{ab}{\frac{d^3}{F_D} + \frac{b^3}{F_V} + \frac{a^3}{12}\left(\frac{1}{F_O} + \frac{1}{F_u}\right)} \tag{2.4}$$

där

F_V är arean av transversalerna (vertikalerna) i fackverket

F_D är arean av diagonalerna i fackverket

F_o och F_u är arean hos underflänsen + en medverkande area hos nedre delen av livet.

I Handboken Bygg ges denna area som ¼ av livarean, men med FEM-beräkningar behöver inte denna approximation användas.

I Finland används en liknande typ av fackverk för nya broar med längre spännvidd, se Figur 5. Texten i de finska kompletterande reglerna [3] för Eurokoderna lyder, grovt översatt:

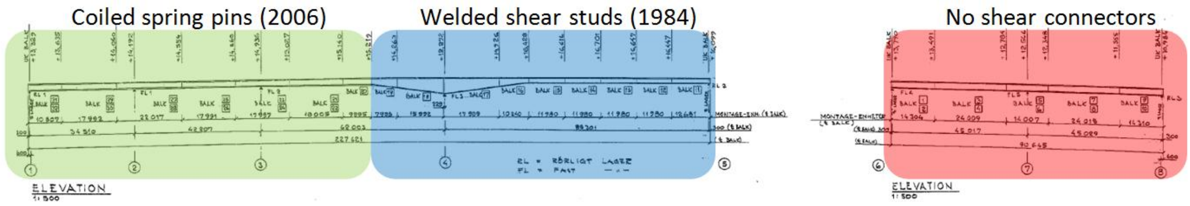
När spännvidden är över 50-70 meter rekommenderas att man lägger till en fackverk bottenkrok nära bottenflänsplanet, vilket i praktiken skapar ett boxliknande beteende som är både funktionellt och ekonomiskt försvarbart. Detta ökar väsentligt torsionsstyvheten hos strukturen, vilket därmed bidrar till att minska välvning samt ger bättre fördelning av lasten mellan balkarna.

Denna praxis har inte författarens kunskap inte antagits i andra länder förutom i speciella fall i Frankrike och Spanien.

Figur 1. Lansering av finsk bro med K-fackverk mellan underflänsarna.

3 Beräkningar bro över Pitsund

Totalt tre broar (broar vid Bergeforsen, Smedjebacken samt Pitsund) har studerats. Resultat från bro vid Bergeforsen redovisas i [4] och [5]. I Bilaga A redovisas resultat från beräkningarna av bron över Pitsund, dessa har även redovisats vid ECCS brokonferens i Prag september 2018 [6]. Då Pitsundsbron (Figur 6) är en äldre bro har inte hela bron ursprungligen dimensionerats för samverkan, men en del förstärktes 2006 med en för Skandinavien ny metod, fjädersprintar (Spirols) som slogs in genom borrade hål i överflänsen och betongfarbanan. På så vis behövde man inte ta bort beläggning och betongen, och trafikstörningarna blev minimala. Som framgår av Bilaga A räknar vi hur som helst med samverkan i de högra 2 spannen i figur nedan.



Figur 6. Pitsundsbron i Norrbotten, byggd 1984, där effekterna av att införa samverkan i efterhand 2006 studerats i SBUF-projektet Förlängning av gamla stålbroars livslängd genom samverkan.

Förutom de i Bilaga A diskuterade resultaten kan följande slutsatser lyftas fram:

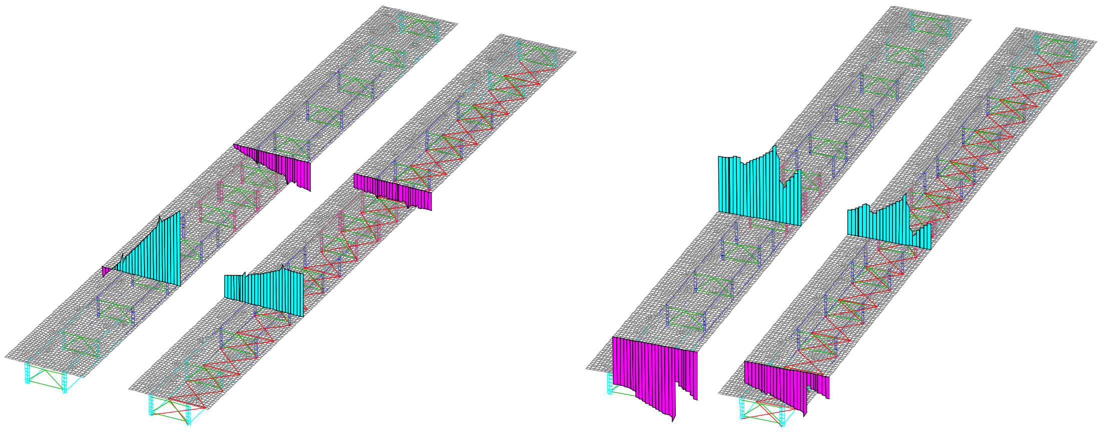
- För studerad värsta utmattningsdetalj vid en livavstyvning x=66 m från broände erhöllls i fallet inget fackverk spänningsvidden 49.8 MPa. Med fackverk mellan underflänsarna erhölls mellan 3 och 12 gånger förlängning av brons återstående livslängd jämfört med fallet ingen förstärkning. Det relativt stora spannet är avhängigt konfigurationen av fackverket samt dimensionen på diagonalerna. Se även tabell 1.
- Alla tre typer (D-, K- samt X-fackverk) verkar avsevärt kunna förlänga brons livslängd med avseende på utmattning.
- Det är viktigt att notera att beräkningarna förutsätter samverkan mellan betong och överflänsar för att skjuvflödet ska kunna verka runt hela det fiktiva lådtvärsnittet.
- En iakttagelse som kan nämnas är att spänningarna i värst belastad balk minskar ungefär lika mycket som nedböjningen av denna. Vidare är summan av spänningarna i vänster+höger balk ungefär konstant med eller utan fackverk, det samma gäller även för nedböjningarna.
- Förutom vridstyvhet kan det extra fackverket bidra med global styvhet till tvärsnittet hos bron, genom att diagonaler adderas i samma plan som underflänsarna, som därigenom skulle kunna få minskade spänningar av trafiklasten. Numeriska studier visar dock att denna effekt dock inte är så stor på grund av att arean hos diagonalerna är mindre än underflänsens area, samtidigt som vinkeln mellan balkar och diagonaler och töjningar i transversaler mellan underflänsarna minimerar effekten. (Utan transversaler i knutpunkterna böjer underflänsarna undan, och effekten uteblir i princip.)

Tabell 1. Böjspänningar av trafik efter förstärkning jämfört med ingen förstärkning.

Studerat fackverk	$\Delta\sigma^{1}_{m+b}$ [MPa]	Ratio
D 100 x 100 x 5	38.5	0.77
D 200 x 200 x 10	32.5	0.65
K 100 x 100 x 5	39.6	0.80
K 200 x 200 x 10	32.8	0.66
X 100 x 100 x 5	33.1	0.66
X 200 x 200 x 10	30.6	0.61
Inget fackverk	49.8	-

För betongfarbanans del kan följande slutsatser dras av beräkningar på Pitsundsbron, se även Figur 7:

- Tack vare bättre lastfördelning mellan balkarna minskade den maximala tryckspänningen av böjmoment i betongfarbanan.
- Skjuvflödet i betongfarbanan för värst belastad balk minskar, samtidigt som det Saint Venantska skjuvflödet mellan balkarna ökar.
- Tack vare det efter förstärkning stora slutna tvärsnittet minskar det vridmoment som tas av bara farbanan till ungefär hälften, det stora tvärsnittet har ju mycket större area än betongfarbanan i sig.



*Figur 7. a) Normalspänningar respektive b) skjuvspänningar i av utmattningsfordon på Pitsundsbrons farbana utan respektive med förstärkning (X-fackverk 150*150*5 mm).*

4 Tester av nya friktionsförband

Ofta vill man av toleransskäl inte använda passförband vid montage på plats. Detta torde gälla i än högre grad om en del konstruktionen redan är på plats, och en del tillverkas i verkstad och det inte finns någon möjlighet att samborra eller provmontera i verkstaden. Ett alternativ är friktionsförband men det innebär att ytorna på befintlig bro måste blästras och förses med nytt ytskikt, vilket i sig kan innebära mycket merarbete och dessutom risk för att arbetet förorenar underliggande vattendrag.

För att studera möjligheten att uppnå hög friktion utan att röra befintligt målningssystem, med användande av kulor som trycks in i materialet vid uppspänning av skruvförbanden, har två unika provserier utförts vid LTU;

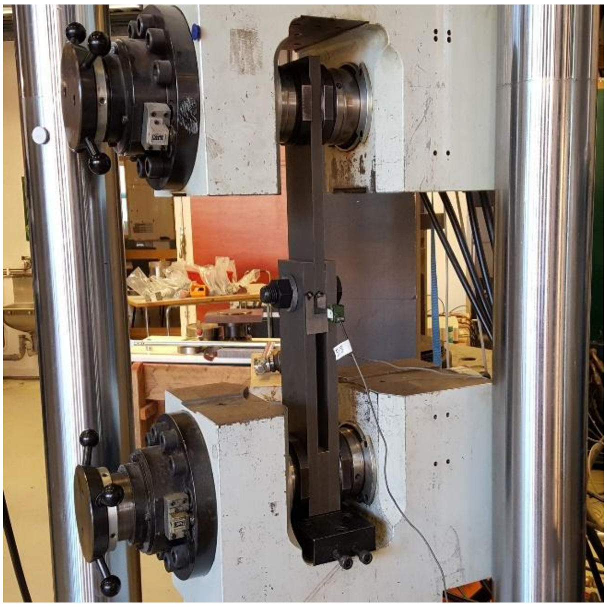
1) En serie i Mattias Varedians examensarbete [7], [8] med omålade plåtar, där såväl nödvändig klämkraft för intryckning som beteendet hos ett tvåskärigt förband belastat med tvärkraft studerades. Förbanden förspändes med 320 kN, skruvar M30.

2) En serie med målade ytor där motsvarande försök gjordes [9]. Förbanden förspändes med 240 kN, skruvar M24.

De cirkulära kulor som studerats har varit utförda i

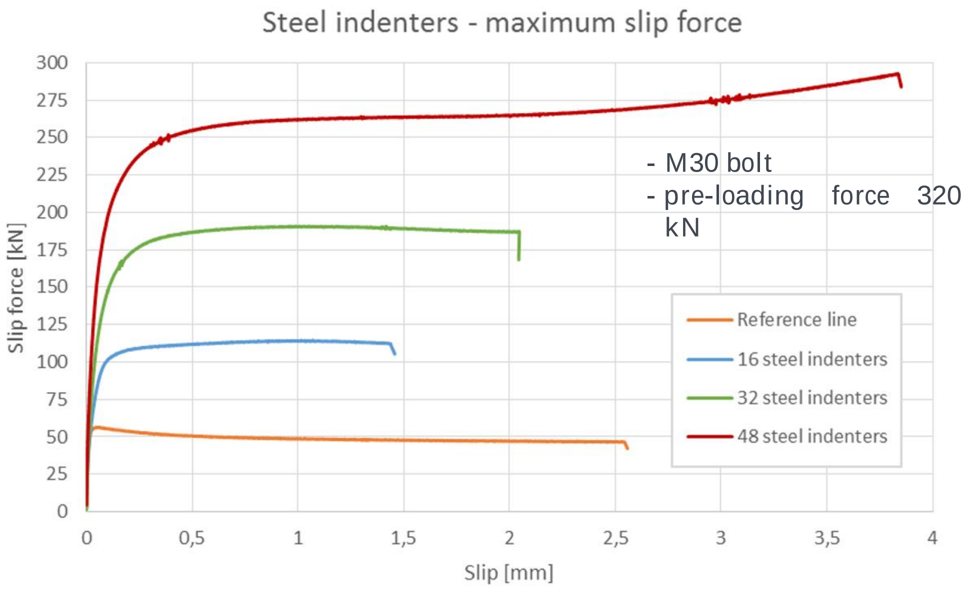
- rostfritt stål
- tungstenskarbid
- aluminiumoxid
- silikonnitrid

Försöksuppställningar och resultat beskrivs i Bilaga C, samt även i Varedians examensarbete. Där framgår även de försök som gjorde avseende nödvändig förspänningskraft för att få kulorna att tryckas ner i plåtarna, eller omvänt hur många kulor en skruv M30 förmår trycka in vid uppspänning. Försöksuppställning för tvåskäriga skjuvförband framgår av figur 8.



Figur 8. Försöksuppställning vid skjuvbelastning av förband med kulor i anliggningsytorna.

I figur 9 visas beteendet i förband för de omålade plåtarna med rostfria stålkulor. För rena plåtar utan kulor fås, med två anliggningsytor, friktion vid maxlast ~ 57 kN/2*320 kN förspänning = 9%. Med 16, 32 respektive 48 2.5 mm stålkulor stiger friktionen avsevärt, se figur 9 nedan.

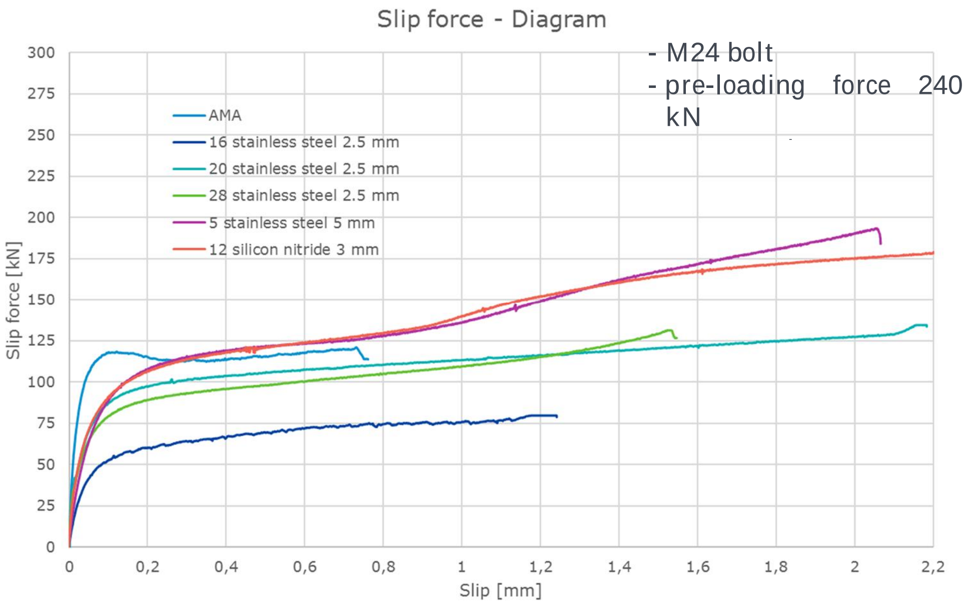


Figur 9. Last-förskjutningsdiagram för försöksuppställning enligt figur 7, omålade plåtar.

En slutsats är att kulorna verkligen höjer lasten innan glidning i förbanden. En annan slutsats är att kulorna i sig verkar ge ett stabilare beteende hos förbanden. Med detta avses att efter att en inledande ”statisk friktion övervunnits minskar lasten för fallet plåt mot plåt. Detta ses inte hos förbanden med kulor, utan där ökar tvärtemot lasten för fallet med 48 kulor.

För att vidare studera konceptet gjorde försök med plåtar målade med AMA-system för svenska broar. Detta består av rostskyddande zink epoxy, järnglimmerpigmenterad epoxy samt täckfärg av polyuretan, totalt drygt 0,3 mm per målad yta. I figur 10 nedan ses skjuvförbandens beteende för fallen

- inga kulor
- rostfria kulor D=2.5 mm
- rostfria kulor D = 5 mm
- kulor av silikonnitrid, D = 3 mm



Figur 10. Last-förskjutningsdiagram för försöksuppställning enligt figur 7, målade plåtar enligt AMA-system.

Av figur 10 framgår att maximal skjuvkraft innan glidning för fallet inga kulor är avsevärt högre för plåt målad med AMA-system än omålad plåt, jfr figur 9. En annan slutsats är att kulorna med 2.5 mm diameter snarast verkar som kullager för fallet med AMA-system, beroende på att de inte förmås tränga ner tillräckligt. De större kulorna med diameter 5 mm verkar dock bita och ger en monotont ökande kraft ända upp till 2 mm förskjutning.

5 Diskussion och slutsatser

- Konceptet att förstärka äldre I-balksbroar av stål + betong genom att med ett fackverk mellan underflänsarna få balkarna att dela på lasten mer broderligt har stor potential. Effekten märks tydligast vid dimensionering mot utmattning, eftersom all egenvikt av stålbalkar, betongfarbana och beläggning redan är jämnt fördelad i brottgränstillstånd. Fackverket kommer vidare bara att göra nytta för tillkommande laster efter att det monterats.
- Även om det finns analytiska lösningar för att räkna ut hur exempelvis lasten av en excentriskt placerad lastbil fördelas mellan balkarna, torde normalfallet vara att utföra beräkningarna med FEM.

- Vilken typ av fackverk som väljes kommer bland annat att bero av brons geometri och befintliga tvärförband, och likaså av de detaljer som konstruktören väljer för knutpunkterna. Lösningarna i Bilaga B ska därför ses som förslag och inte regler, och praxis kommer att utarbetas i samband med att metoden används.
- Vridstyvheten som brotvärsnittet får av fackverket gynnar inte bara stålet utan även betongfarbanan.
- Utan liggande vertikaler i fackverket mellan underflänsarna (och vinkelrät mot dessa) kommer fackverkets diagonaler inte att kunna hjälpa till i global böjning. Även med vertikaler blir hjälpen dock liten p.g.a. mindre dimensioner i diagonaler jämfört med underflänsen, vinkeln mellan underfläns och diagonaler samt de töjningar som uppstår i vertikalerna.
- Kulor som trycks in i plåtar vid förspänning av skjuvförband verkar kunna höja lasten för när förbandet börjar glida (figur 9) samt dessutom ge ett mer stabilt beteende vid stora deformationer. Då det är orimligt att en stålmontör ska behöva hålla reda på enstaka kulor skulle kulorna komma i en matris på ett ark liknande till exempel ett sandpapper, med hål för skruven i mitten. Om man på detta sätt skulle kunna reducera antalet skruvar till häften, utan att behöva blästra/måla på plats, vore mycket vunnet.

Litteraturförteckning:

[1] Roik, K. (1983). Vorlesungen über Stahlbau : Grundlagen. 2nd ed. Berlin: Berlag von W. Ernst & Sohn.

[2] Handboken BYGG (1971). Bygg - Allmänna grunder. Del 1A, Kapitel 156:274.

[3] Liikenneviraston (2016). Eurokoodin soveltmisohje. Teräs - ja liittorakenteiden suunnittelu – NCCI 4, 25.8.2016ISBN 978-952-317-306-4, (in Finnish)

[4] Vestman, V. (2015). Improvement of Fatigue Resistance Through Box Action For I-Girder Composite Bridges, Master Thesis, Luleå University of Technology.

[5] Vestman V; Häggström J; Collin P;. Improvement of fatigue resistance through box-action for I-girder composite bridges, IABSE Stockholm 2016, ISBN 978-3-85748-144-4, s1988-1994

[6] Vestman, V., Collin, P. & Möller, M. (2018). Box-action giving new life-time to old steel bridges, 9th ECCS Prague 2018.

[7] Varedian, M., A New Improved Type of Friction Connection: An Experimental Study Master Thesis, Luleå University of Technology, 2016

[8] Varedian, M., Collin, P., & Eriksson, K. (2017). A New Improved Type of Friction Connection- An Experimental Study, IABSE Vancouver 2017, ISBN 978-3-85748-153-6.

[9] Varedian, M., Collin, P., Eriksson, K. & Andersson, E. (2018). An Experimental Study of Friction Connection for Different Surface Treatments, 9th ECCS Prague 2018.

Bilaga A Beräkningar Pitsundsbron

Box-action giving new life-time to old steel bridges

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Abstract. When strengthening existing I-girder composite bridges one idea is to make the cross section act like a box section, by adding a horizontal truss between the bottom flanges. This means that eccentric loads produce a torque that is transferred by shear forces around the section. The magnitude of the effects coming from introducing such a framework between girders is addressed in this article. The fatigue resistance will be improved by the reduced stress ranges and increased amount of tolerated load cycles and extend the lifetime of the details, and by so the lifetime for the bridge. The work described in the paper is part of the European R&D project Prolonging Life Time of Old Steel and Steel-Concrete Bridges (ProLife), RFCS 2015-00025.

1. Introduction

Many old I-girder steel/composite bridges are too weak and in a need of replacement, repair or strengthening.

The rules for assessment/classification vary between the European countries. In addition, newer bridges designed according to the Eurocodes for example often give much tougher design in fatigue, which for example can give twice as large bottom flange in mid span than the old Swedish codes, which means that very few of the old bridges would survive a check with the Eurocodes for new bridges.

For symmetric I-girder bridges the loads from the weight of the steel and concrete are generally evenly distributed between the girders, just as for box girder bridges. For bridges consisting of two I-girders the concrete deck is often considered as simply supported in the transverse direction on top of the girders, meaning that a concentrated load on top of one girder will be distributed to only that girder, with no help from the second girder. In reality the torsional stiffness of the deck and the warping stiffness of the whole composite section however transfer some of the load so the real distribution can be about 90 % for the loaded girder and 10 % for the other, depending on the geometry of the bridge. For a box section this is not the case, the both halves parts of the box share the eccentric loads almost equally.

When strengthening existing I-girder composite bridges, one concept is to make the cross section act like a box section, by adding a horizontal truss between the bottom flanges. This means that the eccentric loads produce a torque that will be carried by shear forces around the section. The preferred type of truss is a K-truss, since other types will force the diagonals to take part in the global bending, which will make them sensible to buckling between the joints. This means a lot in the Ultimate Limit State (ULS), but even more in the Fatigue Limit State (FLS). In the FLS the fatigue is determined by the stress ranges in certain parts of the structure, for instance the welded details of an I-girder. If the girders can act together, sharing the moment from an eccentric load evenly should a lower stress range can be achieved. The increased amount of load cycles that the bridge can withstand, with the new distribution between the girders (70/30) can be up to six times, compared to an un-strengthened bridge with the distribution (100/0) [1].

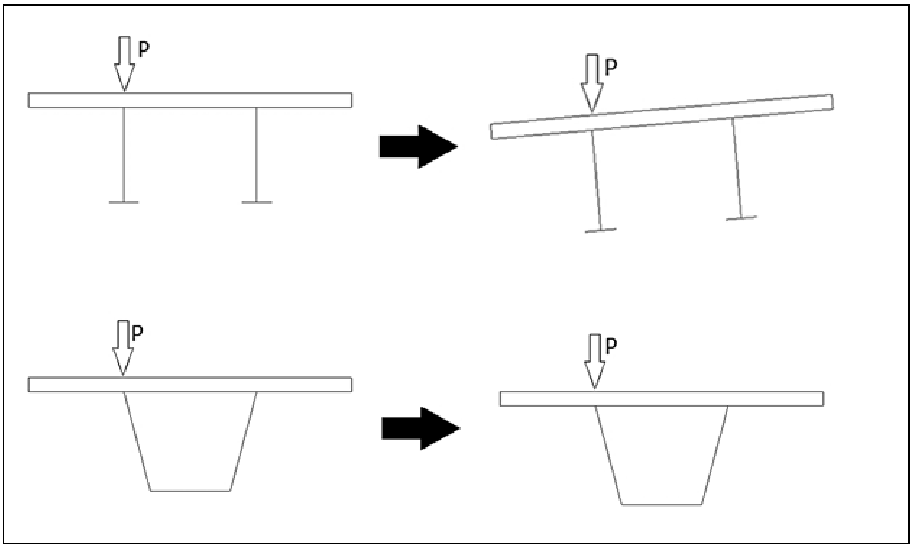


Figure 1. Deflection corresponding to different type of cross section.

2. Case study of the Pitsund Bridge

The Pitsund Bridge is a seven span bridge, with a total length of 399 m and a free width of 9 m, located in the northern part of Sweden outside the city of Piteå. In six of seven spans, the superstructure consists of steel girders with a concrete deck slab on top. The seventh span is a movable span, designed as a bascular bridge with two leafs.

This bridge was chosen for the case study with the strengthening method of a horizontal truss. The bridge has three different types of cross-section compositions. The first part which was strengthened in 2006 with coiled spring pins to achieve composite action. The second part which was original designed as a composite cross section with welded shear headed studs in 1984. The last span is a non-composite section with steel I-girders and a concrete deck with only a few anchoring rebars that connect the steel and the concrete.

The part chosen for this case study is the last one, indicated in red in Figure 2. This span is today like said without any composite action between the girders and the deck. The idea is that if a strengthening should be needed for this part of the bridge, the first part of the strengthening should be with the same method used in the first span, with the coiled sprig pins. There after the bridge could be strengthened further with a horizontal truss. The composite action provided by the coiled spring pins is essential for the cross section with the horizontal truss to work as a box section. This is because of that the non-composite cross section should be useless to be strengthened by a horizontal truss. Without composite action between the steel and the concrete the cross-section would theoretically still be an open cross section instead of a closed section which is the whole idea with the truss between the bottom flanges. In this case study is therefore assumed that this part of the bridge is a composite cross section.

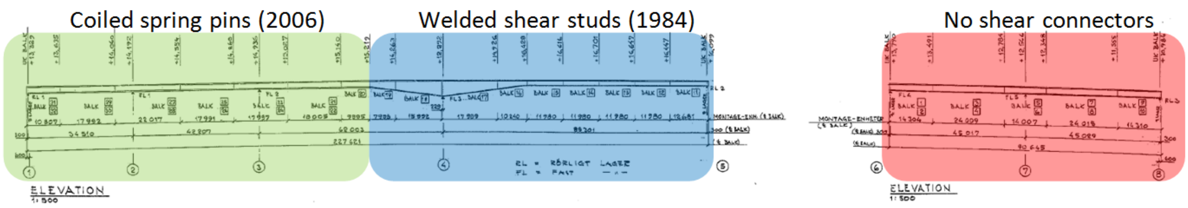


Figure 2. Elevation drawing of the Pitsund Bridge.

This part of the bridge is a continues bridge in two spans both 45 m long with a total length of 90.6 m including 0.3 m extension of the girders in both ends. The elevation drawing of the steel girders in these spans is shown in Figure 3.

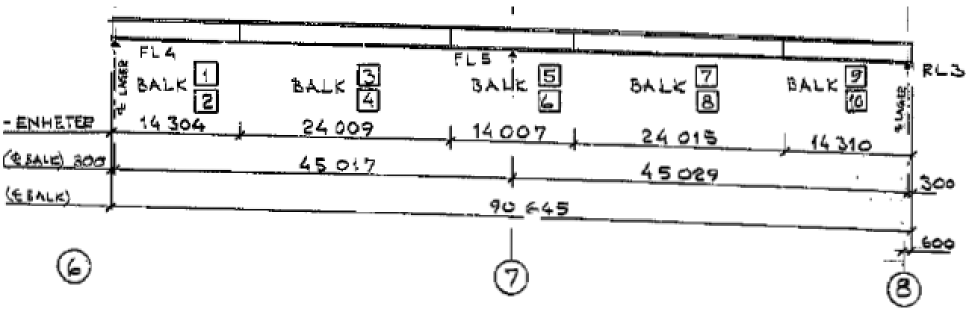


Figure 3. Elevation drawing of the chosen part for the case study.

The cross-section consists of two steel I-girders with a non-composite deck slab on top. Figure 4 illustrates the typical steel cross-section for this part of the bridge. The distance between the centre lines of the webs is 4.5 m, and the height of the steel section is ~2.4 m.

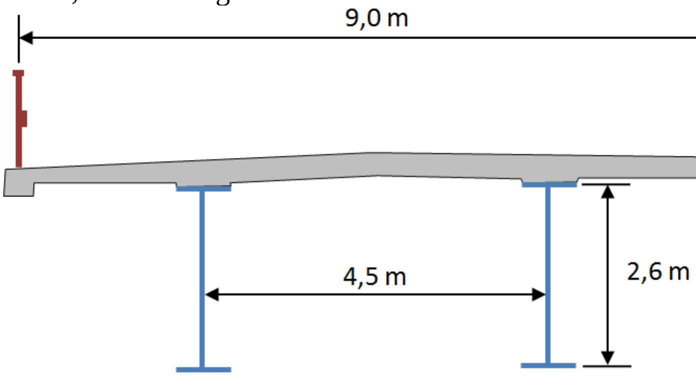


Figure 4. Typical cross-section of the superstructure for the chosen part of the bridge.

2.1. FEM-modelling

The FE-model and the analysis are made in the program ANSYS and the model consists of a concrete slab which is modelled by 8-node solid elements. The steel girders are modelled by 4-node shell elements and the cross-beams and horizontal trusses are modelled by beam elements. In Figure 5 the model is showed with different colours for the steel girders illustrating the different dimensions in the cross sections.

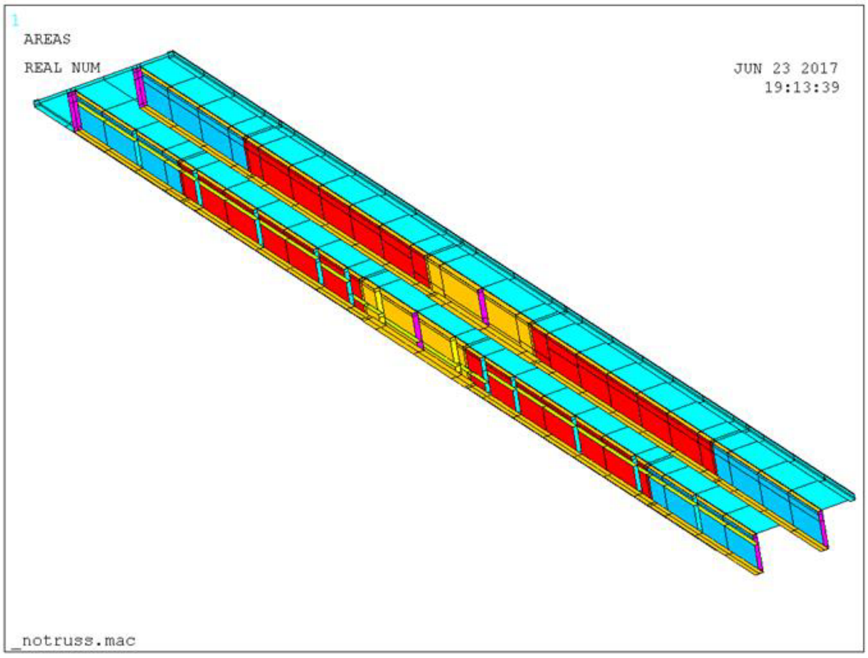


Figure 5. Bridge from below, no cross beams are showed.

The meshing of the model and the configuration of the cross beams are illustrated in Figure 6 and Figure 7.

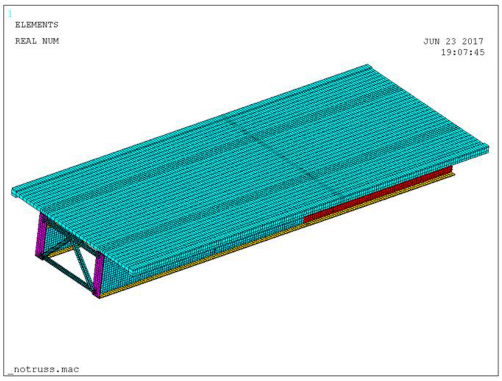


Figure 6. Bridge segment of the bridge from the above, no horizontal truss is showed.

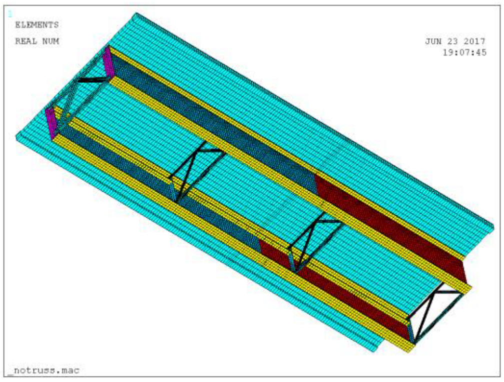


Figure 7. Bridge segment of the bridge from the below, no horizontal truss is showed.

3. Strengthening with a horizontal truss

Three types of shapes for the horizontal truss between the lower flanges are analysed in this study.

- K-shaped horizontal truss, see Figure 8
- X-shaped horizontal truss, see Figure 9
- D-truss, the truss is with diagonal beams between the lower flanges, see FIGUREE

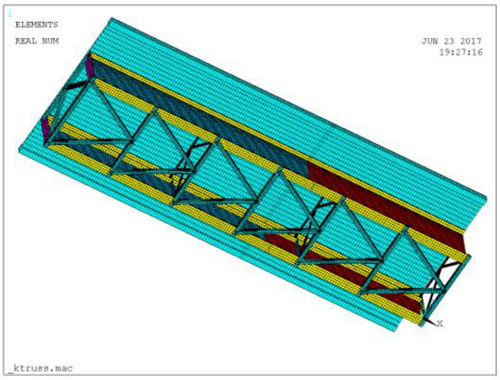


Figure 8. Bridge segment, from beneath, horizontal K-truss, RHS 200 x 200 x 10

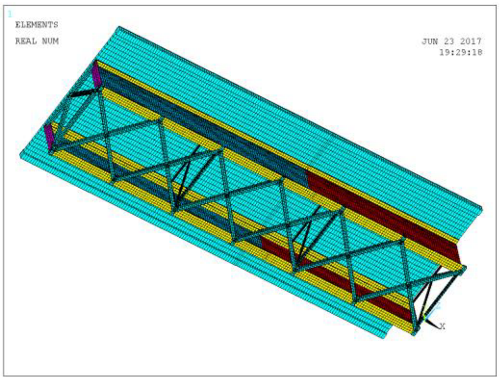


Figure 9. Bridge segment, from beneath, horizontal X-truss, RHS 200 x 200 x 10.

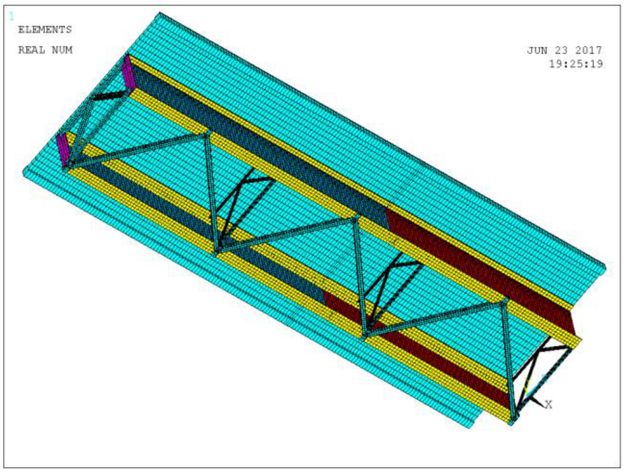


Figure 10. Bridge segment, from beneath, horizontal D-truss, RHS 200 x 200 x 10

4. Fatigue load

The fatigue vehicle used in this case study is the fatigue load model 4 [2]. For this study however the main aim is not to validate the fatigue damage but how much the stress from the fatigue load can be decreased by the strengthening method with horizontal truss.

Two main fatigue details are checked for unstrengthen and strengthened models. The first, located 66 m from the left (support 6), is the web stiffener, see Figure 11 and Figure 12. The other fatigue detail is the on-site welded joint, see FIGUREE AND FIGUREE, which is located at 76 m from support 6.

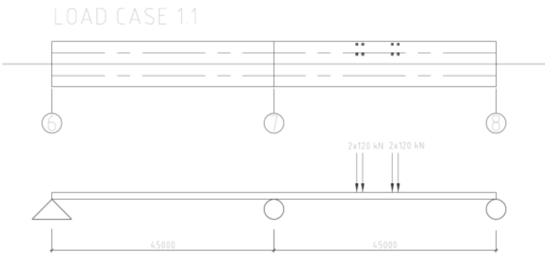


Figure 11. Load case for maximum stress at point 66 m from support 6.

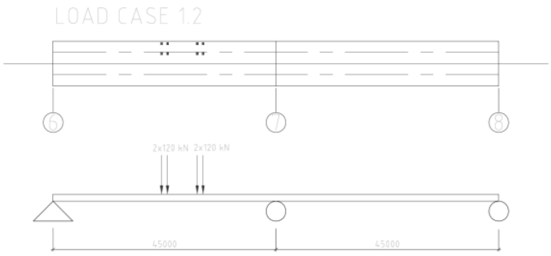


Figure 12. Load case for minimum stress at point 66 m from support 6.

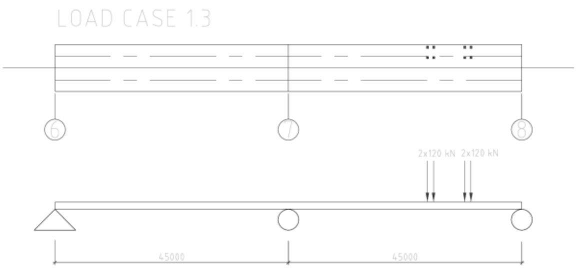


Figure 13. Load case for maximum stress at point 76 m from support 6.

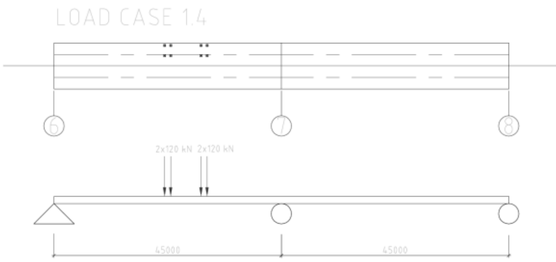


Figure 14. Load case for minimum stress at point 76 m from support 6.

Figure 16 illustrates the fatigue detail of transverse splice, used for on-site welded joints, EN 1993-1-9, Table 8.2.

To be able to get the maximum stress amplitude from the fatigue load two cases for each fatigue detail must be tested. The first load case gives the maximum stress and the other the minimum stress. The loads where placed according to the influence line to get the highest respectively the lowest stress in the detail. The vehicle load was placed as far out as possible, which means 0.5 m from the edge of the traffic lane, wheels furthest out was 4 m from the center line of the bridge.

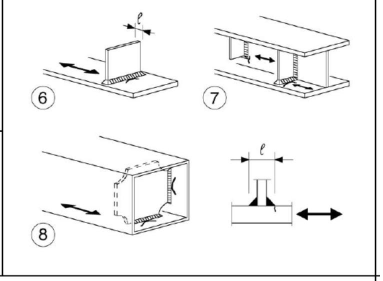
80	ℓ≤50 mm	
71	50<ℓ≤80 mm	

Figure 15. Detail classes according to EN 1993-1-9, for web stiffeners.

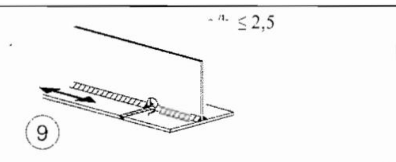
71		9) Longitudinal butt weld, fillet weld or intermittent weld with a cope hole height not greater than 60 mm. For cope holes with a height > 60 mm see detail 1) in Table 8.4	9) Δσ based on direct stress in flange.

Figure 16. Fatigue detail of transverse splice, used for on-site welded joints, EN 1993-1-9, Table 8.2.

5. Results for the case study

The results from Pitsund Bridge are summarized below. The stress range for the location at x=66 m where the fatigue detail are located are listed in following tables. The result for the other location x= 76 m is not presented further in this article due to that it shows the same type of results. Some short explanation for the indexes follows:

$\Delta\sigma^1{}_m$ - stress range, in case of membrane stresses, in the most loaded girder

$\Delta\sigma^2{}_m$ - stress range, in case of membrane stresses, in the girder which not has the load directly above

$\Delta\sigma^1{}_{m+b}$ - stress range, in case of membrane stresses + lateral bending stress, in the most loaded girder

$\Delta\sigma^2{}_{m+b}$ - stress range, in case of membrane stresses + lateral bending stress, in the girder which not has the load directly above

The analysis is based on cracked concrete above the internal support, which is the design case for new bridges. The assumption is that the length of the cracked concrete is 15 % of the span length on both sides of the internal support; the assumption is according to the norms in EN.

Table. 1 Stress ranges bottom flange at x=66 m based on cracked concrete above internal support, loads at lateral coordinate z = 4 & 2 m except for No truss, centric load for which z = 1 & -1 m. Beam 1 is the most loaded beam.

Analysis	$\Delta\sigma_m^1$ [MPa]	$\Delta\sigma_{m+b}^1$ [MPa]	$\Delta\sigma_m^2$ [MPa]	$\Delta\sigma_{m+b}^2$ [MPa]	$\Delta\sigma_m^1 + \Delta\sigma_m^2$ [MPa]
D 100 x 100 x 5	34.8	38.5	14.0	15.6	48.8
D 200 x 200 x 10	30.0	32.5	18.6	19.0	48.6
K 100 x 100 x 5	35.1	39.6	14.2	19.3	49.3
K 200 x 200 x 10	30.2	32.8	19.0	21.9	49.2
X 100 x 100 x 5	32.3	33.1	16.9	23.9	49,2

X 200 x 200 x 10	28.7	30.6	20.6	25.6	49.3
No truss	45.4	49.8	3.8	8.3	49.2
No truss, centric load	24.7	24.8	24.7	24.8	49.4

5.1. Improvement of stress ranges and estimated life time with respect to fatigue

The stress ranges for the strengthened models are here compared with the model without trusses.

Table. 2 Ratio of membrane + lateral bending stress for truss to no truss, most loaded beam, cracked above internal support, x =66 m

Analysis	$\Delta\sigma_{m+b}^{1}$ [MPa]	Ratio
D 100 x 100 x 5	38.5	0.77
D 200 x 200 x 10	32.5	0.65
K 100 x 100 x 5	39.6	0.80
K 200 x 200 x 10	32.8	0.66
X 100 x 100 x 5	33.1	0.66
X 200 x 200 x 10	30.6	0.61
No truss	49.8	-

6. Conclusions

The main purpose of the work carried out in this report has been to further analyse the effect from a horizontal truss system that has been added between the bottom flanges of the main I-shaped girders in a composite bridge. The main idea of applying these extra bracing members is that the connection of the two main girders will change the overall behaviour of the bridges from more or less two independent girders to a “box girder bridge” meaning that an eccentric load can be carried by a combination of bending and torsion in the box girder rather than “pure bending” in the loaded girder only.

Based on the results of the analyses of the Pitsund Bridge, the following results are found:

- The increased life-time for the fatigue detail in x=66m (web stiffener) on Pitsund Bridge various between 2.5 to 12 times depending on the configuration and amount of the horizontal truss.
- All three types of horizontal trusses are almost equalled efficient for purpose of distribute the eccentric load between both girders.
- Without any additional web stiffeners at the locations for the connection of the bracing the global stiffness contribution is limited, even for X-bracings which would have given a decreased stress in total for the global bending.
- Important note, to have any use of this horizontal truss the part of an old bridge without composite action it would first be needed to be strengthened with post installed shear connectors, for example with welded shear connectors or coiled spring pins.

Acknowledgements

Besides RFCS, the authors wish to thank the Transport Administrations of Sweden, Finland and Norway, as well as SBUF and Ramboll, for their support.

References

[1] Vestman, V. (2015). Improvement of Fatigue Resistance Through Box Action For I-Girder Composite Bridges, Master Thesis, Luleå University of Technology.

[2] Vestman V; Häggström J; Collin P;. Improvement of fatigue resistance through box-action for I-girder composite bridges, IABSE Stockholm 2016, ISBN 978-3-85748-144-4, pp. 1988-1994

[3] EN 1993-1-9:2005, Eurocode 3, Design of steel structures, Part 1-9.

Bilaga B

Design guidance for strengthening of I-girder bridges with trusses

There are many old I-girder bridges with a composite concrete deck that will need strengthening for static strength or for fatigue strength. There are many ways to achieve better fatigue performance for old steel bridges, like hammer-peening of the welds to increase the detail category for fatigue, changing the static system to lower cross-sectional forces, or simply adding steel to lower the stress levels. This project has focused on a novel idea to significantly increase the Saint Venant torsional stiffness of the bridge cross-section, by using a horizontal truss to create a box where the top and the sides are formed by the concrete plate and the steel web plates. This requires that the steel girders are connected to the concrete deck, admitting a shear flow in the box-like cross-section. In the case of no composite action, the concept of using coiled spring pins, described in this report, could of course be considered.

In the absence of common European design standards for existing bridges, the design can still be carried out based on the Eurocodes, although there might be national codes and guidelines that cover this topic on a national level.

B.1 General concept

Many old I-girder steel/composite bridges are too weak and in a need of replacement, repair or strengthening. The rules for assessment/classification vary between the European countries.

In addition, newer bridges designed according to the Eurocodes for example often give much tougher design in fatigue, which for example can give twice as large bottom flanges in mid span than the old Swedish codes, which means that few of the old bridges would survive a check with the Eurocodes for new bridges .

For symmetric I-girder bridges the loads from the weight of the steel and concrete are generally evenly distributed between the girders, just as for box girder bridges. For bridges consisting of two I-girders the concrete deck is often considered as simply supported in the transverse direction on top of the girders, meaning that a concentrated load on top of one girder will be distributed to only that girder, with no help from the second girder. In reality the torsional stiffness of the deck and the warping stiffness of the whole composite section however transfer some of the load so the real distribution can be about 90 % for the loaded girder and 10 % for the other, depending on the geometry of the bridge. For a box section this is not the case, the both half parts of the box share the eccentric loads almost equally see Figure B.1.

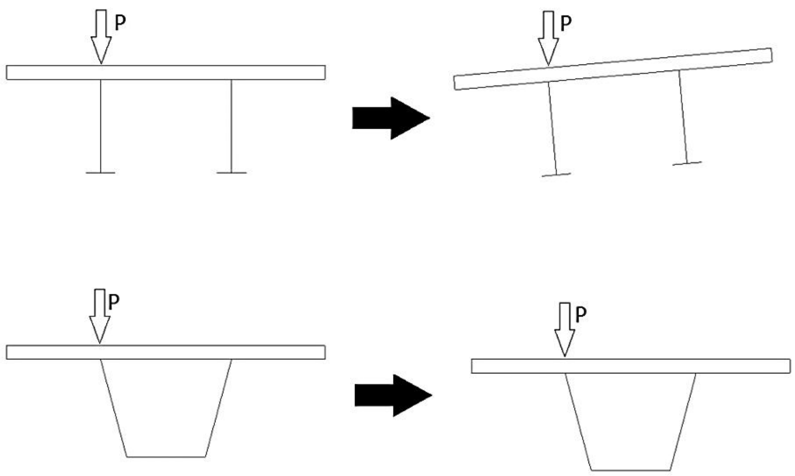


Figure B.1. Deflection corresponding to different types of cross section

When strengthening existing I-girder composite bridges, one concept is to make the cross-section act like a box section, by adding a horizontal truss between the bottom flanges. This means that the torsional moments produced by eccentric loads partly will be carried by shear forces around the cross-section. This means a lot in the Ultimate Limit State (ULS), but even more in the Fatigue Limit State (FLS). In the FLS the fatigue is determined by the stress ranges in certain parts of the structure, for instance the welded details of an I-girder. If the girders can act together, sharing the moment from an eccentric load more evenly, a lower stress range can be achieved. The amount of load cycles that a bridge can withstand, with e.g. a new distribution between the girders (70/30) could be up to six times higher, compared to an un-strengthened bridge with the distribution (100/0), based on the slope m=5 in the Wöhler-diagram.

The effect of the trusses has been described by Roik [B1] by adding a fictitious extra bottom flange to the I-girders, with an equivalent thickness giving the cross-section the same torsional stiffness to an imaginary box as the truss does. Two of the truss types are shown in Figure B.2 below, see also Chapter 2.2 of this report for more information on the effective thicknesses.

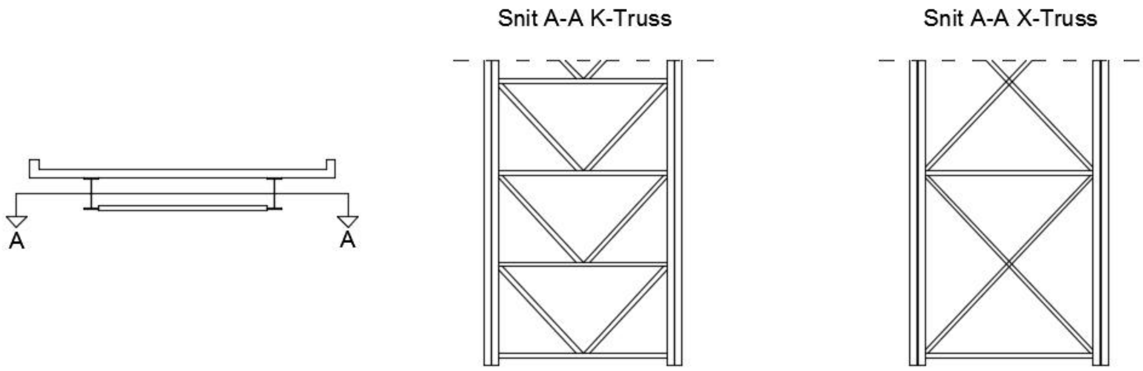


Figure B.2. Two options: K-trusses and X-trusses between the bottom flanges.

B.2 Design guidance

<u>*Configuration of the trusses*</u>

- In general, the effectiveness of the new horizontal trusses (K- or X-bracing) is dependent on the existing cross beams. It is found that both the stiffness and the geometry of cross beams have an substantial influence in the effect – especially in cases where the existing cross beams are non-symmetric profiles such as low UNP-profiles.
- All three types of horizontal trusses (K-, X- and D-trusses) are quite equally efficient for the purpose of distribute the eccentric load between both girders.
- The two types of trusses shown in Figure B.2 can both take torsion from eccentric loading. One big difference between the two types is that the X-truss also will take part in the global bending of the superstructure, while the K-truss, when loaded by global bending, will escape by deformations of the members perpendicular to the bridge. In order to achieve double composite action for X-trusses, transversal members are needed at the joints between diagonals and bottom flanges, otherwise the bottom flange will deform perpendicularly to the longitudinal axis of the bridge, introducing some lateral bending stresses. These stresses will of course be included in the results of a FE-analysis of the bridge modelled with shell elements. In many cases the cross girders can be close and strong enough to take care of this issue.
- Without any additional members at the locations for the connection of the bracing the global stiffness contribution is limited, even for X-bracings which would have given a decreased stress in total for the global bending. However with an additional member between the flanges, or the case with the position of the diagonal-flange connections coinciding with the original cross girders + web stiffeners, the horizontal truss will carry a part of the global bending moment for the case of X-truss or D-truss. For the K-truss the lateral bending of the member perpendicular to the bridge flanges means that this effect will be negligible.
- The angles between the diagonals are another parameter that will be influenced by the distance between the girders, as well as the longitudinal distance between the joints of the truss. If the cross girders are close and the distance between the girders is quite large, it might be enough to have one X-truss per cross girder, meaning that no extra members perpendicular to the bridge are needed in between the cross girders, as the cross girders will keep the distance between the bottom flanges at each joint. The effective thickness depending on the angle between the diagonals and the bridge is illustrated in Figure B.3 for a concept one-span bridge with 6 m between the I-girders [B2].The figure indicates that a suitable angle of the diagonals in a K-truss could be somewhere between 35 and 50

degrees. It must however be said that this could vary with the geometry of the bridge, if there are web stiffeners interfering or other practical reason to be considered.

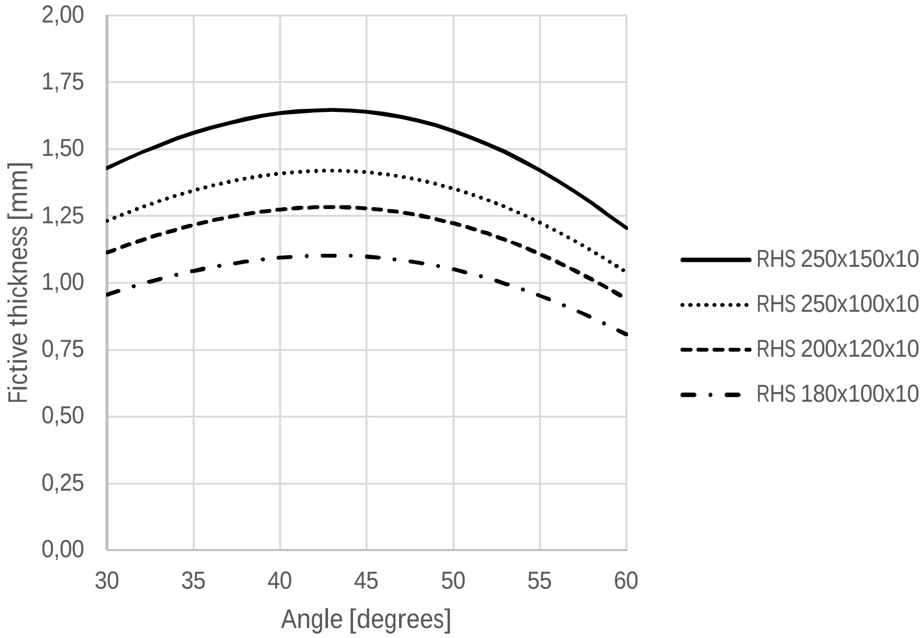


Figure B.3. Fictive thickness with different angle and dimension for K-bracing on a one span bridge with 6 m between the girders.

<u>*Modeling of the bridge*</u>

- It is recommended to apply a FE-model based on shell elements for the steel girders in order to ensure that the relevant effects are included and thereby that the effects of the new bracing members not is overestimated. Special attention should be applied if beams models are applied – especially in cases where the existing cross beams and web stiffeners are sparse, hence not able to prevent some distortion of the box-section that is created when a truss is fastened to the bottom flanges. If beams models are used for analysis, it is recommended that these models should be verified and/or calibrated in order to avoid results that might be unsafe. It is important to include original cross-bracings and web-stiffeners in the model, since they prevent the distortion of the cross-section.
- As for the bridge deck it may be modeled by either shell elements or solid elements. Solid elements increase the model size but give a slightly better representation of the structure because the geometry of the deck may be accounted for precisely. The increase in model size is not dramatic and so the bridge deck can be modeled by solid elements. For bridges with welded shear connectors between deck and girders, perfect bond may be assumed.
- As for cross-bracings in bridges, and horizontal trusses between bottom flanges, these may be modeled by beam elements for the purpose of establishing the static behavior of the bridge. If the connections of the beams to the bridge girders are such that it may be considered hinged, end release of considered rotational degrees of freedom is conducted. It should be noted, that for the static behavior of the bridge, the impact of the connections being hinged or rigid on the behavior of the bridge is small. However, for the resistance of the beam element it matters.
- When it comes to fatigue, the cracking of the concrete above the internal support has little influence on the results. The increased field moment because of the decreased support moment is almost taken out by the decreased field moment from when the load is on the other span. The 15% rule of clause 5.4.2.3 in EC 4-2 is however recommended, together with modelling only half the thickness of the slab in shear [B3]
- The local effects near the new joint connections should be analysed. The present study has not revealed any substantial stress concentration, but it is recommended that the local stresses are studied in the joint connection, adjacent web stiffeners, gusset plates, bottom flange of main girder etc. Such analyses are partly dependent on the exact layout of the joint connections. If considered critical, such analysis could be carried out in a FE-model

using solid elements in order to be able to model the behaviour correctly including local eccentricities, gusset plates, bolts, etc.

- An interesting observation is that the sum of both deformations and stresses of the two girders in an I-girder bridge are the same whether the truck is placed eccentrically or in the middle bridge. Furthermore, both the stresses and the deformations of the loaded girder decrease quite simultaneously when the bridge is modelled by a truss between the bottom flanges. This is however not the case with the vertical shear forces, as the torsional moment will be carried by both a vertical and a horizontal shear flow [B4].
- Only small changes to the shear force in the studs are expected. When the truss systems are applied to the bridge model, the shear force is (marginally) reduced due to a better distribution of the load. However, the torsional stiffness increases the shear flow through the studs. Therefore only a small variation in the shear flow through the shear studs is expected in the normal case. On the other hand for the unloaded girder the relative change of the shear force will increase. Normally, since the shear forces in the studs of the unloaded girder are much smaller than the shear force in the studs of the loaded girder, this will not be critical for the new static system.
- Due to elastic deformation in the diagonals, it is hard to share the load exactly equal between two I-girders strengthened with a horizontal truss. However a reduction of the stresses in the loaded girder with say 20-30 % will however substantially prolong the remaining lifetime of the bridge. If this is not enough, a further step is to introduce double composite action by e.g. a X-truss and members at the joints.

B.3 Some suggested details for installation of trusses for old I-girder bridges

In the following sections, three different types of connection between the flange and truss are suggested. Which type of connection is preferable depends on the existing design and geometry of the bridge. The chosen details however also depend much on the praxis of the country in which the bridge is located, the opinion/preferences of the designer/steel contractor. Keeping that in mind, the details presented should be regarded as possible solutions, not as firm rules.

Two different connections are necessary for installing the truss system. A connection between the bottom flange of the existing girder and the trusses and a connection between the trusses themselves at the “centre of the K“, see Figure B.1.

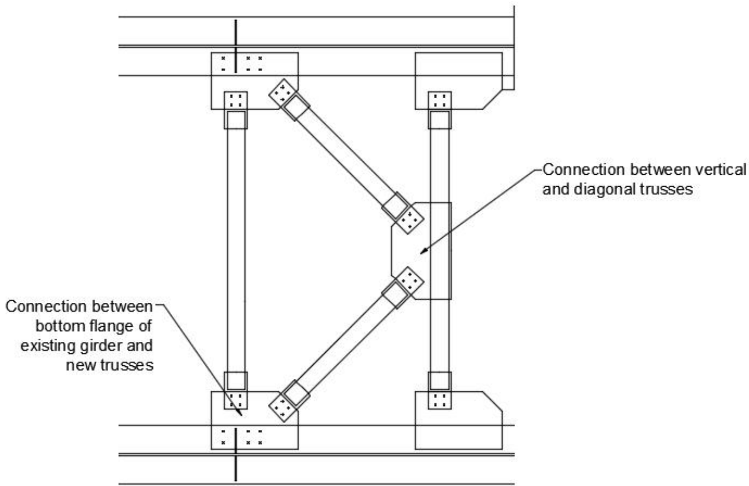


Figure B.4. Overview of the connections in the K-shaped truss system

Similar types of connections are necessary for the X-shaped truss system, where the connection between the trusses themselves is between the two diagonal trusses. The solutions shown in this section are only shown for bolted connections at the site since welded connections are assumed difficult to produce properly on site and additionally might have a negative effect on the detail class of the bridge and thereby reduce the remaining lifetime of the bridge. By using a bolted connection the detail class is not reduced and the trusses thereby have no negative effect on the remaining lifetime of the bridge.

In order to obtain a robust and solid connection and to avoid/minimise gaps between the gusset plates, all bolted joints are shown with a minimum of four bolts per joint in the following examples.

Due to the relatively small forces it can be argued, that the number of bolts can be reduced, e.g. from four to two.

Connection solution 1 for the K-shaped truss system

In the present section the first suggested solution for the connections in the K-shaped truss system are presented. In Figure B.2 the first solution for the connection between the bottom flange of the existing girder and the new trusses is shown.

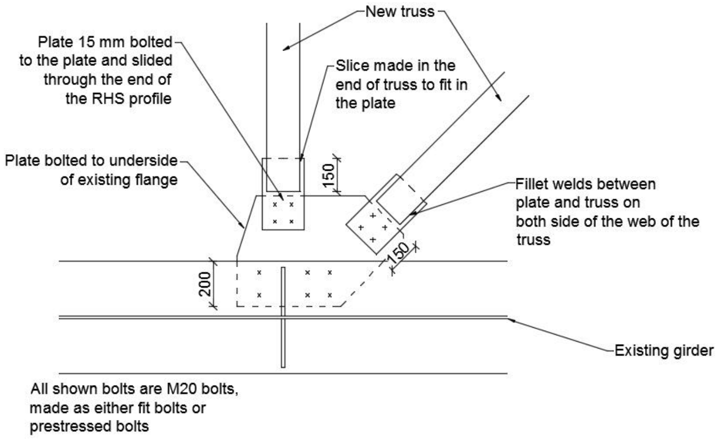


Figure B.5. The first solution for the connection between the K-shaped truss system and the bottom flange of the existing girder

The connection is constructed as a bolted connection where a common gusset plate is bolted to the lower side of the bottom flange of the existing girder. A plate is slided through the end of the truss and connected with fillet welds on both side of the web of the truss. The two plates are connected by bolted connections. All bolts used in the connection are M20 bolts made either as fit bolts or slip resistant (or friction grip) bolts. Tolerances are taken in the bolted connection to fit in the new trusses to the existing conditions.

In the design shown in Figure B.2 it is seen, that the trusses are arranged in such manner that the centre of gravity of the two connecting trusses coincide with the centre of the existing main girder.

In Figure B.3 the suggested solution for the connection between the transverse and diagonal trusses in the K-shaped truss system are shown.

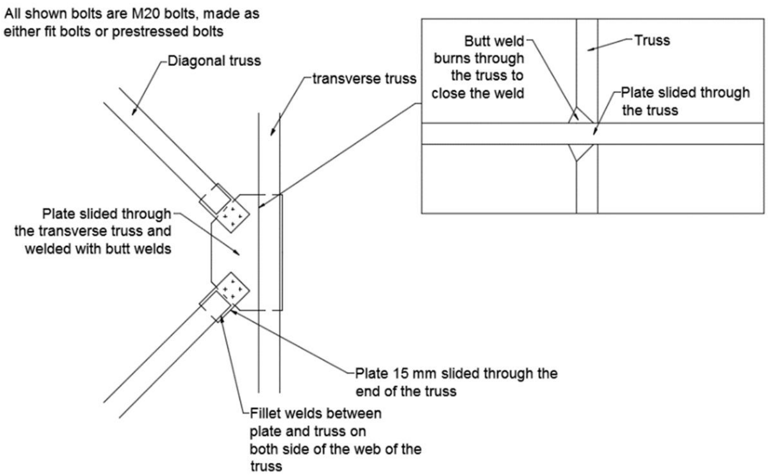


Figure B.6. Solution for the connection between the diagonal and the transverse trusses in the K-shaped truss system

As for the connection between flange and trusses the suggested solution for the connection between the transverse and diagonal trusses are constructed by sliding a plate through the transverse truss.

The connection between the plate and the transverse truss are prepared with butt welds, where the web of the truss is bevelled so the butt weld can burn through. As for the connection showed in Figure B.3, a plate is split through the end of the truss and connected to the truss with simple fillet welds. All bolts shown are M20 bolts designed either as fit bolts or slip resistant bolts.

<u>Connection solution 1 for the X-shaped truss system</u>

The suggested solution for the connection between the bottom flange of the existing girder and the trusses for the X-shaped truss system is designed using the same principles as for the K-shaped truss system. The only modification is that an additional diagonal truss is added whereby the connection is symmetric around the centreline of the transverse truss. A sketch of the connection between flange and truss for the X-shaped truss system is not shown.

In Figure B.4 one suggested solution for the connection between the two diagonal trusses for the X-shaped truss system is shown.

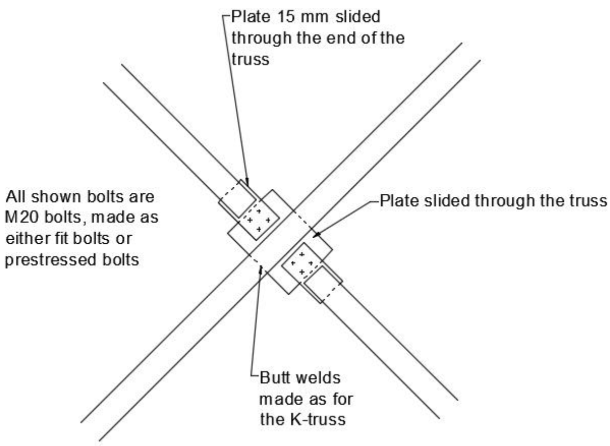


Figure B.7. Solution for the connection between the diagonal trusses in the X-shaped truss system

The suggested solution for the connection in the X-shaped truss system is made as for the K-shaped truss system. One of the diagonal trusses is continuous while the other is cut in half. The Connection is made by sliding a plate through the continuous diagonal truss and connection the other diagonal truss by bolted connections. All shown bolts in Figure B.4 are M20 bolts made either as fit bolts or slip resistant bolts.

<u>Connection solution 2 for the K-shaped truss system</u>

In the solution for the connection showed in Figure B.2, the total number of bolts used is relatively large. Since the cost is strongly dependent on the time used for installation, an alternative solution is proposed, where the transverse trusses are welded directly to the plate that is bolted to the flange. Thereby the number of bolts needed is reduced from 14 to 10, see Figure B.2 and Figure B.5

Also in the design shown in Figure B.5 it is seen that the trusses are arranged in such manner that the centre of gravity of the two connecting trusses coincide with the centre of the existing main girder.

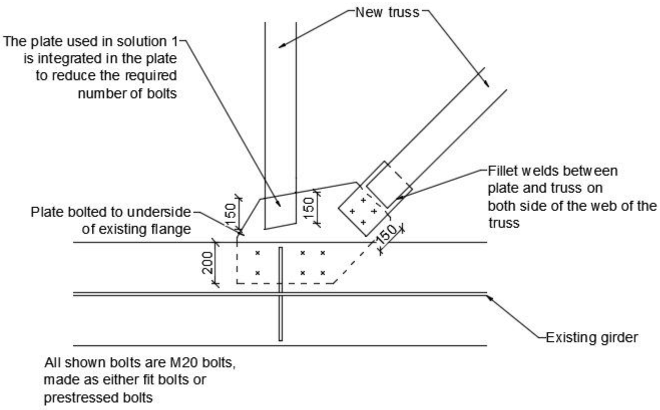


Figure B.8. The second solution for the connection between the K-shaped truss system and the bottom flange of the existing girder

The downside to this solution is that there is less room for tolerances, since all tolerances are now taken in the diagonal trusses. The connection between the plate and the transverse truss is necessary to be welded before installing the trusses. Therefore the weight of the element consisting of plate and transverse truss, that needs to be bolted to the existing flange, is higher than for the first solution.

<u>*Connection solution 3 for the K-shaped truss system*</u>

To reduce the number of bolts needed for the connection, but still obtain the same possibility for fitting and approximately the same weight of the different parts that need to be installed a third solution is suggested, see Figure B.6.

This solution for the connection between flange and truss the small plate fitted at the end of the trusses are bolted directly to the bottom flange. The large gusset plate is thereby avoided and the total number of bolts necessary to make the connection is reduced from 14 to 8. At the same time the total weight of the added steel is reduced. Tolerances can be handled in the bolted connections. The angle of the diagonal trusses can easily be changed to fit the existing conditions off the bridge, see the figure below.

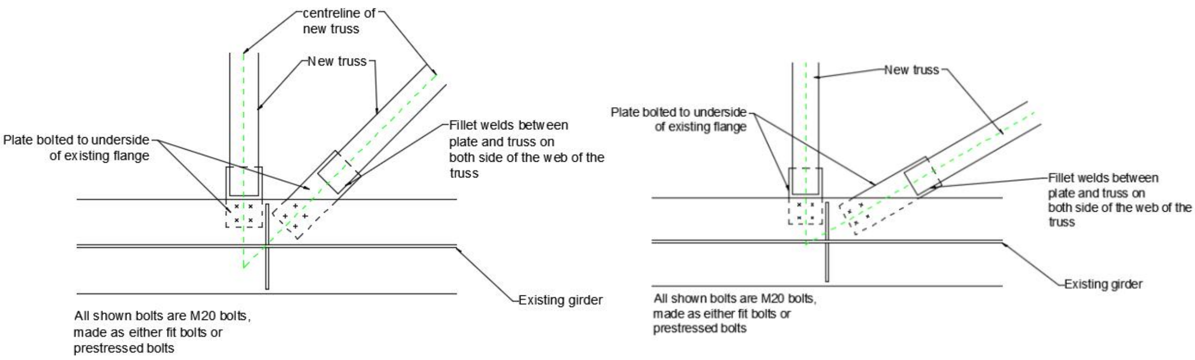


Figure B.9. The third solution for the connection between the K-shaped truss system and the bottom flange of the existing girder

The downside of using this solution is that the centreline of the transverse and diagonal trusses (shown by the green dotted line in Figure B.6) does not intersect at the position of the web stiffener. This means that the centre of the connection is not placed at the position of the web stiffener and therefore this connection has a larger risk of introducing additional local stress increase around the connection than the solutions shown in Figure B.2 and Figure B.5.

Furthermore, in the case where the angle of the diagonals is larger than shown in Figure B.6, it might be impossible to design the point of intersection of the two truss members at the centre line of the main girder.

References

[B1] Roik, K. (1983). Vorlesungen über Stahlbau : Grundlagen. 2nd ed. Berlin: Berlag von W. Ernst & Sohn.

[B2] Vestman V; Häggström J; Collin P;. Improvement of fatigue resistance through box-action for I-girder composite bridges, IABSE Stockholm 2016, ISBN 978-3-85748-144-4, s1988-1994

[B3] EN 1994-2 (2005). Eurocode 4 - Design of composite steel and concrete structures – Part 2: General rules and rules for bridges. European Committee for Standardization; Brussels. Belgium.

[B4] Vestman V; Improvement of Fatigue Resistance Through Box Action For I-Girder Composite Bridges; 2015; Luleå University of Technology

Bilaga C

Experimental testing for development of a new friction grip connection

C.1 Compression and shear tests on unpainted surfaces

The purpose of the first half of the first test series was to investigate whether the spherical indenters inserted between two steel plates is capable of penetrating the plates when pressed together and to determine the force necessary to press an indenter a certain distance into the steel plate. The test was conducted such that the indenters did not interfere with each other. The main purpose of the second half of the first test series is to determine whether the effective friction coefficient (μ) of a friction connection can be improved by indenters when impressed between two plates and subjected to shear force and thereby operate as an enhanced friction connection.

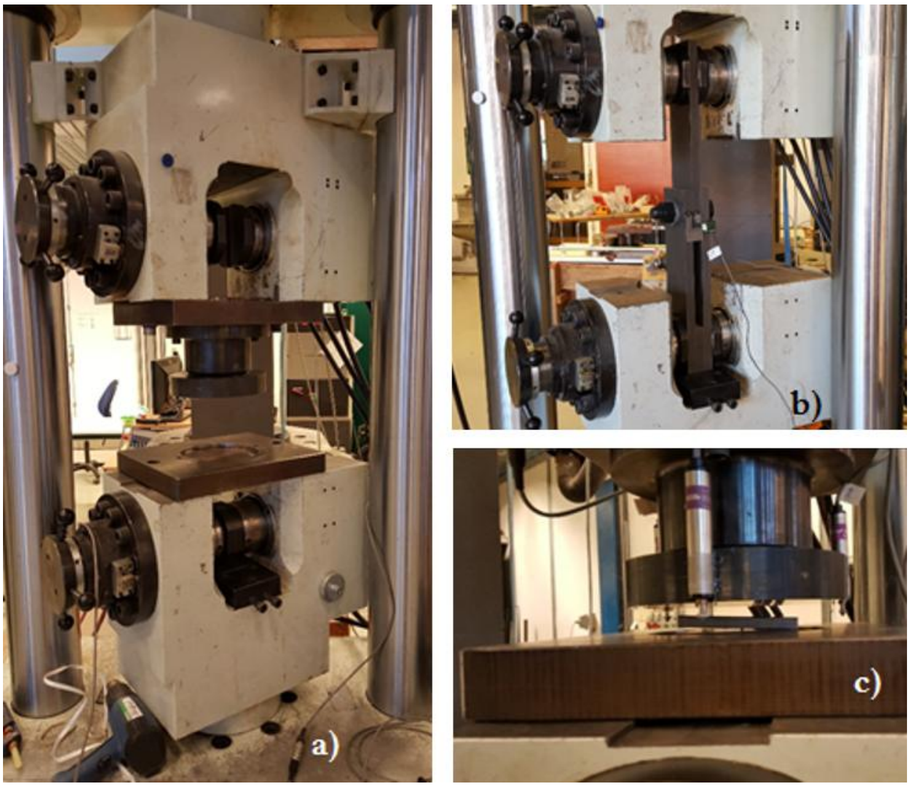


Figure C.1. a) Dartec hydraulic rig, b) Test set for lap joints, c) Test set for compression.

The compression- and lap joint tests were conducted with cold formed structural steel grade S235JRC+C, of different plate thickness between the two tests, and indenters with 2,5 mm diameter. In the first five compression tests a pair of plates, 100x100x10 mm were used. For the lap joint tests plates with the dimension 400x100x20 mm were used. The lap joint test were carried out using M30 class 10.9 bolts, in a 38 mm (oversized) bolt hole, pre-loaded to 320 kN.

During a compression test, the cross-head displacement of the testing machine was increased linearly with time. The depth of indentation and the further compression of the two plates were registered with three circumferentially equidistant displacement gauges attached to the circular end blocks of the loading device. Cross-head displacement, depth of indentation and the corresponding compressive force were registered.

The initial tests in the first test series comprised five different individual tests divided into three groups a) No. 1, a reference test with just two plates compressed, b) No. 2 and 3, tests with one single indenter of tungsten carbide and of stainless steel, respectively, c) No. 4 and 5, tests with 49 indenters of the two materials, respectively.

For the compressive force measured together with a mean value of the three displacement gauges a load-deformation diagram was plotted showing the results from all five tests, with the compressive force [kN] on the y-axis and the deformation [mm] on the x-axis. The scale of the x-axis is chosen according to the size of the indenter showing the impression of the indenter from minus 2,5 mm to 0 mm, thereafter the compression of the plates up to 0,25 mm. At the displacement of minus 2,5 mm the indenter has not yet penetrated the plates. At the displacement 0 mm the indenter is ideally fully pressed into the plates. Beyond 0 mm the deformation of the compressed plates is shown. By dividing the force in test four and five by the number of indenters, the result of all tests in terms of force per indenter can be compared directly.

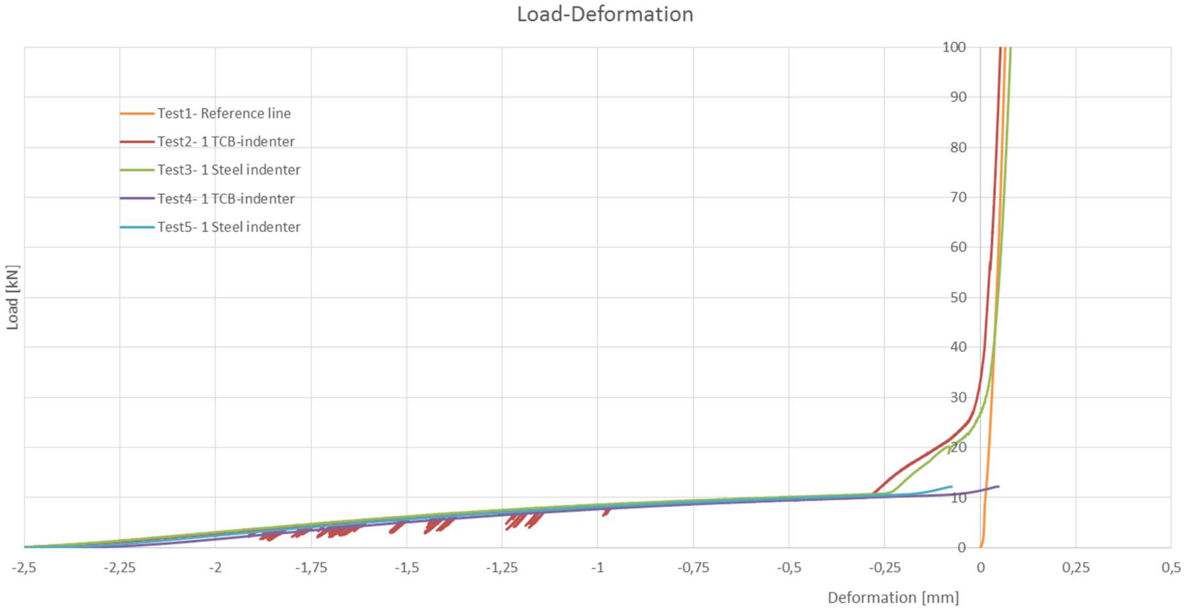


Figure C.2. Load-Deformation diagram with all five tests, where test 4 & 5 are divided with 49 indenters.

In the second half of the first test series a lap joint with two shear planes was loaded longitudinally under a linearly increasing displacement in time until slip failure occurred. Two pairs of knife edges were welded to the plates, one pair on each side of the connection. To register the relationship between the slip force and the slip of the joint, a Crack Opening Displacement (COD) clip gauge (measuring the slip) is mounted to the edges of each pair of knives and then connected to the computer of the testing machine. The computer registered and stored all data from the tests. A clip-gauge mounted to one pair of knives is seen in Figure C.1b).

First a reference joint test was performed with plain plates without any indenters for comparison of force-slip behaviour to the subsequent tests with various indenter arrangements. Thereafter three tests were performed with different number and arrangement of indenters.

For the measured slip force together with a mean value of the two COD clip gauges, a slip force-slip diagram is produced with the results from all steel indenters and the reference curve, with the slip force [kN] on the y-axis and the slip [mm] on the x-axis.

C.2 Results and Conclusions from unpainted tests

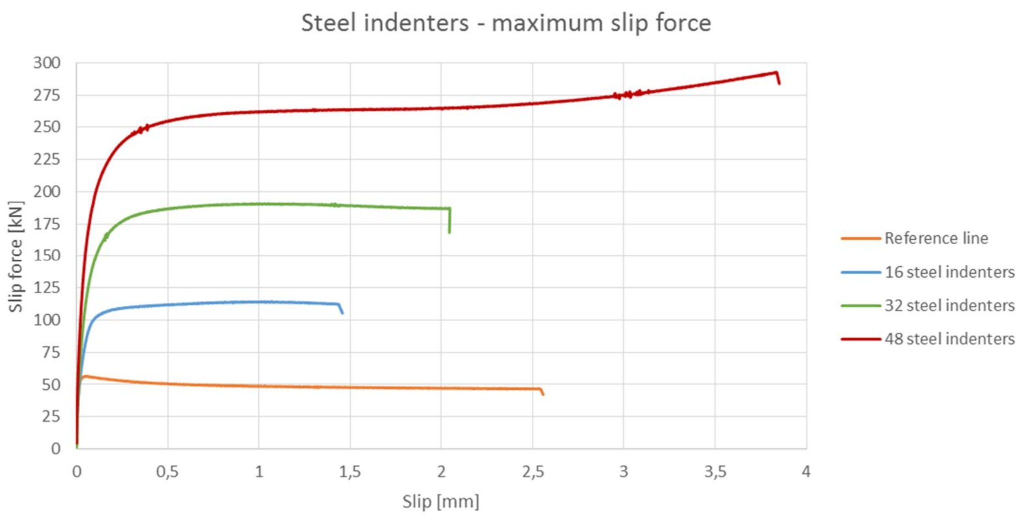


Figure C.3. Slip resistance for respective connection with steel indenters. M30 bolt and pre-loading force 320 kN.

In Figure C.3 the reference curve indicates a weakly but steadily decreasing slip force after the initial slip, while the tests with indenters are characterized by a steadily increasing slip force. It is also seen that the greater the number of indenters in a slip plane, the greater is the slip force of the joint. For a joint characterized by a decreasing slip force, an (accidental) overload inevitably brings about unbounded further slip, or, in other terms, such a joint is unstable and provides no safety against overloading. In contrast, the friction force of joints with indenters increases monotonically with increasing slip. A joint with indenters, as opposed to a plain joint, thus provides safety against overloading as an overload is accommodated by some slip increment. For the connections with 8, 16 and 24 indenters per shear plane, the slip resistance at 0,15 mm slip is 105,9, 162,6 and 217,0 kN, respectively. A joint without any indenters has a slip resistance at 0,15 mm at 54,5 kN.

The effective friction coefficient (μ) can be derived from Eurocode equation 3.6 in 1993-1-8 (2005) and by taking k_s and γ_{M3} equal to unit. The bolt hole coefficient (k_s) and the partial safety factor (γ_{M3}) should not be included when calculating μ as these can be considered as safety factors in dimensioning.

$$\mu = \frac{F_{s,Rd}}{F_{p,C}\, n} \tag{C.1}$$

The effective friction coefficient according to Equation 1 versus the number of indenters is shown in Figure C.4. A prediction of the friction coefficient is made for the highest possible number of indenters with the pre-loading force 320 kN. In the reference test without indenters the friction coefficient was μ=0,09. For the connections with 8, 16 and 24 indenters per shear plane, the effective friction coefficient are 0,17, 0,25 and 0,34 respectively (left figure). The impressions from the test with 24 stainless steel indenters in a shear plane are shown in Figure C.5.

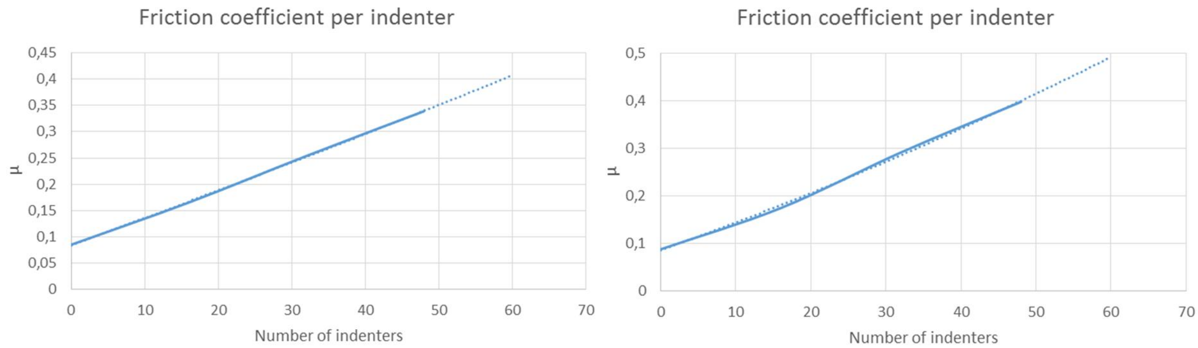


Figure C.4. Friction coefficient versus number of indenters, 0,15 mm and 0,50 mm slip, M30 bolt and pre-loading force 320 kN.

For an assumed number of 29 indenters per slip plane and a pre-loading force of 320 kN, the slip resistance may attain 250 kN at 0,15 mm slip and 310 kN at 0,50 mm slip. The friction coefficient for the 0,15 mm slip criterion is close to 0,40, and 0,50 for the 0,50 mm slip criterion. The slip resistance force of the reference test with “rolled surface” condition is 54,5 kN and the friction coefficient is μ = 0,085. In comparison a value according to Eurocode of the slip resistance force for the reference test was calculated, with the chosen friction coefficient μ = 0,20 for a “rolled surface” according to Eurocode, with the slip resistance 109 kN. This considerable difference indicates that the present “rolled surface” condition in practice may include too wide a range of effective friction coefficients and that subgroups with reliable friction coefficients could be considered.

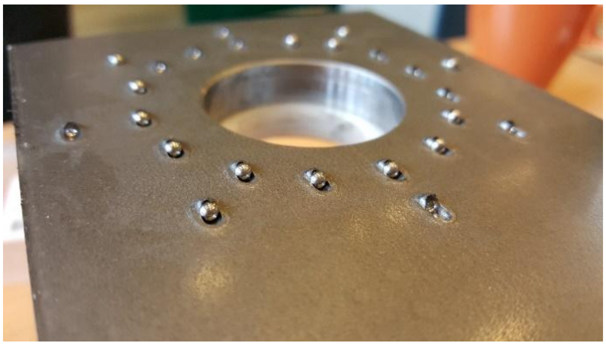


Figure C.5. Impressions per shear plane. 24 stainless steel indenters. M30 bolt and pre-loading force 320 kN.

To conclude whether indenters have a positive effect and can contribute to an enhanced friction connection the friction coefficients obtained are compared with those given in 1993-1-8 (2005), for different surface treatments. Since connections with oversized holes were used in this work the Eurocode coefficients should be multiplied with the factor k_s=0,85. It is now possible to compare the highest value in Eurocode μ=0,43 (for surfaces blasted with shot or grit with loose rust removed, not pitted) and the greatest value for the lap joint tests μ=0,40. The comparison shows that the connection with indenters is almost as effective. A joint with indenters provides however unconditional safety against overloading and the effective friction coefficient, which is related to the number of indenter, is well-defined, while the Eurocode friction coefficient may be uncertain. If the slip criterion of 0,15 mm is increased to 0,50 mm, an effective friction coefficient about 16 % greater than the corresponding Eurocode coefficient is obtained.

C.3 Compression tests on “AMA” treated surfaces

In the second test series, indenters of different size and material were loaded in compression between two AMA treated plates in order to determine the force needed to press down an indenter of a certain size a certain depth into a plate. Thereafter plates with different surface treatments were shear loaded in a lap joint in order to compare their effective friction values in comparison to the Eurocode tabulated friction values. Finally AMA treated surfaces with indenters of different sizes, materials and number per slip plane were shear loaded in a lap joint and compared with the previous tests without surface treatments.

AMA refers to the corrosion class C5-M (A5M.06-EP(Zn)/EP/PUR), defined in ISO 12944-5. According to the ISO standard the surface should first be blasted, if needed, to Sa 2,5 and thereafter be treated with a zinc rich epoxy (40-60 µm), then with epoxy pigmented with micaceous iron oxide (2*75 µm) and finally covered with polyurethane cover paint (2*60 µm). This means that after blasting, the thickness of the surface layer of the AMA treatment is around 0,31 mm.

The compression- and lap joint tests were conducted with structural steel grade S355+N for all plates. All indenters used were spherical but varied in size and material: 2,5 or 5,0 mm diameter of high strength stainless steel, 3,0 mm diameter of silicon nitride and 2,5 mm aluminium oxide indenters. The material of the stainless steel indenters is X46Cr13 according to Eurocode nomenclature and is characterized by ductile, hard and of high strength. The ultimate tensile strength is 1700-1900 MPa and the hardness HRC 52-60. Silicon nitride (Si_3N_4) is a corrosion resistant ceramic material with good mechanical and toughness properties. The ultimate compressive strength is 2300-4000 MPa and the hardness HV 1400-1600 (HRC 74-77). Aluminium oxide (Al_2O_3) is a very hard ceramic material but also brittle with low compressive strength in uni- and biaxial loading. The hardness HV 1356 (HRC ~70). The lap joint test was carried out using M24 class 10.9 bolts, in a 26 mm bolt hole, pre-loaded to 240 kN. For the aluminium oxide test was the earlier configuration with M30 class 10.9 bolts were used.

In the compression test, a pair of plates, 100x100x20 m, with 16 indenters uniformly distributed in a square pattern were compressed, Figure C.6. The displacement increased linearly with time and the depth of indentation was measured with three displacement gauges situated at equidistant positions around a periphery of the loading cylinder, see figure C1.c).

In the compression tests two tests were carried out. The first test with 3 mm silicon nitride indenters and the second with 5 mm stainless steel indenters.

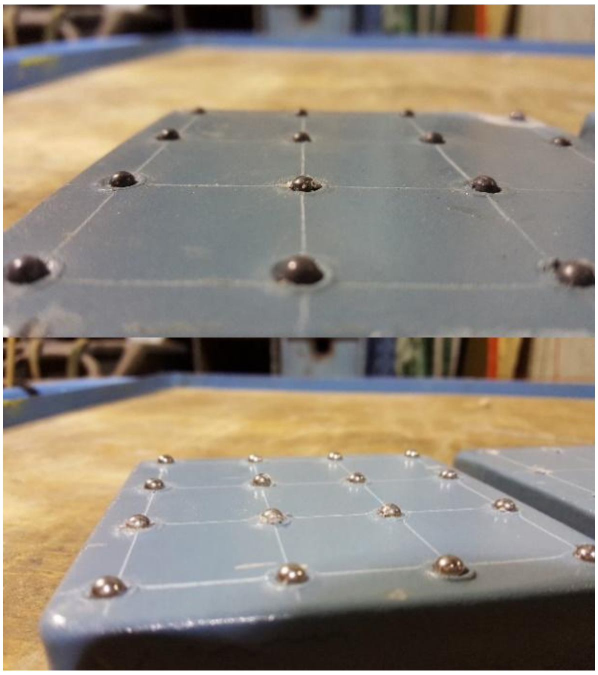


Figure C.6. The two tests with silicon nitride and stainless steel indenters

C.4 Results and Conclusions from "AMA" treated compression tests

The results of the compression tests are shown in a load-deformation diagram Figure C.7, in terms of load per indenter against the mean value of the three measured displacements. The scale of the displacement axis is chosen according to the indenter size. The penetration of the 5 mm indenter from minus 5,0 mm (no penetration) to 0 mm (ideal full penetration) are first shown (beginning from the left), thereafter the compression of the plates themselves up to 1,5 mm compression. As the first 0,62 mm displacement correspond to the indenter penetration of the surface paint layer of the two plates, the load-deformation curves were displaced 0,62 mm to the left in order to illustrate true penetration.

It is seen that the compressive force needed to completely press a 5 mm diameter stainless steel indenter into the plates was ~41 kN. For a 3 mm diameter silicon nitride indenter this value was 17,7 kN. Since the load capacity of the testing machine is limited to 600 kN (or 37,5 kN per indenter), the test result for the stainless steel indenters (41 kN) was estimated by using a second-degree polynomial extrapolation (the dashed part of the blue curve in Figure C.7).

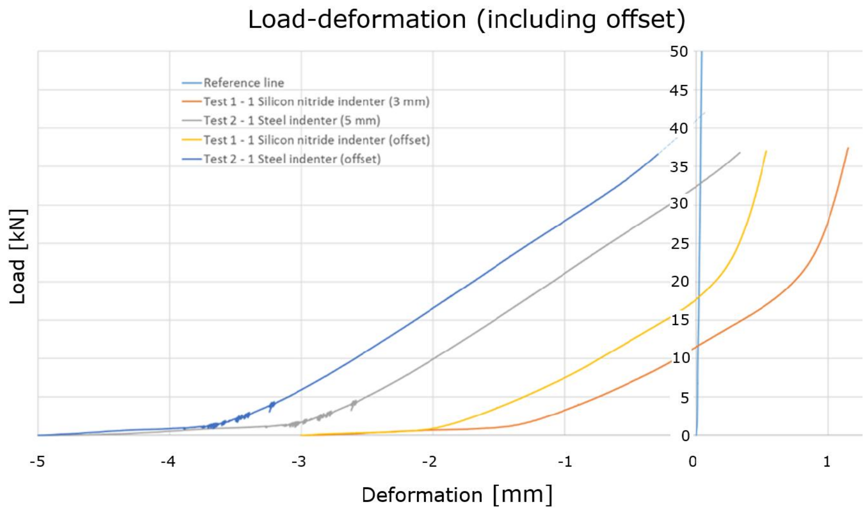


Figure C.7. Force per indenter versus adjusted slip length

C.5 Shear tests on "AMA" treated surfaces

In the lap joint test series a lap joint with two shear planes with and without indenters was loaded longitudinally under a linearly increasing displacement until slip failure occurred. Two pairs of knife edges were welded to the plates (400x100x20 mm) edges, one pair on each side of the joint, Figure C.1b. To register the slip of the joint, a Crack Opening Displacement (COD) clip gauge was mounted to the edges of each pair of knives. The COD displacement and the longitudinal load were registered and stored electronically.

The lap joint tests contain several individual tests divided into three groups. In the first group several tests without indenters were conducted in order to compare the coefficient of friction of different plate surface treatments. The results of these tests were compared to corresponding values given in the Eurocode. In the second group AMA treated plates with varying number of 2,5 mm stainless steel indenters were tested. In the third and last group AMA treated plates also were tested, but with varying indenter size.

C.6 Results and Conclusions from "AMA" treated shear tests

In the first group of tests with different surface treatments three tests were conducted. One test with fine blasting, another with rougher blasting and one further with the AMA treatment. The lap joint test results were then plotted in a slip force versus slip length diagram for all three tests combined with the reference test result from the earlier series with the "rolled surface" condition.

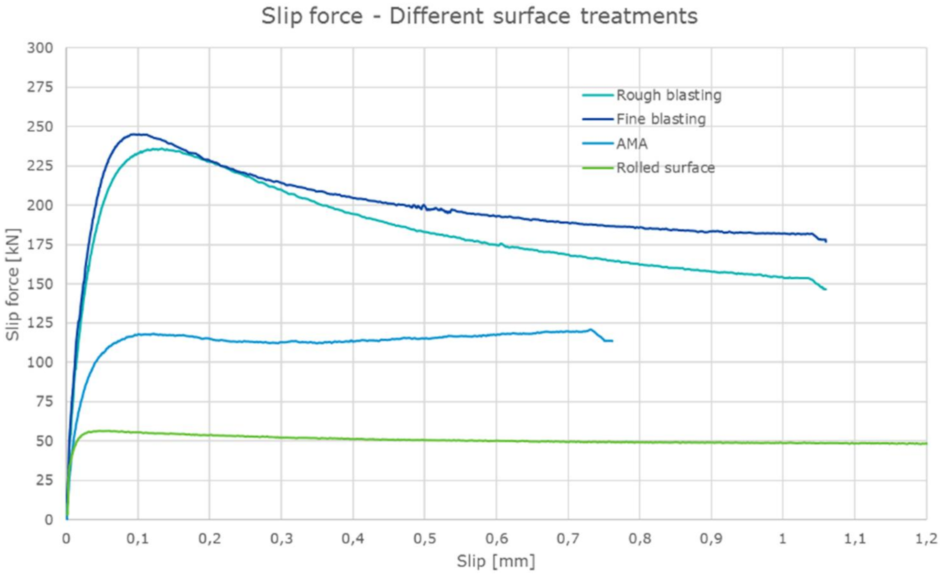


Figure C.8. Slip resistance for connections with different surface treatments. M24 bolt and pre-loading force 240 kN (M30 bolt and 320 kN for rolled surface)

The "rolled surface" reference curve indicates a weakly but steadily decreasing slip force after the initial slip of increasing slip force up to maximum slip force, while the blasting tests are characterized by a much more rapid decrease after maximum slip force. The AMA treatment slip force is almost constant after maximum load. Note however that the "rolled surface" condition test was performed with a different pre-setting; with an M30 bolt and a pre-loading force of 320 kN. It is therefore more relevant to compare friction coefficients than slip forces, thereby taking the effect of pre-load into consideration. Friction coefficients calculated with Equation 1 for the different surface conditions in Figure C.8 are tabulated in Table C.1. Pictures on the specimens from the first test series is shown in Figure C.9.

Table C.1 Friction coefficient versus surface treatment

Surface treatment	$F_{s,Rd,0,15}$ [kN]	$\mu_{0,15}$ [-]
Rolled surface	54,5	0,085
AMA	117,4	0,245
Rough blasting	234,8	0,489
Fine blasting	238,2	0,496

Figure C.9. The specimens from the second test series

The present tests verify the Eurocode value μ_{max} = 0,5 for “blasted surfaces”, but also reveal a drawback (mentioned above) affecting this surface treatment, in that immediately after reaching a friction coefficient of $\mu_{0,1}$ = 0,5 at 0,10 mm, the slip resistance decreases rapidly from its maximum value and resistance to further slip is lost. Under controlled displacement condition the friction coefficients of $\mu_{0,5}$ = 0,42 and $\mu_{1,0}$ = 0,38 were obtained for further slip, needless to say that this means that this joint type is unstable under controlled load conditions. The results show that indenter friction joints are at least as good as traditional friction joints regarding slip resistance, but moreover offer they stability and thus safety against overloading. According to the present results it is not wise to use μ = 0,5 for “blasted surface” if slip exceeds 0,1 mm since μ = 0,5 no longer is true, which leads to an unsafe and incorrect slip resistance. Further, the effect of the AMA treatment may be overestimated in that according to the surface treatment description in the Eurocode, designers will most likely use a slip factor of μ = 0,4, while our results indicate that the AMA treatment only reaches a friction coefficient of μ = 0,25.

In the second group of tests, with AMA treated plates and with different numbers of 2,5 mm stainless steel indenters, the purpose was to study the effect, if any, of a remaining gap between the joint plates after compression. If the number of indenters is too great for the indenters to be fully pressed down into the plates by the pre-loading force of the joint bolt, the question arises whether slip resistance will still increase with slip length? The maximum number of indenters in a joint at theoretical zero gap is n_{max}=240/10,6=22,6. In view of this, for this group of tests, joints with 16, 20 and 28 indenters per shear plane were chosen. In that way it was possible to check that the slip force, as expected, indeed increases with slip length for 16 and 20 indenters and further to observe the behaviour of a joint when the number of indenters is so great that a remaining gap between plates can be expected after compression. In Figure C.10 all three tests are plotted together with the AMA reference test.

The number of indenters in a friction joint have a significant effect on the slip resistance. Up to the limit of the maximum number of indenter that can be fully pressed down into the plates by the pre-loading force of the joint bolt, in this case 22 indenters, the slip resistance increases in general with the number of indenters. For a number of indenters exceeding the maximum number, the slip resistance does not increase further with number of indenters. The slip resistance may even be slightly reduced, as seen by comparing the curves for 20 (below the limit) and 28 (above the limit) indenters in Figure C.10. Somewhat surprisingly, this behaviour is in contrast to previous test results from the first test series, in that the slip resistance for all tests with 2,5 mm indenters is smaller than for the friction joint with AMA treated plates (no indenters). The reason for this difference is most likely that the thickness of the AMA paint layer of 0,31 mm for each plate simply was too great in relation to the size of the indenters.

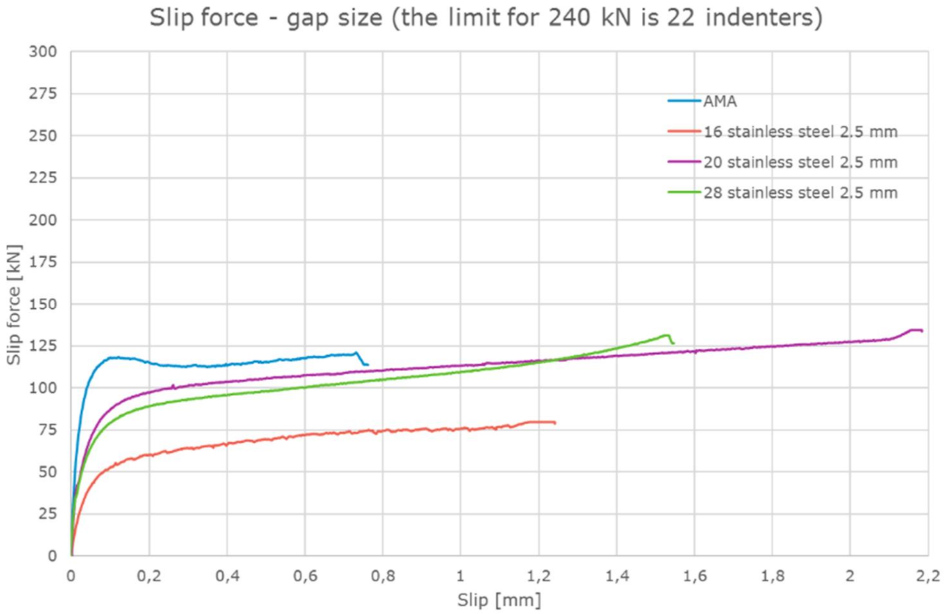


Figure C.10. Slip resistance for connections with different number of indenters. M24 bolt and pre-loading force 240 kN

An example of the effect of the paint layer on the plates in a friction joint upon the effective friction coefficient of joint with indenters, supporting the above difference, can be seen by comparing the friction coefficients at 0,15 mm slip and 20 stainless steel 2,5 mm diameter indenters per shear plane for a) plates with paint, $\mu_{0,15}$ = 0,196, see Error! Reference source not found., and b) no paint, $\mu_{0,15}$ = 0,297, see Figure C.11 (Figure C.4 is here revised showing number of indenters per shear plane in contrast with the original Figure C.4). The effective friction coefficient is reduced by the paint layer, in this case the "painted" value to some 65 % of the plain, "not painted" value.

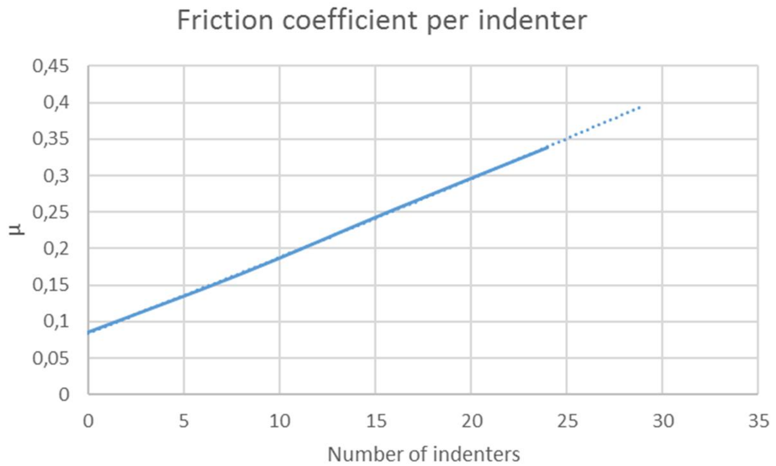


Figure C.11 Friction coefficient versus number of indenters per shear plane, M30 bolt and pre-loading force 320 kN and 0,15 mm slip

In the third group of tests, with different indenter sizes, all joints were provided with almost as many indenters as possible according to the compression tests. 3 mm diameter silicon nitride indenters and 2,5 mm and 5 mm steel indenters were used. The reason for using a ceramic material was to study the indenter behaviour both in compression and in shear, and compare it to steel.

Using the maximum number of indenters it is seen that the indenter size does not affect the slip resistance significantly at initial slip for all joints. After the initial slip, slip resistance increases with indenter size. The slip resistance beyond 1,0 mm slip was found to increase more rapidly for larger indenters. A somewhat smaller slip resistance could be observed for the 2,5 mm indenter compared to the case with no indenters, as shown in Figure C.12. This is believed to be due to the size of the indenter in comparison to the thickness of the paint layer. Also, the larger the indenter diameter, the fewer indenters are needed to obtain certain slip resistance. In Figure C.13 the impressions of indenters on the plates after disassembling of the joints are illustrated.

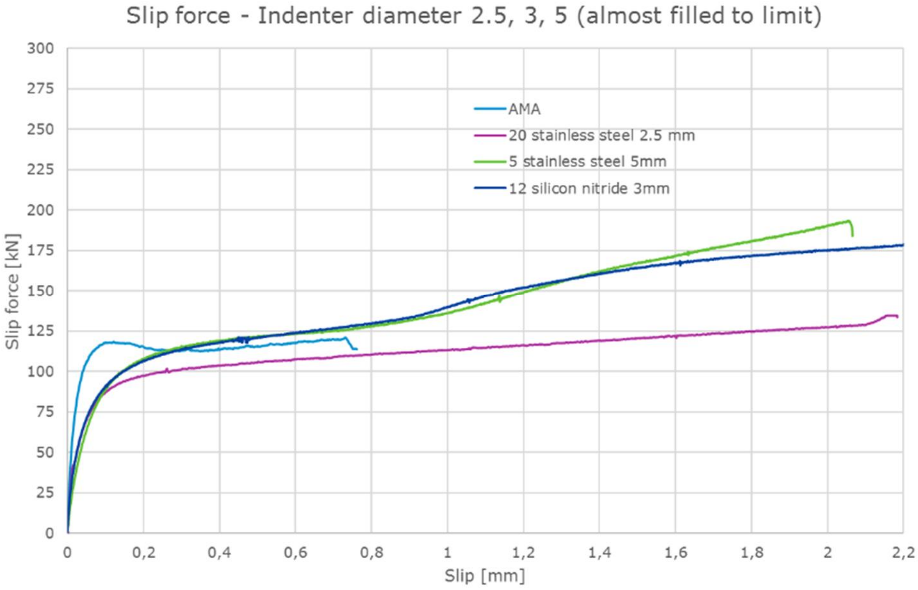


Figure C.12 Slip resistance for connections with different indenter size, filled almost to the maximum number of indenters. M24 bolt and pre-loading force 240 kN

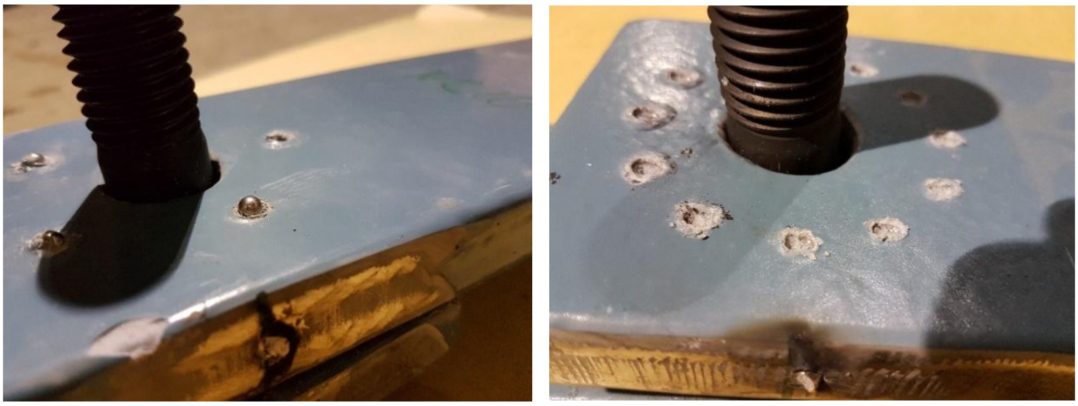


Figure C.13 Lap joint tests, after disassembling