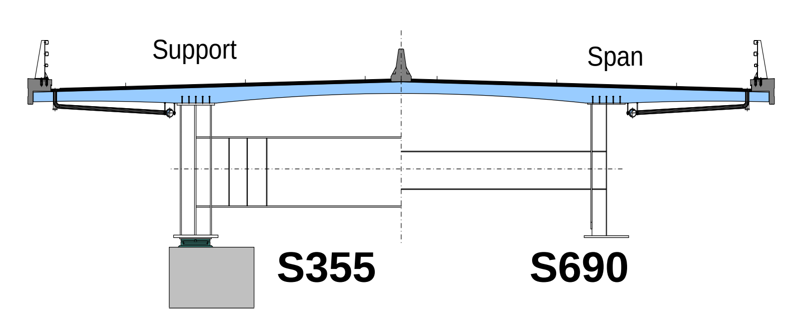
A.M. Habraken, L. Duchêne, C. Bouffioux, C. Canalès



OPTIBRI

Opening and Project Overview

OptiBri-Workshop

"Design Guidelines for Optimal Use of HSS in Bridges„

3 May 2017

Anne Marie Habraken

Research Fund for Coal & Steel

Partners	Core Expertise
University of Liège Be (Ulg)	*(Coordinator)* Material scientist Modelling, Experimental Lab
Industeel Be	Producer of high quality steels
GRID Pt	Civil Engineering
University of Stuttgart Ge (USTUTT)	Bridge, Stability, Euro code, Experimental Lab
University of Coimbra Pt (UC)	Environmental and cost impact assessment
Belgian Welding Institute Be (BWI)	Welding procedure and Post Weld treatments

LIÈGE université

ArcelorMittal

GRID

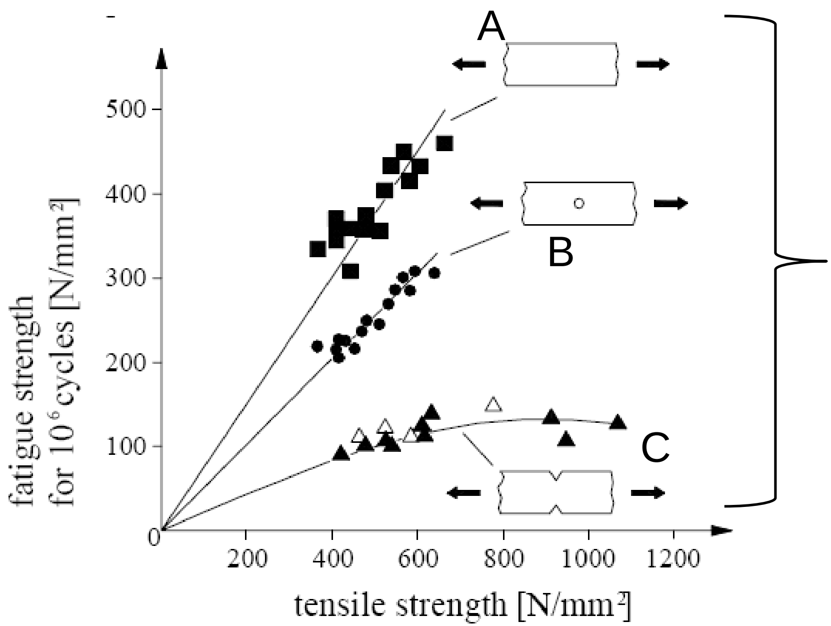
University of Stuttgart Germany

research

How the project was born ?

For a **material scientist**, studying also forming process, High Strength Steel (**HSS**) means

- higher stress value, higher fatigue limit, specific microstructures,
- logical ways to decrease weight (cars, planes: transport industry)



Sheet sample
Bulk sample
→ Material study case A

Beams, Plate with or without welding joints ? Where ?

How the project was born ?

For civil engineers, **HSS** means:

- higher material cost but **potential decrease**
  of the amount of material
  of welding time
  of transport
  of environmental impact…

Objectives of OPTIBRI Project

- **Quantification** of the interest of HSS use under current euro code rules
- Scientific study to define the need of **Eurocode enhancement (Stability, Fatigue)**
- Check **fatigue issues** of post treated weld joint of HSS
- Study **weld joint and post treatment quality** in HSS

My netwok + the one of my Civil Eng. colleagues → Partnership → Brain storming in Summer 2013

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ULG
HSS Material

Civil Eng Market ?
Industeel

GRID
Fatigue issues for bridge if slender structures

IBW
Weld joint quality & effect on fatigue
→ PIT, TIG

USTUTT
Stability issues if slender structures

UC
Tools to assess the interest of HSS
LCA LCC LCP

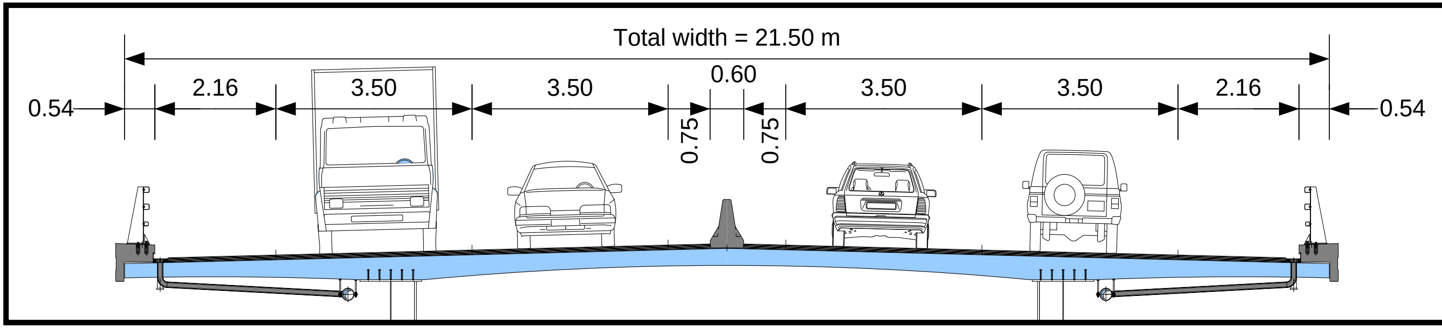
LCA Life cycle Assessment
LCC Life cycle Cost
LCP Life cycle Performance

Case study = road bridge (continuous plate girder steel concrete composite deck, with internal spans 80 meters)

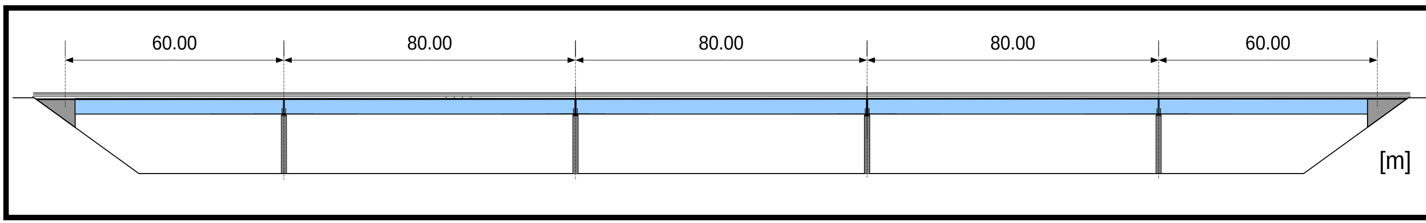
OPTImal use of HSS in BRIdges = OPTIBRI

Case Design

- **Road bridge with four traffic lanes**



- **Five spans: 60 + 3 x 80 + 60 = 360 m**



3 designs for the same bridge

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Design A : classical design using S355 steel
based on current state of Eurocodes and national rules

Design B : design using S690QL steel, where it has an interest
based on current state of Eurocodes and national rules

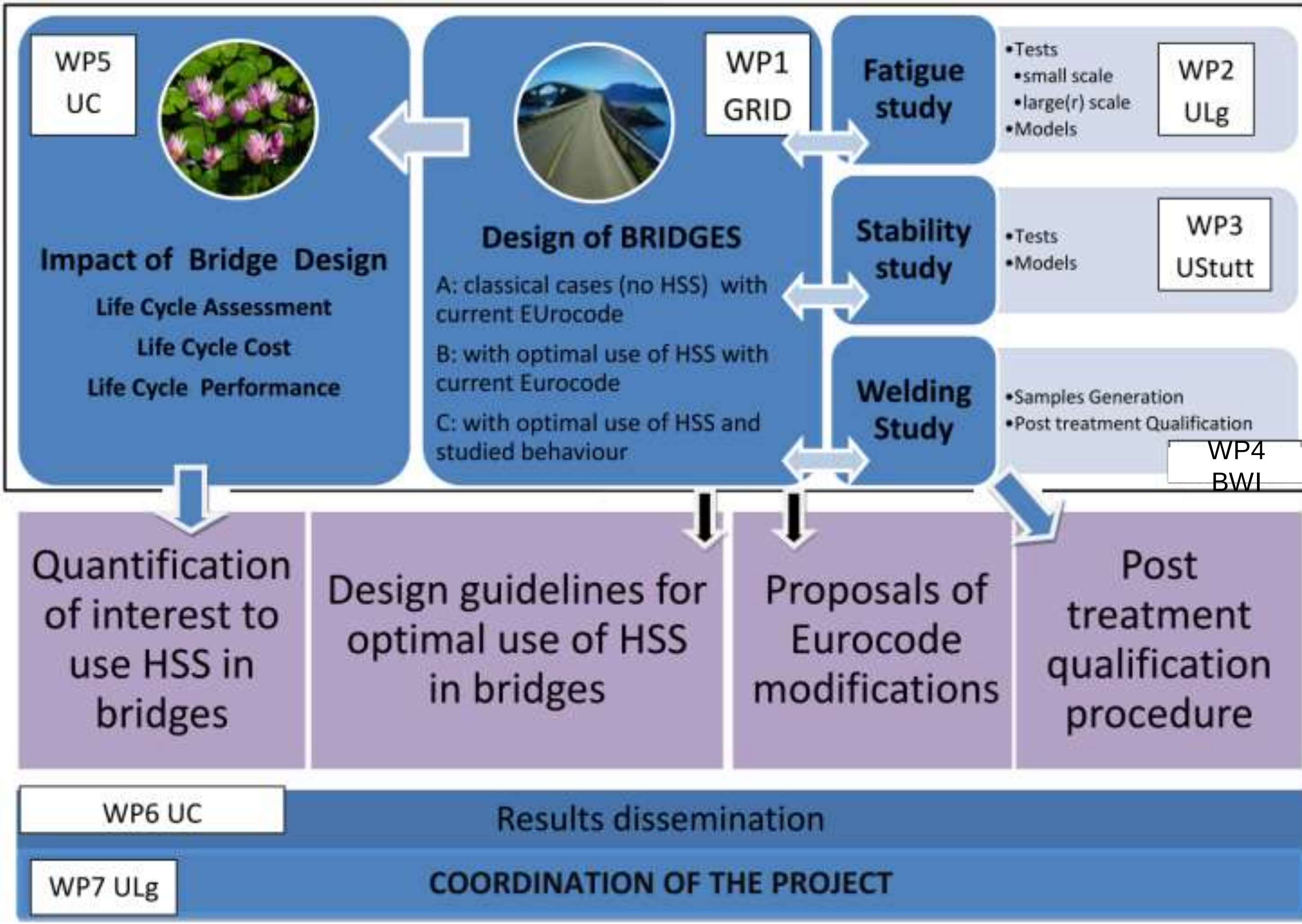
Design C : design using S690QL steel, where it has an interest based on

-real material behavior
(experimental tests and fatigue damage simulations of bridge details)

-advanced stability law
(experimental + FE anlysis of the buckling of multiaxially stressed plates
→ enhanced formula within of the code rules EN 1993-1-5)

J.O Pedro's presentation: Challenges and Benefits of High Strength Steel (HSS) in Highway Bridges

P. Toussaint's presentation: Usual application of High Strength Steel (HSS) Plates with focus on S690



WP5
UC
Impact of Bridge Design
Life Cycle Assessment
Life Cycle Cost
Life Cycle Performance
WP1
GRID
Design of BRIDGES
A: classical cases (no HSS) with current EUrocode
B: with optimal use of HSS with current Eurocode
C: with optimal use of HSS and studied behaviour
Fatigue study
•Tests
•small scale
•large(r) scale
•Models
WP2
ULg
Stability study
•Tests
•Models
WP3
UStutt
Welding Study
•Samples Generation
•Post treatment Qualification
WP4
BWI
DELIVRABLES
Quantification of interest to use HSS in bridges
Design guidelines for optimal use of HSS in bridges
Proposals of Eurocode modifications
Post treatment qualification procedure
WP6 UC
Results dissemination
WP7 ULg
COORDINATION OF THE PROJECT

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WP1 Design of Bridges by GRID

Design A provides a reference

Design B allows investigating different designs based on S690QL use

discussions between USTUTT and GRID
oriented the choices and verifications done (current Eurocode use)

Design C ongoing work based on the results of experimental fatigue curves of welded plates (Ulg) and beams (USTUTT) (with weld post treatments) + new formula of buckling verification (USTUTT)

Delays in material delivery → in test results → in model identification → in the simulation of bridge details → in Design C

C. Batista's presentation: Improved Bridge Design by Use of High Strength Steel (HSS) with OPTIBRI Developments

WP2 Fatigue study (Ulg, USTUTT, BWI)

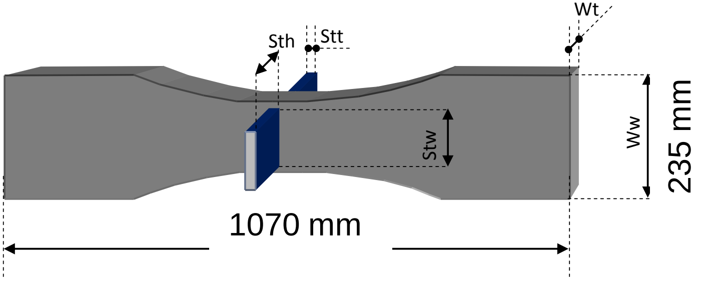
Ulg : *material scientist's approach*

- Static tests ≠ loadings, **B**ase **M**etal, **H**eat **A**ffected **Z**one and **W**eld **M**etal (WBI) - ***3 elasto plastic models (BM, HAZ, WM)***

- Fatigue tests on small specimens (mm)
  → parameters of ***Lemaitre damage model (1)***

- Static and Fatigue tests on plates + welded transversal stiffeners (Ulg) + post treatment (PIT,TIC) (residual stress distribution)
  → parameters of ***Lemaitre damage model (2)***

1st validation of the fatigue simulations with Lemaitre model



C Bouffioux's presentation: Characterization of Fatigue Behaviour, from Material Science to Civil Engineering Applications

WP2 Fatigue study (Ulg, USTUTT, BWI)

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- Fatigue tests on Beams + welded transversal stiffeners (USTUTT)

-2st validation of the fatigue simulations with Lemaitre model

Simulations of Bridge C detail:

Loading from Eurocode FLM5

→ 1 stress history

→ 1 damage distribution of the studied bridge detail

→ detail category confirmed or not

→ sensitivity analysis *not performed* : 1st approach of real behavior in HSS in bridges, *ongoing work*

-Representative HSS bridge potential rupture

S. Breunig's presentation: Categorization of Fatigue Details in View of Post-Weld Treatments

WP3 Stability study (USTUTT)

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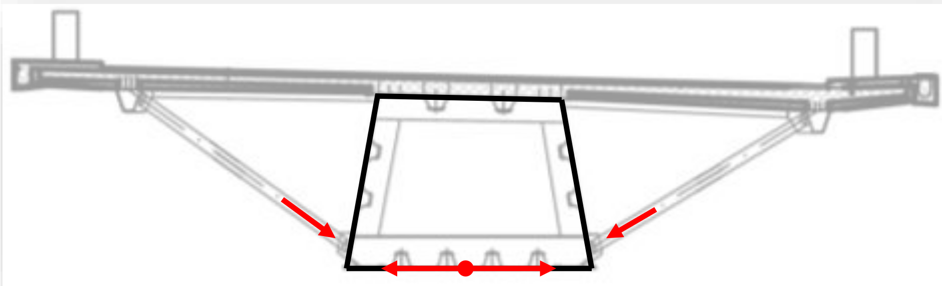
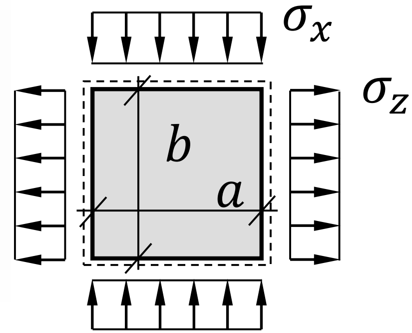
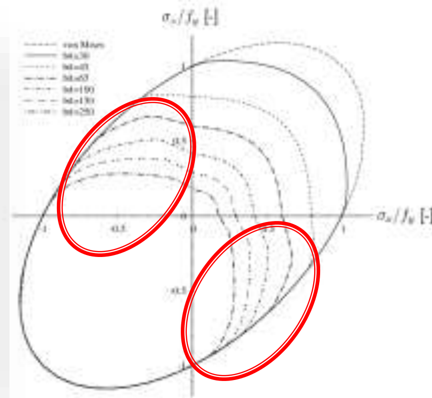


Panel with bi axial loading





V. Pourostad's presentation: Buckling Behavior of Slender Plates under Multiaxial Stresses

- → FE element simulations that are validated by experiments
- → Parametric study
- → Enhancement of the reduced stress method, introduction of V factor in Eurocode formulae

WP4 Welding study (BWI)

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Study of Fatigue crack
and microstructure
to identify optimal welding
procedure and Post Treatment
Qualification.

Welding of all plates and beams

PIT (Pneumatic Impact Treatment)
TIG (Tungsten Inert Gas) remelting
were used as Post Treatments.

Initial choice LTT (Low Temperature
Transformation filler material) dropped

LTT could not reach required toughness
values (50 to 60 J) in bridges
(results of FATWELDHSS project 2015)

T. Baaten's presentation: Welding and Post-Welded Treatments of High Strength Steel (HSS) joints

WP5 Impact of Bridge Design (UC)

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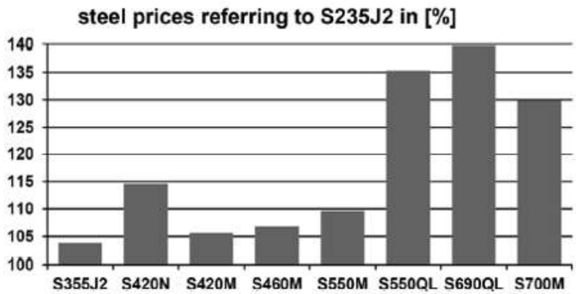
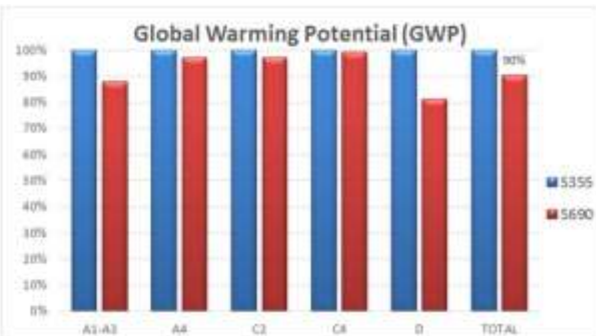


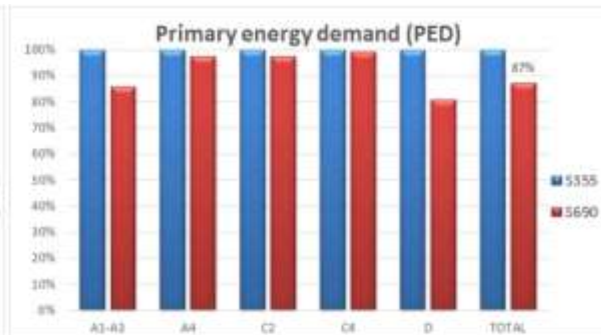
Stroetmann R. Eurosteel 2011

Work on
LCA Life cycle Assessment
LCC Life cycle Cost
LCP Life cycle Performance

Design A // B : on going work,

Design C = future





C. Rigueiro's presentation:
Comparative Life-Time Assessment of the Use of High Strength Steel (HSS) in Bridges

Thank you for your attention!

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GRID

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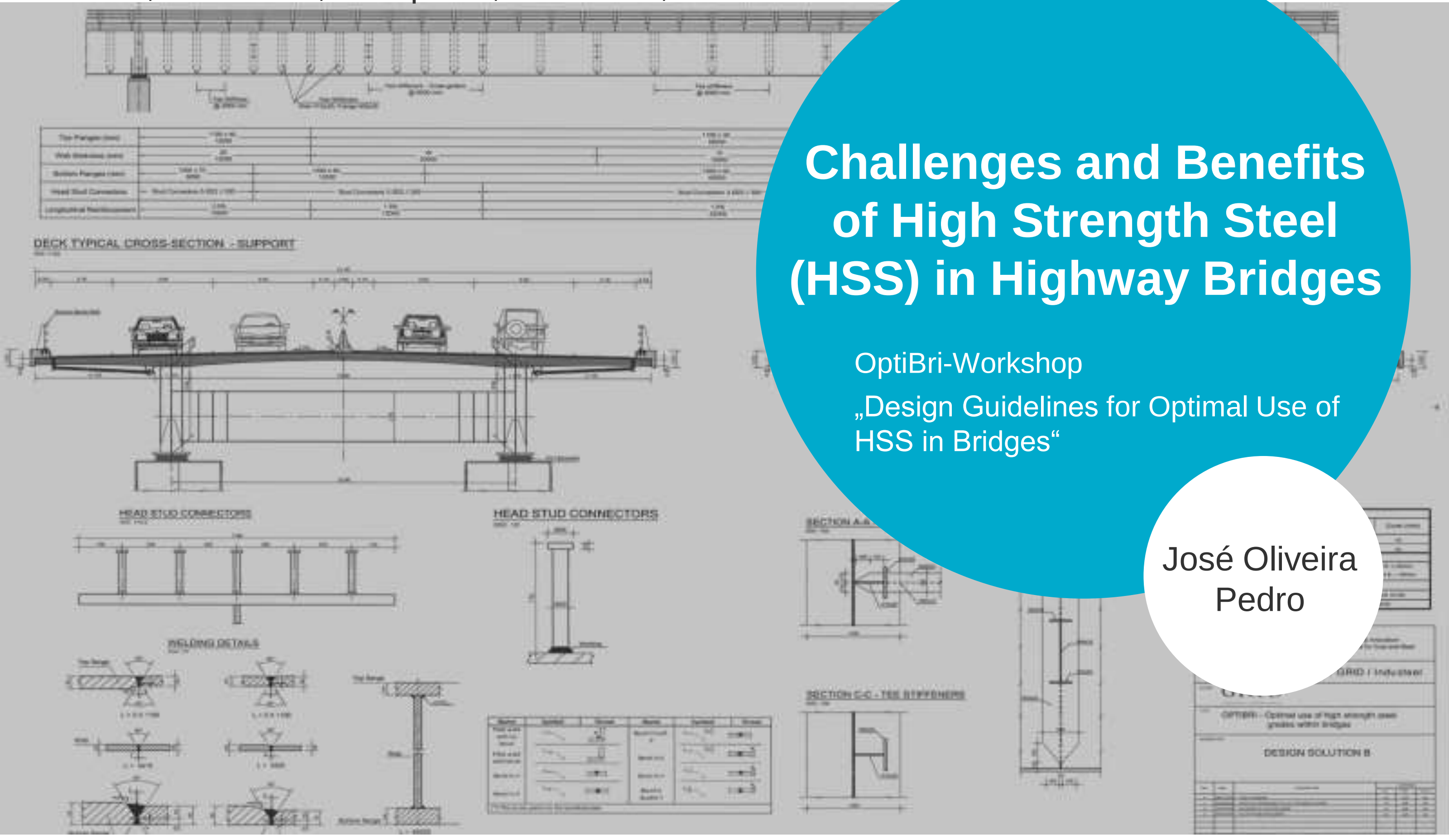
A.Reis, J.O.Pedro, C.Baptista, F.Virtuoso, C.Vieira



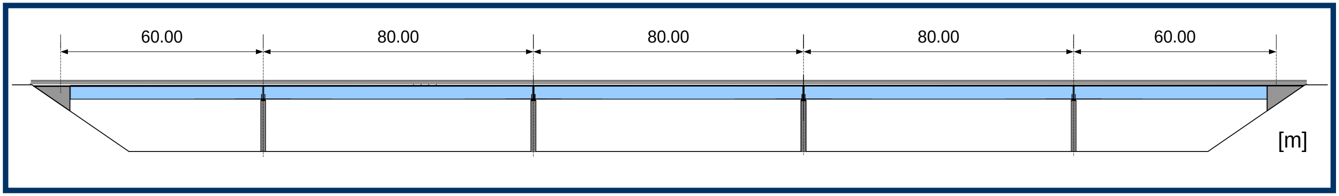
Challenges and Benefits of High Strength Steel (HSS) in Highway Bridges

OptiBri-Workshop

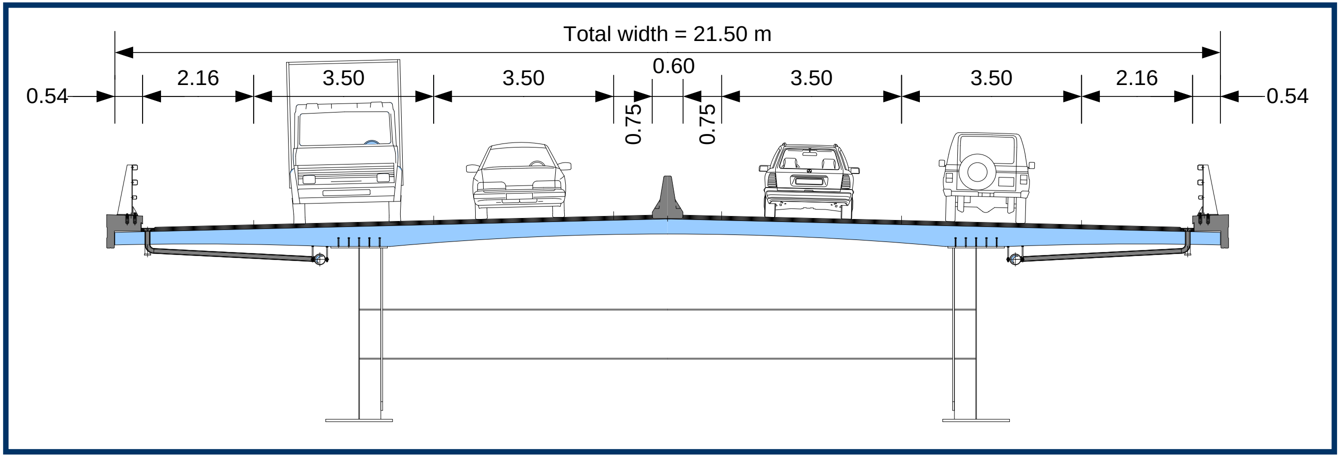
„Design Guidelines for Optimal Use of HSS in Bridges“

José Oliveira Pedro

Overview – Case study: general layout

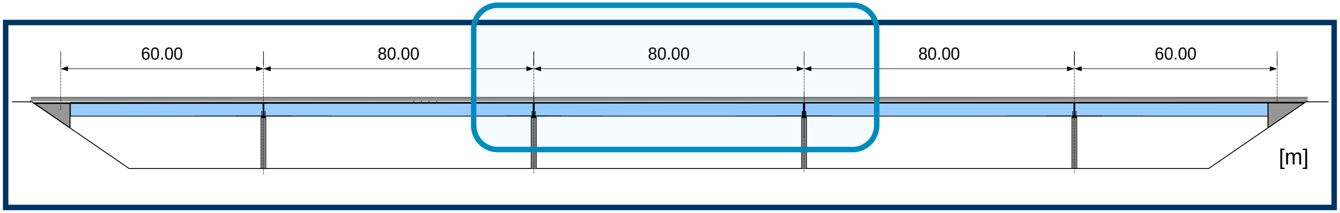


- **Five spans: 60 + 3 x 80 + 60 = 360 m**



- **Highway bridge with four traffic lanes**

Overview – Case study: construction



- Studied span: typical 80m inner span

- Executed by incremental launching of the steel structure

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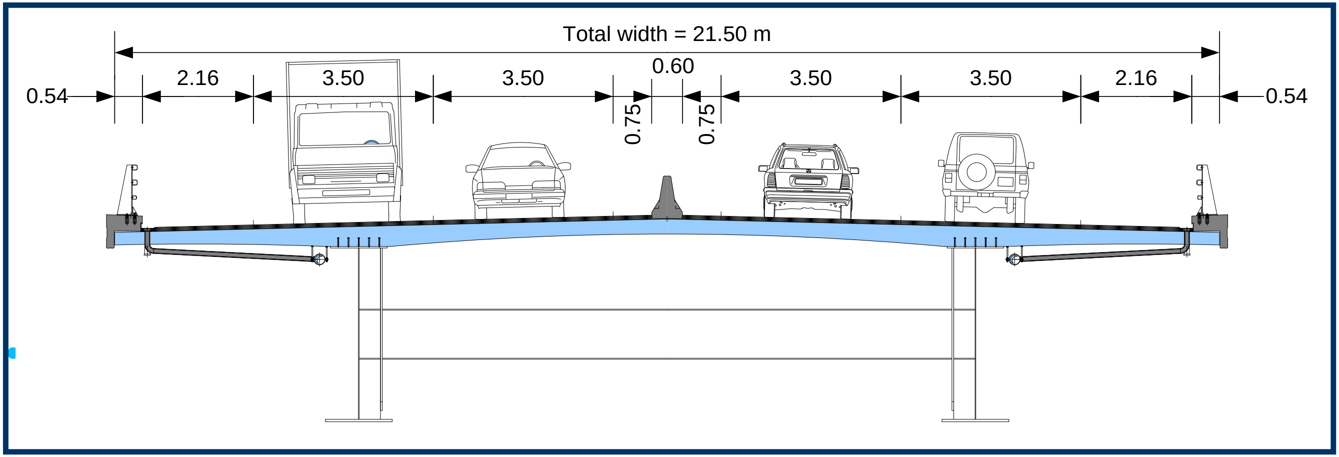
Overview – Case study: Design A and B

- **Design A –** using standard S355 NL and present Eurocodes
- **Design B –** using HSS S690 QL and present Eurocodes
- **Design C –** using HSS S690 QL, welding treatment and possible upgrades to the EC 3-1-5



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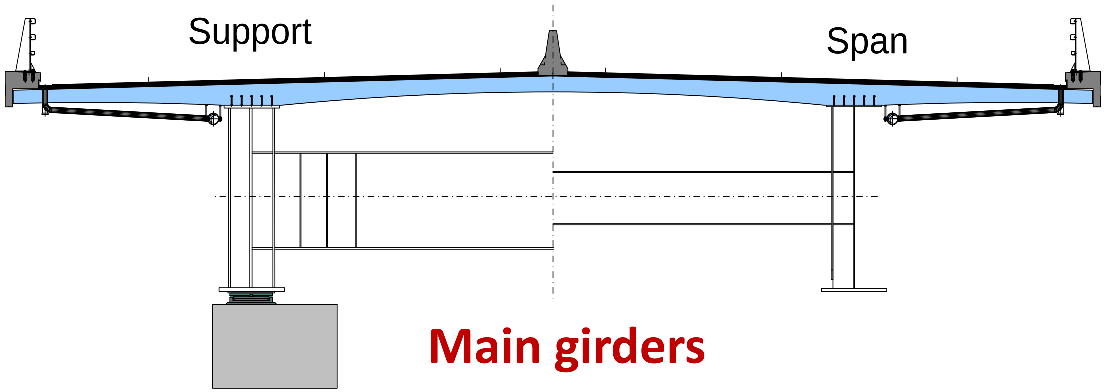
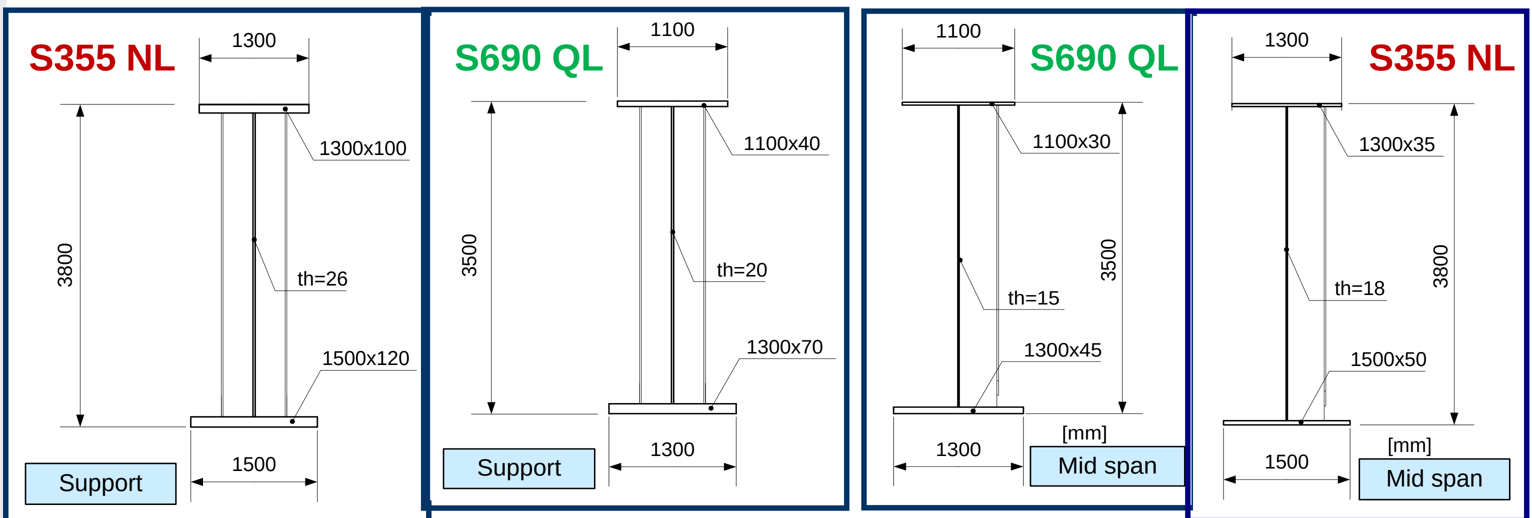
Design A – S355 NL <> Design B – S690 QL





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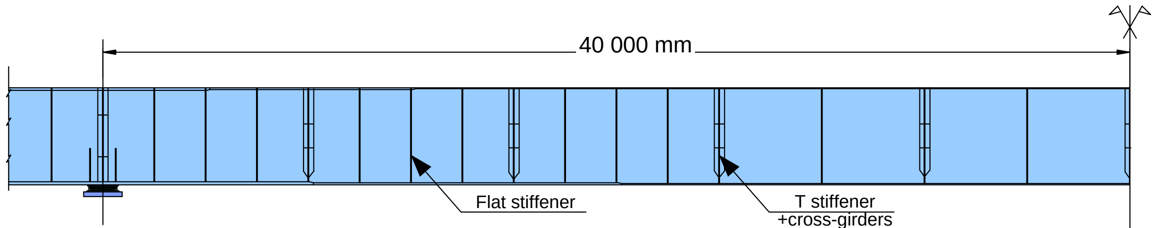
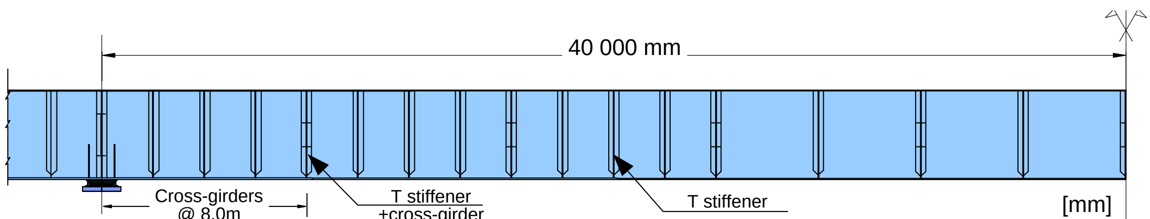
Design A – S355 NL <> Design B – S690 QL

S355 NL



Top Flanges (mm)	1300 x 100 4000	1300 x 70 8000	1300 x 35 28000
Web thickness (mm)	26 12000	22 20000	18 8000
Bottom Flanges (mm)	1500 x120 8000	1500 x 80 12000	1500 x 50 20000
Head Stud Connectors	Stud Connectors 5 ϕ22// 300	Stud Connectors 5 ϕ22// 400	Stud Connectors 3 ϕ22// 400
Longitudinal Reinforcement	2.0% 12000	1.5% 12000	1.0% 16000

S690 QL



Top Flanges (mm)	1100 x 40 12000	1100 x 30 28000	
Web thickness (mm)	20 12000	18 20000	15 8000
Bottom Flanges (mm)	1300 x 70 8000	1300 x 60 12000	1300 x 45 20000
Head Stud Connectors	Stud Connectors 5 ϕ22// 200	Stud Connectors 5 ϕ22// 300	Stud Connectors 3 ϕ22// 300
Longitudinal Reinforcement	2.0% 12000	1.5% 12000	1.0% 16000

Structural steel distribution for the typical 80 m span

Design A – S355 NL <> Design B – S690 QL

CONCRETE:

Designation	EN 206-1	Exposure Classes	Cover (mm)
Piers and Foundations	C30/37	XC3 / XF1	45
Deck - Slab	C35/45	XC4/ XF4	40

STEEL:

Structural Steel	EN10025-2 S355 J2 (Z15 if th. ≤ 30mm) EN10025-3 S355 N (Z15 if 30 < th. ≤ 80mm) EN10025-3 S355 NL (Z25 if th. > 80mm)
Reinforcement	B500B (EN 10080)
Prestress Cables	$f_{p0,1k} \geq$ 1637 MPa / $f_{puk} \geq$ 1860 MPa (EN 10138)
Stud Connectors	EN10025 S235 J2 + C450 (EN ISO 13918)

Structural Steel	EN10025-6 S690 QL (40J,-40°C) (Z15 if th. ≤ 40mm) EN10025-6 S690 QL1 (40J,-40°C) (Z15 if th. > 40mm)

Structural Materials

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Bridge design criteria

- ✓ **<u>Deck steel design and detailing</u>** is performed using European standards EN 1990, EN 1991, EN 1993 and EN 1994
- ✓ **<u>Structural behaviour</u>** at ultimate and serviceability limit states (ULS, SLS), evaluated by finite frame element models, with due account for rheological effect from concrete
- ✓ **<u>Construction stages</u>** are taken into account by superposition of results from:
  - steel structure frame model, for the application of its own weight and the slab concrete weight
  - composite structure frame models with modular ratios for concrete, assessed for short-term actions, permanent actions and shrinkage effects (following EN 1994-2)

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Bridge design criteria

- ✓ **Longitudinal safety verifications** included namely:
  - ULS – bending and shear girders resistance
  - SLS – stress limitations on structural steel, reinforcement and concrete slab, and deflections
  - ULS – fatigue of girders structural steel and stud connectors (welded joint between the transverse stiffeners and the bottom tension flange proves to be the most relevant detail for the design of the composite steel-concrete twin plate girder deck)
- ✓ **Flange induced buckling** following formulation from EN 1993-1-5
- ✓ **Transverse stiffeners** designed also according with EN 1993-1-5 (plate buckling of the webs near supports is a key issue when using HSS; close intermediate transverse stiffeners are used to increase web shear buckling resistance)

ULS bridge deck design – Bending resistance (EN 1993-1-5)

Mid-span section Class 1 – *Plastic section analysis*	Design A S355	Design B S690
$M_{\mathrm{Ed}} / M_{\mathrm{pl.Rd}} < 1$	**0.74**	**0.54**
$\{\sigma_{\mathrm{Ed}} / (f_{\mathrm{yf}} / \gamma_{\mathrm{M0}})\}$ Bottom flange < 1	**0.93**	**0.65**

- ULS bending resistance is not a critical design issue for S690
- All span sections can still be designed elastically

ULS bridge deck design – Bending resistance (EN 1993-1-5)

Support section Class 4 – *Elastic analysis with effective section*	Design A S355	Design B S690
$\{\sigma_{\mathrm{Ed}} / (f_{\mathrm{yf}} / \gamma_{\mathrm{M0}})\}_{\text{Eff. bottom flange}} < 1$	**0.95**	**0.88**
$\{\sigma_{\mathrm{Ed}} / (\chi_{\mathrm{LT}} f_{\mathrm{yf}} / \gamma_{\mathrm{M1}})\}_{\text{Eff. bottom flange}} < 1$ (*) (*) at 0.25 L_k = 5 m from the support	**0.92**	**0.97**

- ULS bending resistance > elastic analysis for both designs
- For S690 also the bottom flange is in class 4 since $\varepsilon = \sqrt{235/f_y} = 0.584$
- For S690 > web under compression with ρ = 0.49, bottom flange reduction ρ = 0.94; Lateral torsion buckling χ_{LT} = 0.72

ULS bridge deck design – Shear resistance (EN 1993-1-5)

Effective Width Method (EN 1993-1-5, sections 4 to 7)

Support section (transversal stiffeners @ 2m)	Design A S355	Design B S690
h_w x t_w (mm²)	3590 x **26**	3390 x **20**
$\bar{\lambda}_w = 0.76\sqrt{f_{yw}/\tau_{cr}}$	0.97	1.77
χ_w	0.86	0.56
$V_{Ed}/V_{bw,Rd} = V_{Ed}/(\chi_w h_w t_w f_{yw}/\sqrt{3}\gamma_{M1})$	**0.86**	**0.91**
(M/V) Interaction with γ_{M1}=1.1 (*) (*) at min $\{t_w/2\,;\,a/2\}$ = 1 m from the support	No interaction	**1.0**

- Using S690, web thickness is reduced from 26 mm to 20 mm
- Interaction (M,V) makes the support panels work at the limit, if consistently a unique safety coefficient γ_{M1} =1.1 is adopted

ULS bridge deck design – Shear resistance (EN 1993-1-5)

Reduced Stress Method (EN 1993-1-5, sections 10)

Support section (transv. stiff. @ 2m; Without long. stiff.)	Design A S355	Design B S690
$h_w \times t_w$ (mm²)	3590 x **26**	3390 x **20**
ρ_x	0.83	0.49
χ_w	0.76	0.52
$(\sigma_{x,Ed}\ ,\ \tau_{Ed})$ (MPa) bottom end of the web	(267.8,135.0)	(519.7,183.2)
$\sqrt{\left(\frac{\sigma_{x,Ed}}{\rho_x\ f_y/\gamma_{M1}}\right)^2 + 3\left(\frac{\tau_{Ed}}{\chi_w\ f_y/\gamma_{M1}}\right)^2} \leq 1$	$(1.07+0.95)^{0.5}=$ **1.42**	$(2.83+0.94)^{0.5}=$ **1.94**
Required t_w (mm)	**34**	**32**

ULS bridge deck design – Shear resistance (EN 1993-1-5)

Why so inconsistency between

effective Width Method < > Reduced Stress Method ?

- ✓ Using the reduced stress method – no partial plastic stress redistributions are allowed (as it is the case for the interaction criterion of section 7, EC3-1.5)
- ✓ Therefore, ULS bending moment and shear force cannot be primarily allocated to the support cross sectional elements:
  - hogging bending moment resisted by the {flanges + reinforcement} alone
  - so that, the web resistance can fully be used for the support shear force
- ✓ Moreover, the reduced stress method consistently uses $\gamma_{M1} = 1.1$ (which is more accurate) for plastic resistance and instability, but verifications are made for the cross-section over the support

ULS bridge deck design – Shear resistance (EN 1993-1-5)

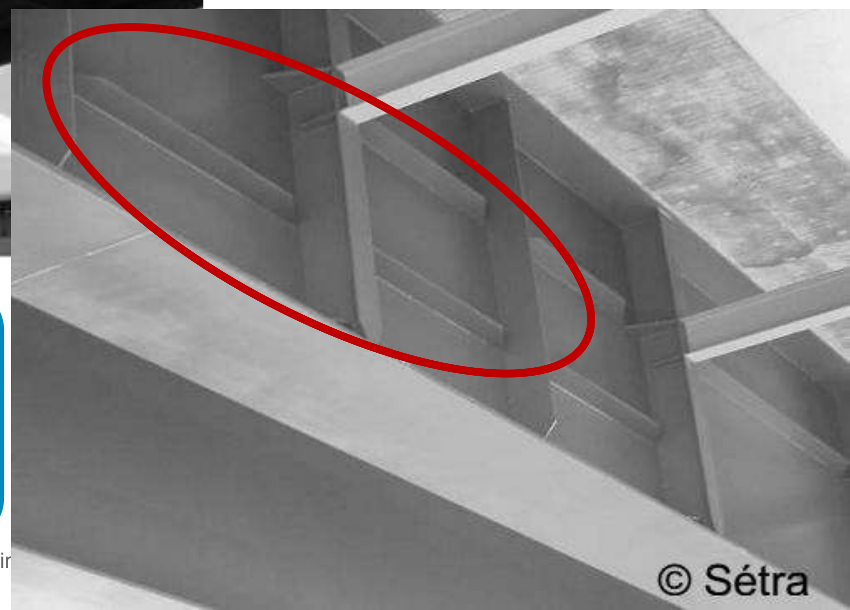
Reduced Stress Method (EN1993-1-5, sections 10)

Longitudinal stiffeners on the outside of the web

Ref. Railway Bridge near Riesa, Germany – COMBRI Design manual

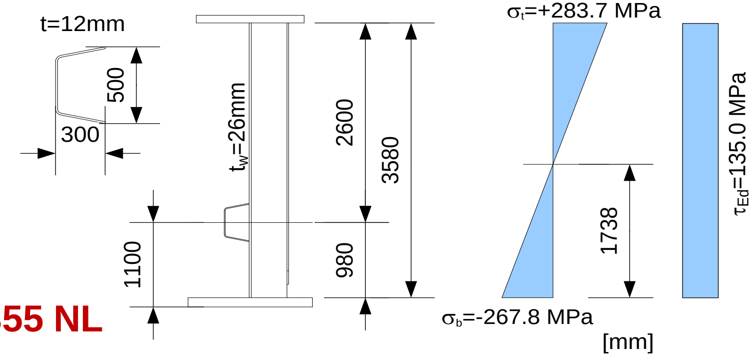
Longitudinal flat stiffeners on the inside of the web

Ref. Twin-girder Bridge in Triel-sur-Seine, France – COMBRI Design manual

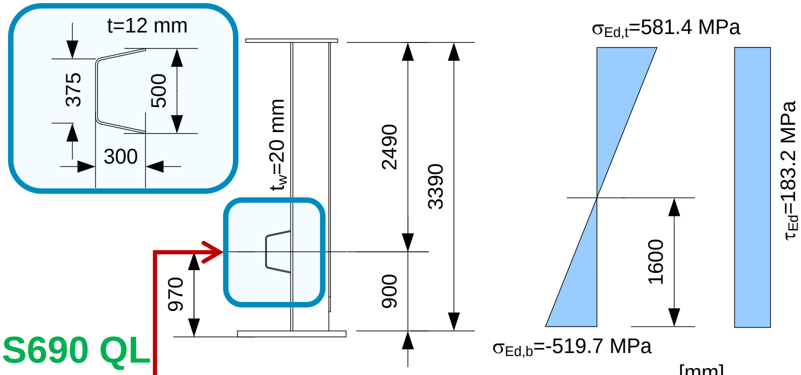


ULS bridge deck design – Shear resistance (EN 1993-1-5)

Reduced Stress Method (EN1993-1-5, sections 10)



S355 NL



S690 QL

Longitudinal closed stiffener

Design solution:

– keep the web thickness

– add a continuous longitudinal closed stiffener in the external compressed bottom part of the web

– extended up to 20 m from both sides of the supports.

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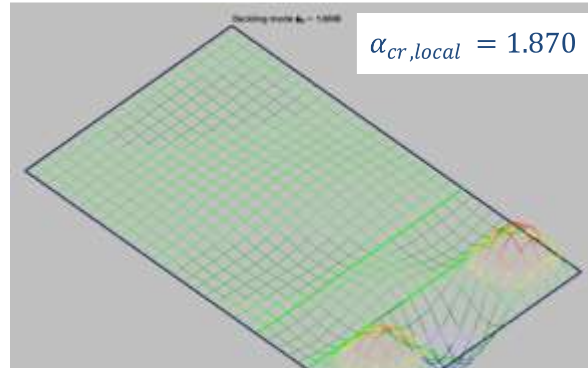
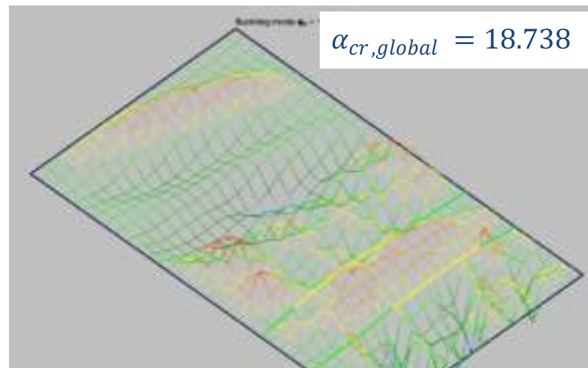
ULS bridge deck design – Shear resistance (EN 1993-1-5)

Reduced Stress Method (EN1993-1-5, sections 10)

S690 QL	Unstiffened Web	Stiffened Web
$\alpha_{ult,k}$	1.13	1.13
$\alpha_{cr,x}$	0.34	---
$\alpha_{cr,\tau}$	0.70	---
α_{cr}	0.31	**1.87** (EBplate)
$\bar{\lambda}_p$	1.92	**0.81**
ρ_x	0.49	**0.96**
χ_w	0.52	**1.00**
$\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)^2$	2.83	**0.75**
$3\left(\frac{\tau_{Ed}}{\chi_w f_y/\gamma_{M1}}\right)^2$	0.94	**0.25**
≤ 1	1.94	**1.00**



Local plate mode due to {bending + shear}



Global plate mode due to {bending + shear}

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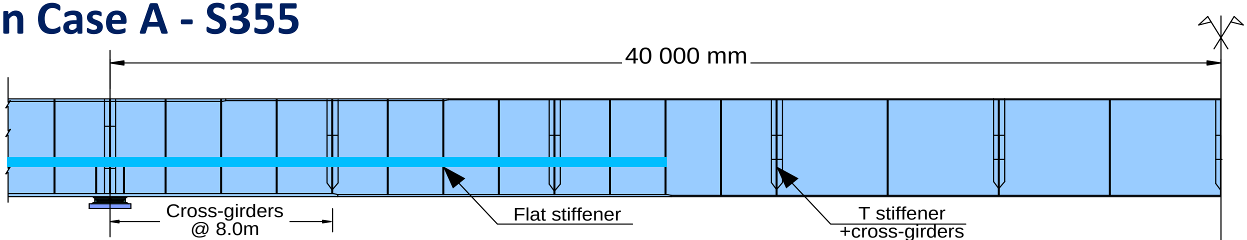
ULS bridge deck design – Transversal stiffeners (EN 1993-1-5)

Design Case A - S355



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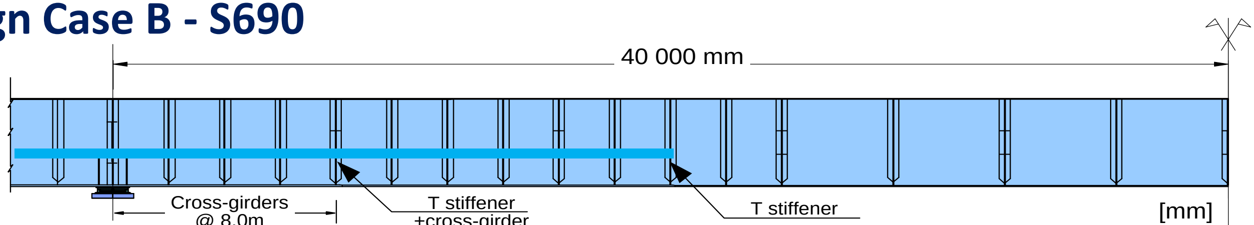
Head Stud Connectors	Stud Connectors 5 ϕ22// 300	Stud Connectors 5 ϕ22// 400	Stud Connectors 3 ϕ22// 400
Longitudinal Reinforcement	2.0% 12000	1.5% 12000	1.0% 16000

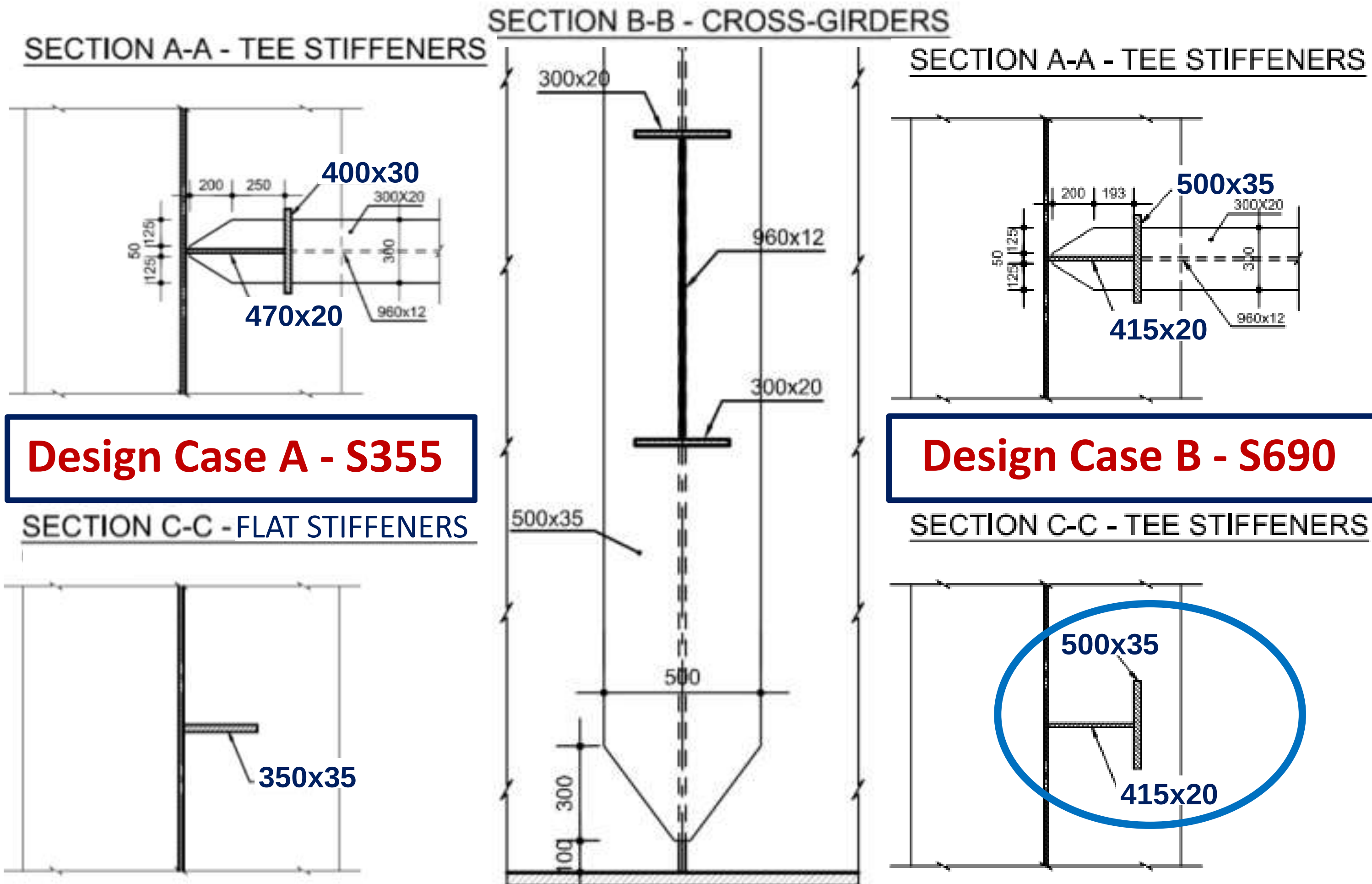
Design Case B - S690



Top Flanges (mm)	1100 x 40 12000	1100 x 30 28000	
Web thickness (mm)	20 12000	18 20000	15 8000
Bottom Flanges (mm)	1300 x 70 8000	1300 x 60 12000	1300 x 45 20000
Head Stud Connectors	Stud Connectors 5 ϕ22// 200	Stud Connectors 5 ϕ22// 300	Stud Connectors 3 ϕ22// 300
Longitudinal Reinforcement	2.0% 12000	1.5% 12000	1.0% 16000

Research Fund for Coal & Steel

ULS bridge deck design – Transversal stiffeners (EN 1993-1-5)



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ULS bridge deck design – Transversal stiffeners (EN 1993-1-5 §9.3.3)

– Minimum stiffness requirements for shear verification of the webs – checked by imposing a second moment of the area of a stiffener I_{st} higher than:

$$I_{st} \geq 1.5\, h_w^3 \cdot t_w^3 / a^2 \quad \text{if} \quad a/h_w < \sqrt{2}$$

$$I_{st} \geq 0.75\, h_w \cdot t_w^3 \quad \text{if} \quad a/h_w \geq \sqrt{2}$$

easily verified and not demanding very strong stiffeners

– Resistance requirement – verified with the axial force N_{st} imposed by the tension field action given by:

$$N_{st} = V_{Ed} - V_{cr,w}$$

considerable more demanding for the intermediate single-sided HSS stiffeners, working with very high V_{Ed}, thus producing high eccentric axial forces N_{st}, to be taken into account in the beam-column verification

V_{Ed} taken at the distance 0.5 h_w from the edge of the panel with the largest shear force;
$V_{cr,w}$ corresponds to the shear buckling resistance of the web without stiffeners.

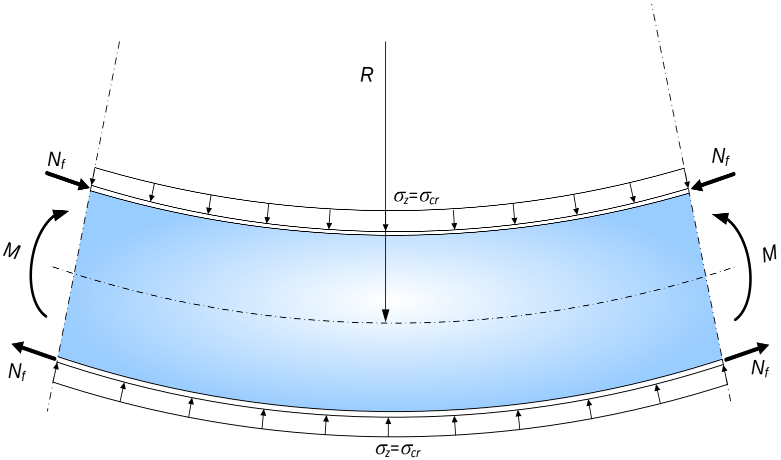
– Safety to torsional buckling – design rules for open stiffeners assume that torsional buckling should be completely prevented when loaded axially; thus EN 1993-1-5 provides the following general requirement for the σ_{cr} the elastic critical stress of open stiffeners:

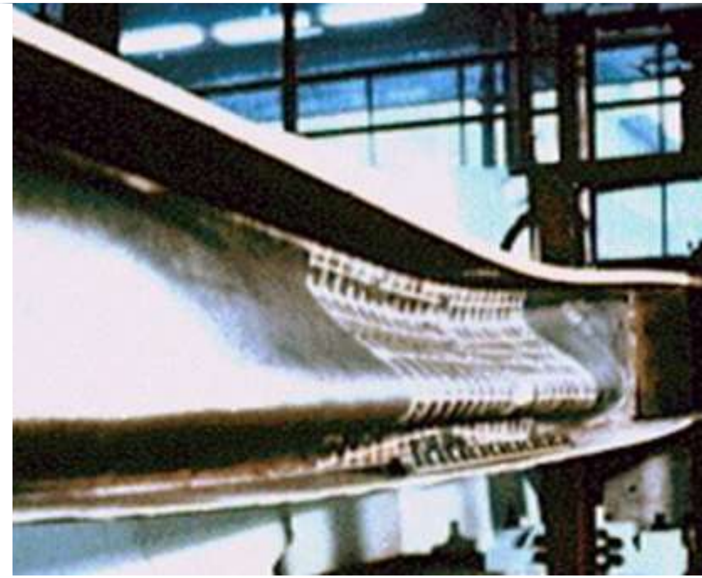
$$\sigma_{cr} \geq \theta\, f_y$$

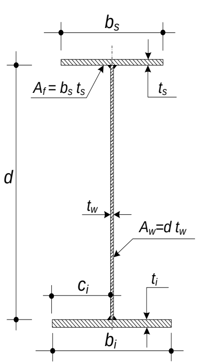
for T stiff. $\theta = 6$
for Flat stiff. $\theta = 2$

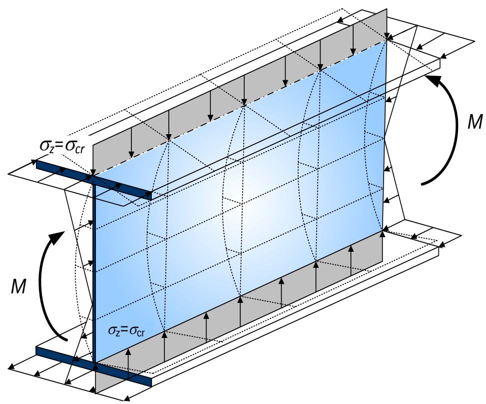
This is the more difficult criteria to achieve; design replaces f_y by the maximum actual stress $\sigma_{max,Ed}$ at the intermediate transverse stiffener

ULS bridge deck design – Flange Induced Buckling (EN 1993-1-5)









$$\frac{h_w}{t_w} \leq K \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}$$

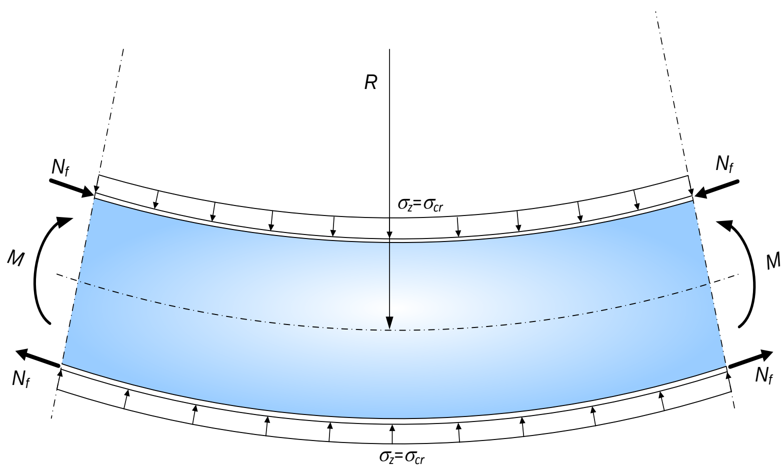
Elastic analysis	$k = 0.55$
Plastic analysis	$k = 0.40$

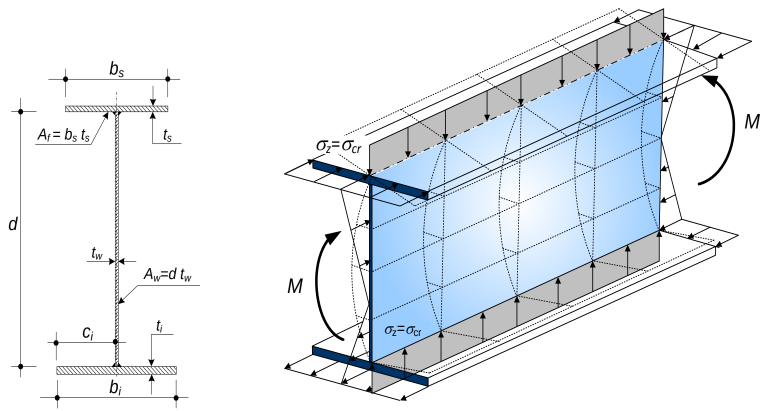
ULS bridge deck design – Flange Induced Buckling (EN 1993-1-5)

Design B - S690 deck section	1-2 support	3	4-5	6-8	9-11 mid-span
k	0.55	0.55	0.55	0.55	0.55
f_{yf} (MPa)	650	650	650	690	690
$A_w = h_w \times t_w$ (mm^2)	3390x20	3400x20	3410x18	3425x18	3425x15
$A_{fc} = b_{f,eff} \times t_f$ (mm^2)	1230x70	1100x60	1100x60	1300x45	1300x45
$k\,E / f_{yf} \sqrt{A_w / A_{fc}}$	**158**	180	**172**	**172**	**157**
h_w / t_w < limit?	**170**	170	**189**	**190**	**228**

Too conservative assumptions!

ULS bridge deck design – Flange Induced Buckling (EN 1993-1-5)



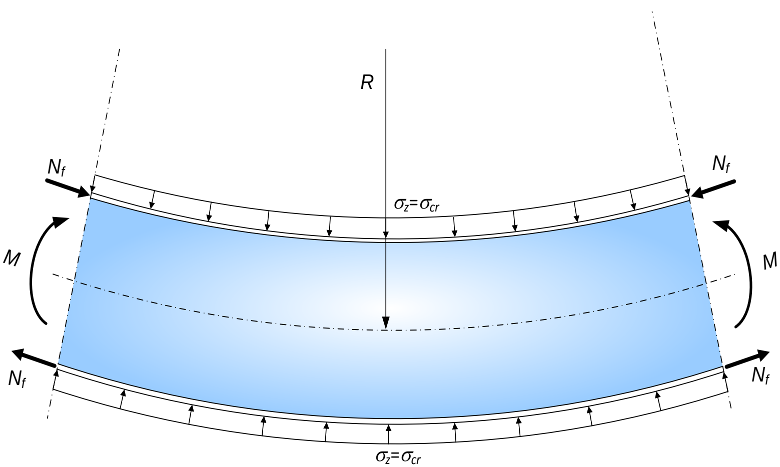


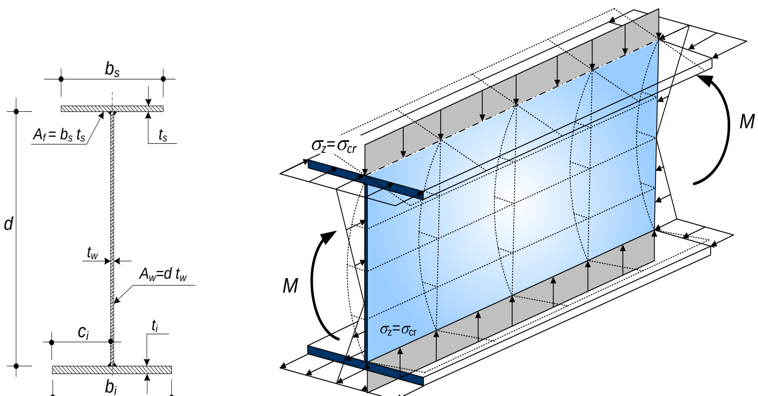
$$\frac{h_w}{t_w} \leq K \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}$$

This expression has indeed several simplified assumptions:

- ✓ Symmetrical I-section girder subjected to pure bending > neutral axis h_i = 1/2 *height*
- ✓ Transversal stiffeners effect ignored
- ✓ Residual stress with a peak of $0.5f_{yf}$ in the region adjacent to the web-to-flange welded joint
- ✓ Both flanges attain yield strength f_{yf}

ULS bridge deck design – Flange Induced Buckling (EN 1993-1-5)





$$\frac{h_w}{t_w} \leq K \frac{E}{\beta f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}$$

Being parameter β function of:

$$\beta^2 = \frac{h\alpha}{3h_i}(\alpha + 0.5) \leq 1\ ,\ \alpha = \sigma_{Ed}/f_{yf}$$

For:

- ✓ Non symmetrical composite girder sections > neutral axis at height h_i from bottom flange (not exactly at 1/2 height h)
- ✓ ULS tension / compression σ_{Ed} installed at the flange, lower that the yielding strength f_{yf}

ULS bridge deck design – Flange Induced Buckling (EN 1993-1-5)

Design B - S690 deck section	1-2 support	3	4-5	6-8	9-11 mid-span
k	0.55	0.55	0.55	0.55	0.55
$\sigma_{Ed}\,(\mathrm{MPa})$	570	411	302	346	447
$\sigma_{Ed}/\,f_{yf}$	0.88	0.63	0.46	0.50	0.65
$h\,/h_i$	2.07	1.82	1.95	1.51	1.59
β	**0.91**	**0.66**	**0.54**	**0.50**	**0.63**
$k\,E\,/\,(\beta f_{yf})\sqrt{A_w/A_{fc}}$	**173**	**274**	**318**	**342**	**250**
$h_w/\,t_w$ < limit?	**170**	**170**	**189**	**190**	**228**

SLS bridge deck design – Deflections and Stresses

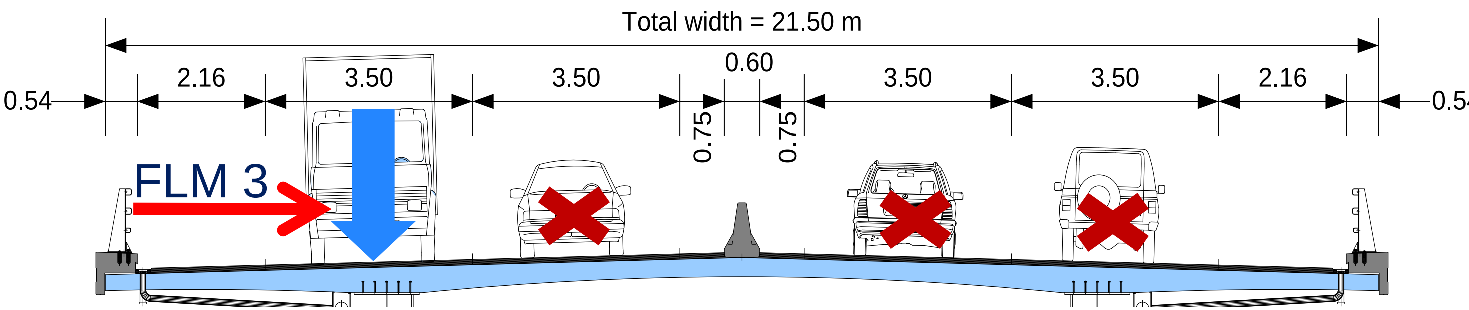
Deflection for frequent Highway Live Loads (LM1)

Condition	Design A – S355	Design B – S690
$\delta(\psi_1 Q_{k1}) \leq L/500 = 160$ mm (*) (*) $L/500$ *imposed by SIA 260*	49 mm (= L/1632)	74 mm (= L/1081)

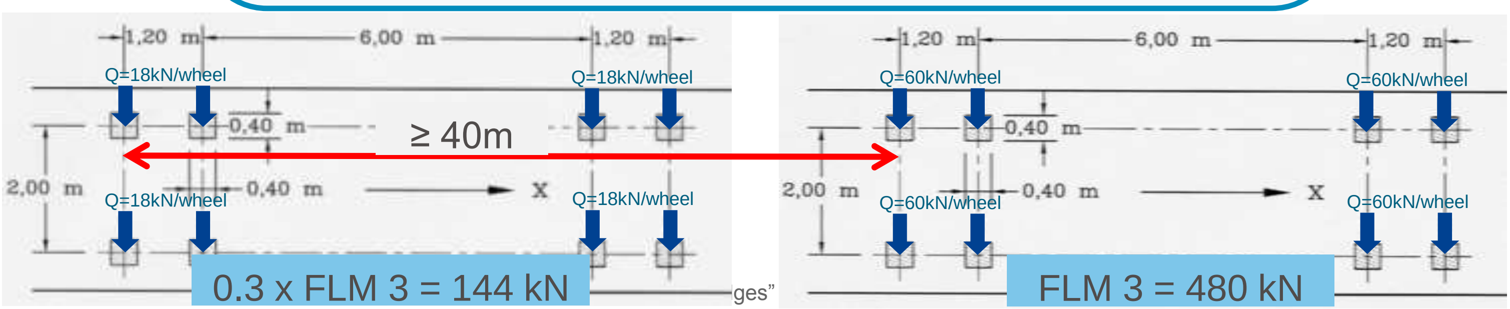
Stress ratios in structural steel ($\sigma_{Ed,ser,max}$ / f_y), concrete slab ($\sigma_{c,ser,max}$ / 0.6 f_{ck}), and slab reinforcement ($\sigma_{rs,ser} \leq$ 0.8 f_{sk})

	Design A – S355		Design B – S690	
Section	Support	Mid-span	Support	Mid-span
Concrete slab / reinforcement	0.49	0.27	0.61	0.32
Top flange	0.71	0.35	0.59	0.26
Web	0.75	0.65	0.61	0.47
Bottom flange	0.73	0.68	0.53	0.48

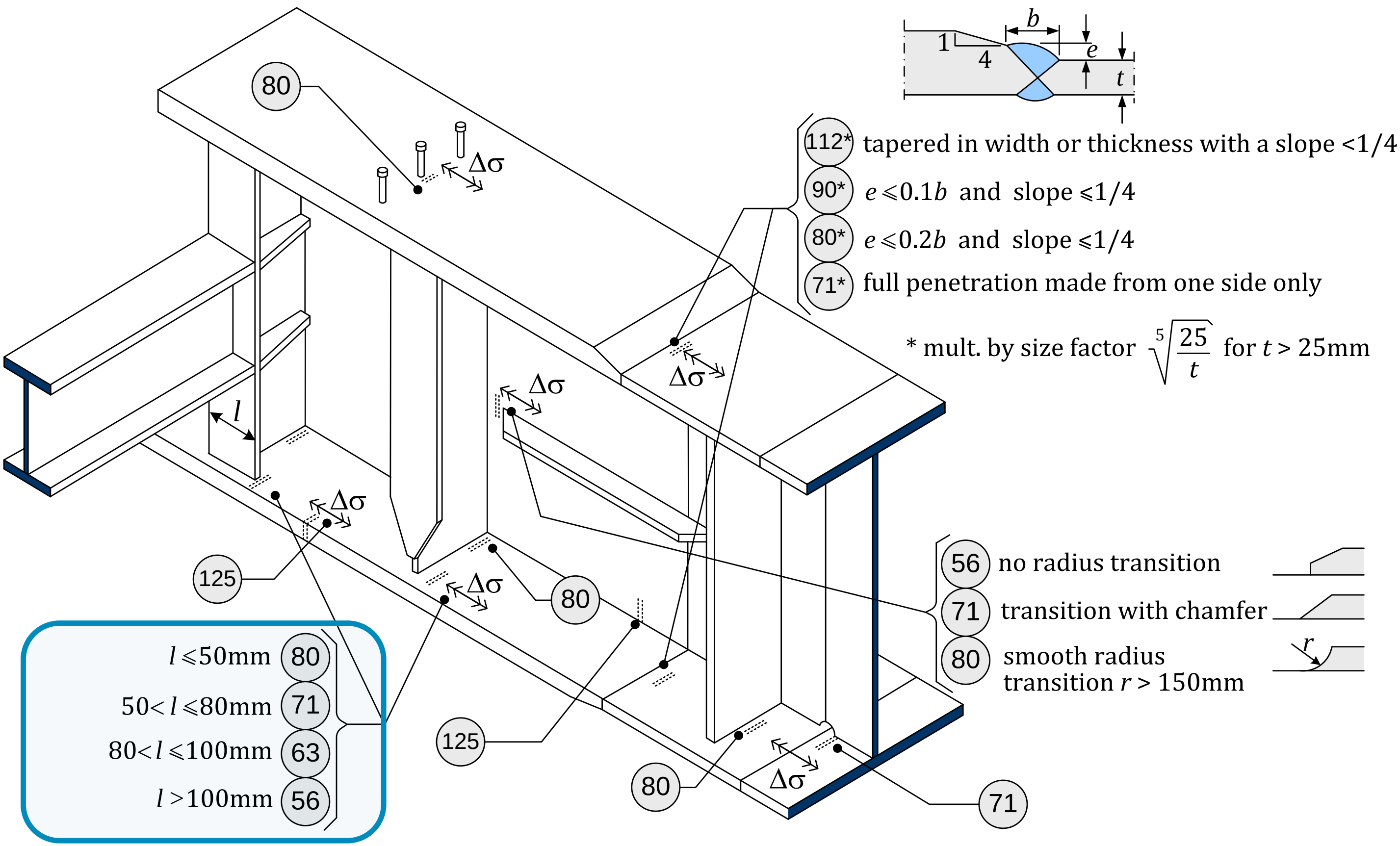
ULS bridge deck design – Fatigue assessment (EN 1993-1-9)



- ✓ Two vehicles FLM3 in the same lane (EN1991-2) (48 ton + 14.4 ton at 40m)
- ✓ Slow lanes in the actual position
- ✓ $N_{obs} = 2.0 \times 10^6$ Lories/year/slow lane
- ✓ Bridge design life of 100 years

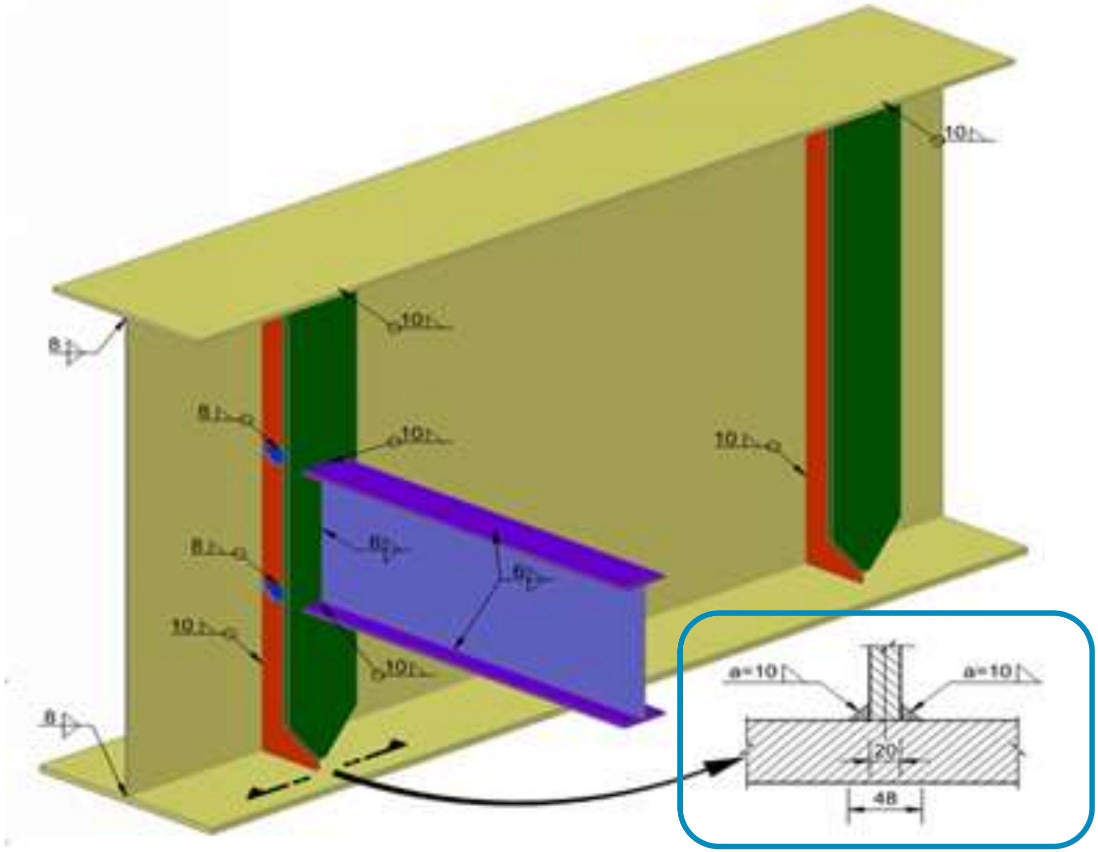


ULS bridge deck design – Fatigue assessment (EN 1993-1-9)



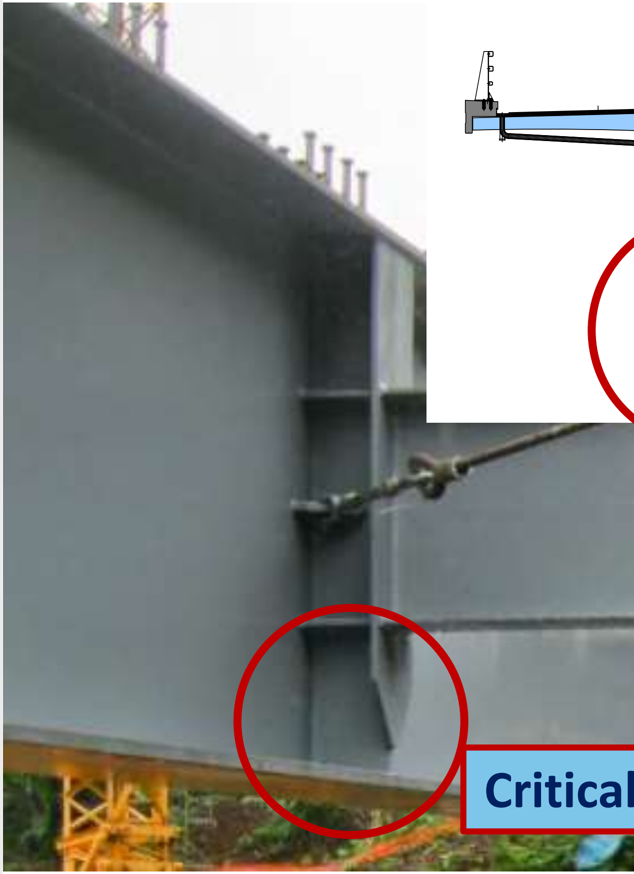
FAT Detail categories

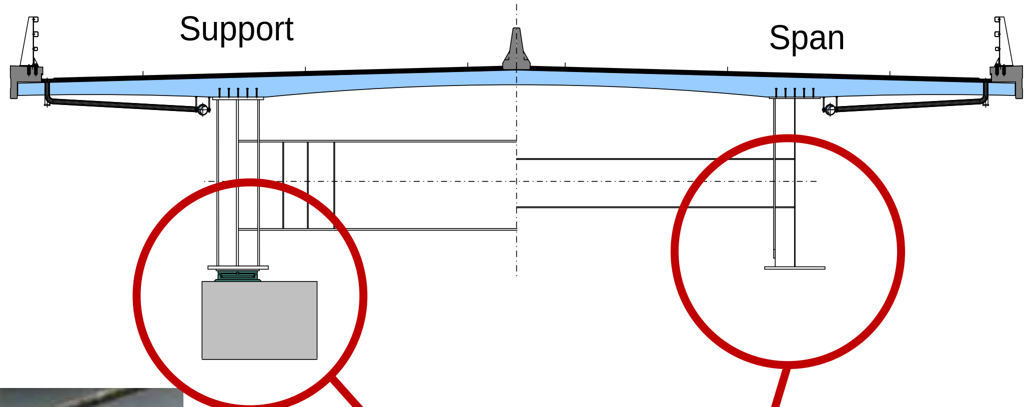
ULS bridge deck design – Fatigue assessment (EN 1993-1-9)



Critical fatigue detail = FAT 56 support; FAT 80 span

ULS bridge deck design – Fatigue assessment (EN 1993-1-9)





$\Delta\sigma_R$= 50 MPa < FAT 56
with $l > 100$ mm

$\Delta\sigma_R$= 78 MPa < FAT 80
with $l = (t_{plate} + 2t_{welding}) < 50$ mm

Critical detail

ULS bridge deck design – Fatigue assessment (EN 1993-1-9)

$\Delta\sigma_R = \Delta\sigma_E \, \gamma_{Mf} \, \gamma_{Ff} < \text{FAT (detail)}$

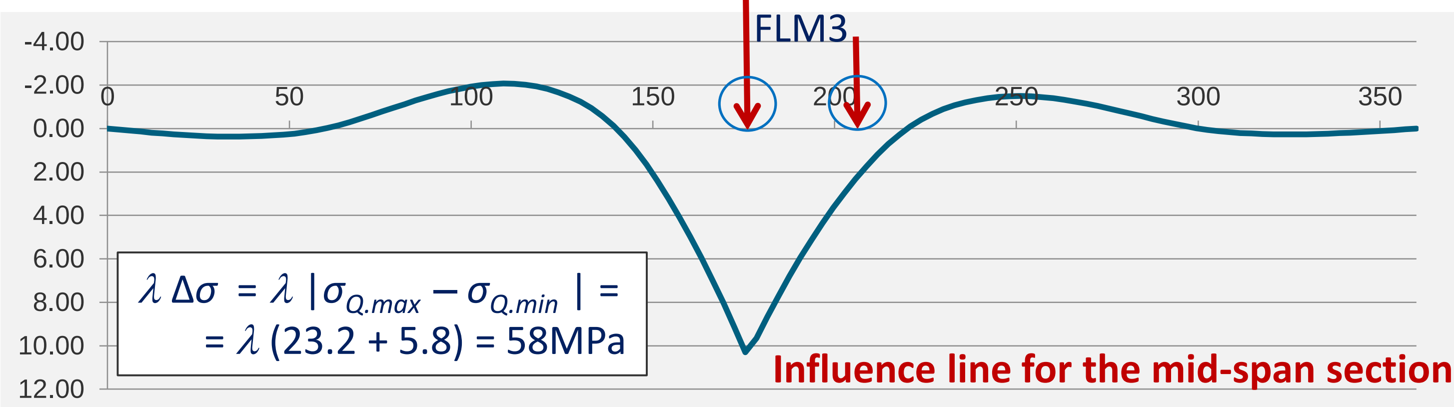
$\Delta\sigma_E = \lambda \, |\sigma_{Q.max} - \sigma_{Q.min}|$

$\gamma_{Mf} = 1.35 \qquad \gamma_{Ff} = 1.0$

Damage equivalent factor

$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \leq \lambda_{max}$

Damage equivalent factor $\lambda <> \lambda_{max}$					
Support	$\lambda_1 = 2.20$	$\lambda_2 = 1.224$	$\lambda_3 = 1.00$	$\lambda_4 = 1.00$	$\lambda = $ **2.69** $< \lambda_{max} = 2.70$
Span	$\lambda_1 = 1.85$	$\lambda_2 = 1.224$	$\lambda_3 = 1.00$	$\lambda_4 = 1.00$	$\lambda = 2.26 > \lambda_{max} = $ **2.00**



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ULS bridge deck design – Fatigue assessment (EN 1993-1-9)

$\Delta\sigma_R = \Delta\sigma_E\,\gamma_{Mf}\,\gamma_{Ff} < \text{FAT (detail)}$

$\Delta\sigma_E = \lambda\,|\sigma_{Q.max} - \sigma_{Q.min}|$

$\gamma_{Mf} = 1.35 \qquad \gamma_{Ff} = 1.0$

Damage equivalent factor

$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \leq \lambda_{max}$

Damage equivalent factor $\lambda <> \lambda_{max}$					
Support	$\lambda_1 = 2.20$	$\lambda_2 = 1.224$	$\lambda_3 = 1.00$	$\lambda_4 = 1.00$	$\lambda =$ **2.69** $< \lambda_{max} = 2.70$
Span	$\lambda_1 = 1.85$	$\lambda_2 = 1.224$	$\lambda_3 = 1.00$	$\lambda_4 = 1.00$	$\lambda = 2.26 > \lambda_{max} =$ **2.00**

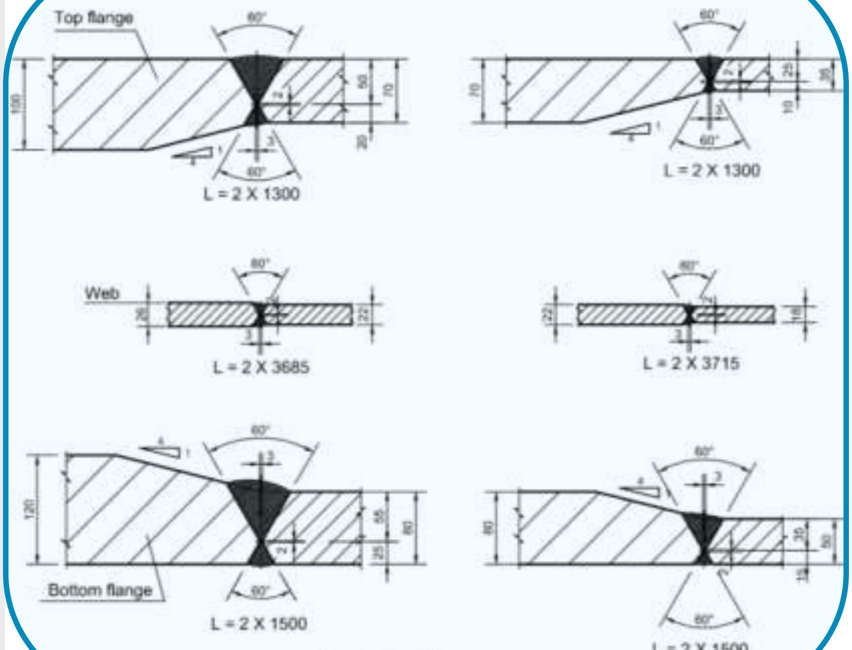
$\Delta\sigma_E\,\gamma_{Mf}\,\gamma_{Ff}$ [MPa]	Design A – S355		Design B – S690		Limit
Section	Support	Mid-span	Support	Mid-span	FAT
Top flange	23	14	36	7	**56**
Bottom flange	26	57	**50**	**78**	**56 / 80**

Bridge deck design – Structural steel weight

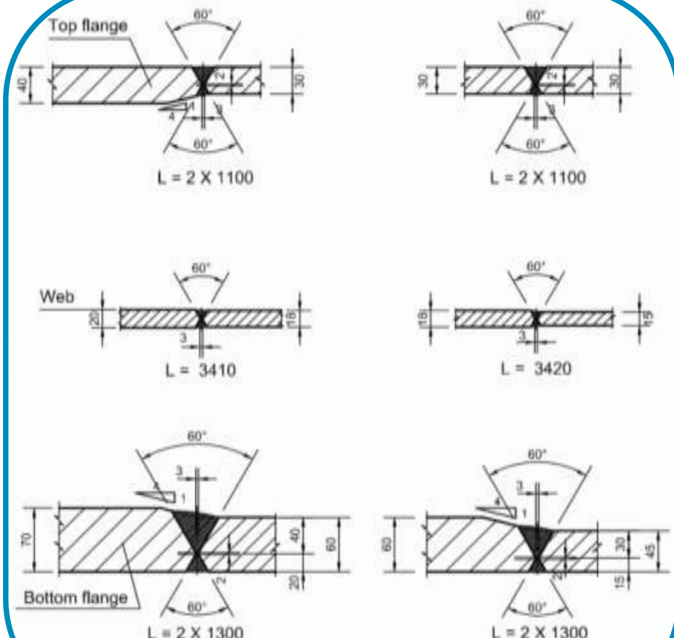
Obtained values	Design A S355 NL (kg/m² deck)	Design B S690 QL (kg/m² deck)	Variation (%)
Structural steel	**219**	**165**	**-25%**
Main girders	186	123	-34%
Cross girders + Stiffeners	33	42	+27%

The use of HSS thinner plates enables an **overall reduction of the structural steel weight of about 25%**

Bridge deck design – Volume of welding



a) Design A



b) Design B

The use of HSS thinner plates **reduces the volume of full penetration welding joints in 65%**, which is quite significant it terms of production benefits

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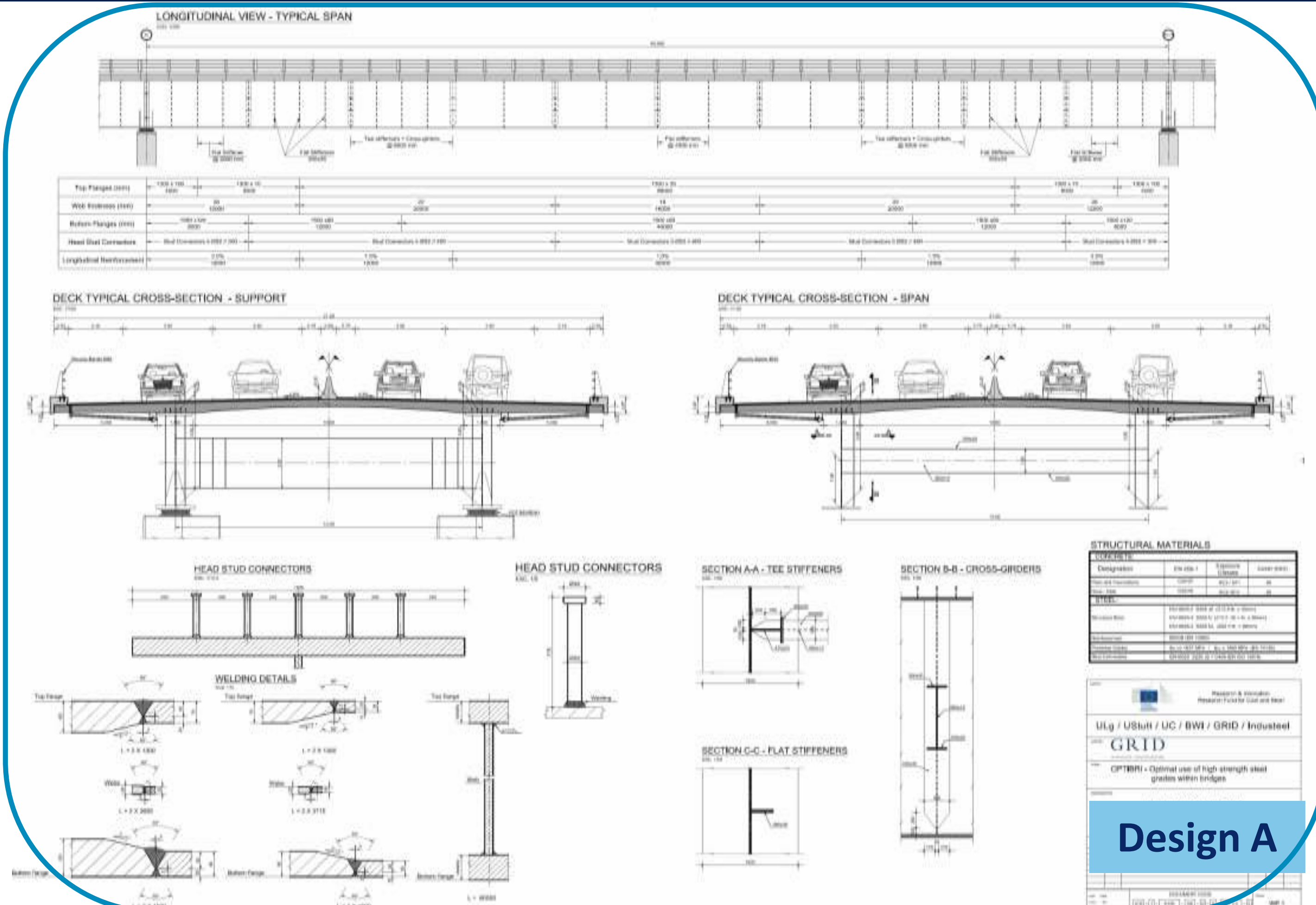
Bridge deck design – Drawings



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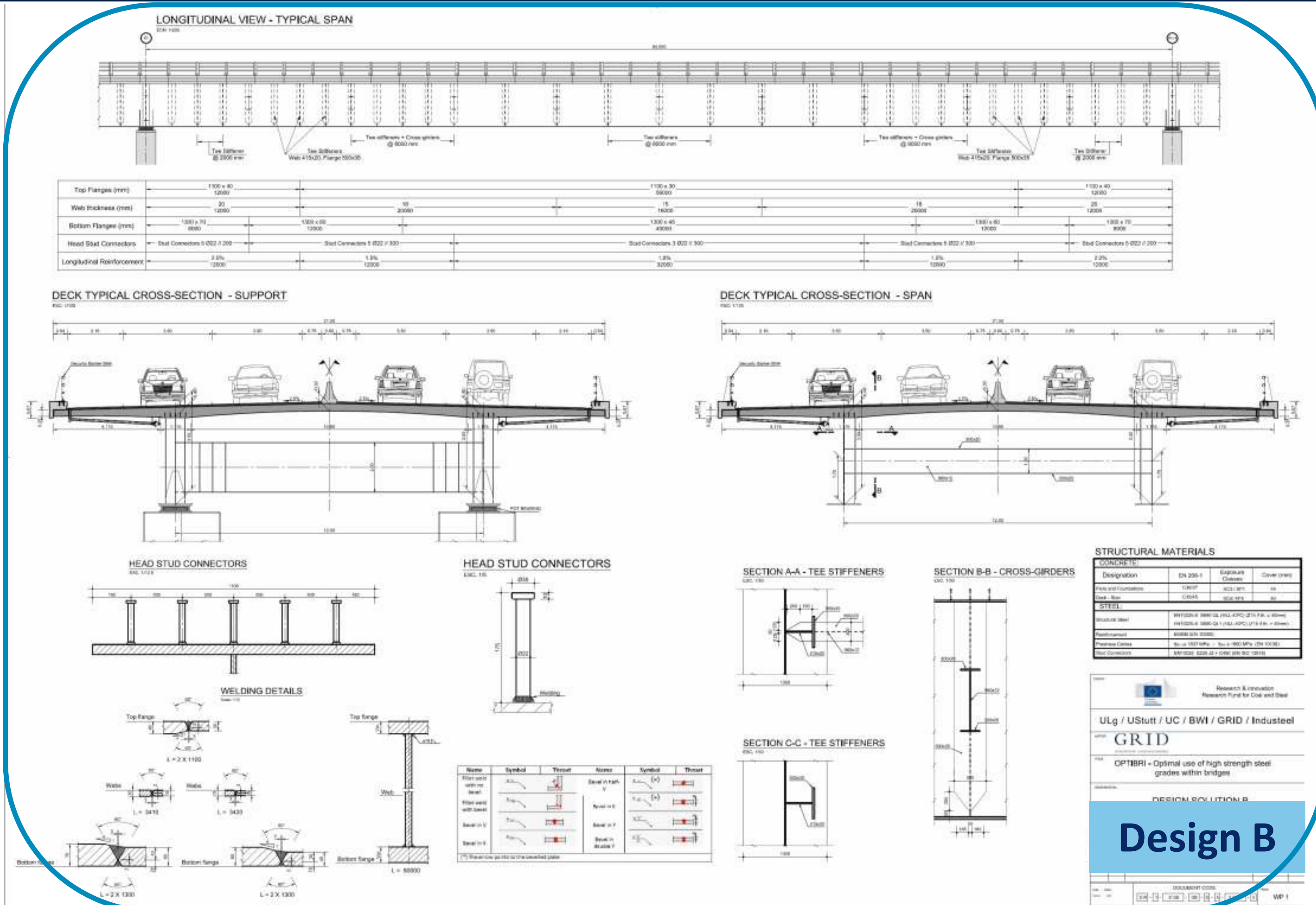
Bridge deck design – Drawings



Bridge deck design – Final considerations

Comparison between the two designs shows that:

- ✓ The use of HSS S690 QL enables a reduction of 25% of the steel weight compared to the standard plate girder deck in S355 NL;
- ✓ Using HSS the deck can be slender and with thinner plates, but more susceptible to local buckling phenomena;
- ✓ Longitudinal stiffeners can be used to increase the web resistance and profit from the use of HSS thinner webs;
- ✓ A substantial cut on the volume of full penetration welding is obtained by using thinner plates;
- ✓ Girders in HSS are much more prone to fatigue, that proves to be the main issue of the design together with buckling phenomena;
- ✓ The critical fatigue detail is the FAT80 at the welded joints between the bottom flange and the transverse stiffeners.

GRID

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Thank you for your attention!

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Research Fund for Coal & Steel

ArcelorMittal

Industeel High Strength Steel Plates

OptiBri-Workshop

„Design Guidelines for Optimal Use of HSS in Bridges“

Dr Ir Patrick Toussaint

What are High Strength Steel plates ?

Quenched and Tempered
High Strength Steel plates mostly for structural applications with minimum **yield strength of 690, 890, 960 and 1100 MPa**

Quenching and tempering provides the steel with high strength and ductility.
Quenching and tempering consists of a two-stage heat-treatment process.
Stage 1 includes hardening, in which the plate is austenitized to approximately 900°C and then quickly cooled. The material is water-quenched while somehow clamped to avoid warping.
Stage 2 consists of tempering the material to obtain the intended material properties.

High Strength Steel plates products portfolio

Industeel produces all HSS grades according to international norms

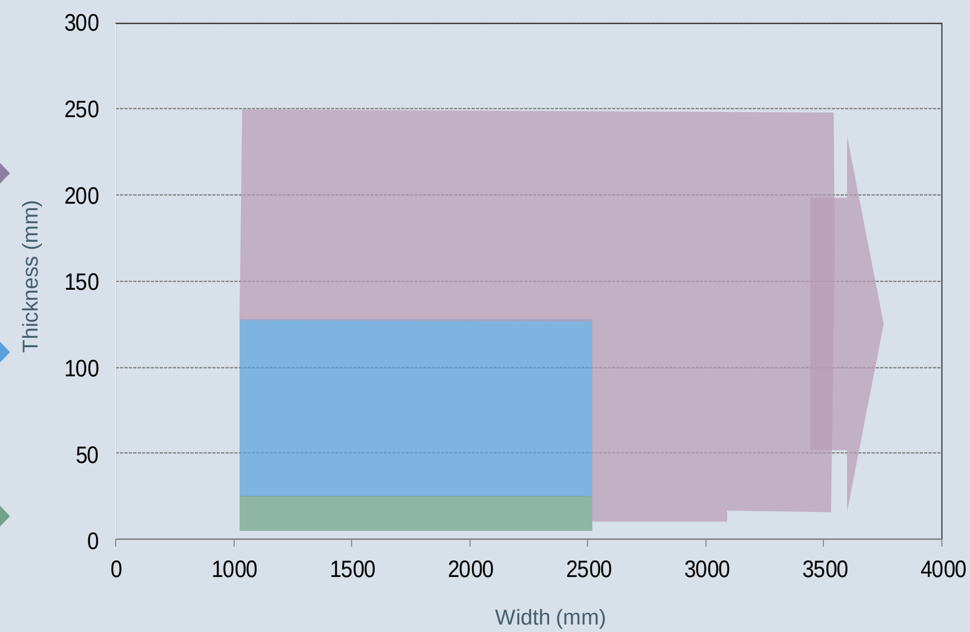
Industeel trademark	Standard
Amstrong® Ultra 690 SuperElso® 690 CR	S690Q - S690QL - S690QL1 according to **EN 10025-6** P690Q -P690QH -P690QL1 -P690QL2 according to **EN 10028-6** ASTM A514 Grades B, E, F, H, Q / ASTM A517 Grades B,E,Q… ASME SA-514 Grades B, E, F, H, Q / ASME SA-517 Grades B,E,Q… ABS, DNV-GL, LRS,… EQ70, VLF690, FH69,…
Amstrong® Ultra 890	S890Q - S890QL - S890QL1 according to **EN 10025-6**
Amstrong® Ultra 960	S960 Q - S960 QL according to **EN 10025-6**
Amstrong® Ultra 1100	Industeel specification, no international standard at this strength level

High Strength Steel plates typical size ranges

- Amstrong® Ultra 690
- A 514 Grades
- A 517 Grades

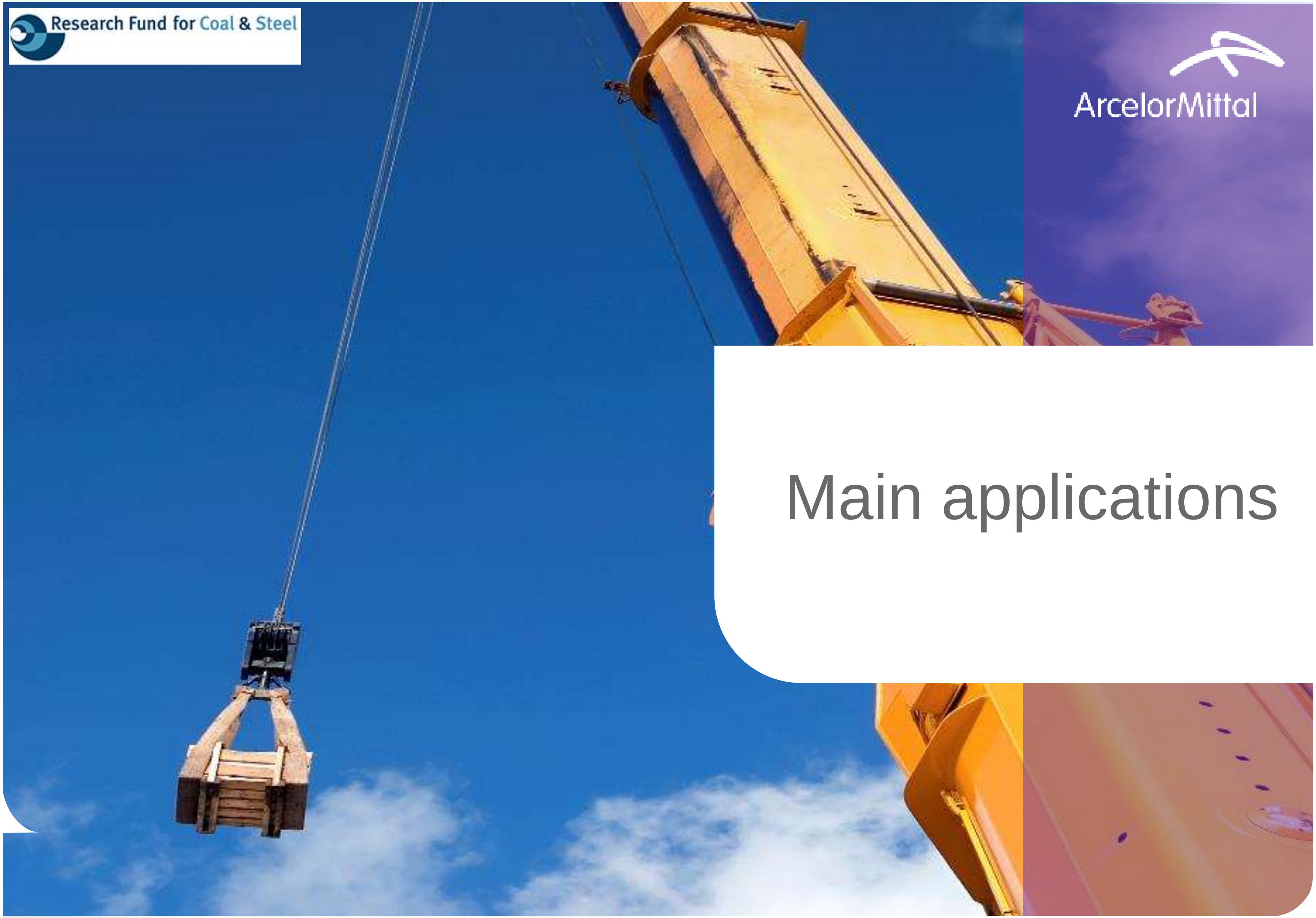
- Amstrong® Ultra 890
- Amstrong® Ultra 960

- Amstrong® Ultra 1100



Industeel has the largest range of sizes and thicknesses available nowadays on the market

- Thickness : 5 to 300mm
- Length : up to 17 metres
- Width : up to 4350mm
- Weight : up to 80 tonnes



Main applications

Yellow Goods & Green Goods

Lifting & Handling

Mobile crane

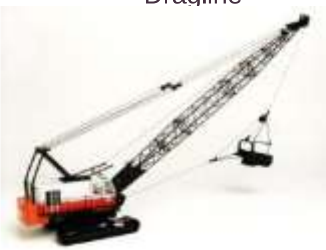
Telescopic Handler

Mining & Construction & Transport

Earth Moving Equipment, construction vehicles, quarrying and mining

Excavator

Dragline



Rigid hauler

Wheeled loader

Articulated dump truck

Hydraulic Excavator

Agricultural & Forestry

Tractors

Harvesters

Tractors

Feller Buncher

Skidder

Forwarder

Lifting & Handling

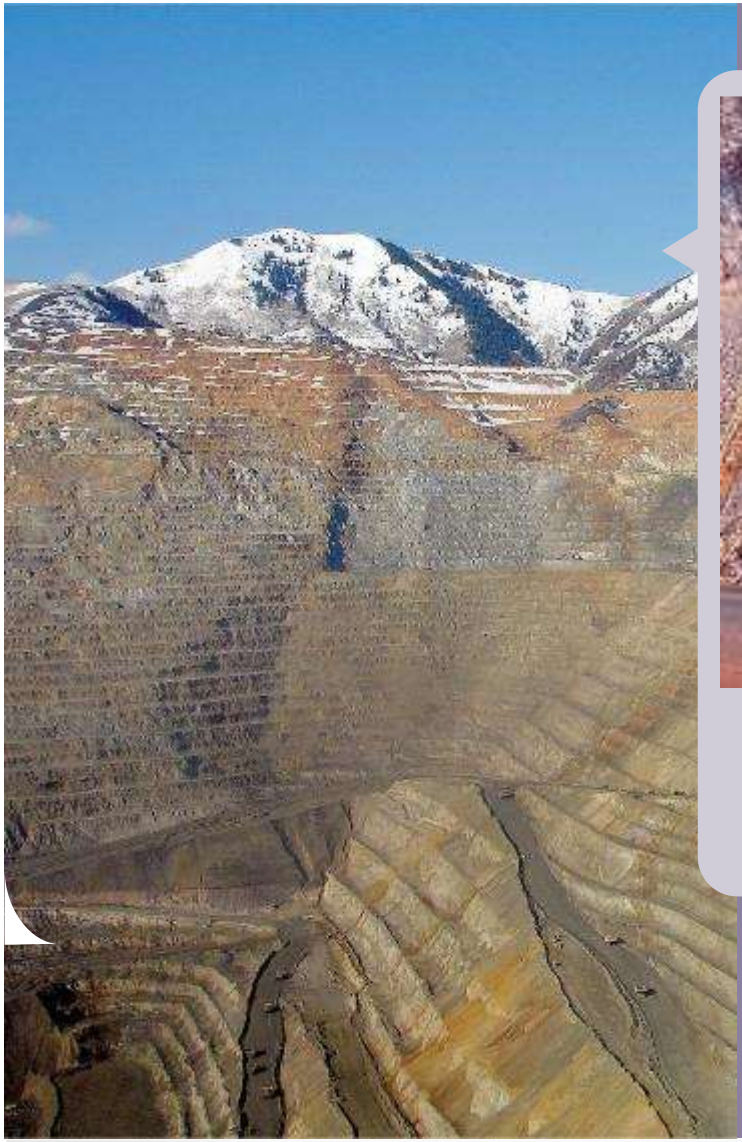
- **Mobile cranes**
- **Chassis**

  Amstrong® Ultra 960 : thickness = 8-60mm

  Amstrong® Ultra 1100 : thickness = 8-15mm

Lighter and more innovative structures

Mining & Construction & Transport



- **Dumpers**
- **Chassis - Canopy**

  Amstrong Ultra®690 : thickness = 8-50mm

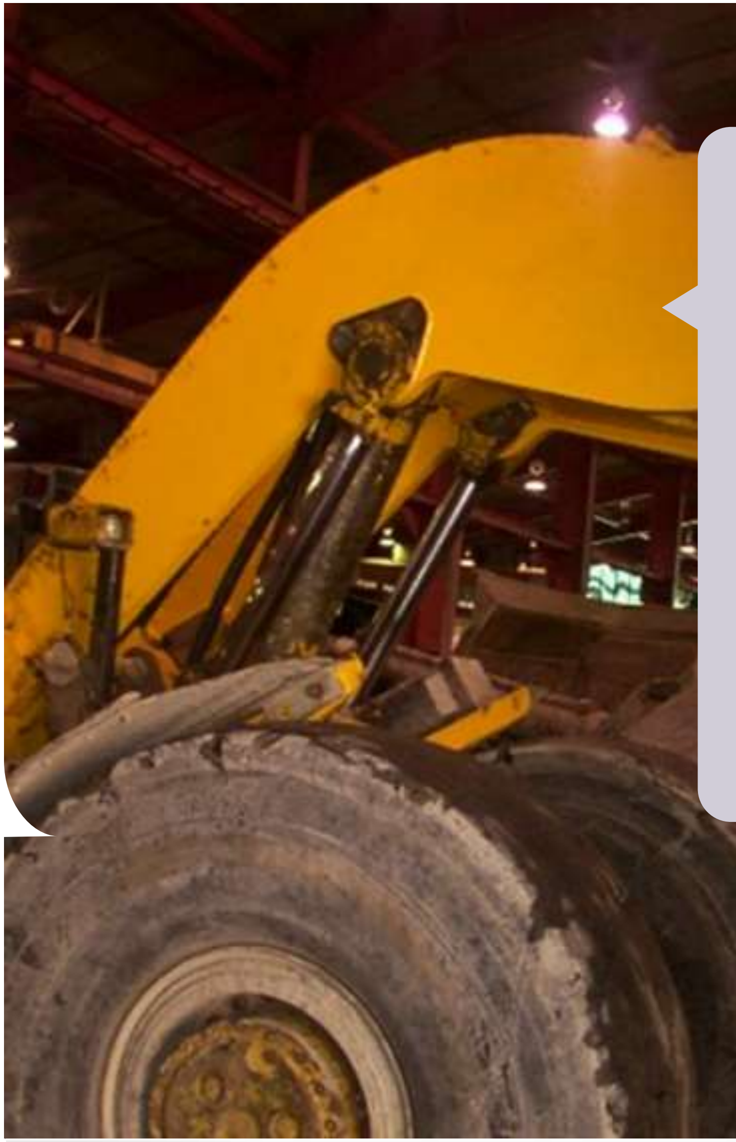
Reduced vehicle weight, reduced fuel consumption, heavier payload

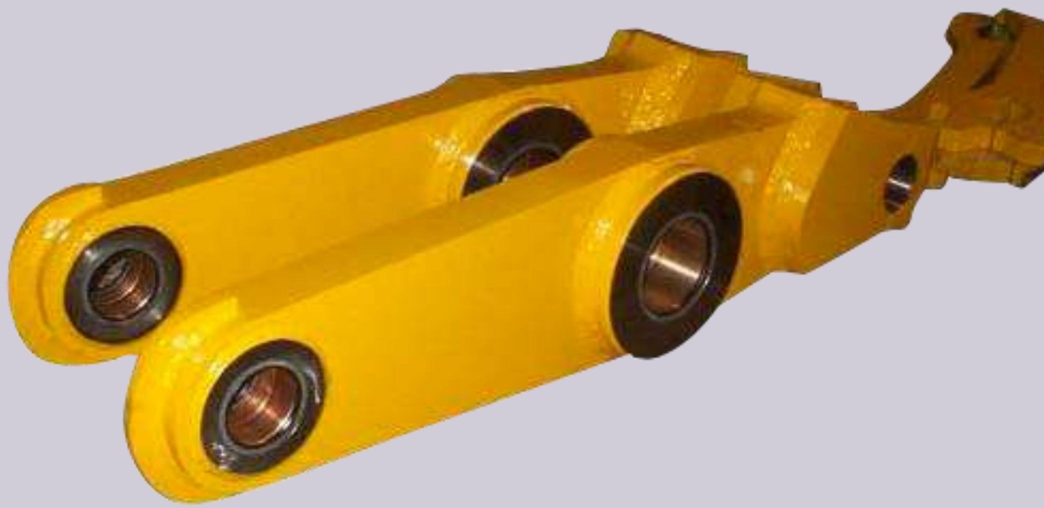
Mining & Construction & Transport

Public work (demolition) - Jaw crushers

More maneuverable cranes and tools

Mining & Construction & Transport





- **Lifting arm**
  Amstrong Ultra® 690 thickness = 60-80mm

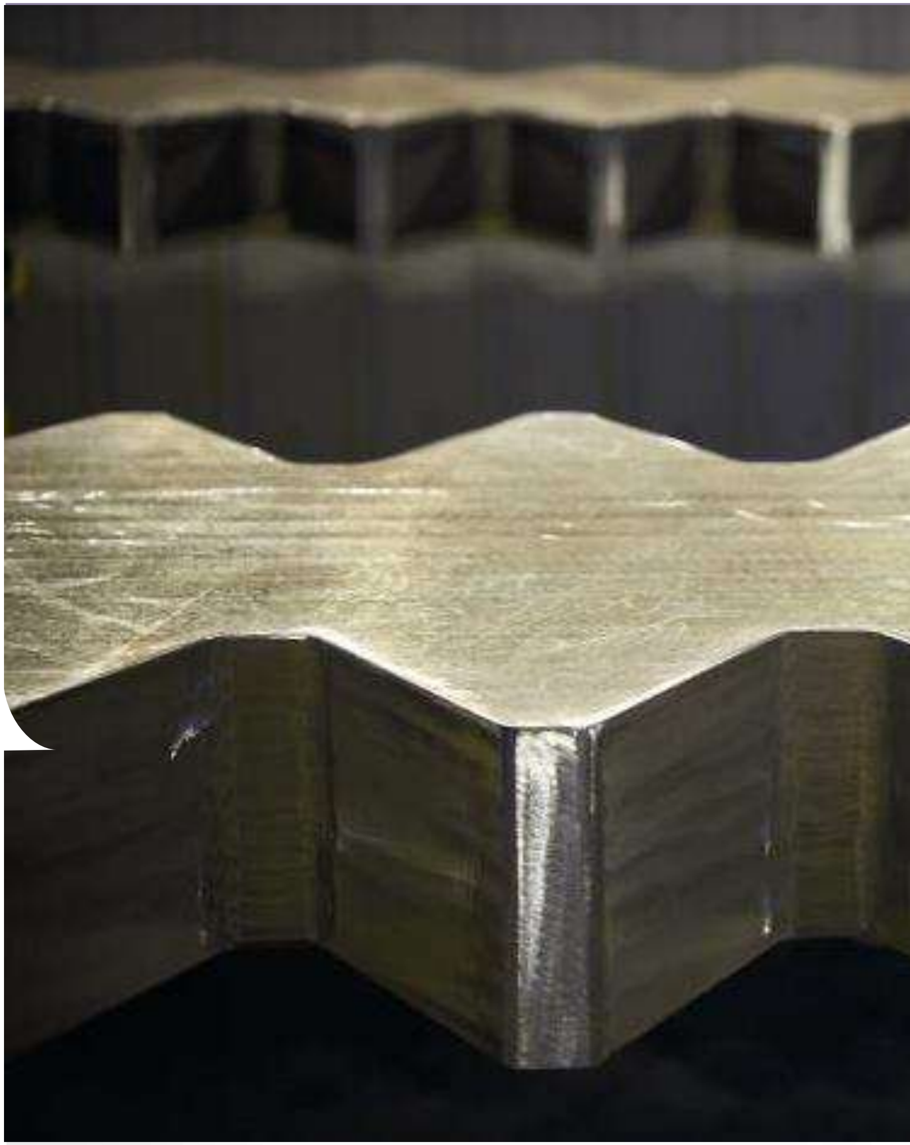
Ability to lift heavier loads than before

Jack-up

West Elara Jack Up
SuperElso ® 690 CR

Offshore Wind Mills

Elements for jack-up rigs



Racks

Length : 8 m up to 15,5 m
Thickness : 160 mm up to 210 mm
Width : 775 mm up to 1060 mm
Weight : up to 23 tonnes

Chords

Length : 4 m up to 10 m
Thickness : 80 mm up to 120 mm
Width : 380 mm up to 680 mm

Welded elements

Length : 8 m up to 24,5 m
Weight : 11 tonnes up to 70 tonnes

ArcelorMittal

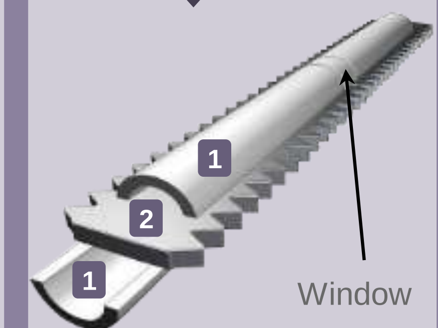
Welded elements for jack-up rigs

Transportation

Jacking system

Legs

Welded element



1 Chords 2 Racks

Research Fund for Coal & Steel

Offshore cranes

ArcelorMittal

Lifting capacity
up to 10 000 tonnes

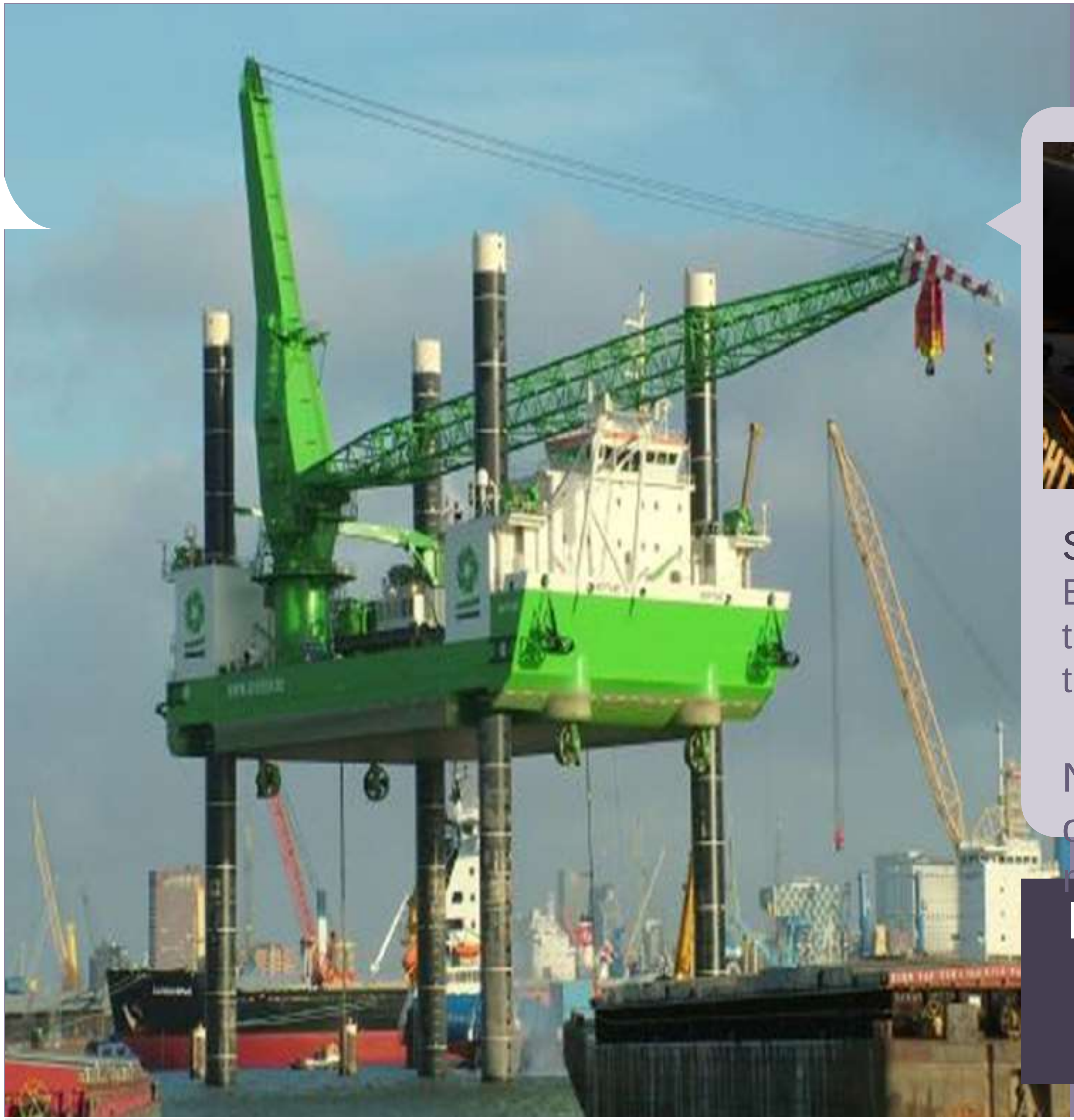
Structure

Amstrong Ultra® 690 (QL and QL1 qualities) tuned to the particular specifications

thickness = 10–100+mm

Technical solutions adapted to customer requirements

Offshore liftboats and spud poles



Spud poles

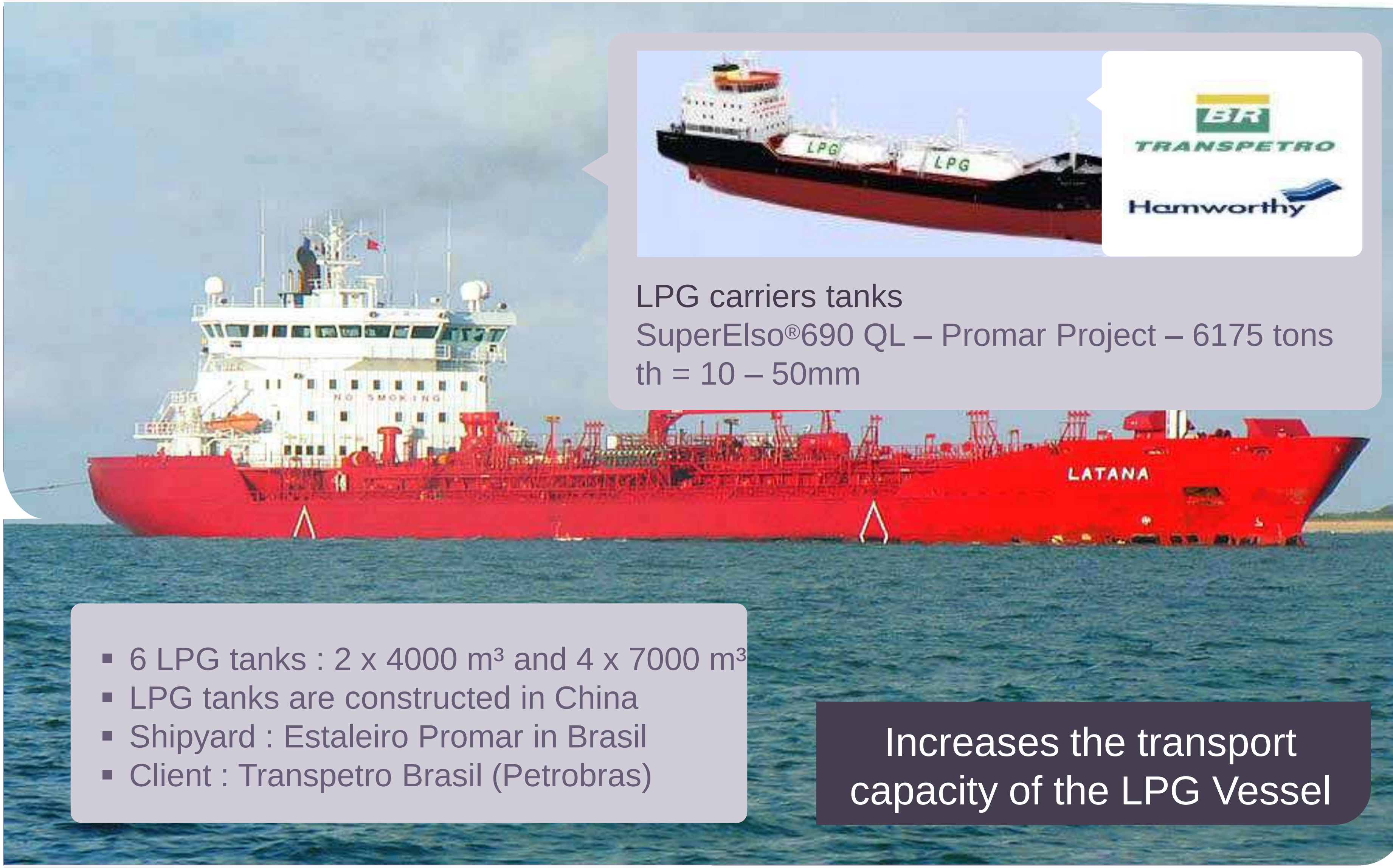
EQ70 (ABS) - Neptune project - 1590 tonnes

thickness = 58 mm

Neptune will work mainly in the offshore windfarm installation market

By increasing the strength of steel, the structural sections can be reduced

LPG transportation vessel



LPG carriers tanks
SuperElso®690 QL – Promar Project – 6175 tons
th = 10 – 50mm

- 6 LPG tanks : 2 x 4000 m³ and 4 x 7000 m³
- LPG tanks are constructed in China
- Shipyard : Estaleiro Promar in Brasil
- Client : Transpetro Brasil (Petrobras)

Increases the transport capacity of the LPG Vessel

Mechanical construction

- Architecture, bridges,
- Steel buildings
- Penstocks
- Chassis of industrial machines
- …

Reduction of wall thickness and weight with increasing strength of steel

Thank you for your attention!

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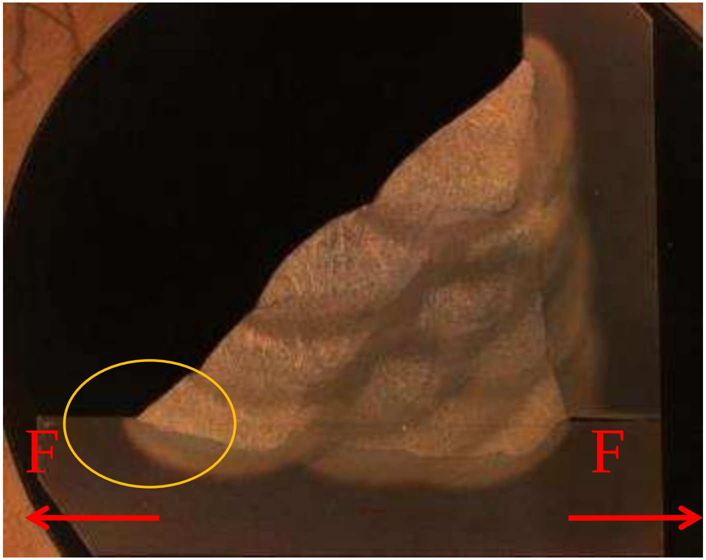
Welding and Post Weld Treatment of High Strenght Steel Joints

Thomas Baaten

- Introduction
- Welding Procedure Qualification
- Physical simulation of thermal history, characterisation, generation of samples
- Welding of high strength steel
- Post Weld Treatment Qualification
  - Parameters
  - Imperfections caused by HFMI
  - Indentation map
  - Finite element model of PIT proces
  - Is Post Weld Treatment Qualification needed?
- Conclusions

Introduction



- Geometry
- Residual **tensile** stress
- Possible softening of the HAZ (f.e. S700MC, aluminium, …)

Welding Procedure Qualification

- Goal: make a weld method which fulfils EN 15614-1 requirements

- The mechanical and metallurgical properties of the weld metal and the heat affected zone are determined by:
  - Pre heat temperature
  - Welding parameters

- Tests needed for fillets welds:
  - Visual examination
  - Dye penetrant/magnetic examination
  - Cross section (looking for metallurgical changes in the HAZ as well)
  - Hardness measurements
  - Additional charpy impacts tests

Welding Procedure Qualification – pre heating

- Pre heating is done to avoid brittle zones (sensitive for hydrogen cracking)
- 4/5 factors are taking into account:
  - Hydrogen content of the filler metal
  - Heat-input of the welding process
  - Chemical composition of the base metal
  - Material thickness
  - Limitions/recommendations from fabricant

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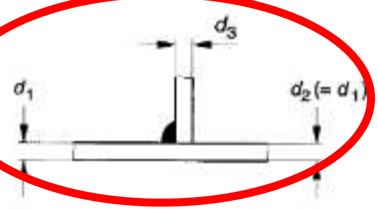
Welding Procedure Qualification – pre heating

- Solid ER100-SG welding wire -> scale **D**

Table C.2 — Hydrogen scales

Diffusable hydrogen content ml/100 g of deposited metal			Hydrogen scale
	>	15	A
10	≤	15	B
5	≤	10	C
3	≤	5	D
	≤	3	E

- Heat-input: 1,5 kJ/mm
- Base material: CEV max. : 0,67
- Combined thickness: 10+2*15 = **40 mm** and 10+2*40 mm= **90 mm**



For simultaneously deposited directly opposed twin fillet welds, combined thickness $= \frac{1}{2}(d_1 + d_2 + d_3)$

$$\text{Combined thickness} = d_1 + d_2 + d_3$$

Welding Procedure Qualification – pre heating

- 40°C for 15 mm base plate and 90°C for 40 mm base plate

Figure C.2 k)

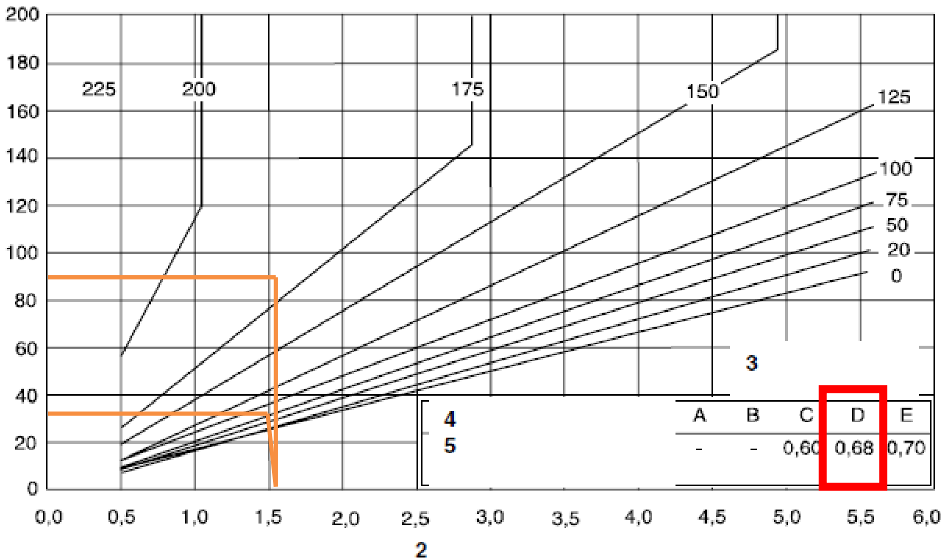


Figure C.2 l)

Key

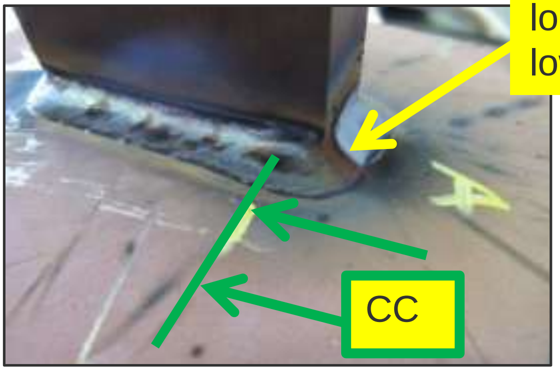
1 Combined thickness, mm
2 Heat input, kJ/mm
3 Minimum preheating temperature, ° C

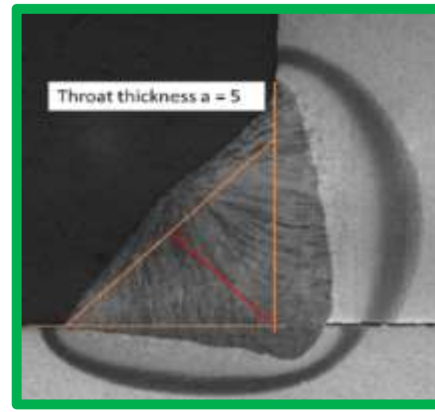
4 Scale
5 To be used for carbon equivalent not exceeding

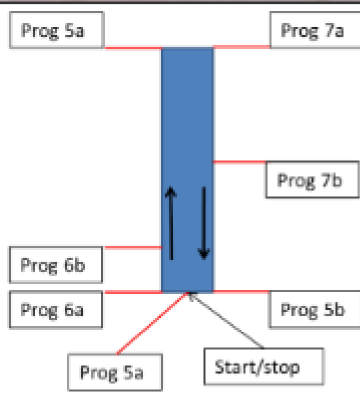
Figure C.2 — Conditions for welding steels with defined carbon equivalents

Welding Procedure Qualification

- Weld Procedure Qualification of welding case A and H was done according to EN ISO 15614-1







weldprog	wire (m/min)	arc length corr. (%)	dynamic or pulse corr.	mode	welding speed m/min
5a	6	10	4	std	0,18
5b	6	10	4	std	0,24
6a	9,5	7	3	puls	0,3
6b	9,5	7	3	puls	0,24
7a	9	5	4	puls	0,3
7b	9	5	4	puls	0,25
	end time: te (s)	2	end curr: Ie (%)	50	
Parameters in source	slope Si1 (s)	0.5	slope Si2 (s)	0.9	

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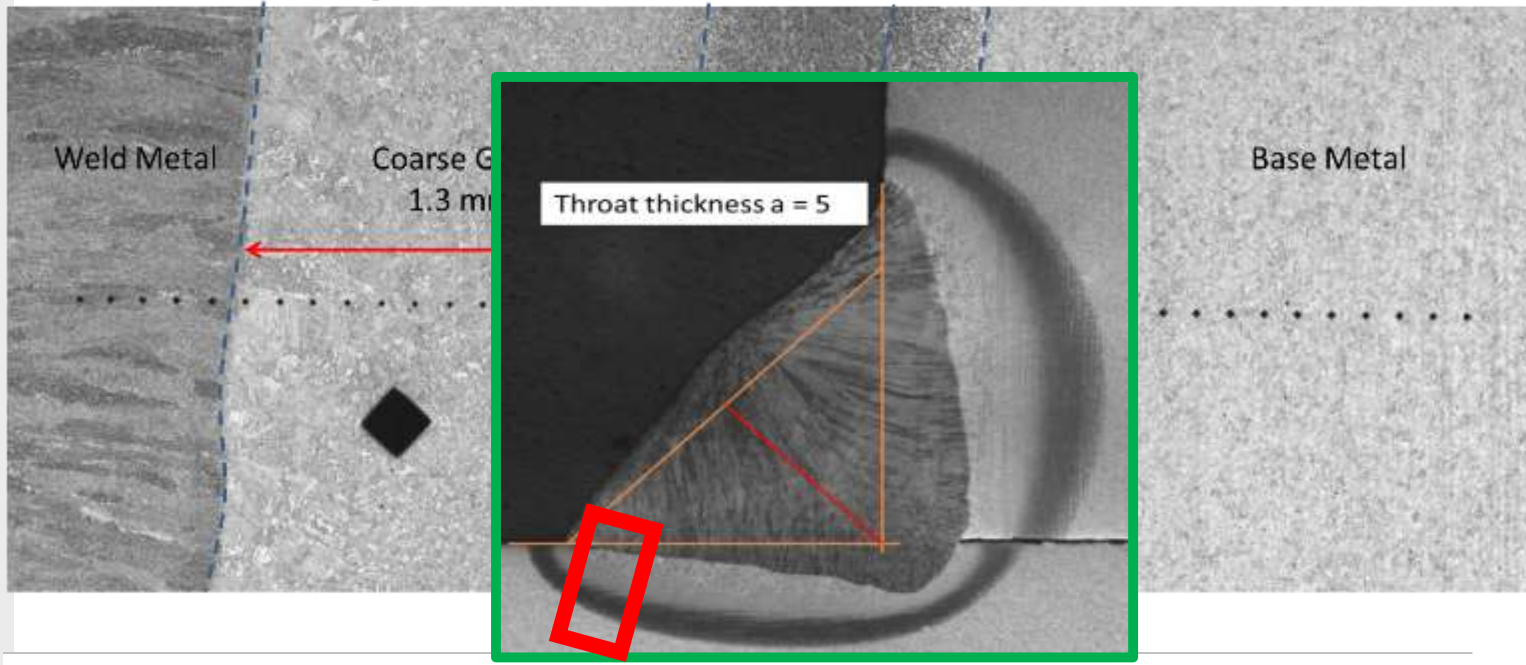
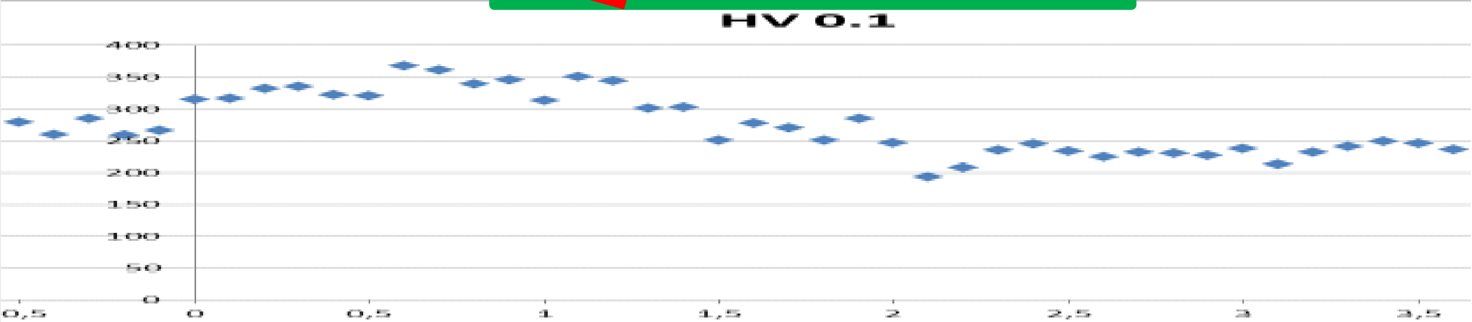
HAZ 1 OP_A_optim_00a





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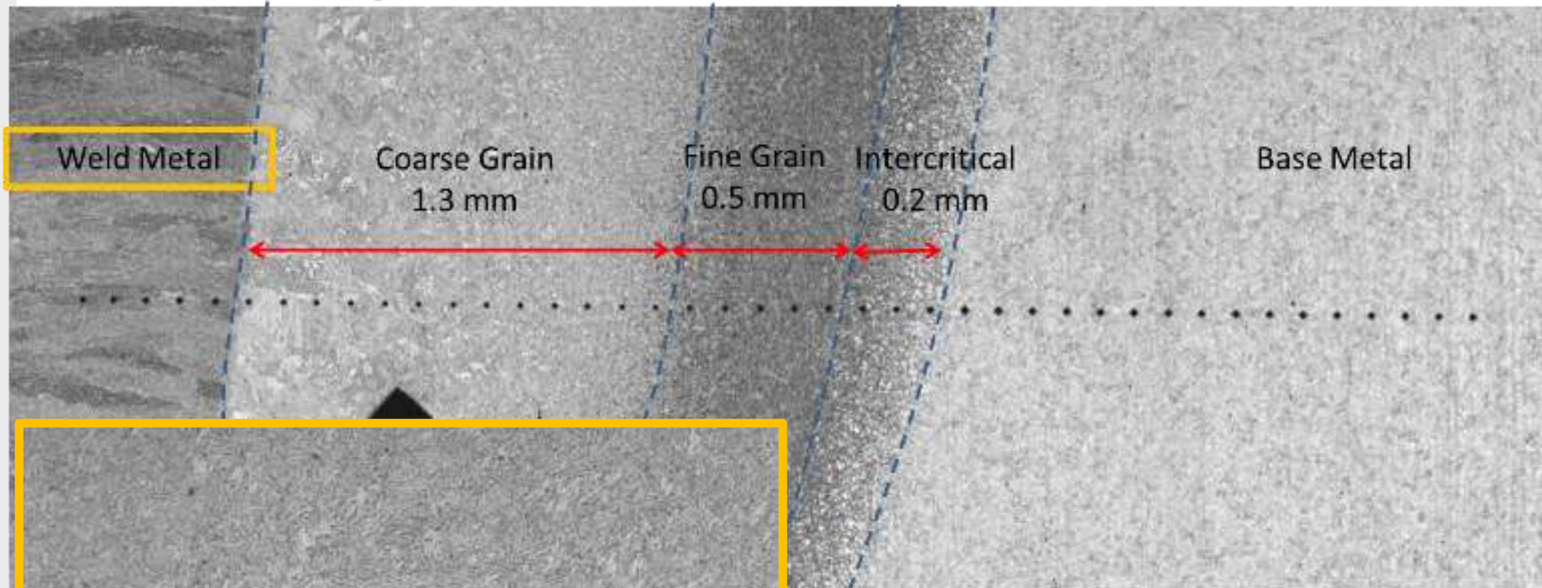
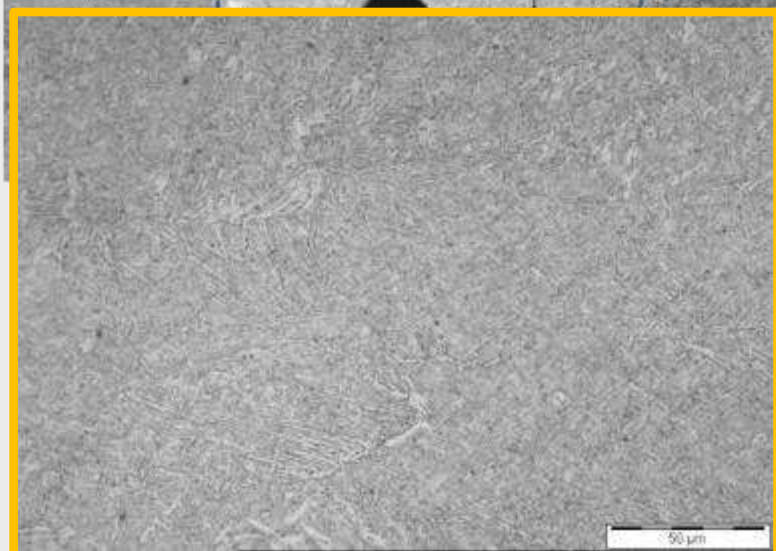
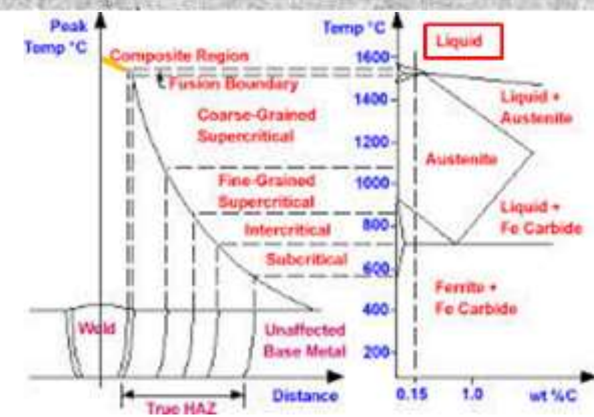
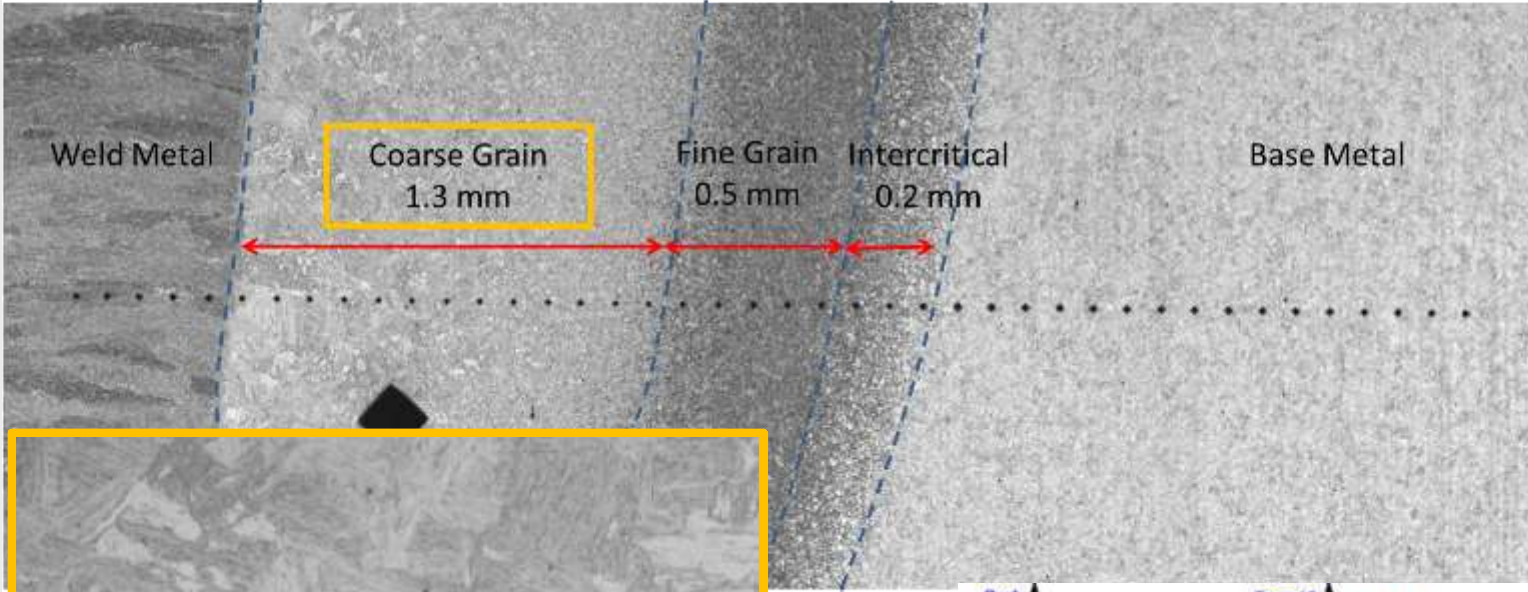
HAZ 1 OP_A_optim_00a

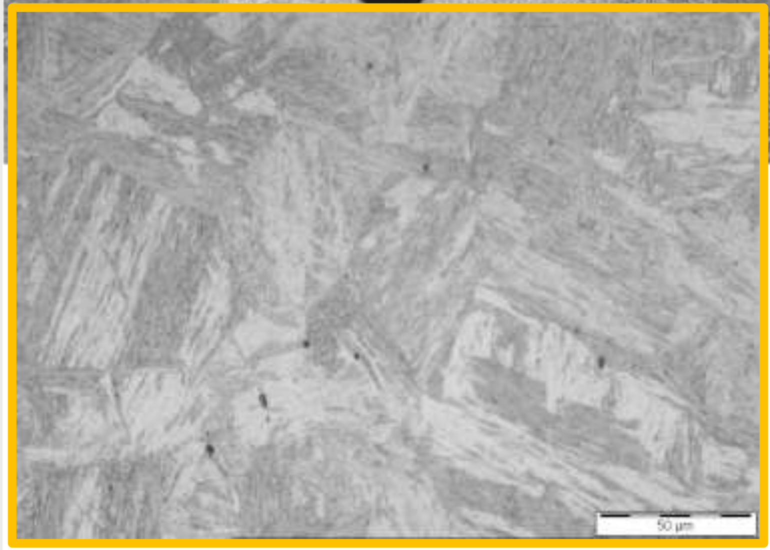


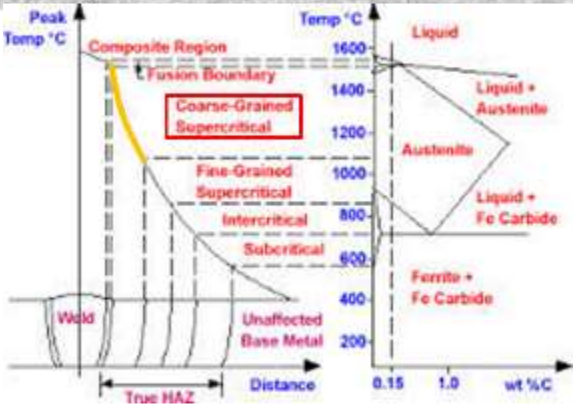




HAZ 1 OP_A_optim_00a







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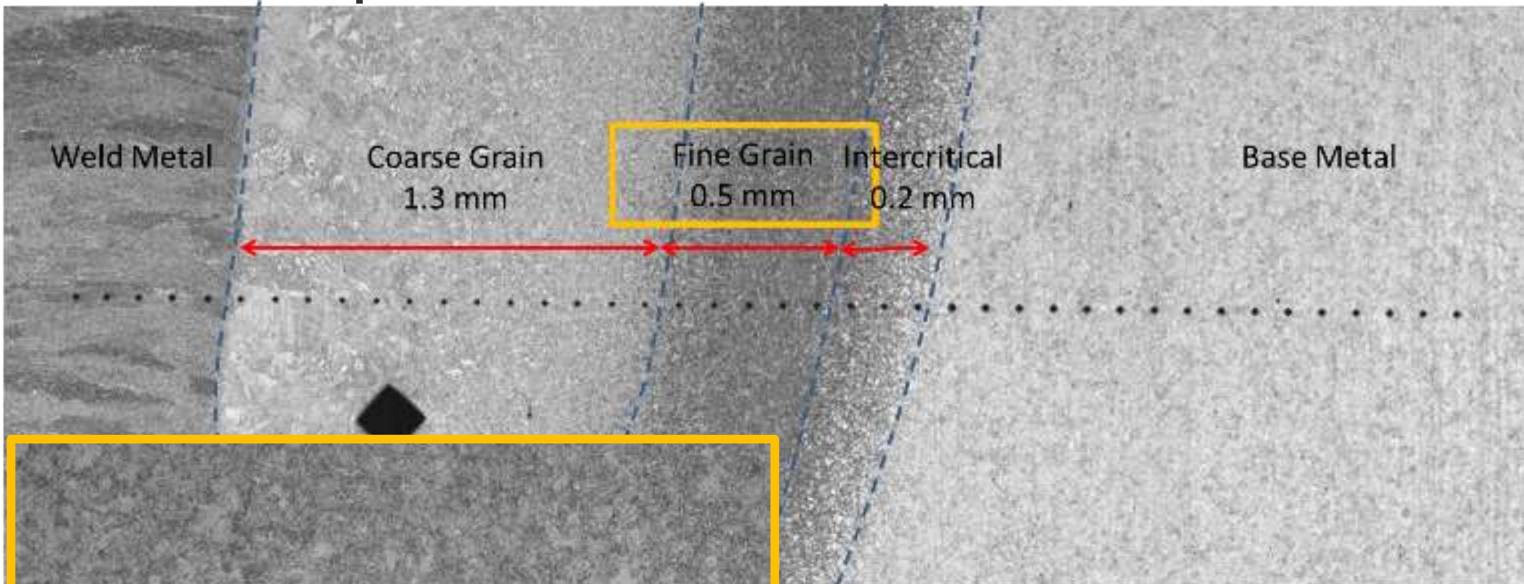
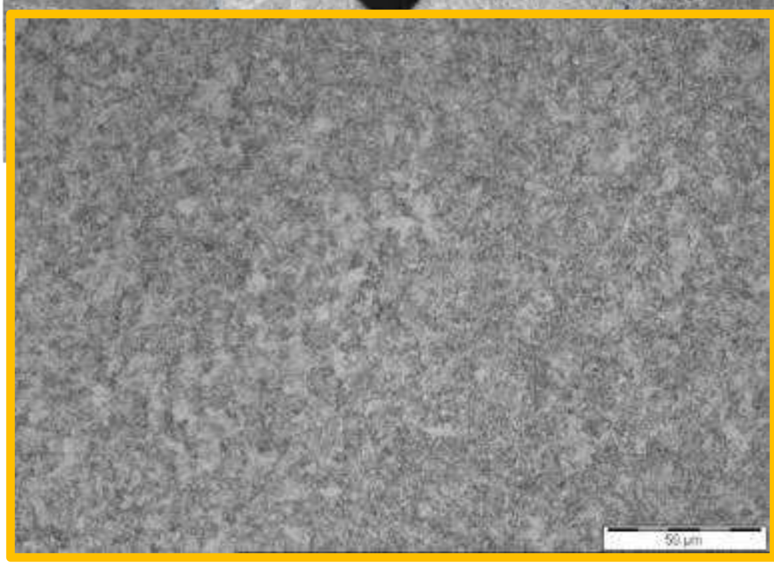
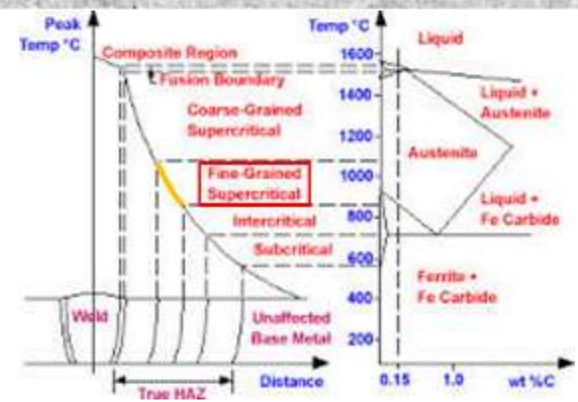
HAZ 1 OP_A_optim_00a







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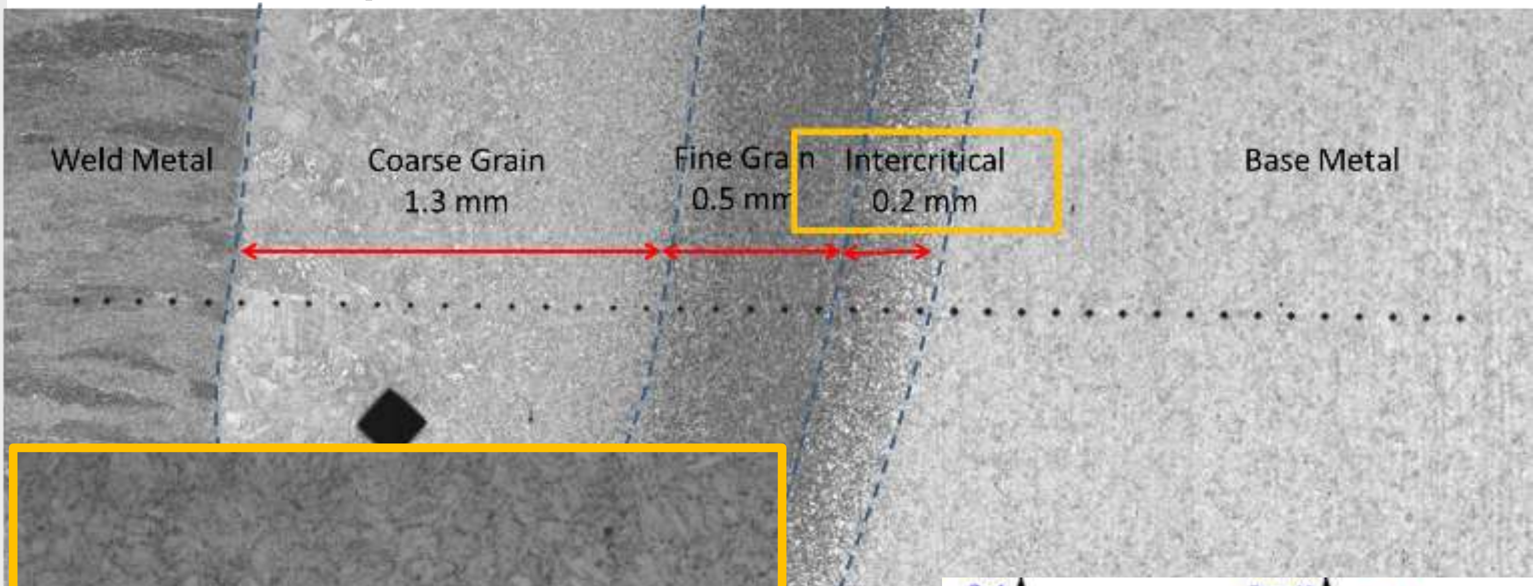
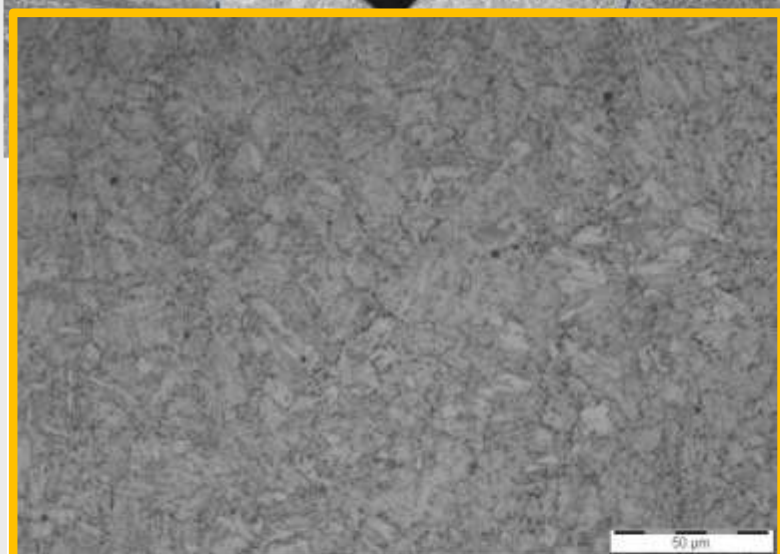
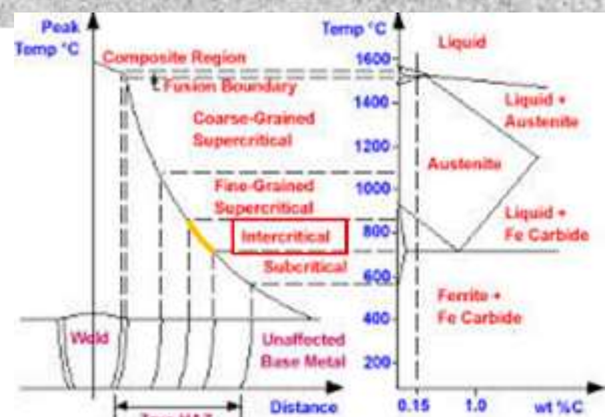
HAZ 1 OP_A_optim_00a







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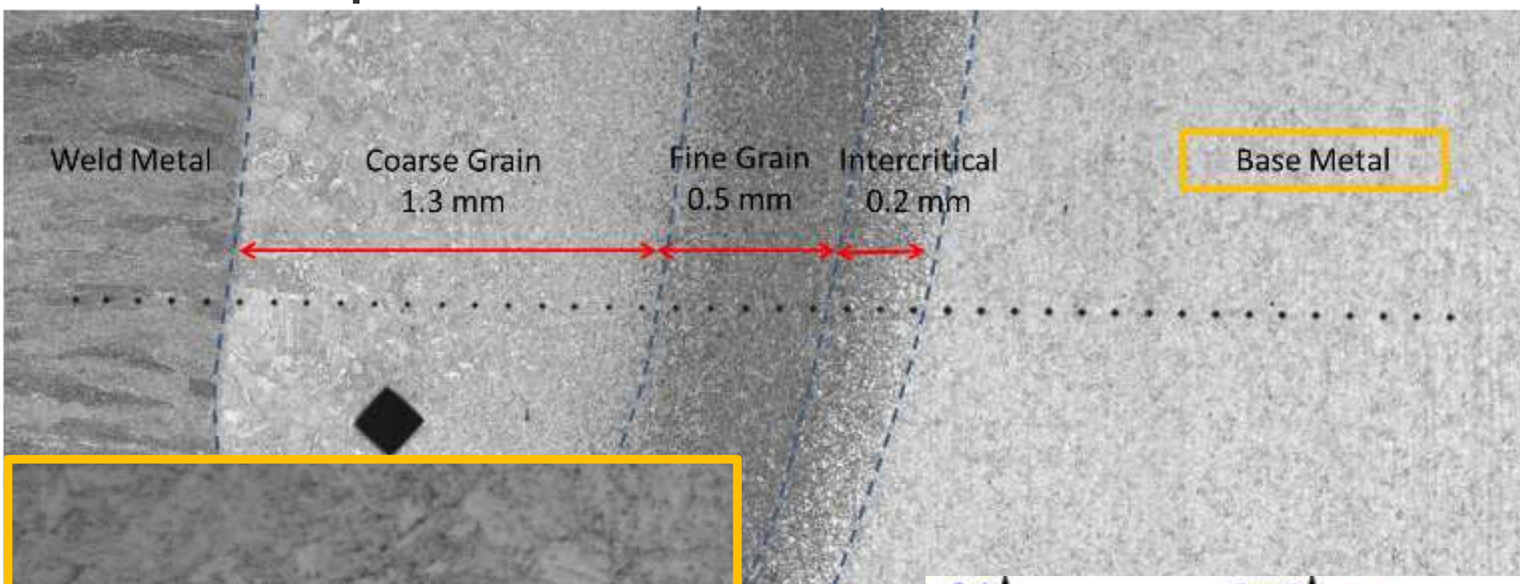
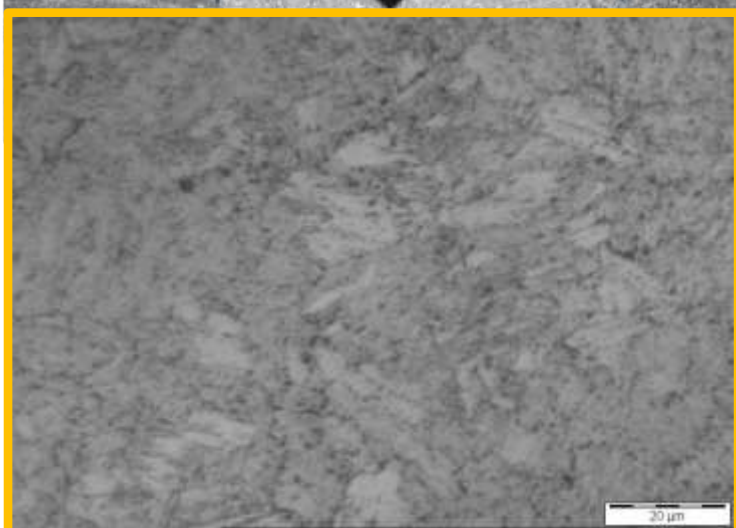
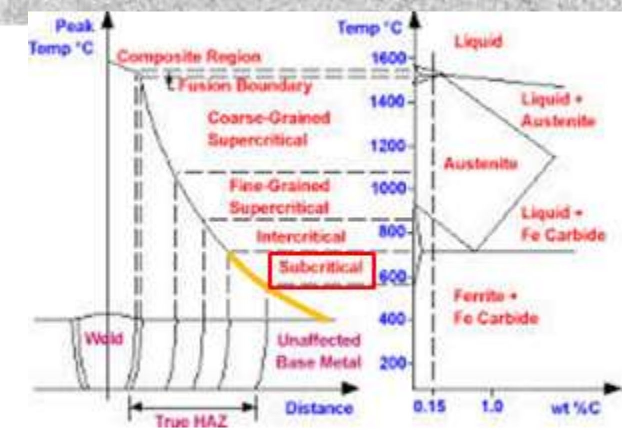
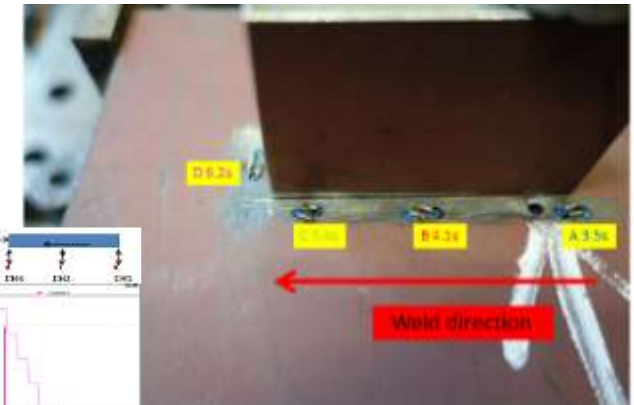
HAZ 1 OP_A_optim_00a

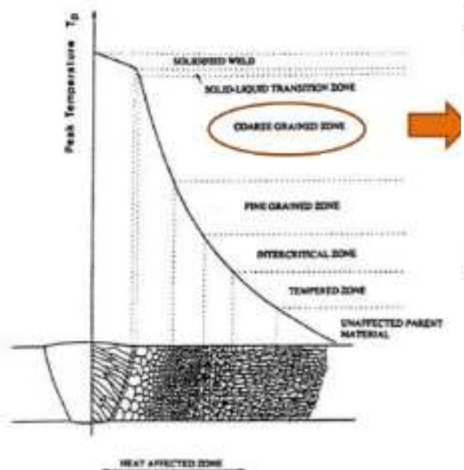


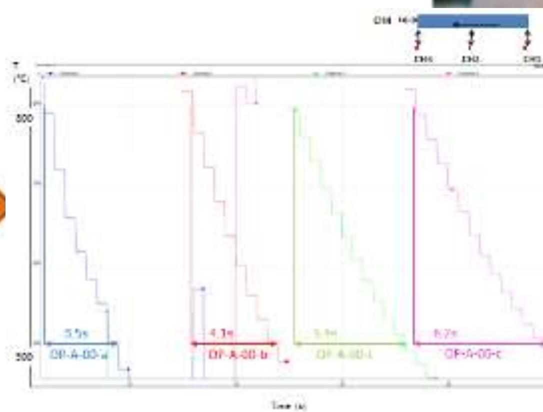


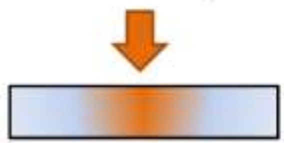


- input for physical weld simulations of thermal history of HAZ 1 and 2 (representing SC A and SC H)
- Δt 8/5 HAZ 1 (welding case A) = 4.4s
- Δt 8/5 HAZ 2 (welding case H) = 7 s



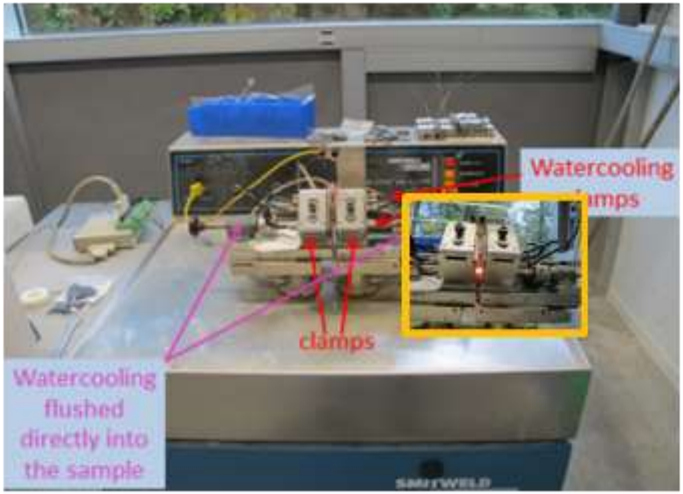




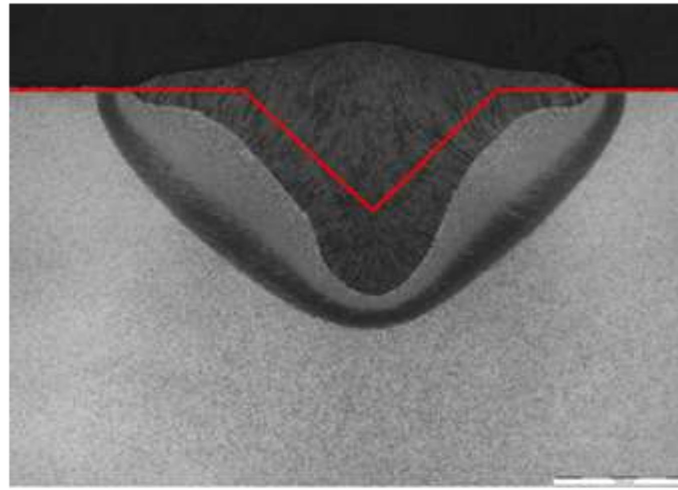


Physical simulation of thermal history, characterisation, generation of samples

Physical simulation of thermal history



Welding Weld Metal samples



Machining test samples of HAZ and WM:

- Tensile test samples
- Bauchinger shear samples
- Fatigue test samples

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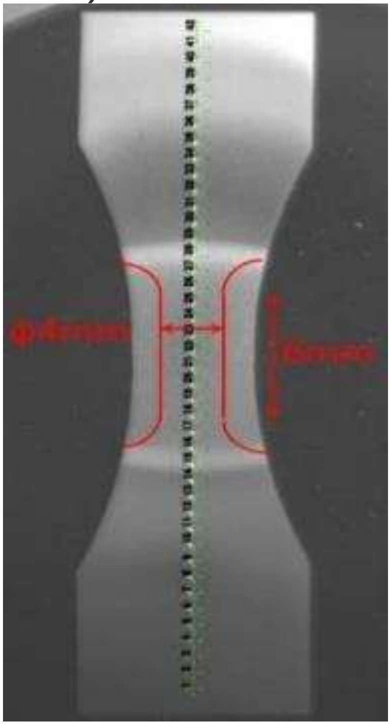
Physical simulation of thermal history, characterisation, generation of samples

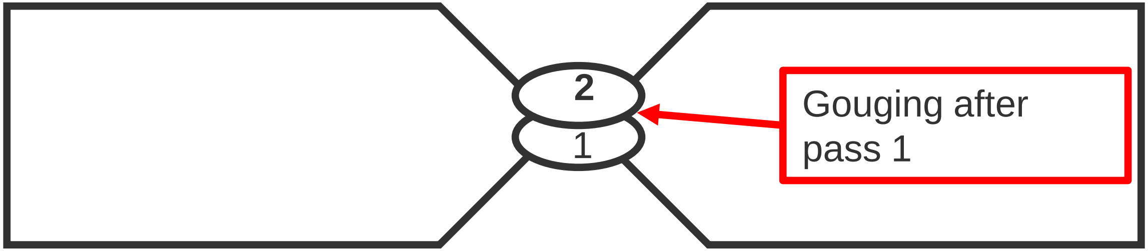
- Tensile tests on weld simulation test samples of HAZ1 and HAZ2 (resp. welding case A and H).



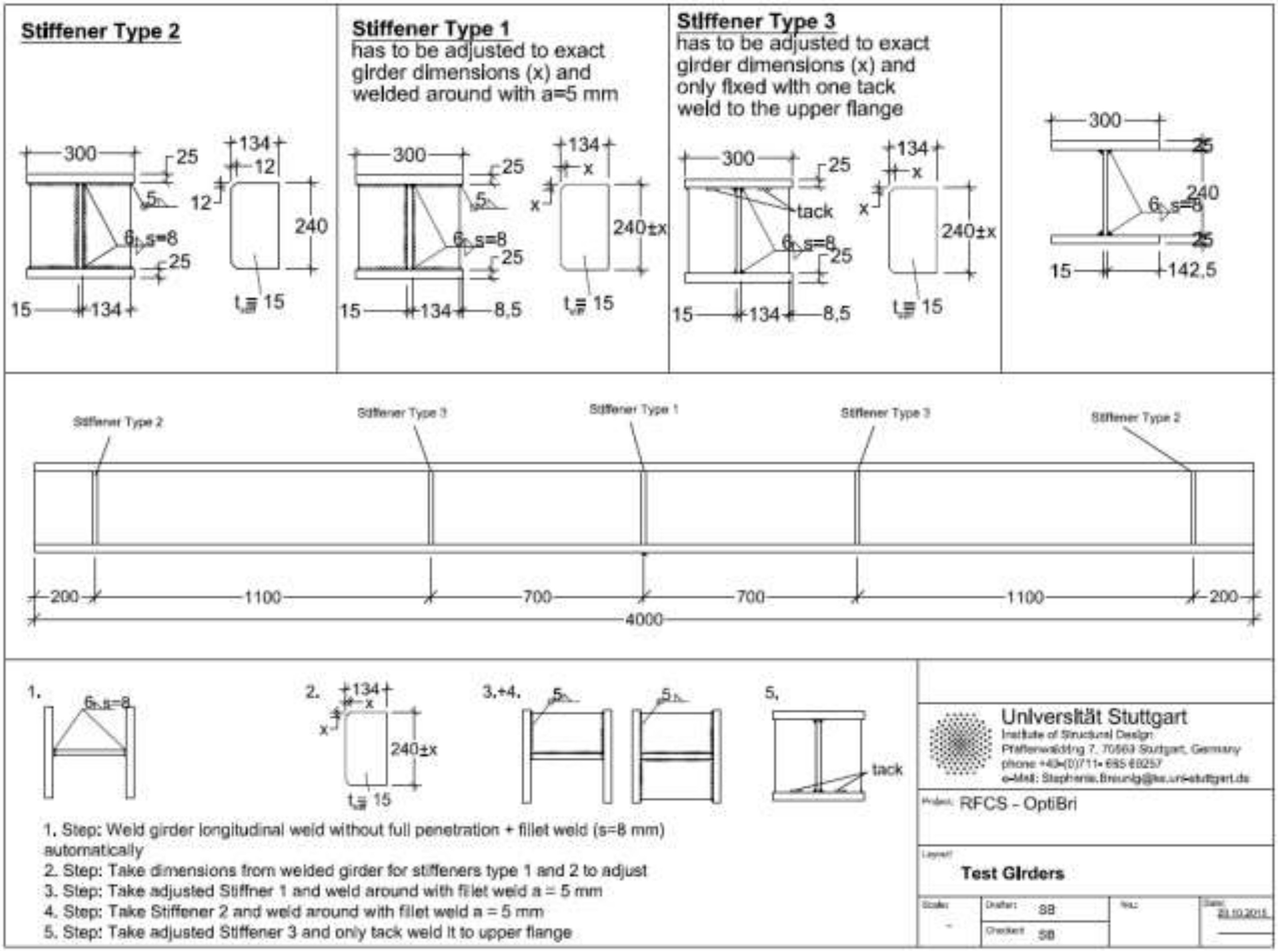
Welding of HSS

- Quenched and tempered (Q&T)
  - S690QL
  - Thickness up to 200 mm
  - Low heat input can cause excessive hardness
  - Often preheating is needed
  - High heat input can cause softening
  - Centre of X joints is critical point in WPQ
  - Generaly good to weld

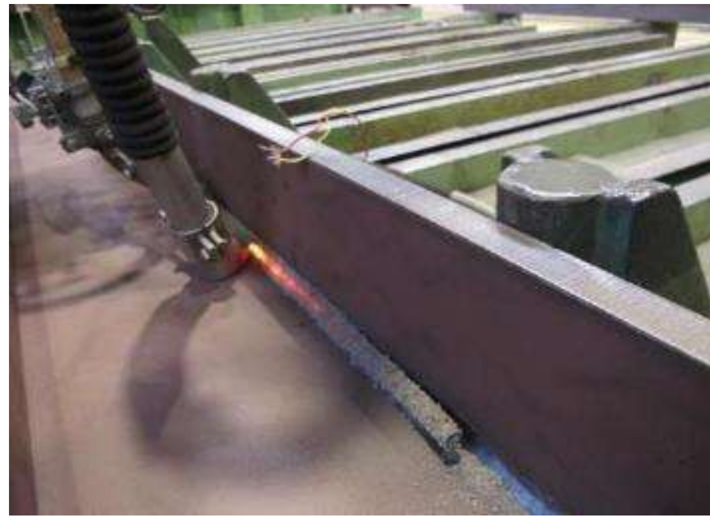
$$Heat\ Input = k.\frac{U.I}{v}\ \text{[J/mm]}$$



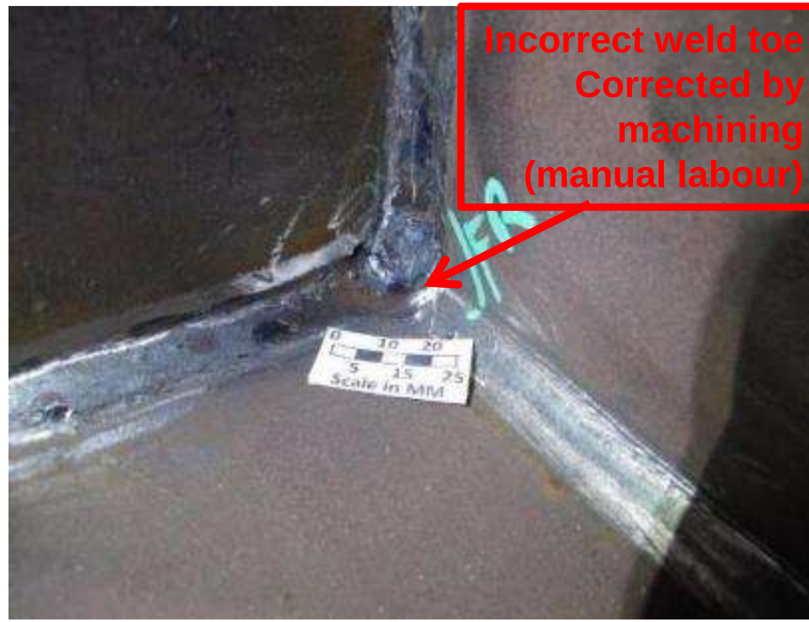
Welding of HSS



Welding of HSS



Welding of HSS



Welding of HSS

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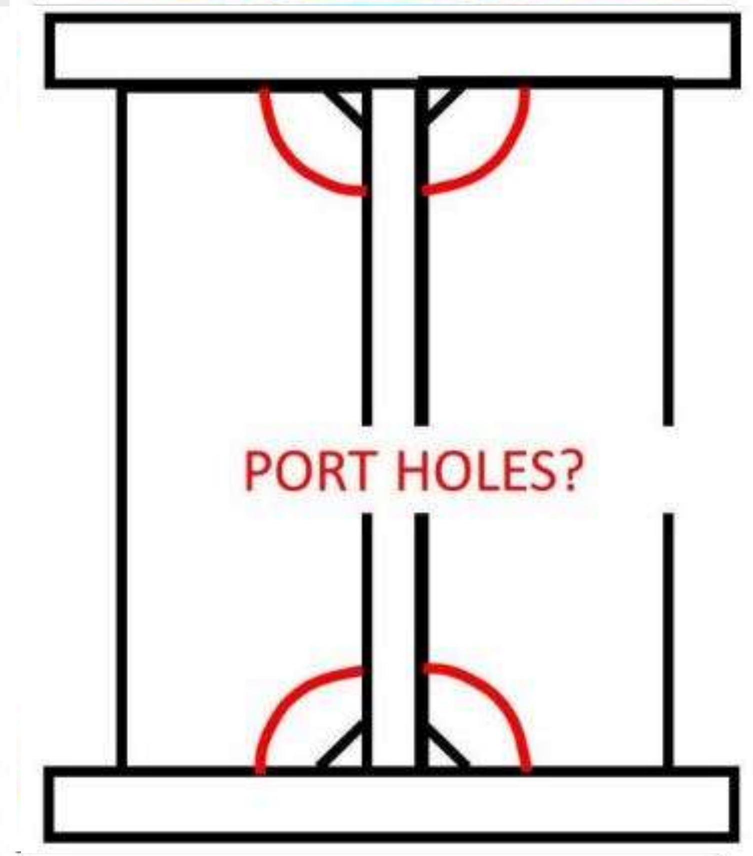


Welding of HSS

- Cutting of the web and flanges was already done in steel factory
- Cutting stiffeners: 1,5h
- Mounting, tackwelding: 12h
- SAW welding + 100°C preheating: 14h
- Flame straightening: 3h
- MAG welding of stiffeners: 24h
- Grinding edges: 3,5h
- Visual examination + MPI central stiffeners: 4h
- Machining incorrect weld toe in corners: 8h
- Extra visual examination + MPI after repair: 2h
- Project management: 18h
- PIT treatment: 2h

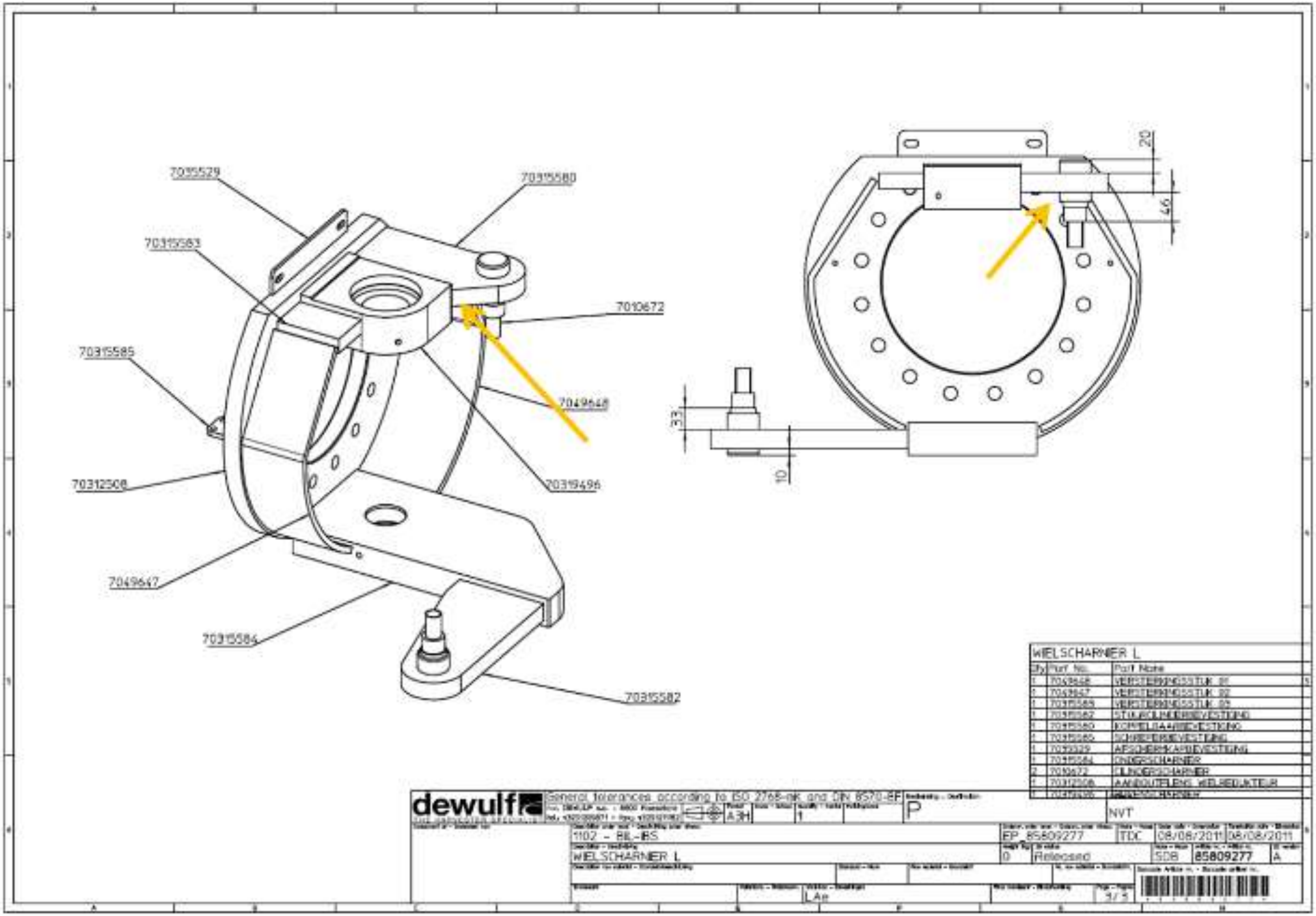
Ref. : Optistraight

- 2 hours of PIT treatment in a total of 92 hours labour

Welding of HSS: Considerations/arguments to skip preheating and concerns

- recommendations for preheating of EN 1011-2 (2010), mainly based on hydrogen cracking, are conservative.
- At the side of the **filler metal** fabricants, seamless flux cored wires were developed in the 90ties (as a better alternative for folded FCW). Entrance of hydrogen is limited massively since 1990.
- At the side of the **steel makers** CE equivalents and %C of HSS can be kept low. The maximum Vickers hardness of the steel used in Optibri is estimated 422 HV10, which is far below the maximum limit of 450HV10 of the EN ISO 15614-1 standard for welding procedure qualification.
- Recent experience of BWI: no hydrogen damage analysis
- Cost saving

Post Weld Treatment of HSS



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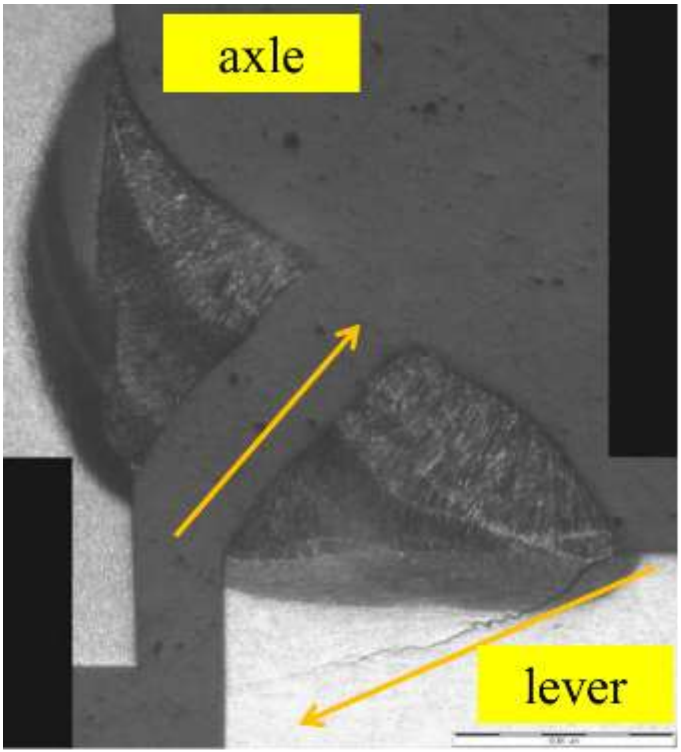
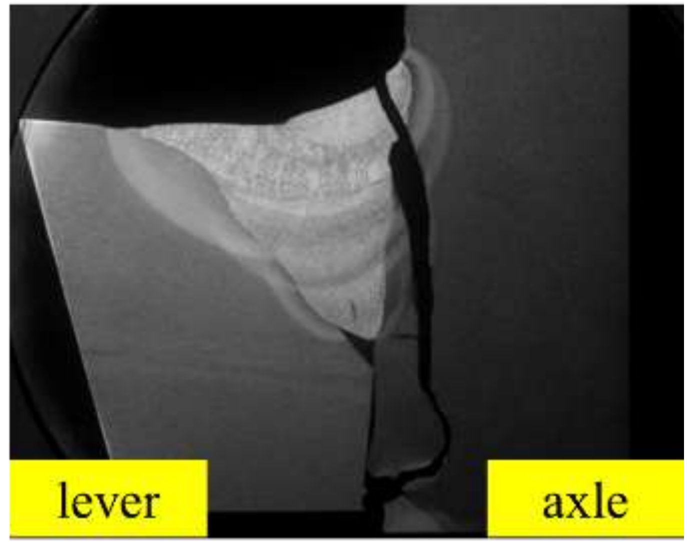
Post Weld Treatment of HSS

- S355 original (not treated)



- S690QL PIT treated



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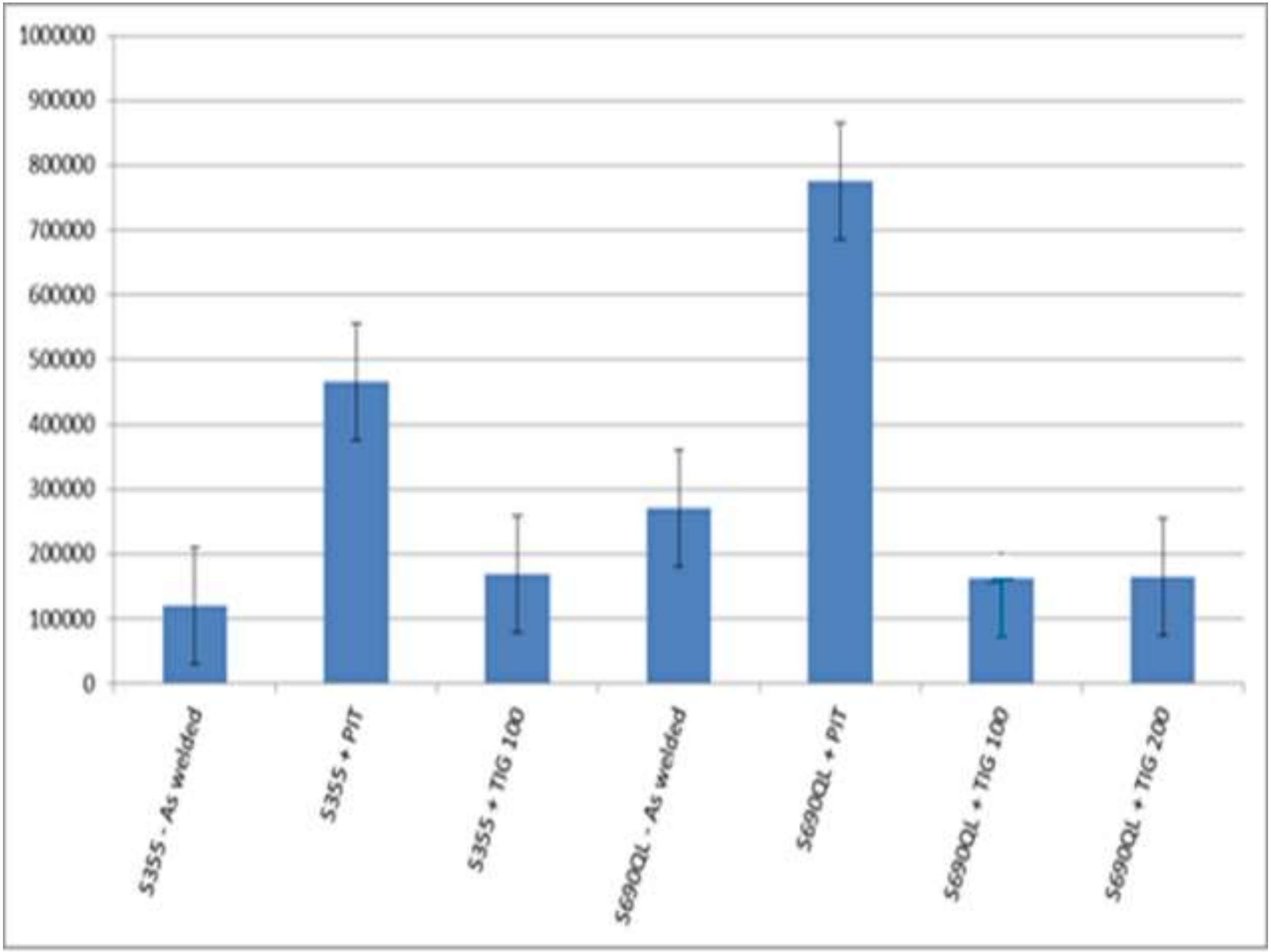
Post Weld Treatment of HSS



Post Weld Treatment qualification

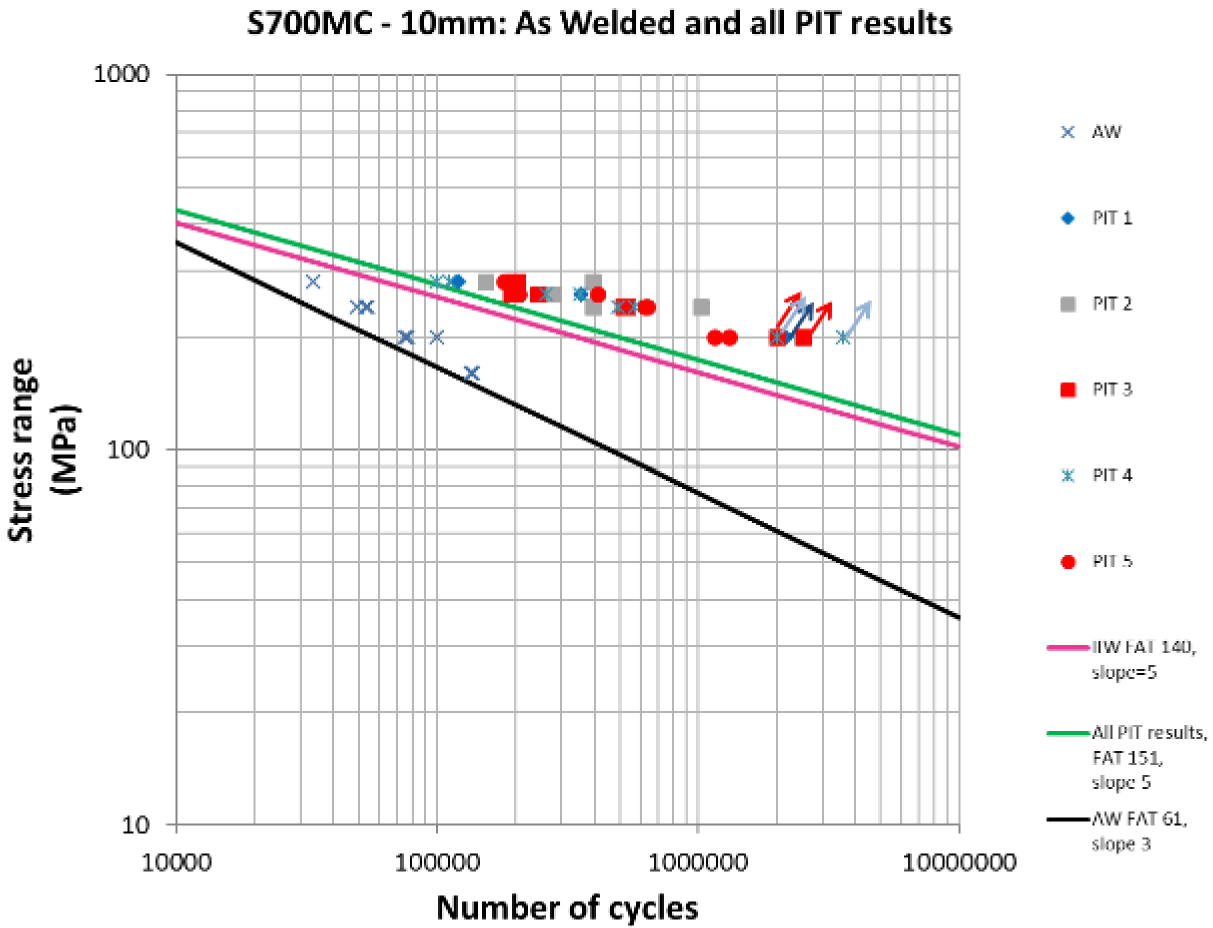
- Available finished fatigue tests samples with longitudonal stiffeners in S420MC and S700MC grades – thickness range 5 - 10 mm
- 5 PIT-parameters were applied (variations in pressure, diameter, frequency) (only 2 parameters were Ok for Pitec, based on their experience.)

Condition	PIT parameters
1	6 bar, 90 Hz, r = 2 mm
2	6 bar, 90 Hz, r = 4 mm
3	6 bar, 90 Hz, r = 1,5 mm
4	6 bar, 120 Hz, r = 2 mm
5	4 bar, 90 Hz, r = 2 mm

- Examination for Post Weld Treatment qualification:
  - Metallographic examination
  - Dimensional check
  - Hardness measurements

Research Fund for Coal & Steel

Post Weld Treatment qualification



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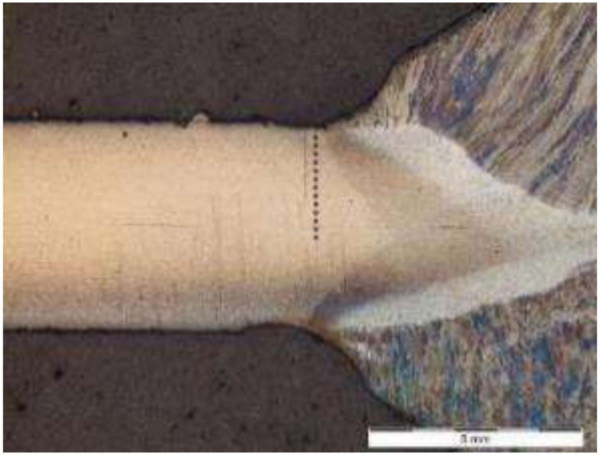
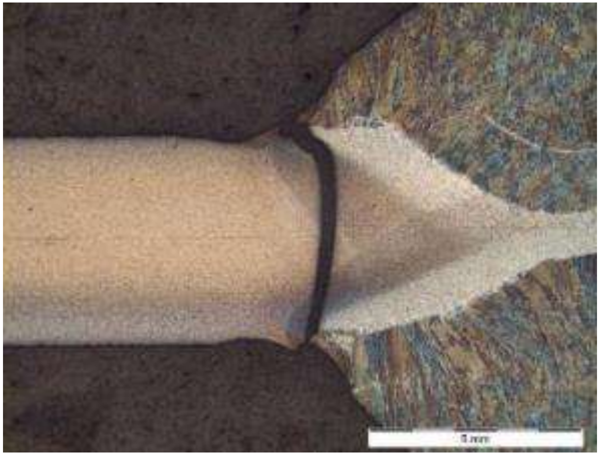
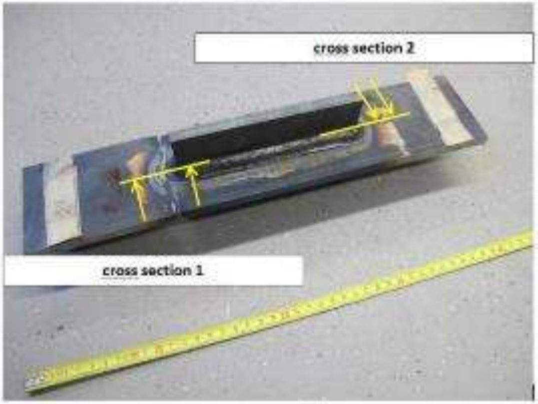
Post weld treatment qualification

- Metallographic examination example S700MC-5-087-PIT1



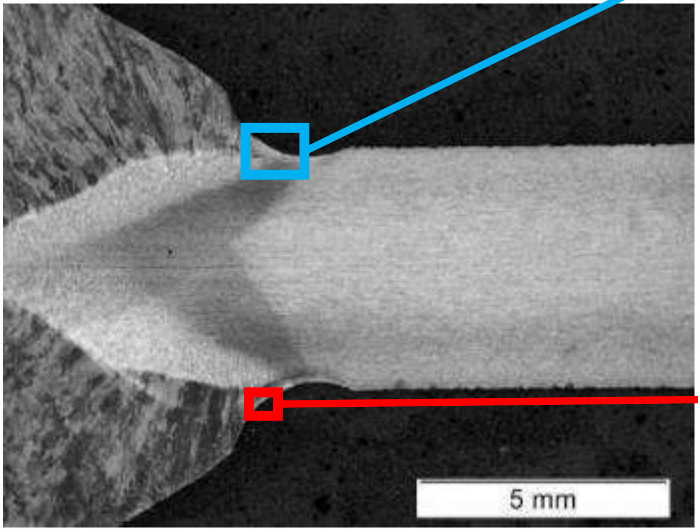


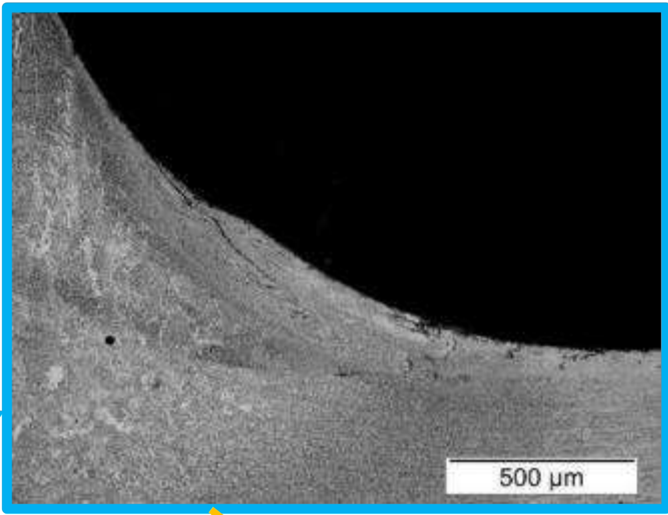


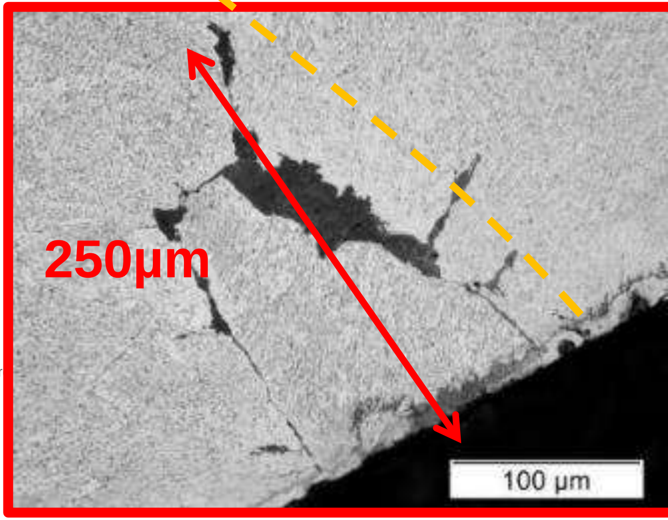
Post weld treatment qualification

3 imperfections were found on 21 samples:

1. Spread out
2. Inclusion of oxides
3. Sharp notch







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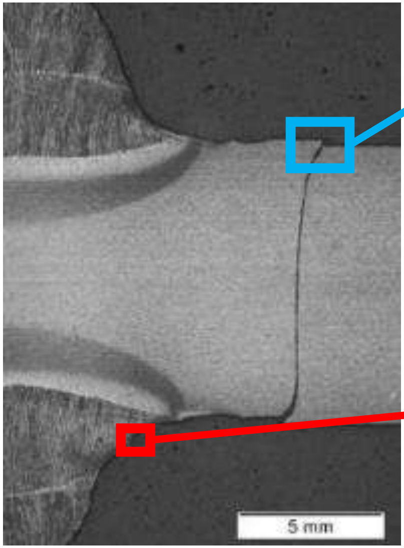

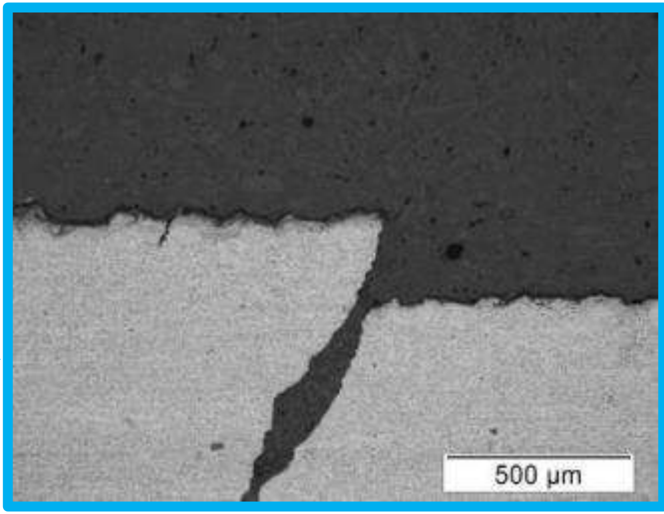

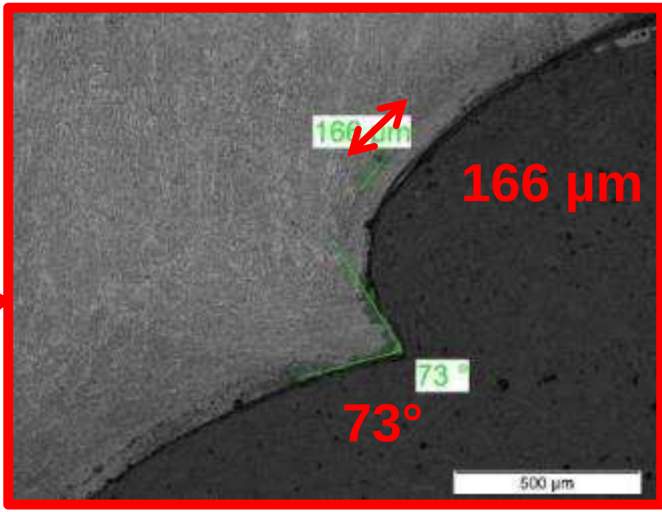

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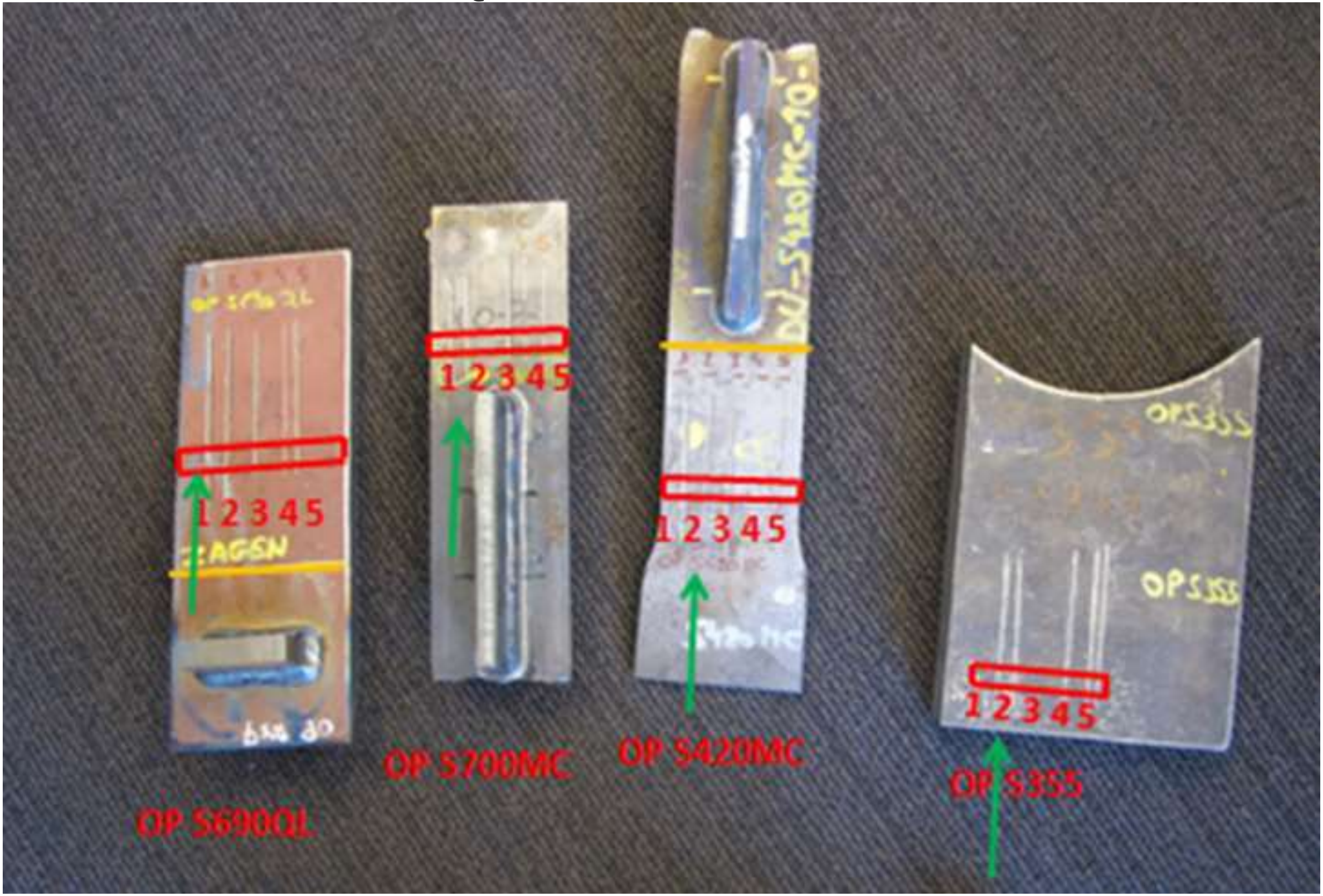
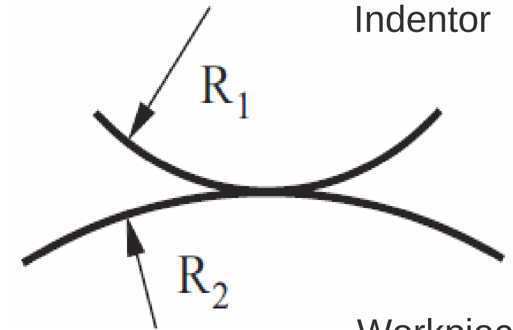
Post Weld Treatment Qualification



Post weld treatment qualification: Hertz theory

$$E^* = \left(\frac{1-\nu_1^2}{E_1} + \frac{1-\nu_2^2}{E_2}\right)^{-1}$$

Indentor

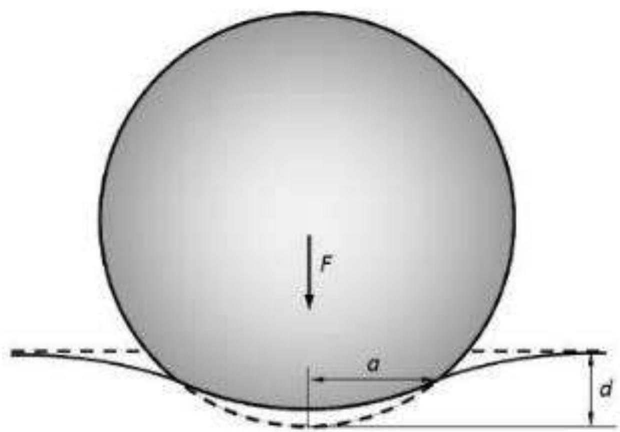


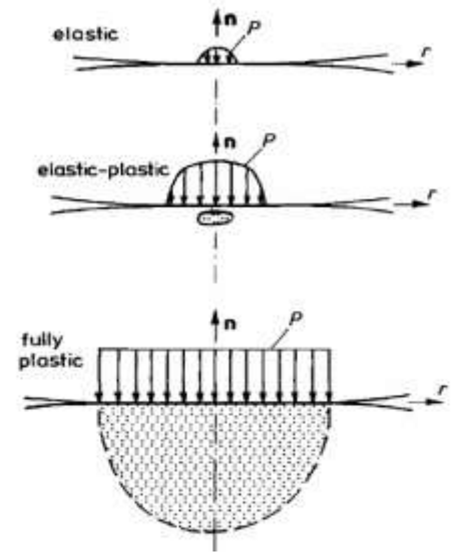
Workpiece

$$R^* = \left(\frac{1}{R_1} + \frac{1}{R_2}\right)^{-1}$$

$$p_0 = \frac{2}{\pi} E^* \left(\frac{d}{R}\right)^{1/2}$$

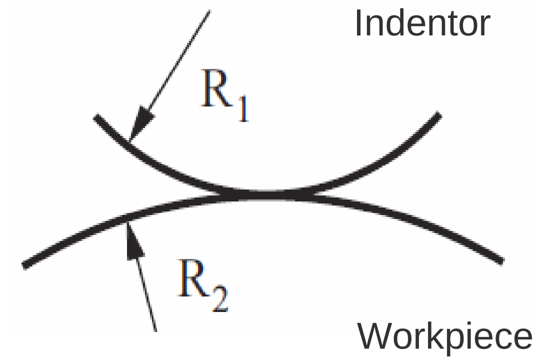
if $p_m < 1.1\ \sigma_y$, elastic deformation occurs





Post weld treatment qualification: Hertz theory

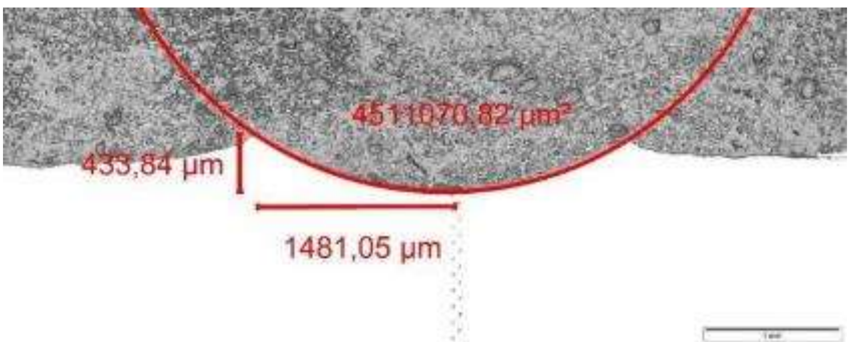
$$E^* = \left(\frac{1-\nu_1^2}{E_1} + \frac{1-\nu_2^2}{E_2}\right)^{-1} = 224.000 \text{ N/mm}^2$$



$$R^* = \left(\frac{1}{R_1} + \frac{1}{R_2}\right)^{-1} = 1.5 \text{ mm}$$

$$p_0 = \frac{2}{\pi} E^* \left(\frac{d}{R}\right)^{1/2} = 76.617 \text{ N/mm}^2 > 2,8 * 355 \text{ N/mm}^2$$

if $p_m > 1.8\, \sigma_y$, contained plastic deformation occurs

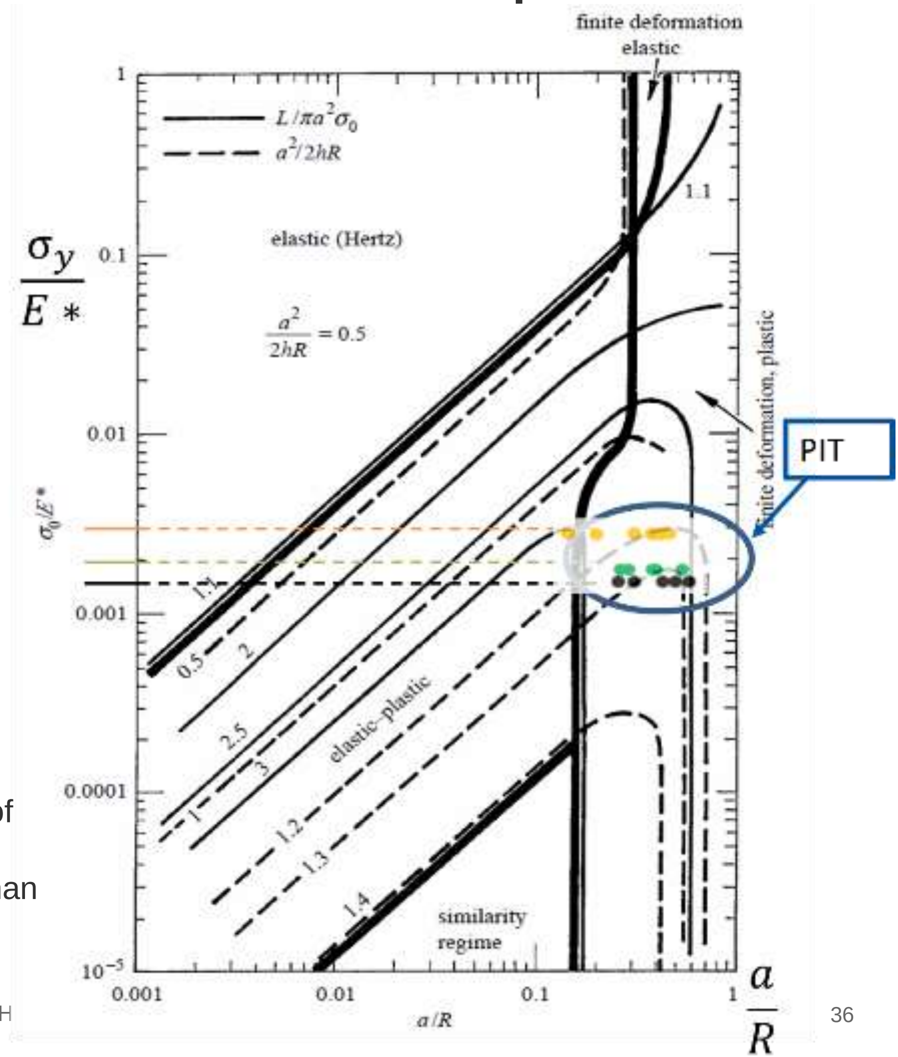


PIT treatment on S355 base material
Indentor: compressed air 6 bar – indentor radius r = 2mm – frequency f= 90Hz

Post weld treatment qualification: indentation map

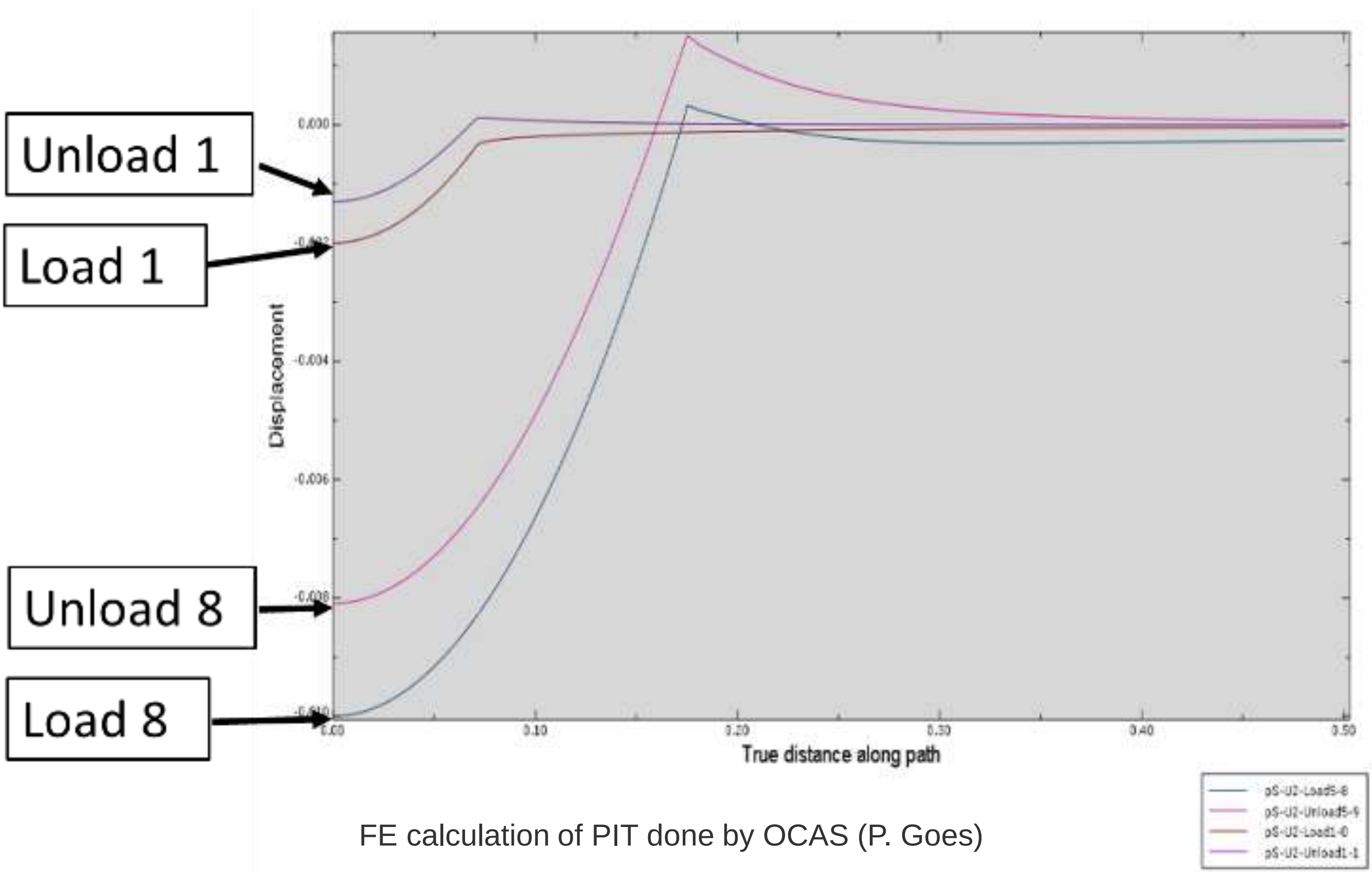
	a/R
S355 PIT 1	0,54
S355 PIT 2	0,25
S355 PIT 3	0,69
S355 PIT 4	0,19
S355 PIT 5	0,43
	a/R
S420 PIT 1	0,38
S420 PIT 2	0,23
S420 PIT 3	0,61
S420 PIT 4	0,19
S420 PIT 5	0,35
	a/R
S690 PIT1	0,41
S690 PIT2	0,36
S690 PIT3	0,38
S690 PIT4	0,13
S690 PIT5	0,39
	a/R
S700 PIT1	0,35
S700 PIT2	0,25
S700 PIT3	0,48
S700 PIT4	0,07
S700 PIT5	0,25

Indentation map of University of Cambridge (Norman Fleck)



Belgian Welding Institute – ing. Thomas Baaten, IWE.

Post weld treatment qualification: dimensional check



FE calculation of PIT done by OCAS (P. Goes)

Conclusions

- Welding of HSS depends on the chemical composition and the fabrication method
- The robustness of PIT is proven by means of fatigue test of different parameters, cross sections and dimensional checks
- **<u>If</u>** a Post Weld Treatment Qualification (PWTQ) is needed for Eurocode, a simple cross section is needed to show that a/R>0,2.
- New ‘IIW Recommendations for the HFMI Treatment For Improving the Fatigue Strength of Welded Joints’ is interesting.

Thank you for your attention!

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A.M. Habraken, L. Duchêne, C. Bouffioux, C. Canalès

Characterization of Fatigue Behaviour, from Material Science to Civil Engineering Applications

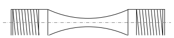
OptiBri-Workshop

"Design Guidelines for Optimal Use of HSS in Bridges"

3rd May 2017

Chantal Bouffioux

Outline

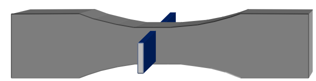


Small samples

Static & fatigue tests

Material behaviour

- Static
- Fatigue (small size)



Large welded plates

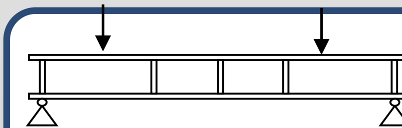
Fatigue tests
Residual stress

Effect of:

- size & machining
- welding
- post-treatments

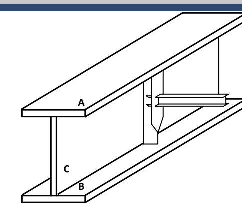
Fatigue behaviour

Future applications



Beams

- Size effect
- Laws validation



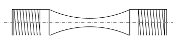
Critical bridge detail

- interest of HSS

University of Liège - Argenco department – MS²F sector – MSM division

Outline

Future applications

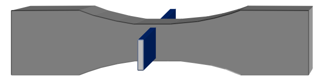


Small samples

Static & fatigue tests

Material behaviour

- Static
- Fatigue (small size)



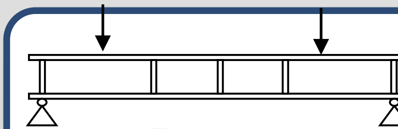
Large welded plates

Fatigue tests
Residual stress

Effect of:

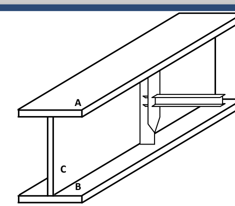
- size & machining
- welding
- post-treatments

Fatigue behaviour



Beams

- Size effect
- Laws validation

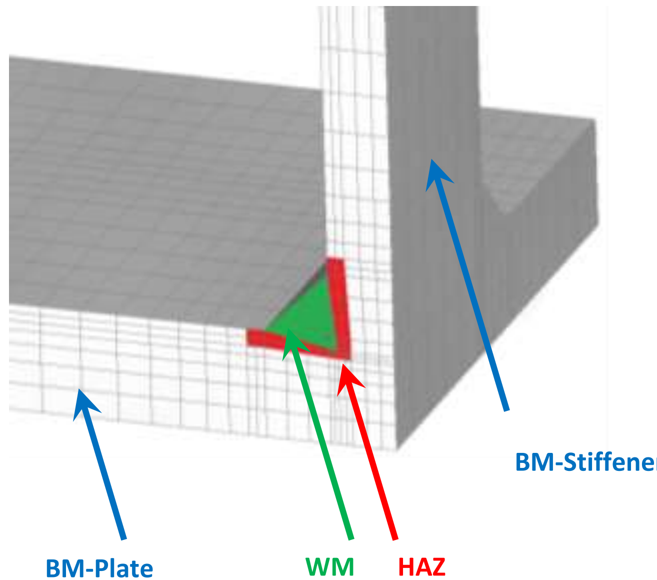


Critical bridge detail

- interest of HSS

Small samples

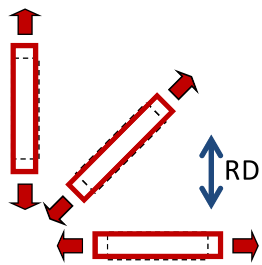
- **4 materials:**
  - Base Material: **BM** **(HSS - S690QL)**
  - Heat Affected Zone:
    - **HAZ1** (25 mm thick)
    - **HAZ2** (40 mm thick)
  - Welded Metal: **WM**

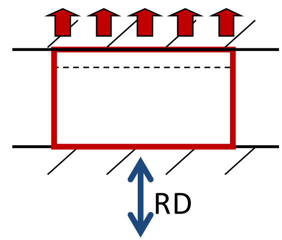


Small samples

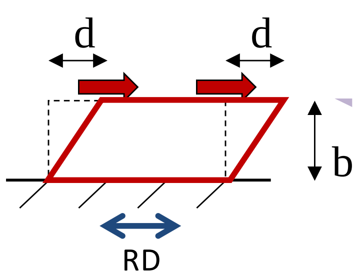
- **Static tests:**

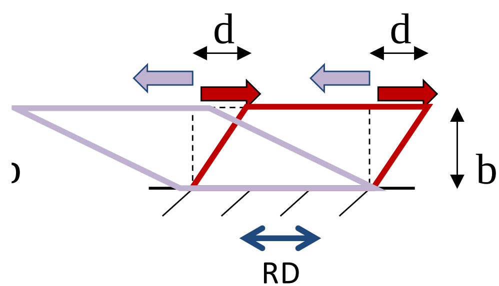
Tensile test **or** **Large tensile test**





+ Shear test **+ Bauschinger shear test**





Small samples

- **Static behavior – material laws & parameters :**

Elastic part: Hooke's law: E, ν

Plastic part: Hill's law (Hill48):

$$F_{HILL}(\underline{\sigma}) = \frac{1}{2}\left[H(\sigma_{xx} - \sigma_{yy})^2 + G(\sigma_{xx} - \sigma_{zz})^2 + F(\sigma_{yy} - \sigma_{zz})^2 + 2N(\sigma_{xy}{}^2 + \sigma_{xz}{}^2 + \sigma_{yz}{}^2)\right] - \sigma_F^2 = 0$$

Isotropic hardening: Voce formulation:

$$\sigma_F = \sigma_0 + K(1 - exp(-n.\varepsilon^{pl}))$$

Back-stress (kinematic hardening): Armstrong-Frederick's equation:

$$\underline{\dot{X}} = C_X(X_{sat}\underline{\dot{\varepsilon}^{pl}} - \dot{\bar{\varepsilon}}^{pl}.\underline{X})$$

E & **ν:** defined by tensile tests

F, G, H: defined by tensile tests in 3 directions (RD, TD, 45°)

N, σ_0, K, n, C_x, X_{sat}: defined by Optim

Small samples

- **Static behavior – material data (inverse method):**
  - **BM**: hardening fully kinematic
  - **HAZ1 ≈ HAZ2**: same static behaviour
  - **WM**

For fatigue tests: $\sigma_{u,eng} = F_i / A_0$

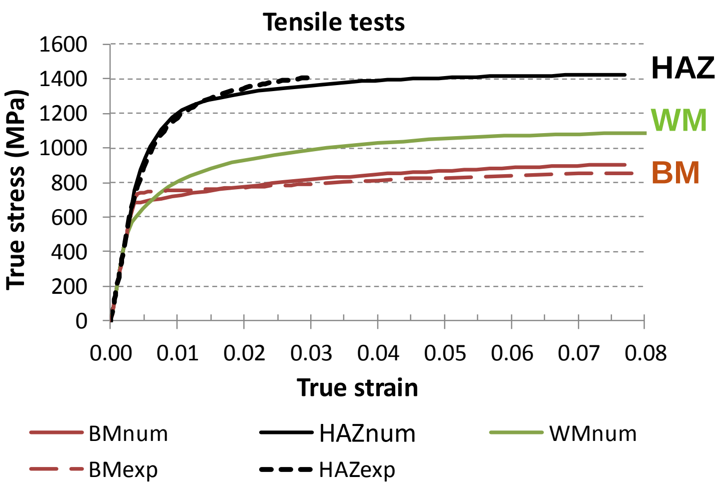
For FEM: $\sigma_{u,true} = F_i / A_i$

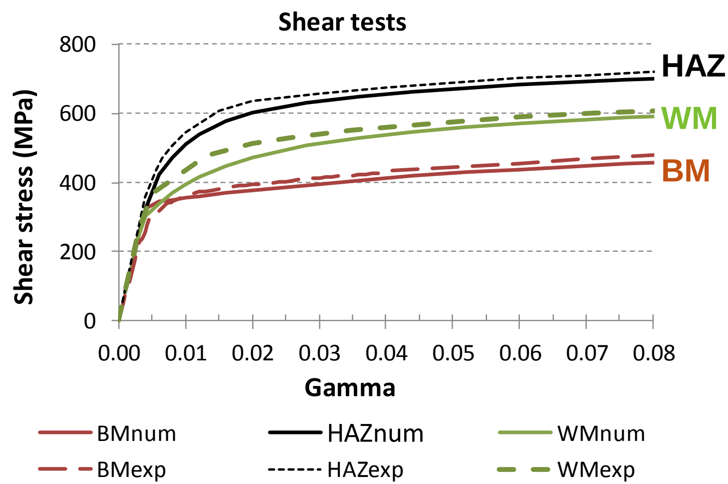
	Ultimate tensile strength (Mpa)	
	$\sigma_{u,eng}$	$\sigma_{u,true}$
BM (S690QL)	838	905
HAZ1, HAZ2	1338	1424
WM	1008	1101

Data for Hooke, Hill, Voce and Armstrong-Frederick laws (units: MPa, s)											
	Elast. data		Yield locus				Isotropic hardening			Kinematic hardening	
Material	E	ν	F	G	H	*N=L=M*	*K*	σ_0	*n*	C_X	X_{sat}
BM (S690QL)	210 116	0.3	1	1	1	*3.9*	*0*	*674*	*0*	*31.9*	*167*
HAZ1, HAZ2	210 000	0.3	1	1	1	*4.45*	*371*	*827*	*511*	*52.5*	*152*
WM	210 000	0.3	1	1	1	*3.2*	*241*	*531*	*285*	*42.6*	*218*

Small samples

- **Static behavior – comparison of material behavior:**





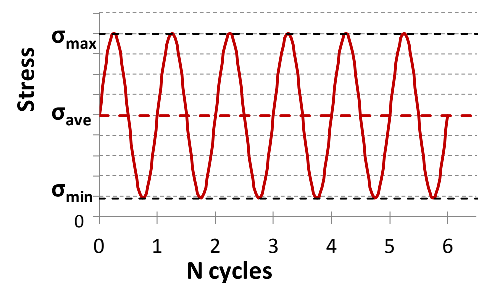
Small samples

- **Fatigue tests:**
  - On vibrophore
  - Axial loading
  - Frequency: 100 – 150 Hz
    (→ correction factor)

R= σ_{min} / σ_{max} = 0.1 or 0.2 or …

Material	Smooth	Notch
BM (S690QL)	4 R	3 geom.
HAZ1	1 R	1 geom.
HAZ2	2 R	1 geom.
WM	2 R	1 geom.

Vibrophore



Small samples

- **Fatigue behavior – material laws & parameters :**

Multiaxial Lemaître Chaboche fatigue model

$$\frac{\partial D}{\partial N} \begin{cases} = 0 & \text{if } f_D < 0 \\ = \left[1-(1-D)^{\beta+1}\right]^{\alpha} \left(\frac{\tilde{A}_{II}}{M}\right)^{\beta} & \text{if } f_D \geq 0 \end{cases}$$

$$f_D = A_{II} - A_{II}^{*}$$

$$A_{II} = \frac{1}{2}\sqrt{\frac{3}{2}\left(\hat{\sigma}_{ijmax} - \hat{\sigma}_{ijmin}\right)\left(\hat{\sigma}_{ijmax} - \hat{\sigma}_{ijmin}\right)} \quad \text{with } \hat{\sigma}_{ij} = \sigma_{ij} - \sum_k \frac{1}{3}\sigma_{kk}$$

$$A_{II}{}^{*} = \sigma_{l0}\left(1 - 3.b.\sigma_{Hm}\right) \quad \text{(Sines' criterion)}$$

$$\tilde{A}_{II} = \frac{A_{II}}{1-D}$$

$$M = M_0\left(1 - 3.b.\sigma_{Hm}\right)$$

$$\alpha = 1 - a\left\langle \frac{A_{II} - A_{II}^{*}}{\sigma_u - \sigma_{eqmax}} \right\rangle$$

$$\sigma_{Hm} = \frac{1}{3}\left[\frac{1}{T}\int_T \mathrm{Tr}\left(\underline{\sigma}(t)\right) dt\right]$$

D:	damage val., 0: sound material, 1: rupture
N:	number of cycle
A_{II}:	2nd invar. of amplit. deviator of σ tensor
$A_{II}{}^{*}$:	fatigue limit
f_D:	damage yield locus
σ_{Hm}:	mean hydrostatic stress
σ_{eqmax}:	maximum Von Mises stress per cycle
<x>	= x if x > 0 else = 0
$b = 1/\sigma_u$	
σ_u:	ultimate tensile stress
σ_{l0}:	endurance limit= fatigue limit at null σ_{mean}
a, M0, β:	other material data to define

Small samples

- **Fatigue behavior – material laws & parameters :**

Volume averaged stress gradient method

For each element, variables χ_{ip}: replaced by an average value of all the elements with their integration point inside the circle with a radius Ra

$$\overline{\chi_{ip}} = \frac{1}{V} . \sum_{i=1}^{Nelem} \chi_{ip,i} . V_i$$

$$\chi_{ip} = \{A_{II}, \sigma_{eqmax}, \sigma_{Hm}\}$$

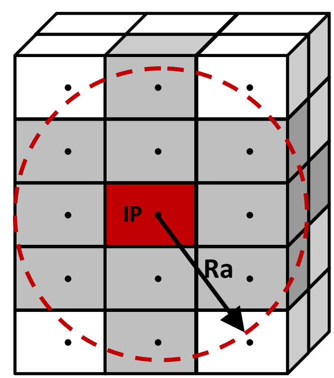
$$V = \sum_{i=1}^{Nelem} V_i$$

A_{II}: 2nd invar. of amplit. deviator of σ tensor

σ_{eqmax}: maximum Von Mises stress per cycle

σ_{Hm}: mean hydrostatic stress

Ra: material data to define



Small samples

- **Fatigue behavior – material data (inverse modelling):**

HAZ1 ≈ HAZ2: same fatigue behavior

Material	σ_U (Mpa)	σ_{l0} (Mpa)	b	β	a	M0	Ra (mm)	$a^*(M0^{-\beta})$
BM	905.0	580.0	1.10 E-03	0.17	1	5.385 E+30	0.06	5.966E-06
HAZ1, HAZ2	1424.0	428.4	7.02E-04	2.094	1	4.410E+05	0.00	1.516E-12
WM	1101.0	319.4	9.08E-04	0.161	1	7.245E+32	0.00	5.182E-06

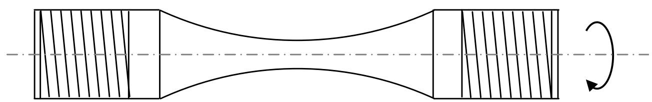
Small samples

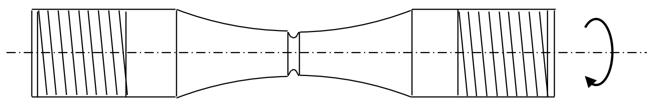
- **Fatigue behavior – comparison experiments & fatigue law:**

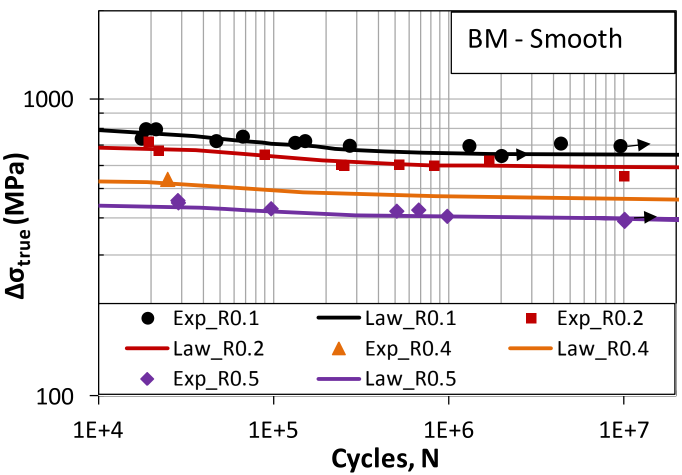
Base material (BM)

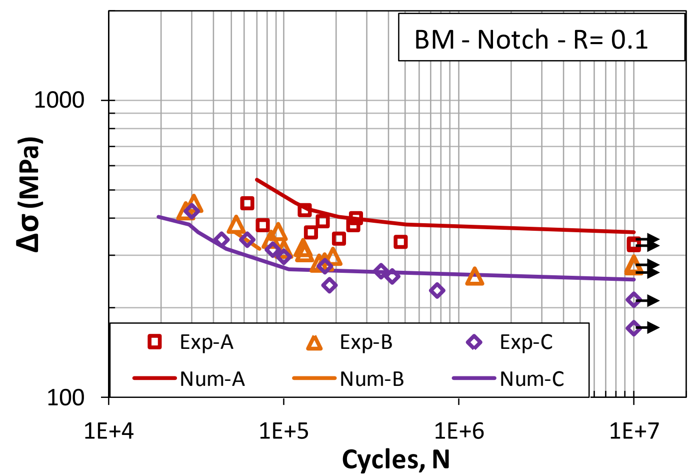
Smooth samples, 4 R

Notched samples, 3 geom.









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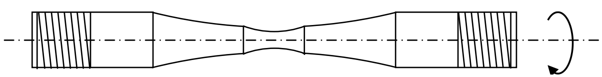
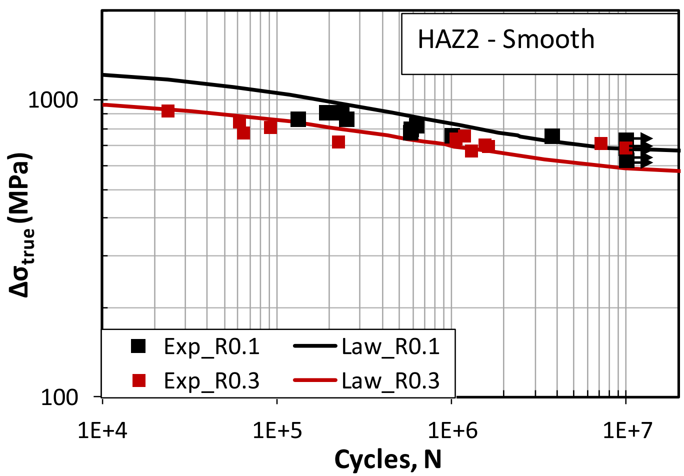
Small samples

- **Fatigue behavior – comparison experiments & fatigue law:**

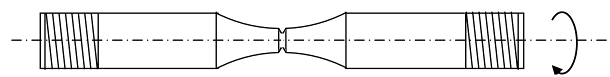
Heat affected zone (HAZ) with HAZ1 ≈ HAZ2

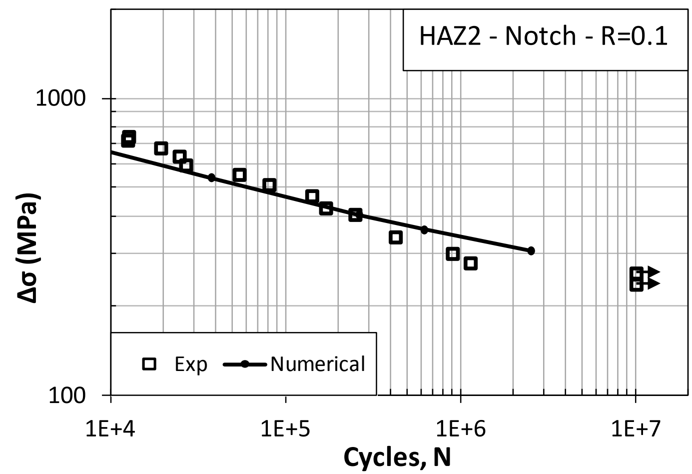
Smooth samples, 2 R





Notched samples, 1 geom.



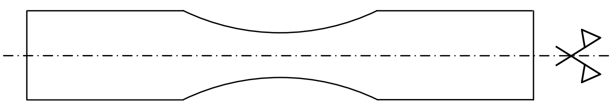


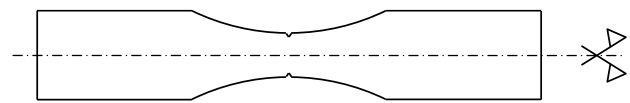
Small samples

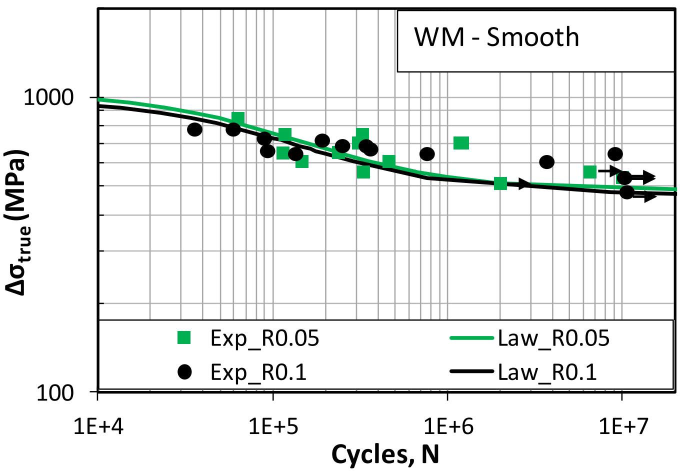
- **Fatigue behavior – comparison experiments & fatigue law:**

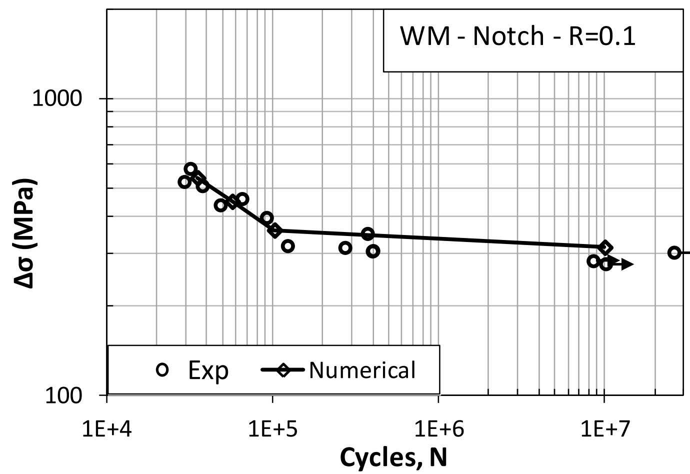
Weld metal (WM)

Smooth samples, 2 R	Notched samples, 1 geom.









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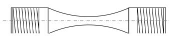
Outline

Future applications

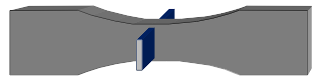


Small samples

Static & fatigue tests

Material behaviour

- Static
- Fatigue (small size)



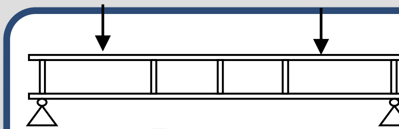
Large welded plates

Fatigue tests Residual stress

Effect of:

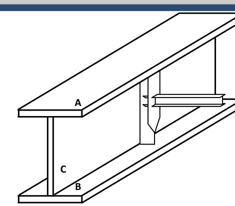
- size & machining
- welding
- post-treatments

Fatigue behaviour



Beams

- Size effect
- Laws validation



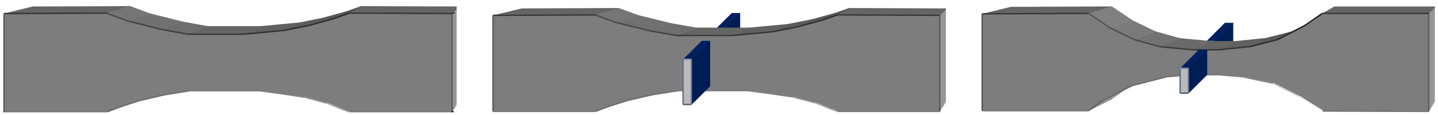
Critical bridge detail

- interest of HSS

University of Liège - Argenco department – MS²F sector – MSM division

Large welded plates

- **Fatigue tests on plates (length= 1070 mm):**



Case	Post-treatment	plate thickness (mm)	Stiffener thickness (mm)	Stiffener length (mm)	distance to edge	stress ratio R
Plate	No weld	25	-	-	-	0.1
A (ref case)	PIT	25	15	60	✓	0.1, 0.3, 0.5
B	PIT	15	15	60	✓	0.1
E	PIT	25	15	60	no	0.1
H	PIT	40	15	60	no	0.1
C	TIG remelting	15	15	60	✓	0.1
D	TIG remelting	25	15	60	✓	0.1
F	TIG remelting	25	15	40	✓	0.1
G	TIG remelting	15	6	60	✓	0.1
I	No post-treatment	15	15	60	✓	0.1

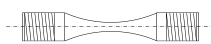
Large welded plates

- **Fatigue behavior – material data (inverse modelling):**

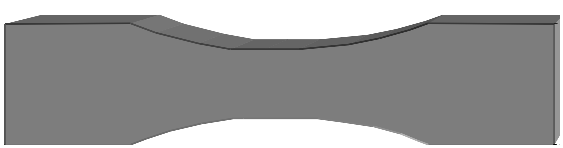
Material	σ_U (Mpa)	σ_{l0} (Mpa)	b	β	a	M0	Ra (mm)	a*(M0$^{-\beta)}$
BM-SS	905.0	**580.0**	1.10 E-03	0.17	1	5.385 E+30	0.06	5.966E-06
BM-plate	905.0	**203.0**	1.10 E-03	0.17	1	5.385 E+30	0.06	5.966E-06

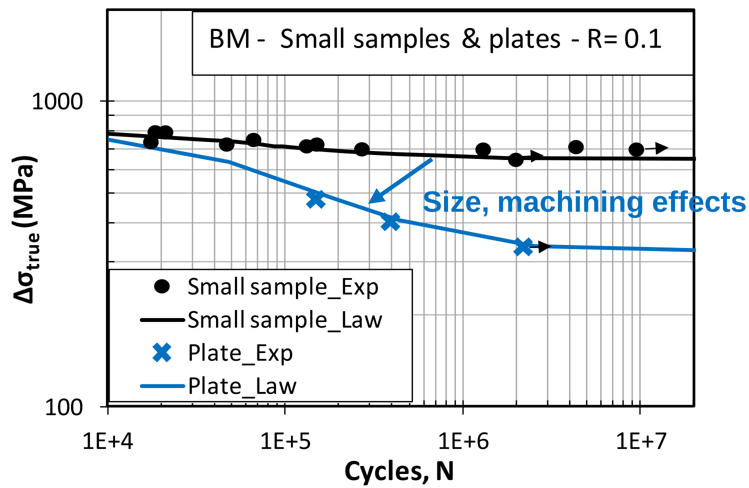
σ_{l0} = endurance limit= fatigue limit at null σ_{mean}

Small samples, Ra= 0.8 µm, length: 96 mm



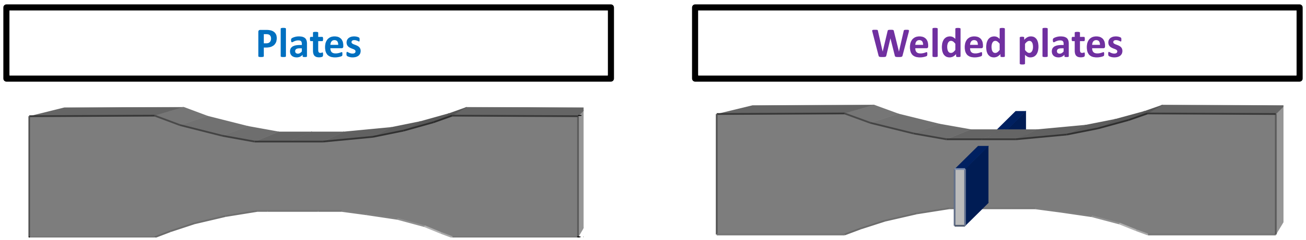
Plates, as produced, length: 1070 mm

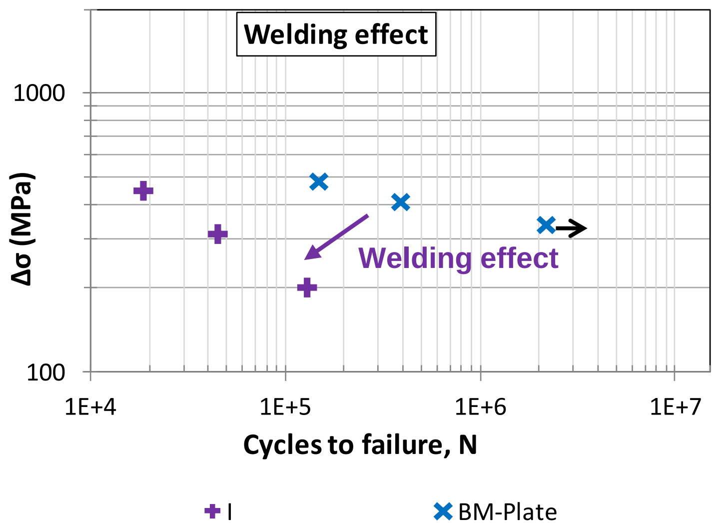




Large welded plates

- **Fatigue tests: welding effect**

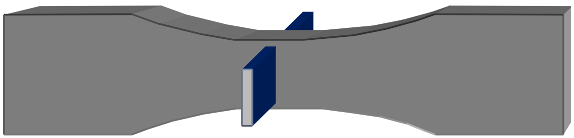




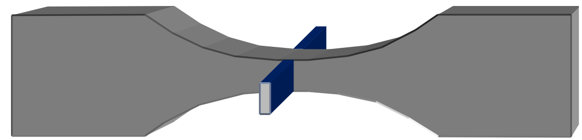
Large welded plates

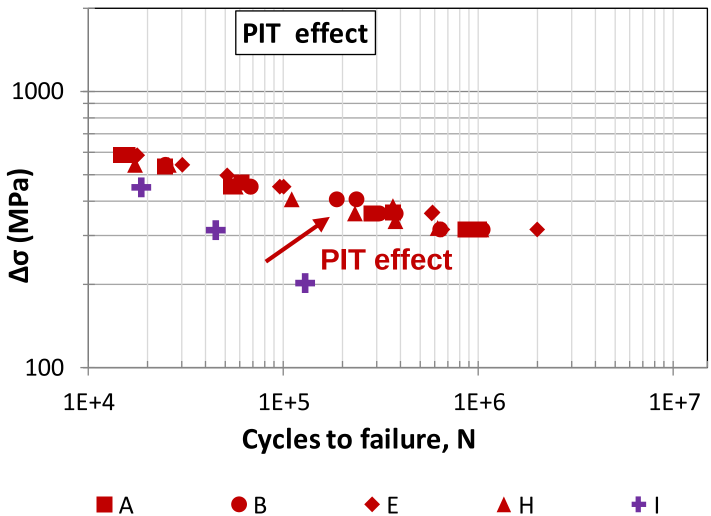
- **Fatigue tests: PIT post-treatment effect**

Welded plates



Welded plates + PIT, ≠ geom.





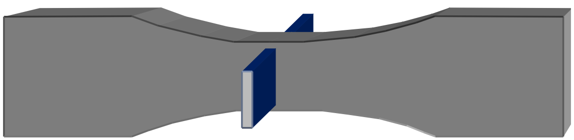
PIT: 4 cases

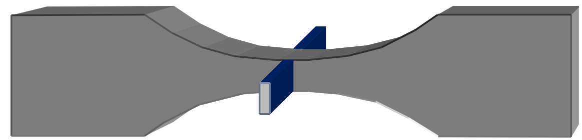
low effect of geometry

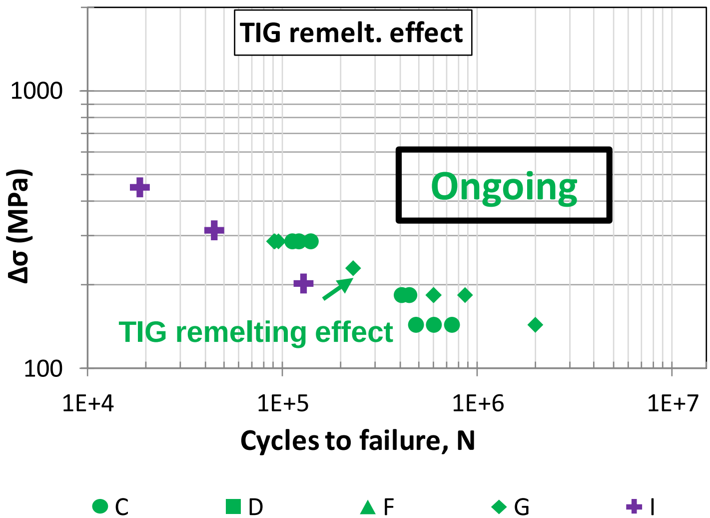
Large welded plates

- **Fatigue tests: TIG remelting post-treatment effect**

Welded plates	Weld. plates + TIG rem. , ≠ geom.



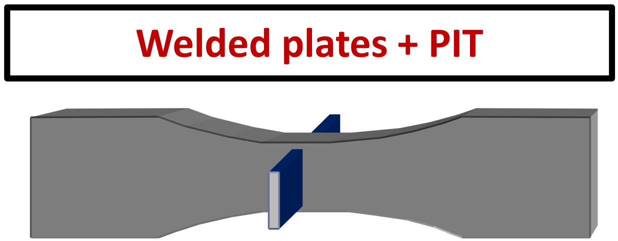


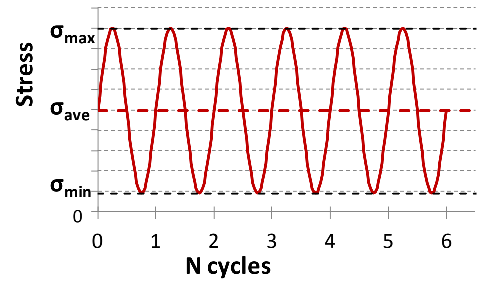


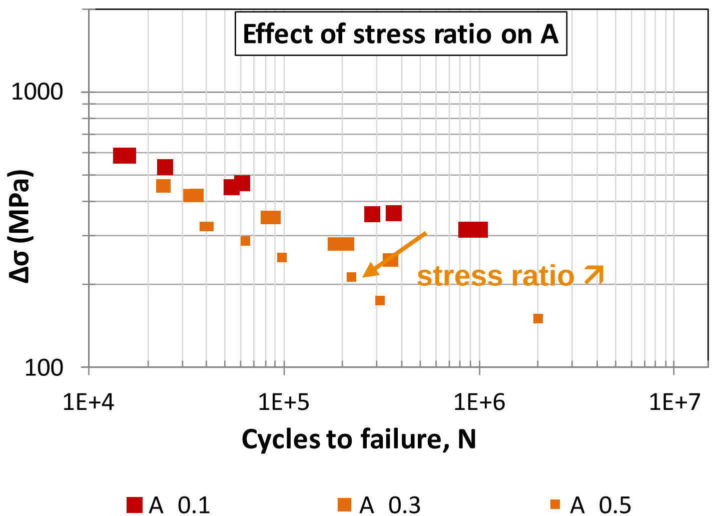
Large welded plates

- **Fatigue tests: Stress ratio effect**

$R = \sigma_{min} / \sigma_{max} = 0.1$ or 0.3 or 0.5

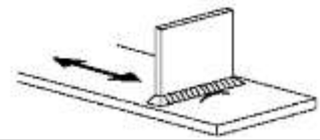






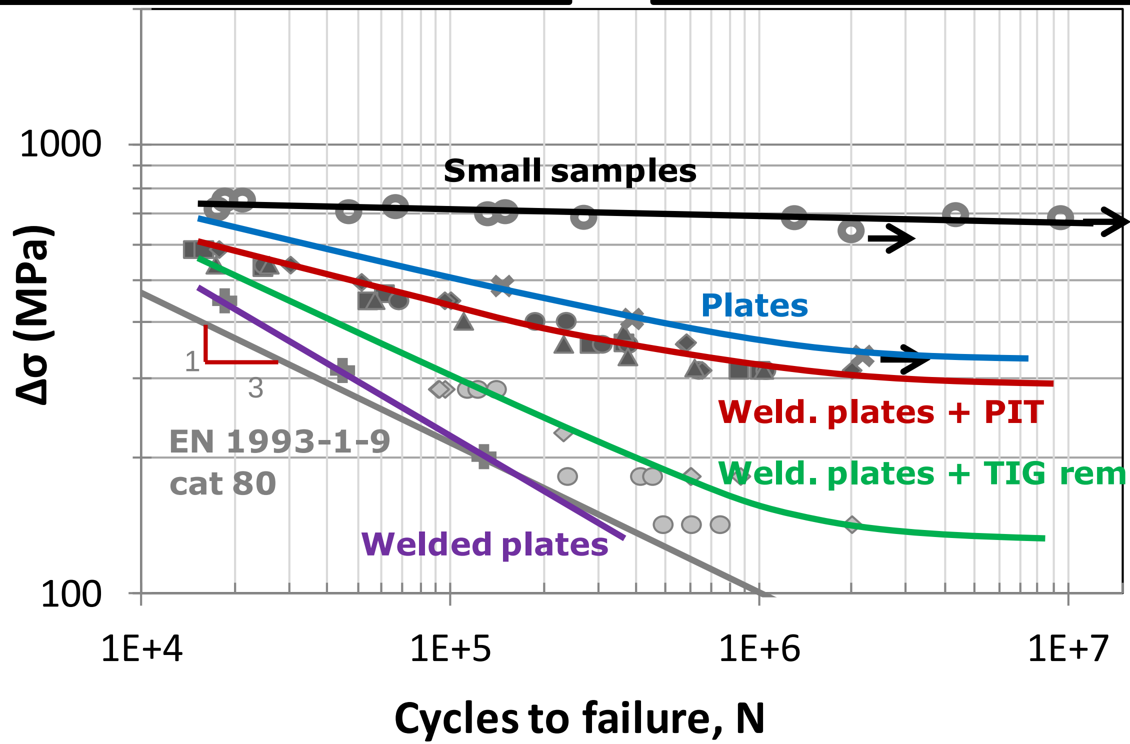
Large welded plates

- **Fatigue tests**



Summary, all cases, R= 0.1

EN 1993-1-9, $\Delta\sigma c$ = 80 MPa

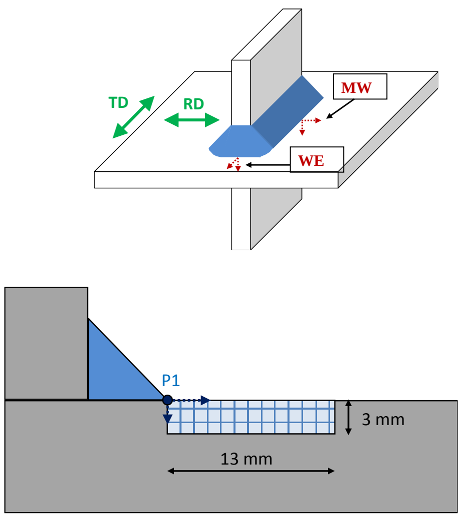


Large welded plates

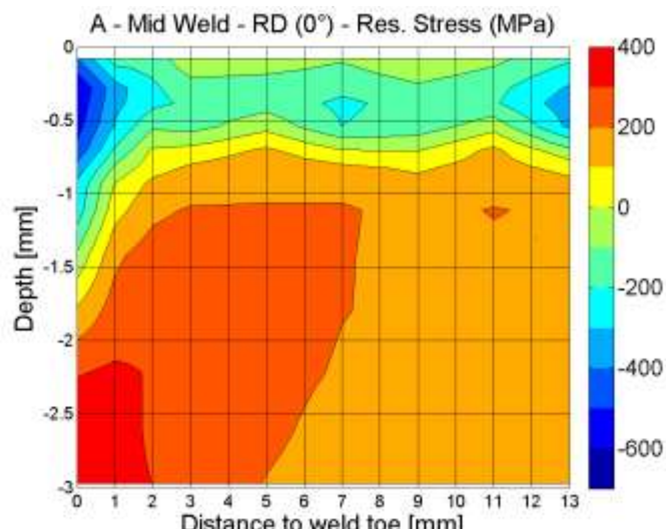
- **Residual stress measurement**

Several cases:
- 3 geometries
- 3 cases: PIT, TIG remelting, no post-treat.
- Mid-weld (MW), weld edge (WE), RD, TD
- X-ray, neutron diffraction

Welding & post-treatment effects:
- up to depth ≈ 3-4 mm
- up to weld toe distance ≈ 6-7 mm



Ex: ref. case: A, with post-treatment (X-ray)



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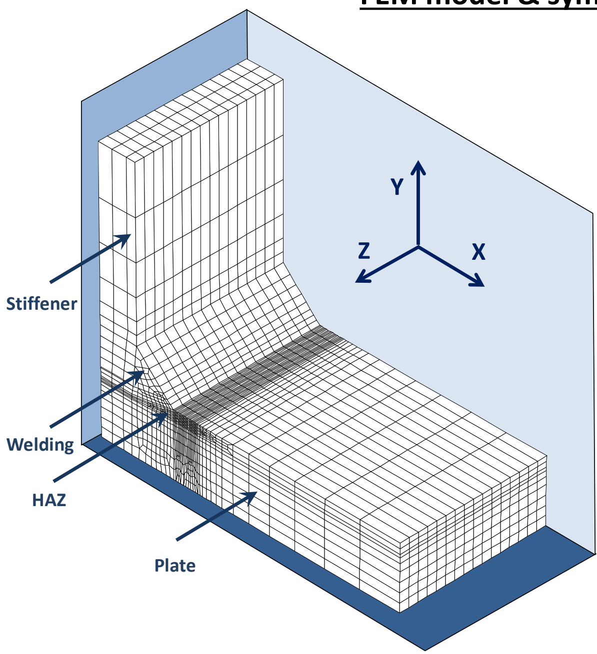
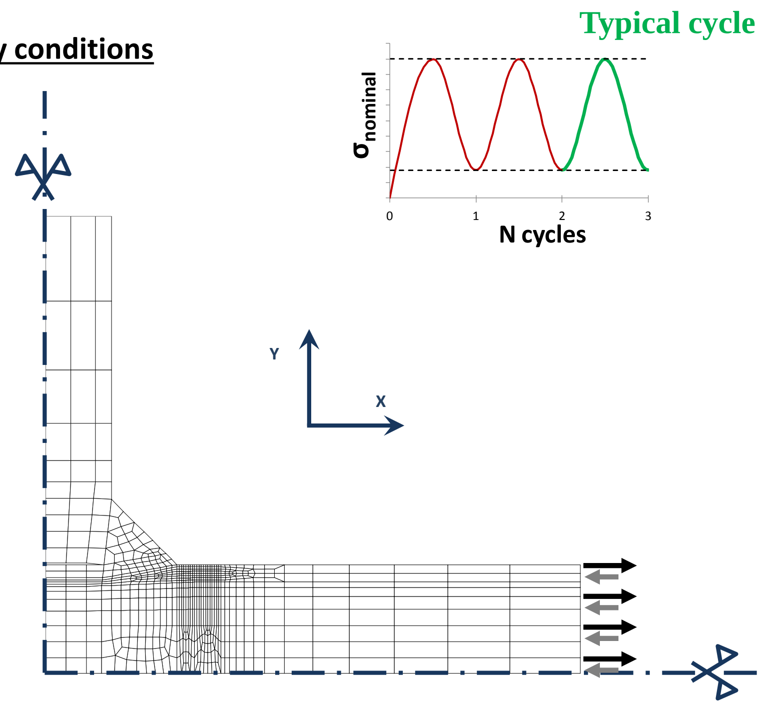
Large welded plates

- **Fatigue - numerical analysis**

FEM model & symmetry conditions





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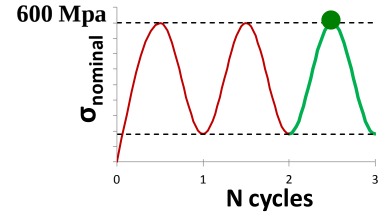
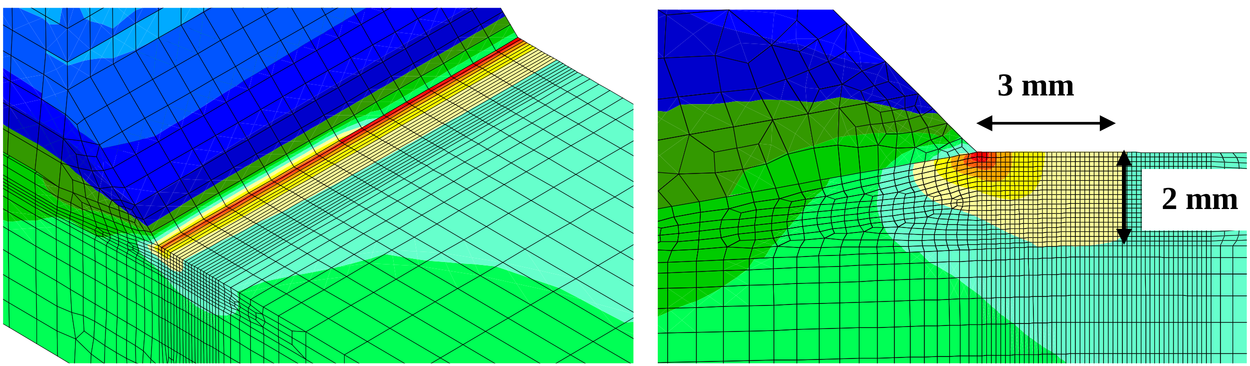
Large welded plates

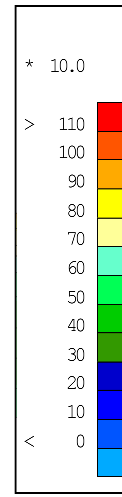
- ## Fatigue - numerical analysis



Example of results without post-treatment , no σ_{res}

σ_{xx} (Mpa) for nominal stress = 600 Mpa (σ_{nom}= F/A_0 and A_0= section of web)





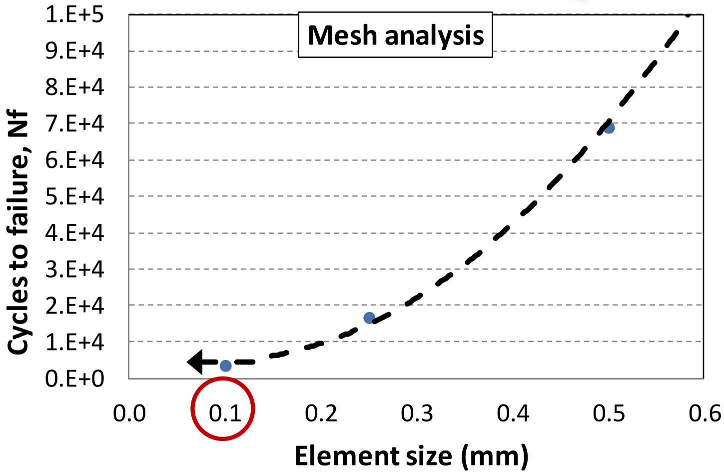
<u>Stress concentration at weld toe:</u>

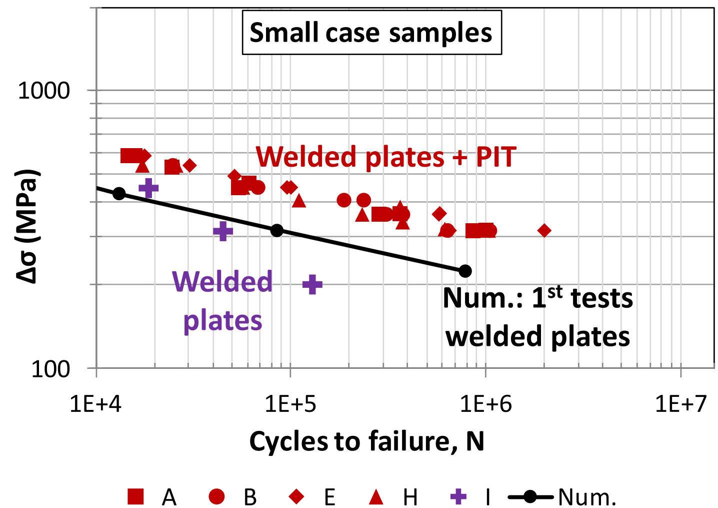
- **up to depth ≈ 2 mm (in HAZ)**
- **up to weld toe distance ≈ 3 mm**

Large welded plates

- **Fatigue - numerical analysis**

1. **Mesh analysis** ➡ **element size at weld toe: 0.1 mm (results not mesh dependent)**
2. **For several stress ranges and a specified stress ratio (here: 0.1):**
   - **Numerical analysis** ➡ **Stress distribution** ➡ **number of cycle at rupture**





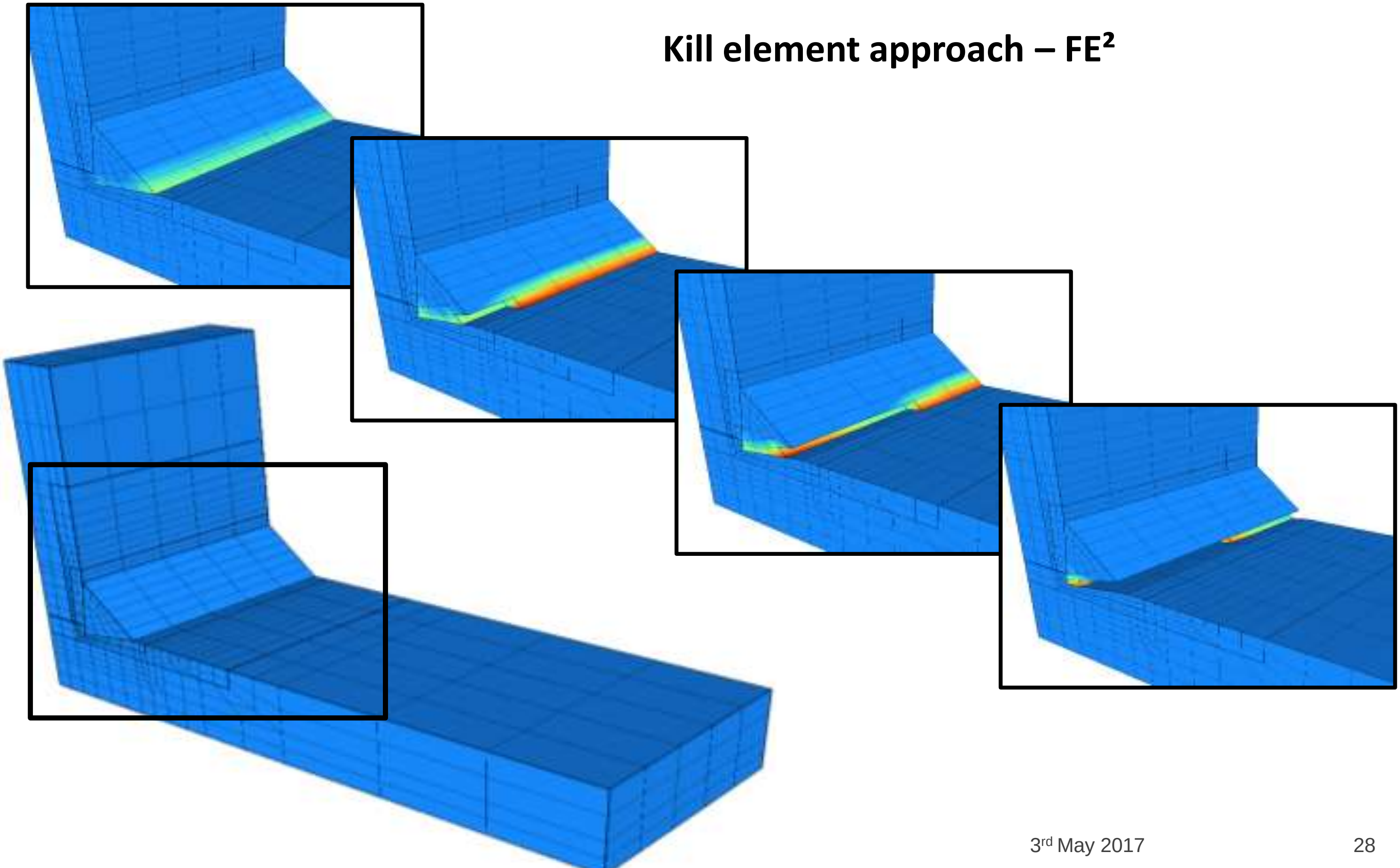
Next steps:

- **to add σ_{res} to model (welding + post-treatment)**
- **to improve fatigue mat. data of HAZ (σ_{l0})**
- **to study beams and critical bridge detail**

Large welded plates

- **Fatigue - numerical analysis, crack propagation**

Kill element approach – FE²



Summary & conclusions

Important test campaign has been done to prepare numerical fatigue study of bridge details:

- **Static tests** / **Fatigue tests** **on small samples** ➡ **Static behaviour** / **Fatigue behaviour (small size)** **BM, HAZ, WM**
- **Fatigue tests on large welded plates** ➡ **Effects of size, surface roughness, welding, geometry, post-treatments**
- **Residual stresses measurements** ➡ **Effect of post-treatments for num. analysis**
- **Mesh analysis** ➡ **welded plates: elem. size at weld toe: 0.1 mm**
- **Crack propagation** ➡ **deep analysis of fatigue study**

➡ **Positive effect in fatigue life is shown on welded plates**

Fatigue characterisation almost ready for analysis on critical bridge detail

Thank you for your attention!

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Categorization of Fatigue Details in View of Post-Weld Treatments

OptiBri-Workshop

„Design Guidelines for Optimal Use of HSS in Bridges“

Stephanie Breunig

Overview

„Categorization of Fatigue Details in View of Post-Weld-Treatments“

1.) General information on **High Frequency Mechanical Impact** (HFMI) **Treatment**

2.) Improvement and categorization of appropriate construction details

a) Benefits and influences on fatigue resistance of HFMI-treated construction details (example: transverse stiffener)

b) Possible existing approaches

3.) Beam Tests

a) Motivation of test series

b) Experimental procedure

c) Results of beam tests

4.) Conclusions and outlook

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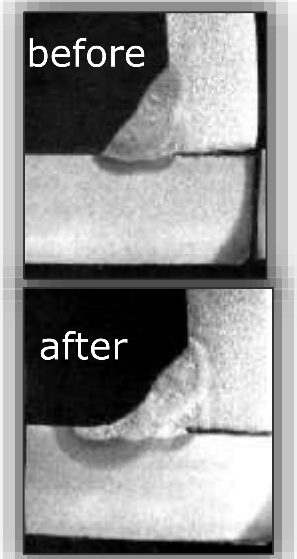
1.) General information on High Frequency Mechanical Impact (HFMI) Treatment

a) Classification of **Post-Weld Treatments**

- Post-Weld Treatment
  - Improvement of notch geometry
    - Grinding
    - TIG dressing
    - Plasma dressing
  - Improvement of residual stress state
    - Hammering
    - Needle
    - Shot peening
    - High Frequency Mechanical Impact (HFMI) Treatment



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1.) General information on High Frequency Mechanical Impact (HFMI) Treatment

b) Mechanism and variants of HFMI

Pneumatic Impact Treatment (PIT)

- By pneumatic pressure mechanical impacts are given with a pneumatically controlled muscle over a hardened pin into the construction
- The intensity is not depending on the applied compressive force due to an integrated spring system

PITec

UIT

HiFIT

1.) General information on High Frequency Mechanical Impact (HFMI) Treatment

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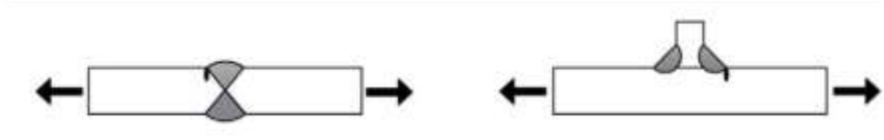
c) Technical requirements

Suitable for HFMI	
Not suitable HFMI	

- For welded construction details with fatigue **failure from weld toe** HFMI-treatment can improve fatigue resistance
- If fatigue failure cracks come from **weld root**, HFMI application is not successful
- **Accessability** to the welds, weld toe is needed

2.) Improvement and categorization of appropriate construction details

a) Benefits and influences on fatigue resistance of HFMI-treated construction details (example: transverse stiffener)

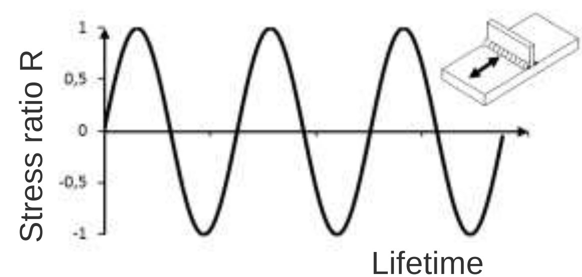
Investigated construction details:

- Butt weld and variants (a.v.)
- Transverse stiffener (a.v.)
- Longitudinal stiffener (a.v.)
- …

→ Amount of improvement depends on construction detail

Quantity of improvement depends on **further parameters**:

- Yield strength $\mathbf{f_y}$
- Stress ratio **R**
- Type of loading (height, quantity, time …)
- Plate thickness **t**



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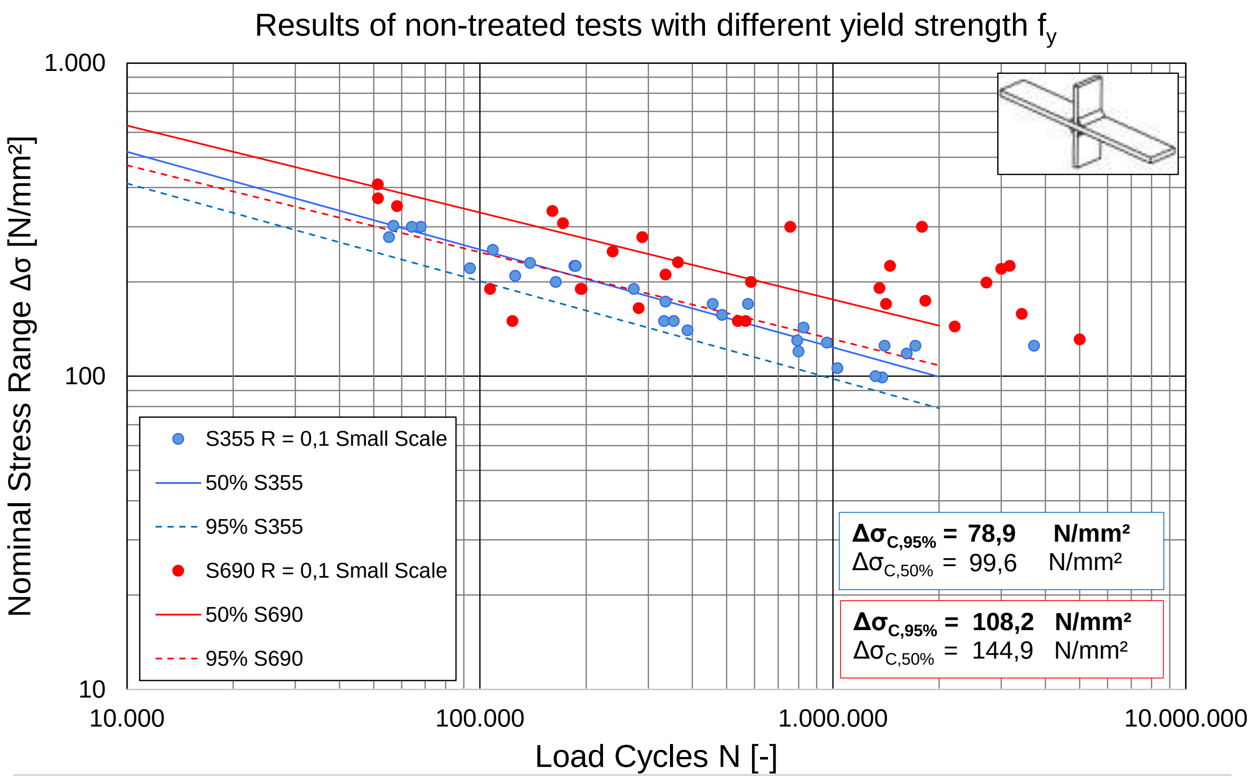
2.) Improvement and categorization of appropriate construction details

a) Benefits and influences on fatigue resistance of HFMI-treated construction details (example: transverse stiffener)



- Higher fatigue resistance for higher yield strength

2.) Improvement and categorization of appropriate construction details

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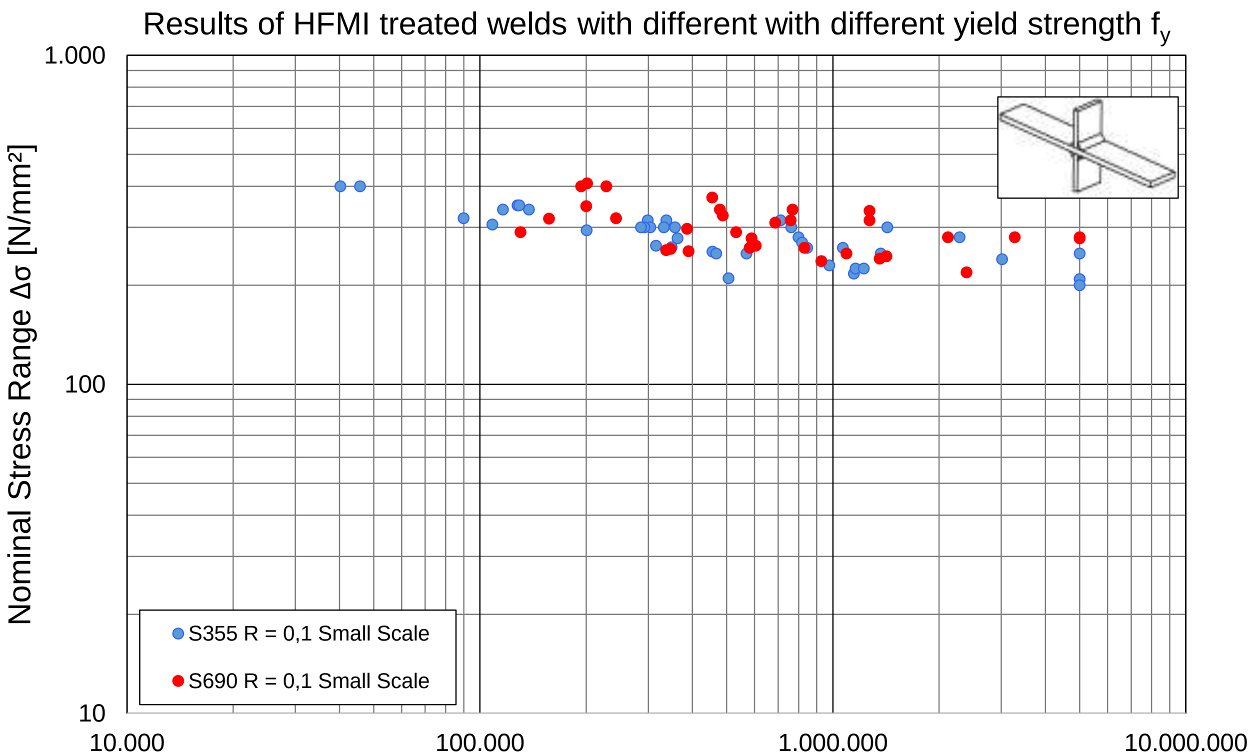
a) Benefits and influences on fatigue resistance of HFMI-treated construction details (example: transverse stiffener)



- Higher fatigue resistance for higher yield strength
- **Benefit** in using HFMI-treatment on welded details of **S690 steels**

2.) Improvement and categorization of appropriate construction details

b) Possible existing design approaches

Mainly 2 different procedures of existing design approaches considering HFMI-treatment on welded details

Stepwise improvement in accordance with **existing FAT-Classes**

+1 FAT-Class

+2 FAT-Class

….

+ x-FAT-Class depending on conditions (R, S355 - S690,…)

Consideration of increasing fatigue resistance by **improvement factor k**

$$\Delta\sigma_{C\cdot Imp} = \Delta\sigma_C \cdot \mathbf{k}$$

$$\mathbf{k} = k_f * k_L * k_R$$

k_i depending on conditions (R, S355 - S690,…)

according to [Dürr] or [Weich]

3.) Beam Tests

a) Motivation of beam test series

- Improvement of fatigue resistance of several construction details is proved by **small scale tests** under **laboratory conditions**
- Component tests show **drop of improvement of fatigue** resistance due to:
  - More complex **residual stress state**
  - More complex **welding conditions**

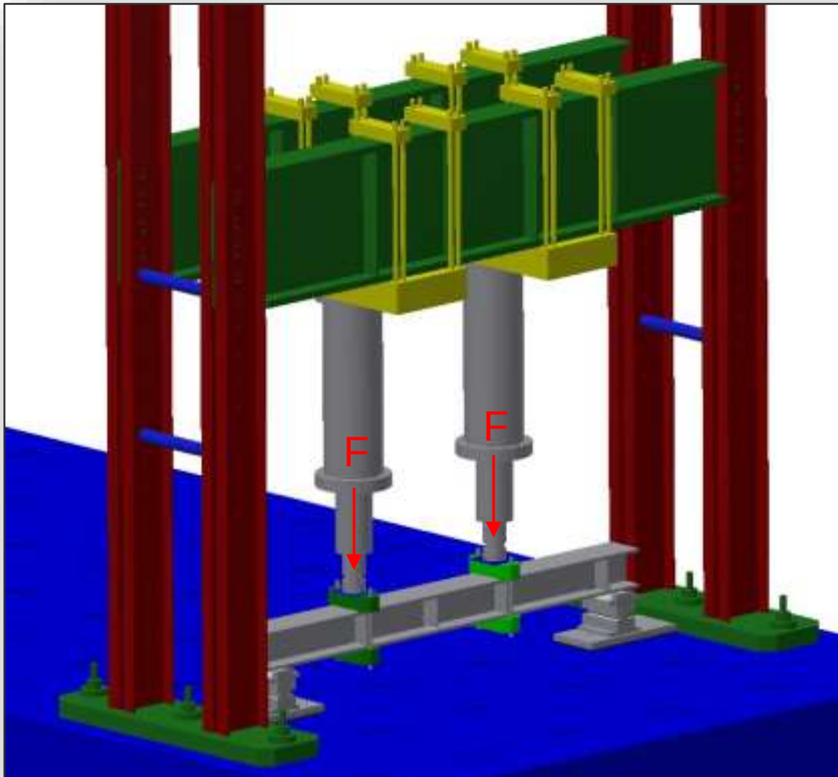
→ Differences in fatigue resistance of small specimen and true scale specimen

- **drop of fatigue** strength according to (Duerr, 2006)

As welded	Small specimen	16% less →	Beam tests
UIT-treated	Small specimen	15% less →	Beam tests

3.) Beam Tests

b) Experimental procedure – test setup



F_{max} = 800kN

➡ Δσ = 400N/mm²

3.) Beam Tests

b) Experimental procedure

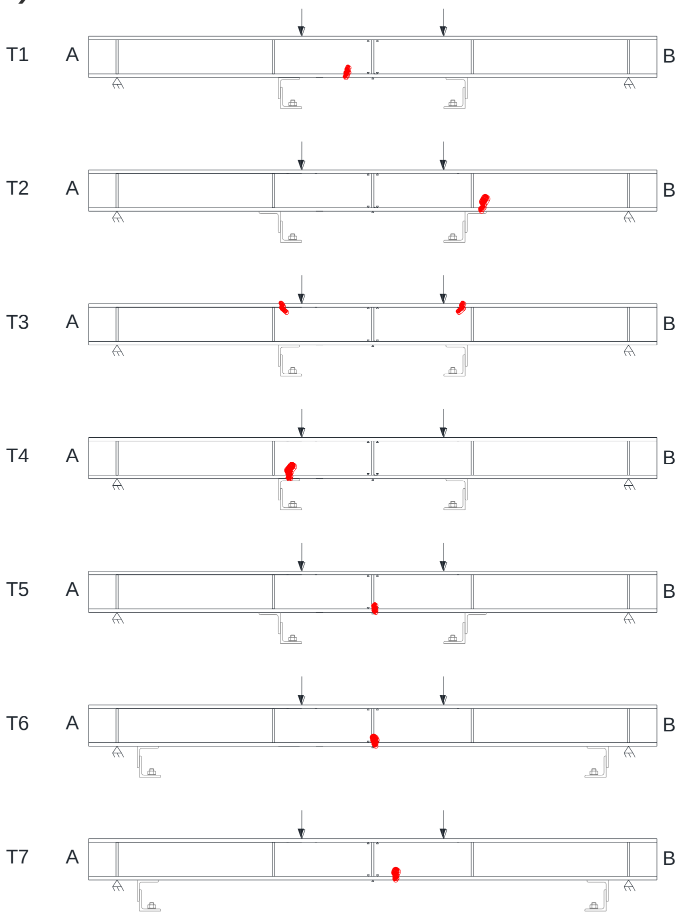
T1

HFMI treatment of stiffener welds leads to **improved detail**, so that **other details** become **relevant**

T1$_{\text{(HFMI: Stiffener)}}$ and T2$_{\text{(HFMI: Stiffener)}}$ failure: Crack crossing the longitudinal fillet weld

3.) Beam Tests

c) Results

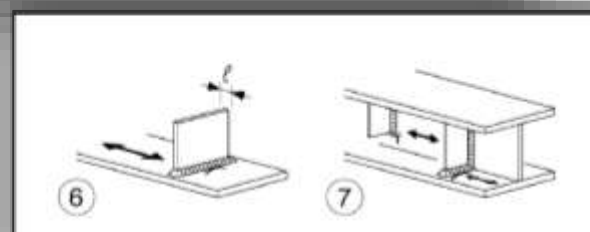


- T1: Crack at longitudinal fillet weld Use as minimum result
- T2: Crack at fixing point on longitudinal fillet weld
  Use as minimum result
- T3: HFMI-treated bottom longitudinal fillet weld; Crack at upper longitudinal fillet weld
- T4: Crack at fixing point
  Use as minimum result
- T5: ✓ AW transverse stiffener
  Crack at transverse stiffener
- T6: ✓ HFMI-treated transverse stiffener
  Crack at transverse stiffener
- T7: HFMI-treated longitudinal fillet weld

3.) Beam Tests

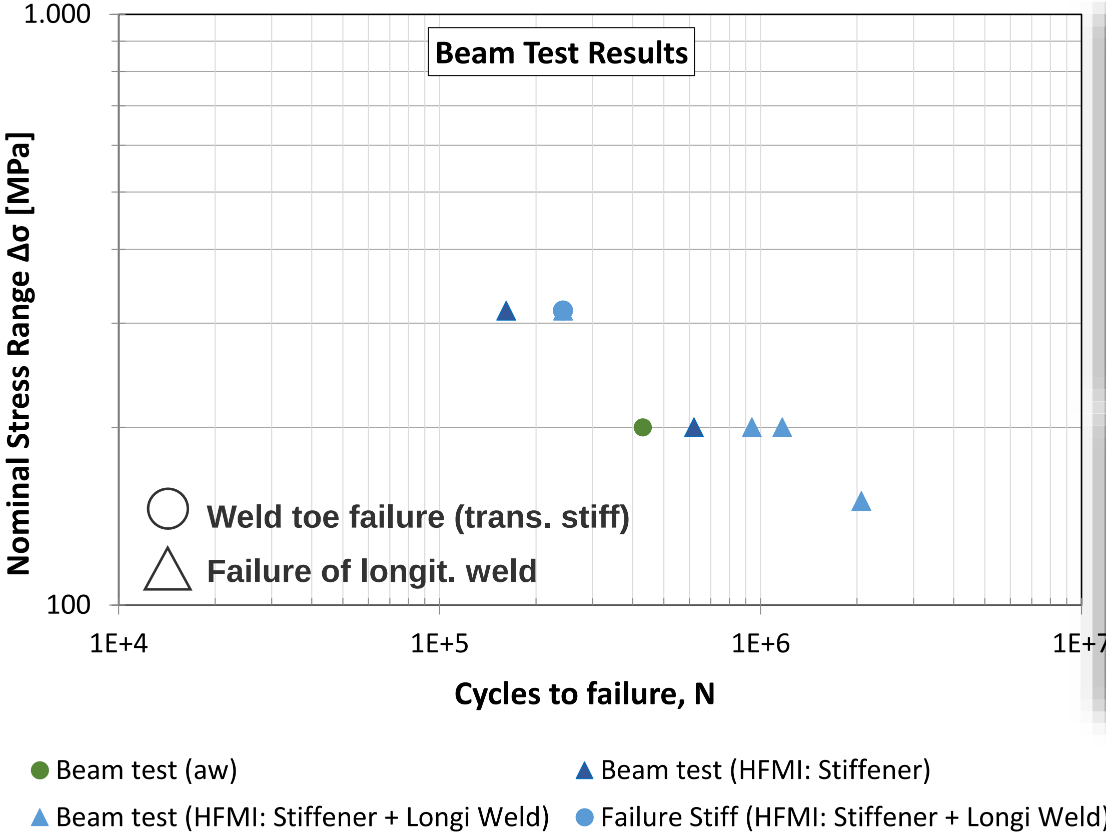
c) Results

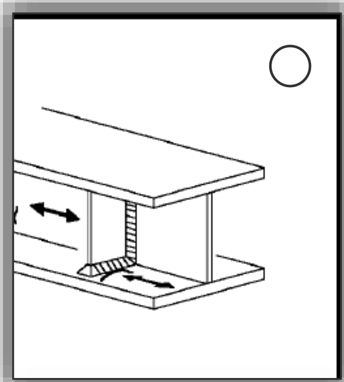
T5 and **T6** show **typical failure from weld toe** as for small scale tests and EC3-1-9

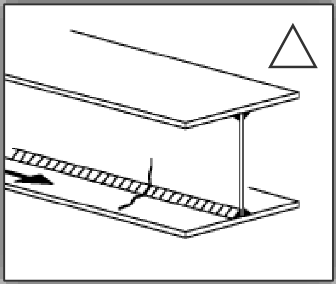


3.) Beam Tests

c) Test results – depending on **failure modes**







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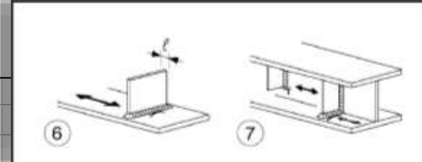
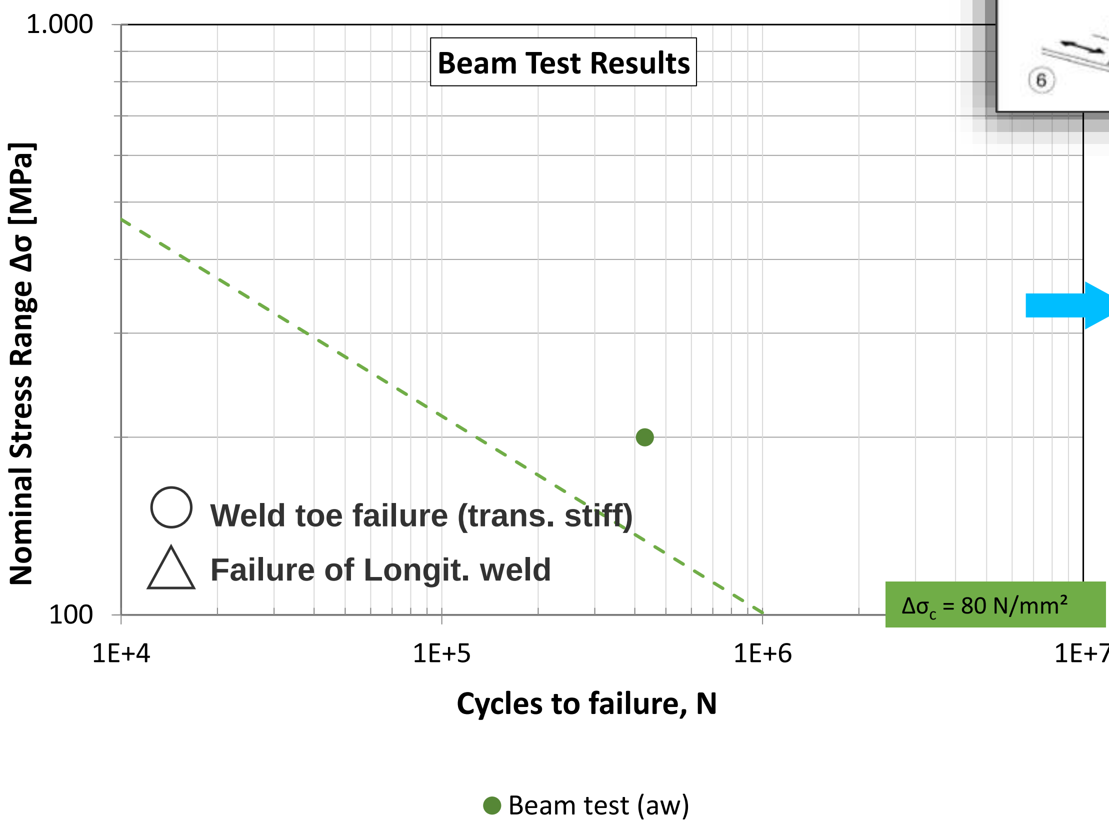
3.) Beam Tests

c) Results for **transverse stiffener**

$\Delta\sigma_c = 80\ N/mm^2$





- Very high fatigue resistance for single **aw-beam test,** compared to **FAT 80**

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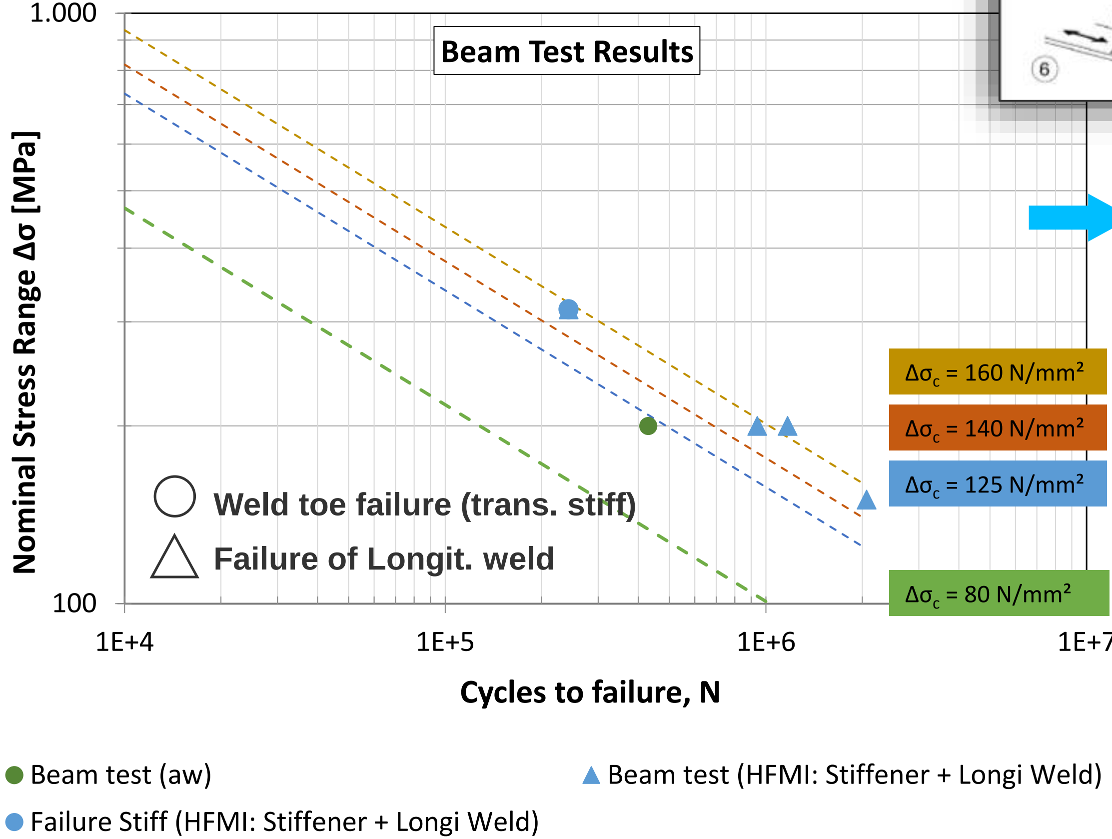
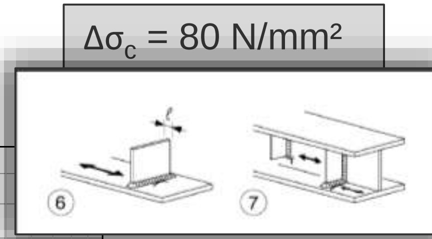
3.) Beam Tests

c) Results for **transverse stiffener**

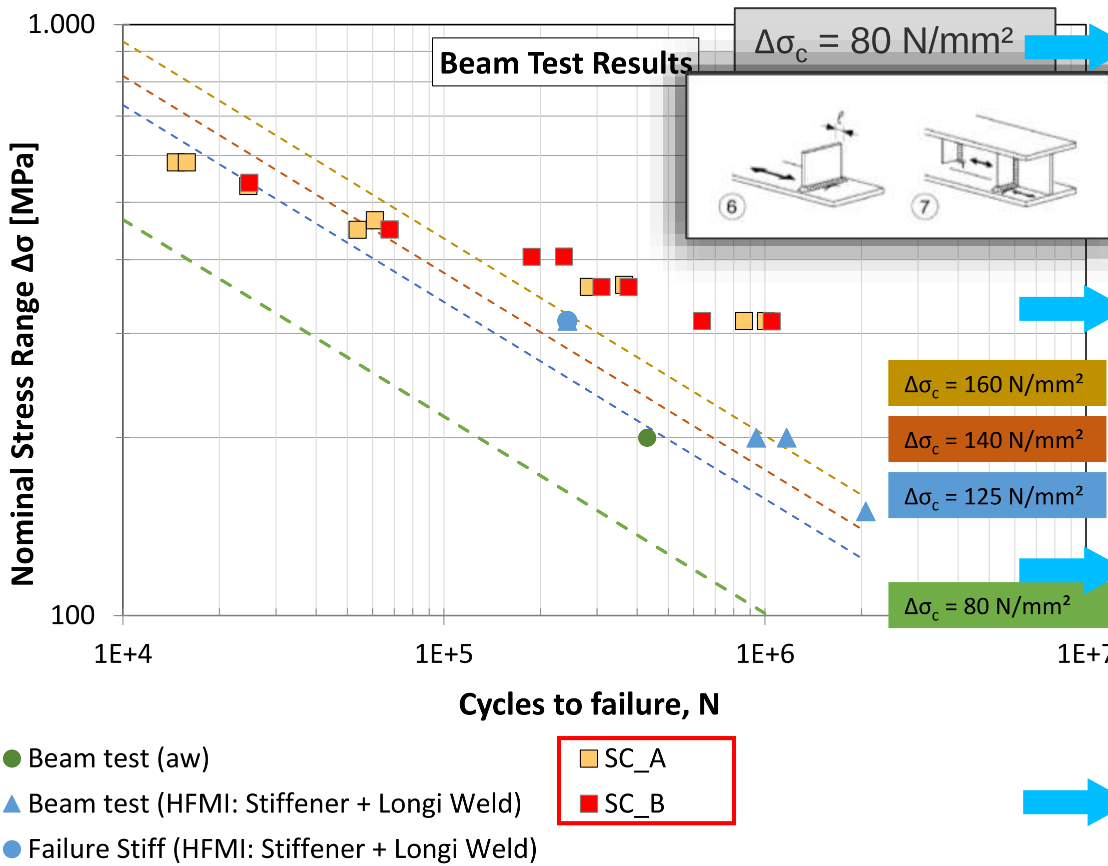




- **Positive influence** by HFMI treatment for transverse stiffener
- Results between FAT **140 and 160**

3.) Beam Tests

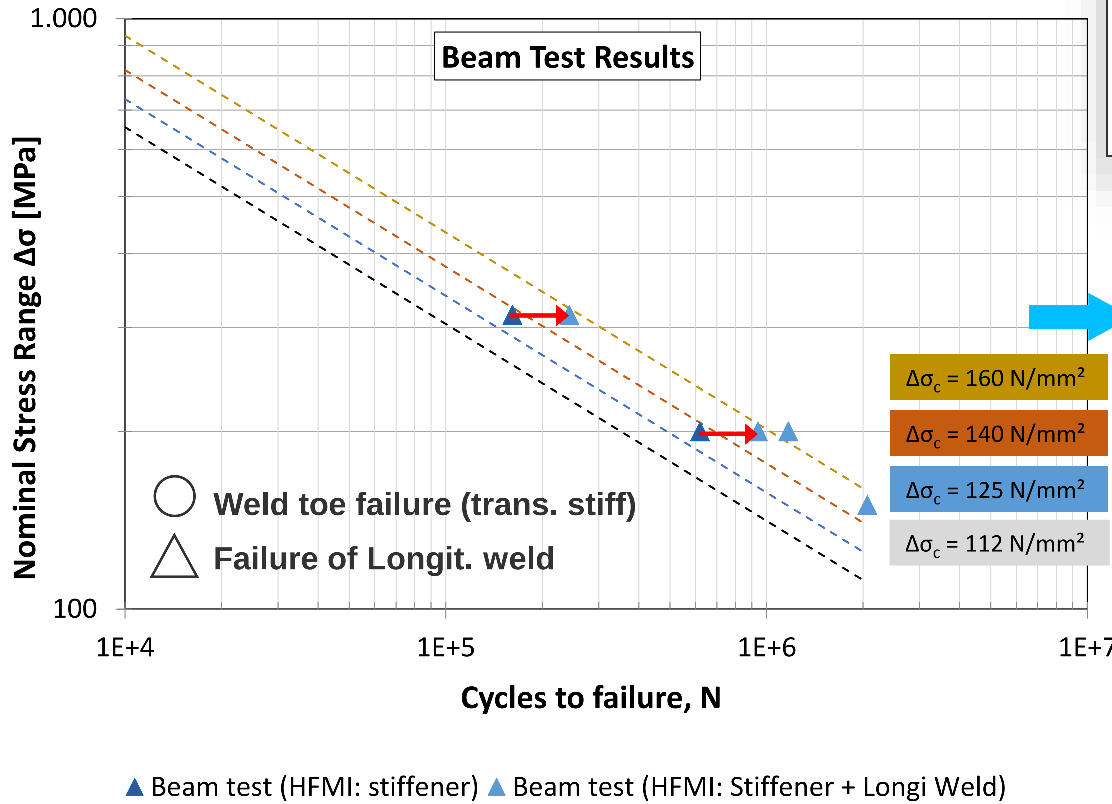
c) Results for **transverse stiffener – comparison to small scale tests**



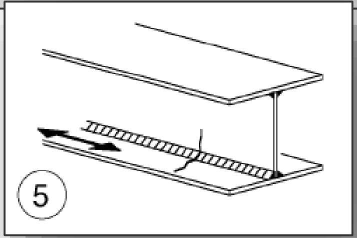
- **Comparison** to small **scale tests shows:** Beam tests have **lower fatigue resistance**
- Compared to EC3-FAT Class: improvement is valid from **around 100.000 Load Cycles**
- Due to **longitudinal failure, no statistically verified scale factor derivable**
- **Slope** seems to be close to **3** for beams

3.) Beam Tests

c) Results for **longitudinal weld failure**

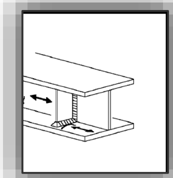


$\Delta\sigma_c$ = 112-125 N/mm²

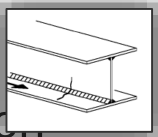


Positive influence by HFMI-treatment can be seen **for longitudinal weld failure**

4.) Conclusions and Outlook



- Effectiveness of HFMI treatment could be shown for
  - Transverse stiffener beam tests (results between FAT 140 – 160)
  - Longitudinal fillet weld
- General uncritical construction details, such as **longitudinal fillet welds,** become decisive



- There is still **improvement potential** by HFMI for construction details not yet investigated (see longitudinal fillet welds)
- **Clear** and **verified design guidelines** have to be integrated into EC 3-1-9

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Thank you for your attention!

Any questions???

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Buckling Behavior of Slender Plates under Multiaxial Stresses

OptiBri-Workshop
„Design Guidelines for Optimal Use of HSS in Bridges“

Vahid Pourostad

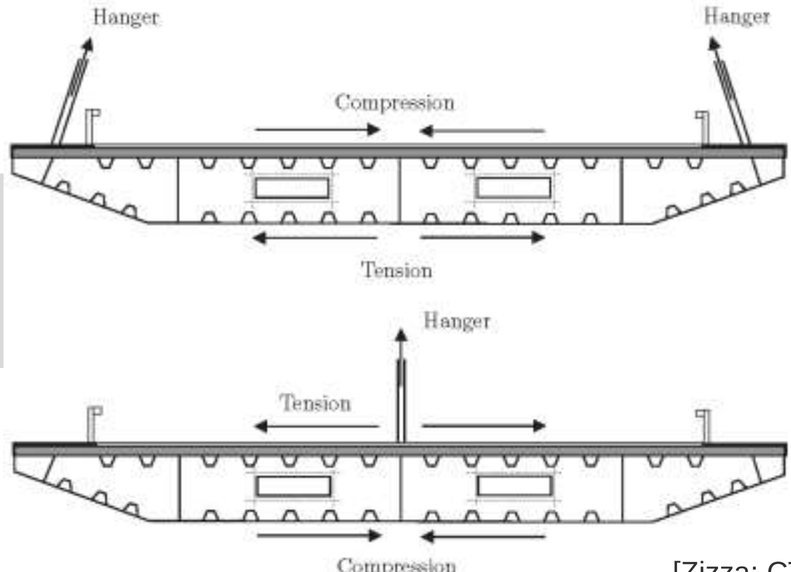
Introduction

Motivation

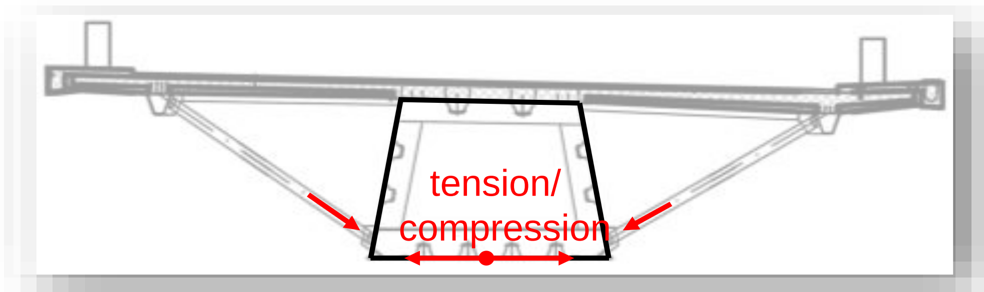
→ Biaxial stress states

tension/compression or compression/compression

Influence of tension stresses?



[Zizza; CTICM]



[Lenneteal Bridge, Hagen]

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Stability behavior of flat plates

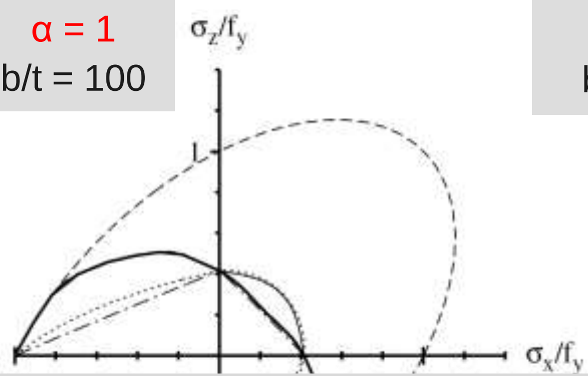
Biaxial compression

- Investigations conducted by (Braun, 2010)
- Proposal of „V-Factor“ in the domain of biaxial compression:

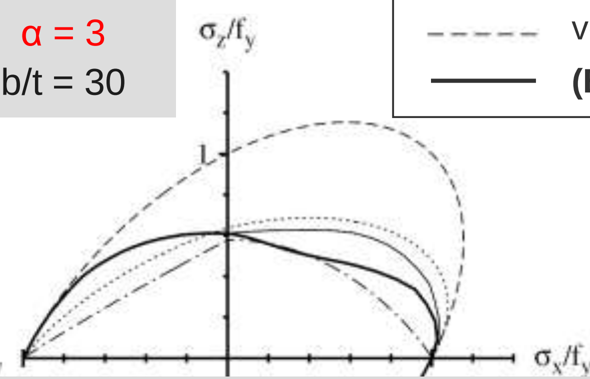
$$\left(\frac{\sigma_{xEd}}{\rho_x \cdot f_y / \gamma_{M1}}\right)^2 + \left(\frac{\sigma_{zEd}}{\rho_z \cdot f_y / \gamma_{M1}}\right)^2 - V \cdot \left(\frac{\sigma_{xEd}}{\rho_x \cdot f_y / \gamma_{M1}}\right) \cdot \left(\frac{\sigma_{zEd}}{\rho_z \cdot f_y / \gamma_{M1}}\right) \leq 1$$

DIN 18800
DnV-RP-C201
Ch. 10, EN 1993-1-5
von-MISES
(BRAUN 2010)

α = 1
b/t = 100



α = 3
b/t = 30



→ Verified by numerical calculations for biaxial compression and unstiffened plates

→ Existing EN 1993-1-5 partly unsafe. Meanwhile official amendment is added.

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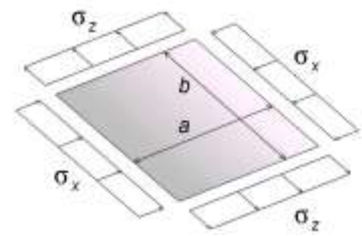
Stability behavior of flat plates

Consideration of tensile stresses?

- EN 1993-1-5, Ch. 10(5)) Note 2: …….*In case of panels with tension and compression it is recommended to apply equations (10.4) and (10.5) only for the compressive parts.*
- That means: “on the safe side the positive effect of tension stresses should be neglected when calculating the reduction factors”

$$\left(\frac{\sigma_{xEd}}{\rho_x \cdot f_y / \gamma_{M1}}\right)^2 + \left(\frac{\sigma_{zEd}}{\rho_z \cdot f_y / \gamma_{M1}}\right)^2 - \left(\frac{\sigma_{xEd}}{\rho_x \cdot f_y / \gamma_{M1}}\right) \cdot \left(\frac{\sigma_{zEd}}{\rho_z \cdot f_y / \gamma_{M1}}\right) + 3 \cdot \left(\frac{\tau_{Ed}}{\chi_w f_y / \gamma_{M1}}\right)^2 \leq 1$$



$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}}$$

→ The assumption leads to conservative results.

Stability behavior of flat plates

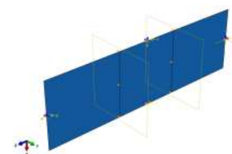
Investigation in the frame of OptiBri

Buckling verification becomes more important for HSS plates:

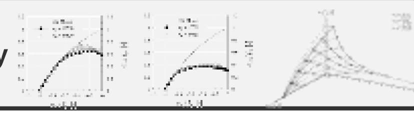
AIM: To allow for taking account of positive effects of tension stresses

WP3.1 Experimental Investigations

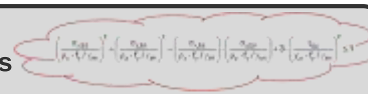
WP3.1 Numerical model



WP3.2 Parametric study



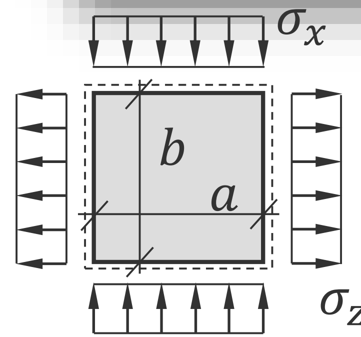
WP3.3 Evaluation of behavior and development design rules

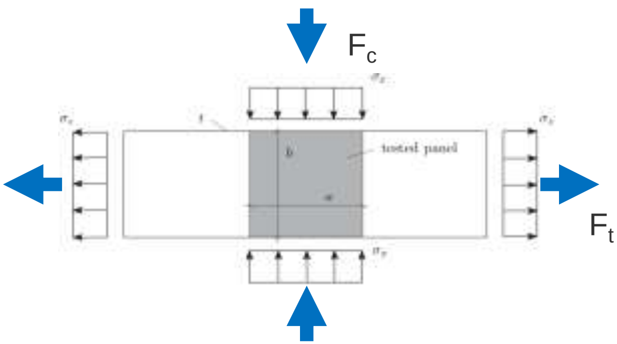


Experimental investigations

Test program and setup

Test	A1	A2	A3	B1	B2	B3
a [mm]	900	900	900	1500	1500	1500
b [mm]	900	900	900	500	500	500
α	1	1	1	3	3	3
t [mm]	6	6	6	6	6	6
b/t	150	150	150	83	83	83
β	0	-0.25	-0.5	0	-1.5	-1



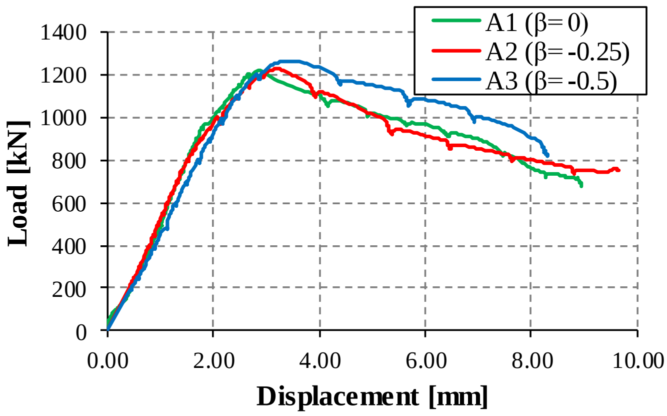


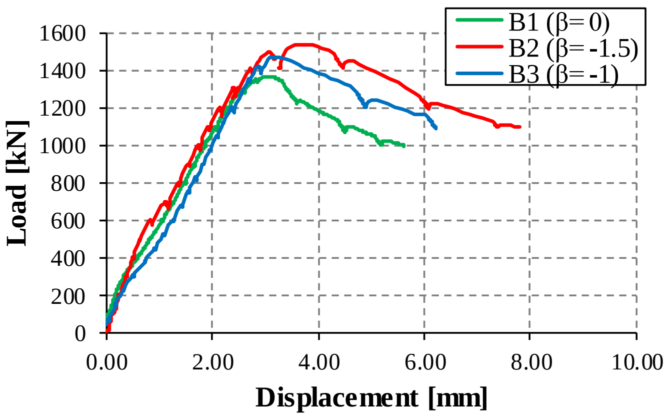
- Variation of aspect-ratio α and slenderness $\frac{b}{t}$
- Variation of stress-ratio $\beta = \frac{\sigma_z}{\sigma_x}$
- Material: S690

→ β = 0 as reference tests for the evaluation of the influence of tension stresses

Experimental investigations

Test results





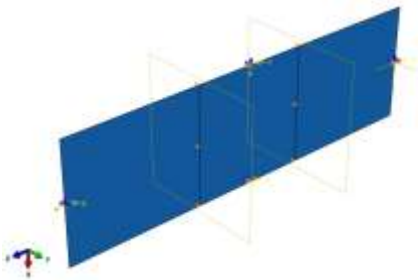
→ Evaluations show increase of loading capacity by increased tension stresses

→ Evaluation of the deformations shows the influence of tension stresses on the buckling shape

Numerical investigation

Recalculations of tests

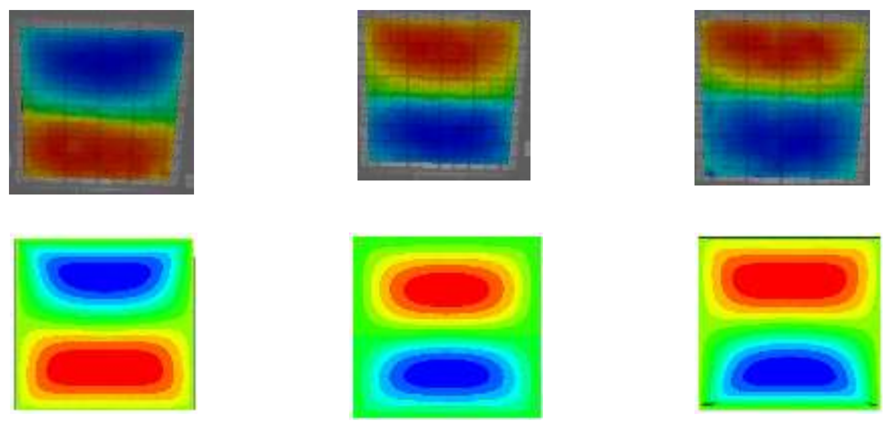
Numerical model in ABAQUS



→ The numerical model has been developed using the material curve from tensile tests and the measured imperfections

Comparison of failure modes

	A1	A2	A3
Experiment			
Numerical model			



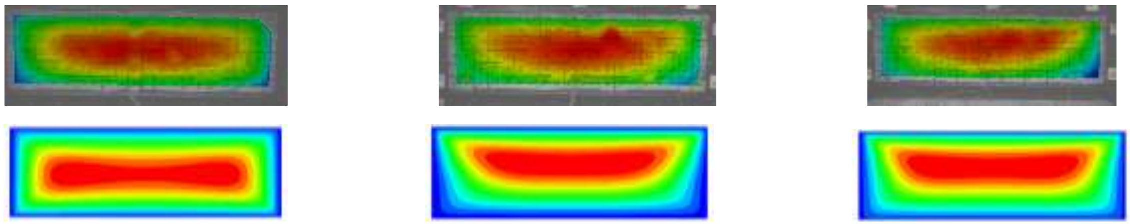
Numerical investigation

Recalculations of tests

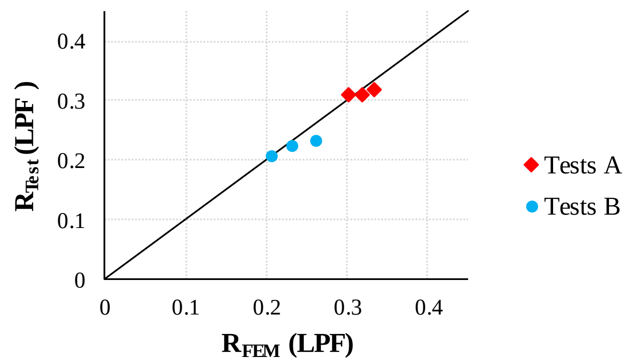
B1 B2 B3

Experiment

Numerical model



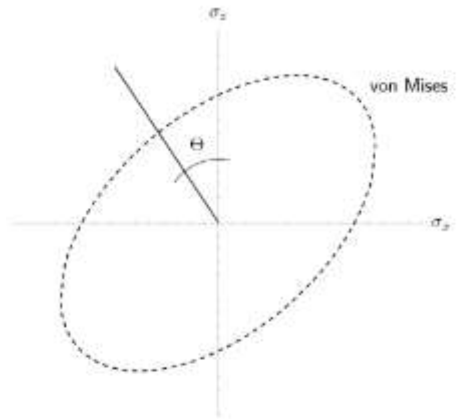
→ Good agreement between numerical and experimental buckling shapes



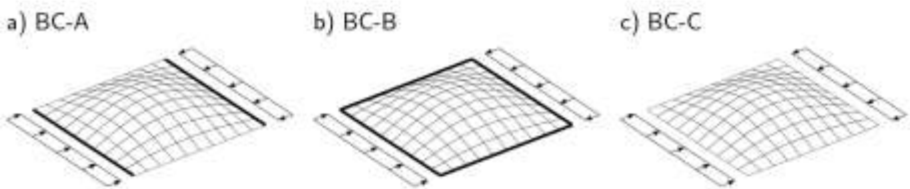
→ Good agreement between numerical and experimental ultimate load

Parametric study

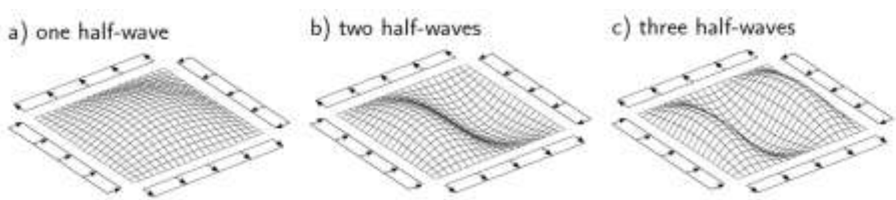
Parametric study using ABAQUS (WP3.2)



- Influence of b/t- and aspect-ratio
- Influence tension stresses on compression
- Influence of boundary conditions



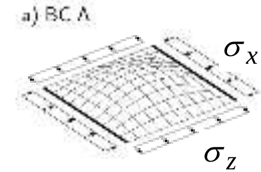
- Influence of imperfection shape and amplitude



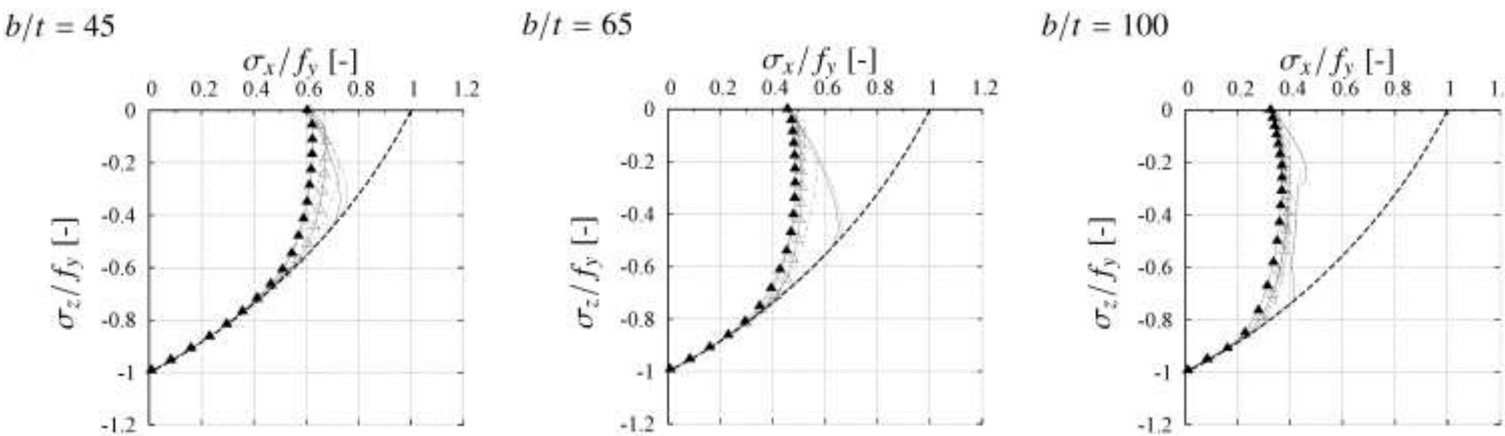
→ Calculations for S690

Parametric study

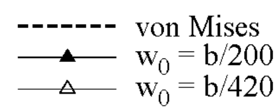
Parametric Study on Square Plates



- Investigated parameters:
  - **interaction angle $\theta = \tan^{-1}\frac{\sigma_z}{\sigma_x}$**
  - **b/t-ratio**
  - **imperfection shape and amplitude**
  - boundary conditions



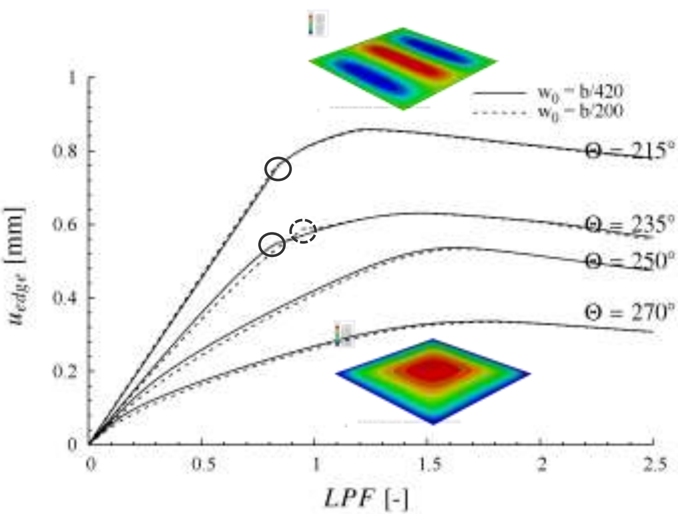
→ Lower envelope considers decisive imperfection shape

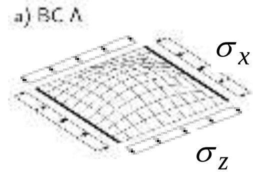


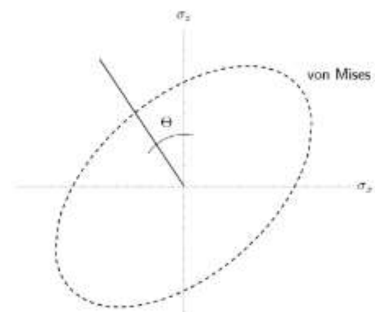
Parametric study

Parametric Study on Square Plates

- Investigated parameters:
  - interaction angle θ
  - b/t-ratio
- **imperfection shape and amplitude**
- boundary conditions







BC-A; α=1; b/t=100
1 half-wave imperfection shape

→ With increasing tension buckling shape changes from one half-wave to 3 half-wave

Enhancement of the reduced stress method

For compression-tension

- Verification formula acc. to EN 1993-1-5 for direct stresses:

$$\left(\frac{\sigma_{xEd}}{\rho_x \cdot f_y / \gamma_{M1}}\right)^2 + \left(\frac{\sigma_{zEd}}{\rho_z \cdot f_y / \gamma_{M1}}\right)^2 - \left(\frac{\sigma_{xEd}}{\rho_x \cdot f_y / \gamma_{M1}}\right) \cdot \left(\frac{\sigma_{zEd}}{\rho_z \cdot f_y / \gamma_{M1}}\right) \leq 1$$

- Modification of verification formula with „V-Factor“ for direct stresses:

$$\left(\frac{\sigma_{xEd}}{\rho_x \cdot f_y / \gamma_{M1}}\right)^2 + \left(\frac{\sigma_{zEd}}{\rho_z \cdot f_y / \gamma_{M1}}\right)^2 - V \cdot \left(\frac{\sigma_{xEd}}{\rho_x \cdot f_y / \gamma_{M1}}\right) \cdot \left(\frac{\sigma_{zEd}}{\rho_z \cdot f_y / \gamma_{M1}}\right) \leq 1$$

- „V-Factor“ in case of biaxial compression proposed by (Braun, 2010)

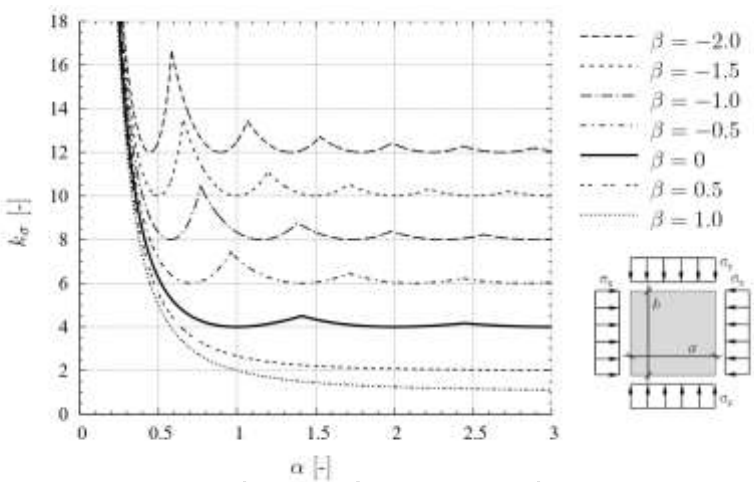
$$V = \rho_{c,x} \cdot \rho_{c,z}$$

- „V-Factor“ in case of compression-tension proposed by (Zizza, 2016)

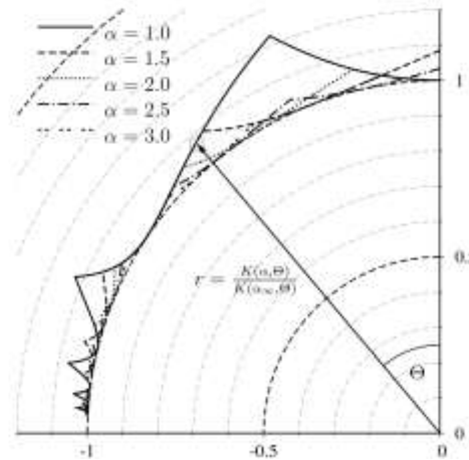
$$V = 1/(\rho_{c,x} \cdot \rho_{c,z}{}^{2-\xi_z})$$

Enhancement of the reduced stress method

Proposed formula for calculation of buckling coefficient by Zizza for interaction of tension and compression



compression and compression



tension and compression

$$\beta = \frac{\sigma_z}{\sigma_x}$$

→ Neglecting the peaks in the calculation of the buckling coefficient for tension compression using:

$$k_\sigma^{\min} = 4(1-\beta)$$

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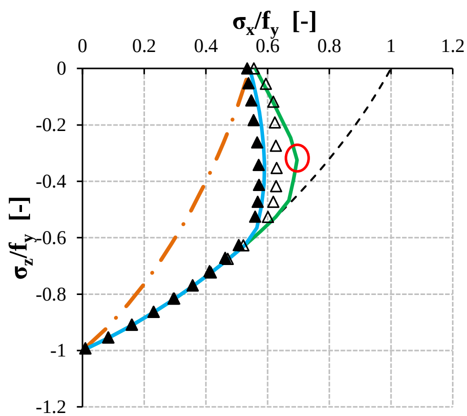
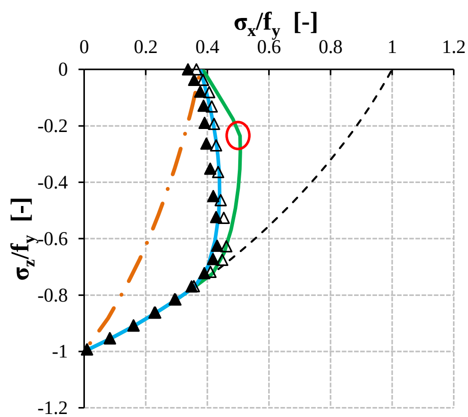
Enhancement of the reduced stress method

Comparison of current design rules with proposed V-Factor (tension-compression)

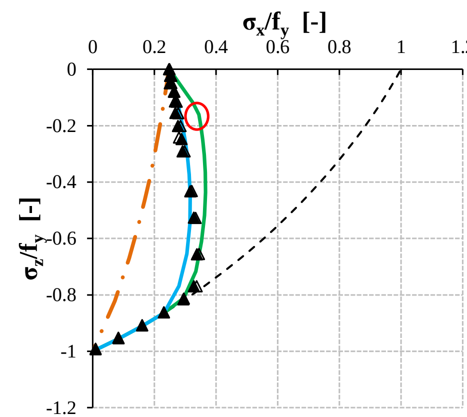
b/t=45



b/t=65



b/t=100



MISES

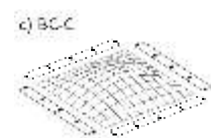
FE (w0=b/200)

FE (w0=b/420)

EN 1993-1-5:2006, neglecting tension stresses (without V factor)

EN 1993-1-5:2006, considering tension stresses (without V factor)

Proposal of Zizza (with V factor)



→ Current design rules neglecting tension stresses lead to conservative results

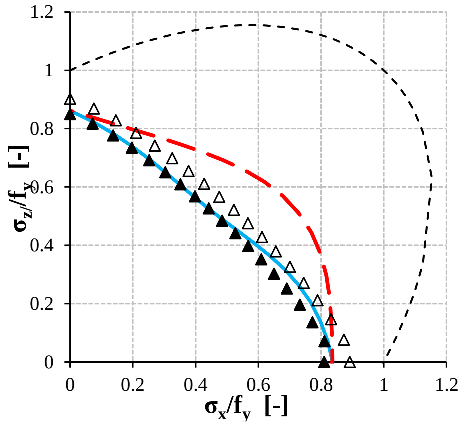
→ Current design rules applying tension for calculating ρ without V factor partially on unsafe side

→ Proposal considering of V factor and neglecting the peak of buckling coefficient leads to good results

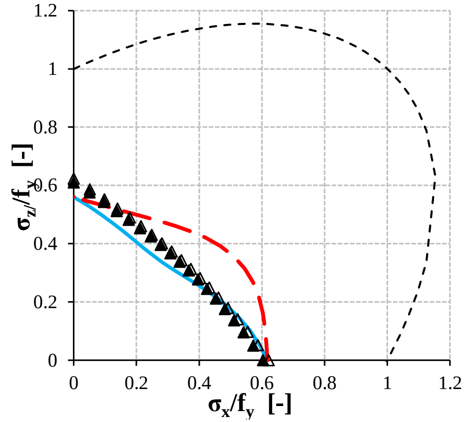
Enhancement of the reduced stress method

Comparison of current design rules with proposed V-Factor (compression-compression)

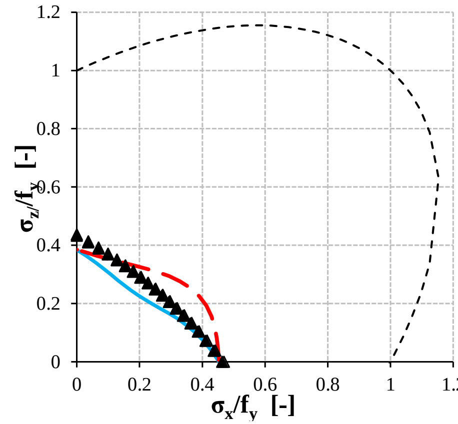
b/t=30



b/t=45



b/t=65



→ Proposed verification with V factor corresponds very well to FE results

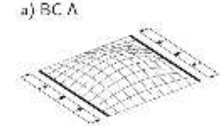
- - - - MISES

▲ FE (w0=b/200)

△ FE (w0=b/420)

EN 1993-1-5:2006 (without V factor)

Proposal of Braun (with V factor)



Flowchart of using MRS (sec. 10)

[reference numbers refer to EN 1993-1-5]

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{crit}}}$$

[eq. (10.2)]

For transverse stresses (z-direction)

$$\varphi_p = 0{,}5 \cdot \left(1 + \alpha_p \cdot (\bar{\lambda}_p - \bar{\lambda}_{p0}) + \bar{\lambda}_p\right)$$

[Annex B]

$$\rho_{p,z} = \frac{1}{\varphi_p + \sqrt{{\varphi_p}^2 - \bar{\lambda}_p}}$$

[Annex B eq.(B.1)]

$$\xi_z = \frac{\sigma_{cr,p,z}}{\sigma_{cr,c,z}} - 1$$

[4.5.4]

$$\rho_{c,z} = (\rho_{pz} - \chi_c) \cdot \xi_z \cdot (2 - \xi_z) + \chi_c$$

[4.5.4(1) eq.(4.13)]

Stiffened plates

$$\alpha_e = \alpha + \left(\frac{0{,}09}{\frac{i}{e}}\right)$$

[4.5.3 (5) eq.(4.12)]

$$\Phi = 0{,}5 \cdot \left(1 + \alpha_e \cdot (\bar{\lambda}_p - 0{,}2) + \bar{\lambda}_p^2\right)$$

[EN 1993-1-1, 6.3.1.2]

$$\chi_c = \frac{1}{\phi + \sqrt{(\phi^2 - \bar{\lambda}_p^2)}}$$

[EN 1993-1-1, 6.3.1.2]

For longitudinal stresses

(x-direction)

$$\rho_{p_x} = \frac{\bar{\lambda}_p - 0{,}055 \cdot (3 + \psi)}{\bar{\lambda}_p^2}$$

[4.4(2) eq.(4.2)]

$$\xi_x = \frac{\sigma_{cr,p,x}}{\sigma_{cr,c,x}} - 1$$

[4.5.4]

$$\rho_{c,x} = (\rho_{px} - \chi_c) \cdot \xi_x \cdot (2 - \xi_x) + \chi_c$$

[4.5.4(1) eq.(4.13)]

Shear stresses

$$\chi_w$$

[Tabelle 5.3]

$$\sqrt{\left(\frac{\sigma_{x,Ed}}{\rho_{c,x} \cdot f_y/\gamma_{M1}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{\rho_{c,z} \cdot f_y/\gamma_{M1}}\right)^2 - V \cdot \left(\frac{\sigma_{x,Ed}}{\rho_{c,x} \cdot f_y/\gamma_{M1}}\right) \cdot \left(\frac{\sigma_{z,Ed}}{\rho_{c,z} \cdot f_y/\gamma_{M1}}\right) + 3 \cdot \left(\frac{\tau_{Ed}}{\chi_w \cdot f_y/\gamma_{M1}}\right)^2} \leq 1$$

[10(5), eq. (10.5)]

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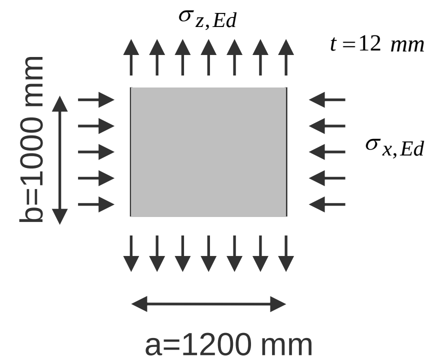
Example

Panels subjected to tension and compression



Acting stresses:

$$\sigma_{x,Ed} = 242\ N/mm^2 \qquad \sigma_{z,Ed} = -89\ N/mm^2$$

Equivalent stress:

$$\sigma_v = \sqrt{(\sigma_{y,Ed})^2 + (\sigma_{z,Ed})^2 + (\sigma_{x,Ed})\cdot(\sigma_{z,Ed})} = 296.69\ N/mm^2$$

Buckling value acc. to proposal of Zizza:

$$\beta = \frac{\sigma_{z,Ed}}{\sigma_{x,Ed}} = \frac{-89}{242} = -0.3678 \Rightarrow k_\sigma = k_\sigma^{\min} = 4(1-(-0{,}368)) = 5.47$$

Elastic critical plate-buckling stress and column-buckling stress

$$\sigma_e = 27.33\ N/mm^2 \Rightarrow \sigma_{cr,p} = k_\sigma \cdot \sigma_e = 5.47 \cdot 27.33 = 149.6\ N/mm^2$$

$$\sigma_{cr,c,x} = \frac{\pi^2 E t^2}{12(1-\upsilon^2)a^2} = 18.98\ N/mm^2$$

Slenderness and reduction factors:

$$\alpha_{cr} = \frac{\sigma_{cr,p}}{\sigma_{x,Ed}} = \frac{149.6}{242} = 0.618 \Rightarrow \alpha_{ult} = \frac{f_y}{\sigma_v} = \frac{690}{296.69} = 2.326 \Rightarrow \overline{\lambda}_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 1.94 \Rightarrow \rho_x = \frac{\overline{\lambda}_p - 0.055\cdot(3+\psi)}{\overline{\lambda}_p^2} = 0.457$$

$$\xi_x = \frac{\sigma_{cr,p,x}}{\sigma_{cr,c,x}} - 1 \geq 1 \Rightarrow \xi_x = 1 \Rightarrow \rho_{c,x} = \rho_x = 0.457$$

$$\rho_{c,z} = 1$$

Example

Panels subjected to tension and compression

Verification acc. to proposal of Zizza:

$$V = 1/\left(\rho_{c,x} \cdot \rho_{c,z}{}^{2-\xi_z}\right) = 1/(0.457) = 2.188$$

$$\eta = \sqrt{\left(\frac{242}{0.457 \cdot 690/1.1}\right)^2 + \left(\frac{-89}{1 \cdot 690/1.1}\right)^2 - 2.188 \cdot \left(\frac{242}{0.457 \cdot 690/1.1}\right) \cdot \left(\frac{-89}{1 \cdot 690/1.1}\right)} = 1 \leq 1$$

Comparison of proposal and current design rule with required thickness of panel

	EC1993-1-5:2006	Proposal of Zizza	EC1993-1-5:2006	Proposal of Zizza
Steel	S690	S690	S355	S355
t [mm]	14	12	24.7	21.2
η	1	1	1	1

→ Proposed verification considering tension stresses and V factor leads to efficient design of the panels

Summary and Outlook

- Six tests have been conducted and recalculated using the FEM
- Tension stresses may change the failure mode of a square panel from one half-wave into more half-waves.
- Tension stresses increase the buckling resistance of the panels.
- The ultimate loads acc. to Sec 10, EN 1993-1-5 with considering the positive effect of tension stresses on the reduction factors and proposed V factor by Zizza, enhance the accuracy of the “reduced stress method” and leads to more efficient design of the panels.

Outlook

- Extension of the numerical investigations for interaction of tension and shear
- Investigations on stiffened plates

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Germany

Thank you for your attention!

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Improved Bridge Design by Use of High Strength Steel (HSS) with OPTIBRI developments

OptiBri-Workshop

„Design Guidelines for Optimal Use of HSS in Bridges“

Cláudio Baptista

OVERVIEW OF IMPROVEMENTS (DESIGN A TO B)

- **Design A –** S355 NL (current Eurocode versions)
- **Design B –** S690 QL (current Eurocode versions)
- **Design C –** S690 QL (upgrade Eurocode versions)

Direct Improvements for Bridge Design:

- **Reduction of maximum steel plate thickness: 120 to 70 mm**
- **Reduction of the welding volume: 65%**
- **Reduction of overall steel weight: 25%**

However…

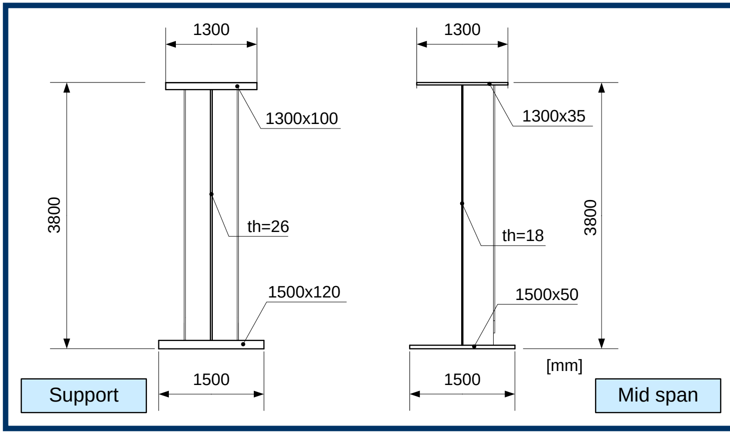
- **Fatigue has become the critical ULS check !**

OVERVIEW OF IMPROVEMENTS (DESIGN A TO B)

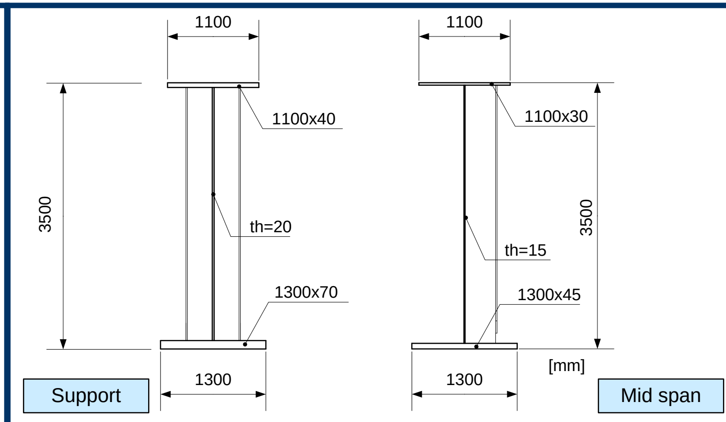
- **Design A –** S355 NL (current Eurocode versions)
- **Design B –** S690 QL (current Eurocode versions)
- **Design C –** S690 QL, (upgrade Eurocode versions)

Direct Improvements for Bridge Design:

- **Reduction of maximum steel plate thickness: 120 to 70 mm**



Design A – S355 NL

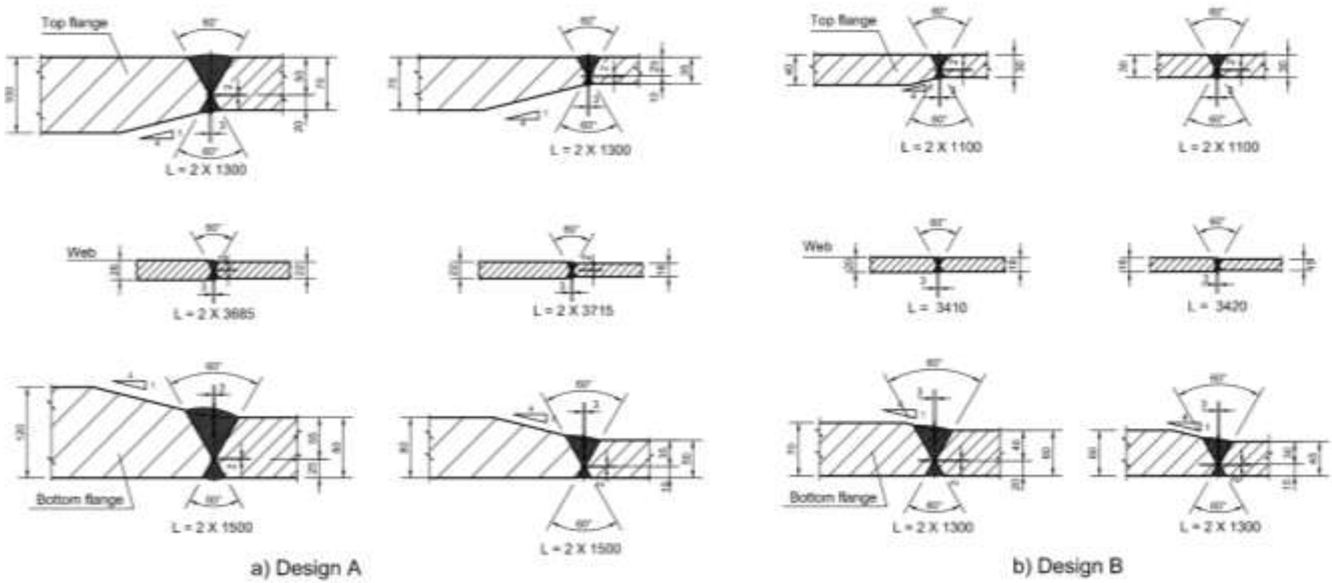


Design B – S690 QL

Research Fund for Coal & Steel

OVERVIEW OF IMPROVEMENTS (DESIGN A TO B)

Direct Improvements for Bridge Design:



Reduction of the welding volume: 65%

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OVERVIEW OF IMPROVEMENTS (DESIGN A TO B)

Direct Improvements for Bridge Design:

- **Reduction of brittle failure risk**

Design A (S355NL): $\sigma_{Ed} = 0.63\ f_y(t)$ -- t_{max} = 92mm (t=120mm)

Design B (S690QL1): $\sigma_{Ed} = 0.53\ f_y(t)$ -- t_{max} = 63mm (t=70mm)

Steel Class	Quality	KV		σ_{Ed} = 0,75 fy(t)	σ_{Ed} = 0,50 fy(t)	σ_{Ed} = 0,25 fy(t)
		at T [°C]	Jmin			
S355	J2	-20	27	40	65	110
	K2,M,N	-20	40	50	80	130
	ML,NL	-50	27	75	110 (120)	175
S690	QL	-20	40	25	45	85
	QL1	-40	40	40	65 (70)	120
	QL1	-60	30	50	80	140

Reference temperature: Tref = -30°C

Design A: t ≤ 120 mm (S355 NL); Design B: t ≤ 70 mm (S690 QL1, 40J at -40°C)

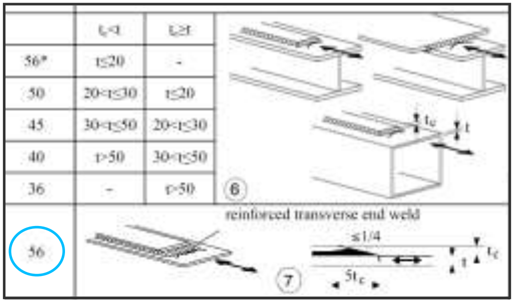
ULS BRIDGE DECK BENDING DESIGN

Mid-span section Class 1 – *Plastic section analysis*	Design A S355	Design B S690
$M_{\mathrm{Ed}} / M_{\mathrm{pl.Rd}} < 1$	0.74	0.54
$\{\sigma_{\mathrm{Ed}} / (f_{\mathrm{yf}} / \gamma_{\mathrm{M0}})\}_{\text{Bottom flange}} < 1$	0.93	0.65

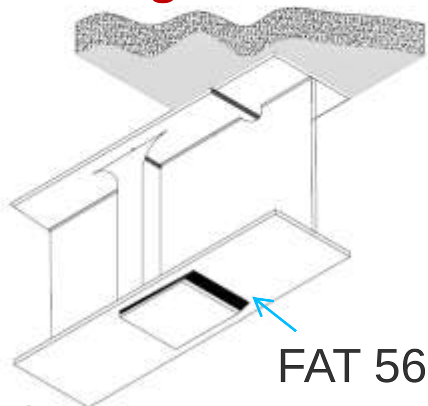
Support section Class 4 – *Elastic analysis with effective section*	Design A S355	Design B S690
$\{\sigma_{\mathrm{Ed}} / (f_{\mathrm{yf}} / \gamma_{\mathrm{M0}})\}_{\text{Eff. bottom flange}} < 1$	0.95	0.88
$\{\sigma_{\mathrm{Ed}} / (\chi_{\mathrm{LT}} f_{\mathrm{yf}} / \gamma_{\mathrm{M1}})\}_{\text{Eff. bottom flange}} < 1$ (*) (*) at 0.25 L_k = 5 m from the support	0.92	0.97

CRITICAL FAT DETAILS (DESIGN A to C)

Bottom flange at support bearings:

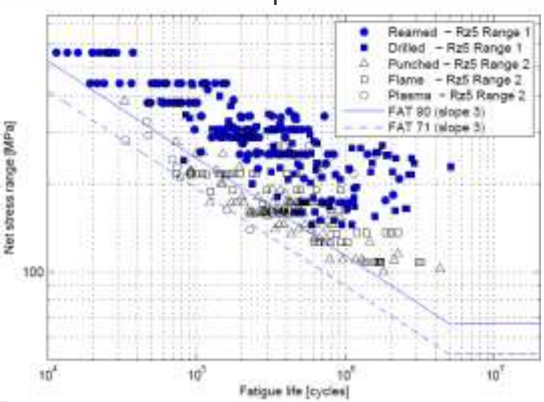


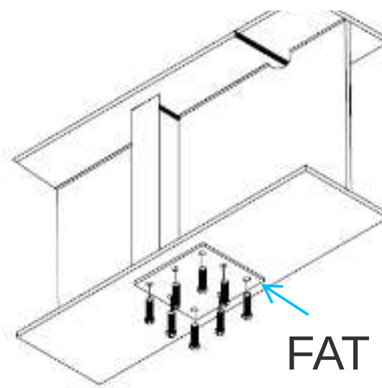
	$t_c<t$	$t_c \geq t$
56*	$t \leq 20$	-
50	$20<t\leq 30$	$t \leq 20$
45	$30<t\leq 50$	$20<t\leq 30$
40	$t>50$	$30<t\leq 50$
36	-	$t>50$
56	reinforced transverse end weld	

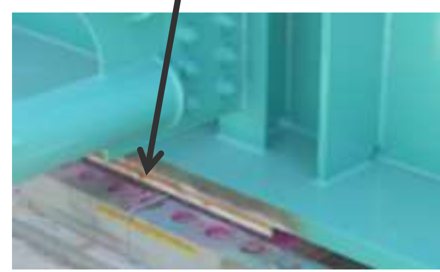


$\Delta s_{E,2}$=50 MPa (Design B S690)

SN curve for plates with holes



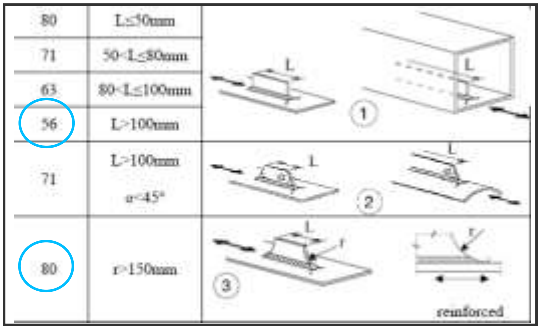




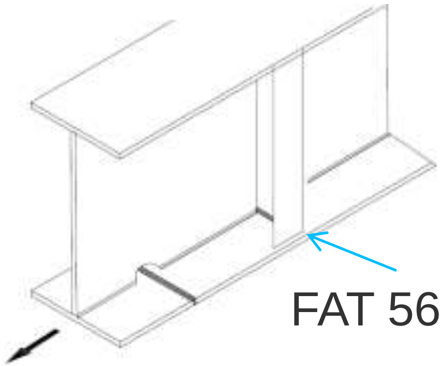
Solution: Bolted Plates

CRITICAL FAT DETAILS (DESIGN A to C)

Transversal stiffener at support:



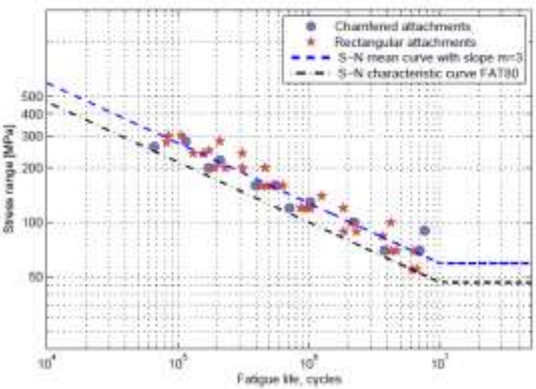
80	L≤50mm	①
71	50<L≤80mm	
63	80<L≤100mm	
56	L>100mm	
71	L>100mm α<45°	②
80	r>150mm	③ reinforced

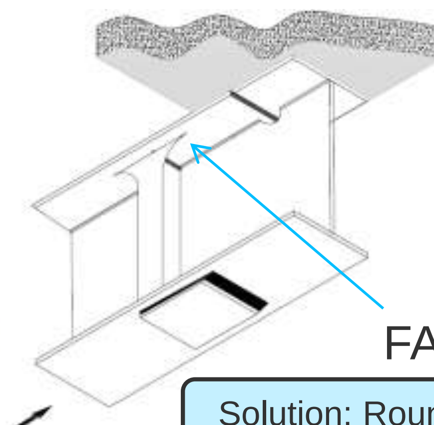


$\Delta s_{E,2}$=50 MPa (Design B S690)

FAT 56

SN curve for plates rounded long attachments





FAT 80

Solution: Rounded stiffener flange

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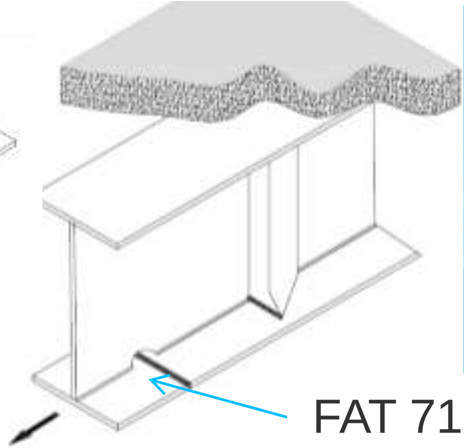
CRITICAL FAT DETAILS (DESIGN A to C)

Cope holes on main beams:



$\Delta s_{E,2}$=57 MPa (Design A S355)

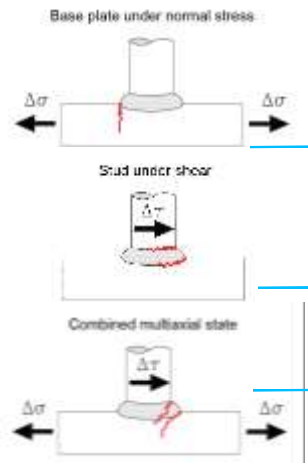
$\Delta s_{E,2}$=78 MPa (Design B S690)

Detail category	Constructional details		Description	Requirements
71	$\Delta\tau/\Delta\sigma$ =	0	10) Longitudinal butt weld, fillet weld or intermittent weld with cope holes, cope holes not higher than 40 % of web.	10) $\Delta\sigma$ and $\Delta\tau$ based on normal stress in flange σ and shear stress τ in web and weld ends.
63		0,0 – 0,2		
56		0,2 – 0,3		
50		0,3 – 0,4		
45		0,4 – 0,5		
40		0,5 – 0,6		
36		> 0,6		

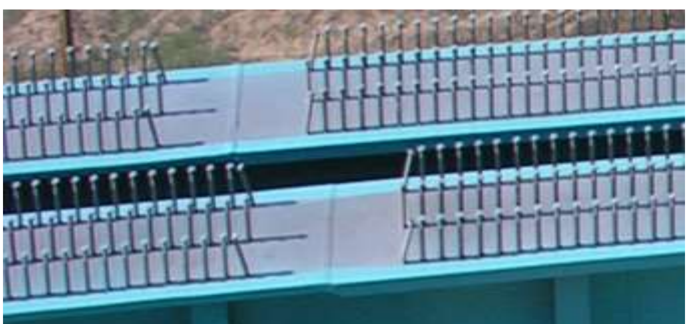
Solution: Avoid Cope holes

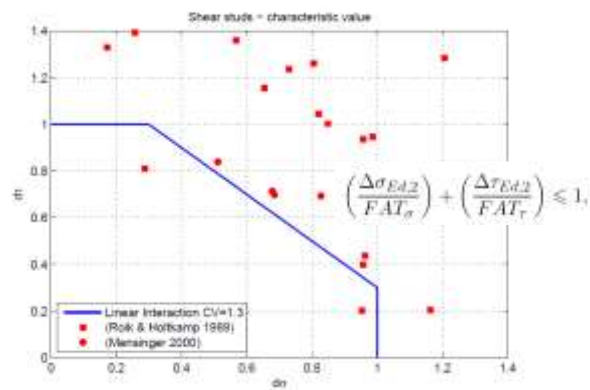
CRITICAL FAT DETAILS (DESIGN A to C)

Shear connectors:



Fatigue check on shear connectors											
Section (X)	1 (0.0)	2 (4.0)	3 (8.0)	4 (12.0)	5 (16.0)	6 (20.0)	7 (24.0)	8 (28.0)	9 (32.0)	10 (36.0)	11 (40.0)
Direct stress on the top flange (FAT 80 m=3)											
$\Delta\sigma_{E,2}$ (MPa)	27.0	23.1	18.6	21.4	15.5	13.3	13.3	5.1	5.1	5.2	5.1
$\frac{\gamma_{Ff}\Delta\sigma_{E,2}}{\Delta\sigma_c/\gamma_{Mf}}$	0.46	0.39	0.31	0.36	0.26	0.23	0.22	0.09	0.09	0.09	0.09
Shear stress on the stud (FAT 90 m=8)											
$\Delta\tau_{E,2}$ (MPa)	16.1	15.5	22.7	21.2	20.2	20.4	33.7	33.5	32.8	32.9	32.7
$\frac{\gamma_{Ff}\Delta\tau_{E,2}}{\Delta\tau_c/\gamma_{Mf,s}}$	0.31	0.30	0.43	0.40	0.38	0.39	0.64	0.64	0.62	0.62	0.62
Interaction: $\frac{\gamma_{Ff}\Delta\sigma_{E,2}}{\Delta\sigma_c/\gamma_{Mf}} + \frac{\gamma_{Ff}\Delta\tau_{E,2}}{\Delta\tau_c/\gamma_{Mf,s}} \leq 1.3$											
Int.exp.≤1.3	0.76	0.69	0.75	0.76	0.65	0.61	0.86	0.72	0.71	0.71	0.71





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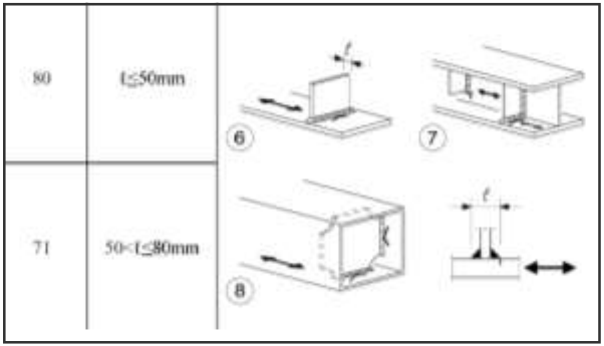
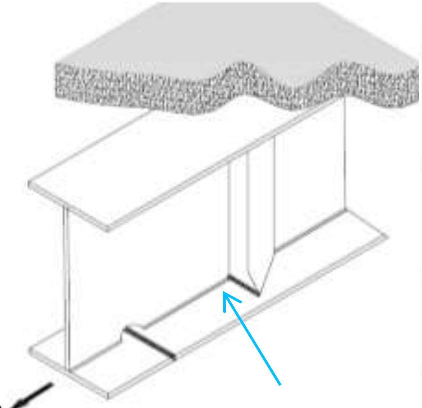
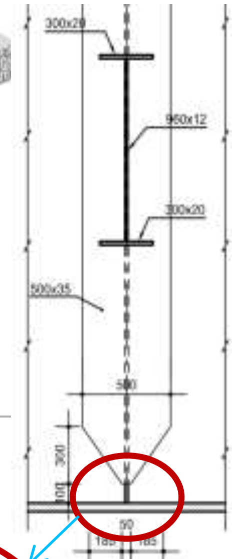
CRITICAL FAT DETAILS (DESIGN A to C)

Transversal attachment on bottom flange of main beams:



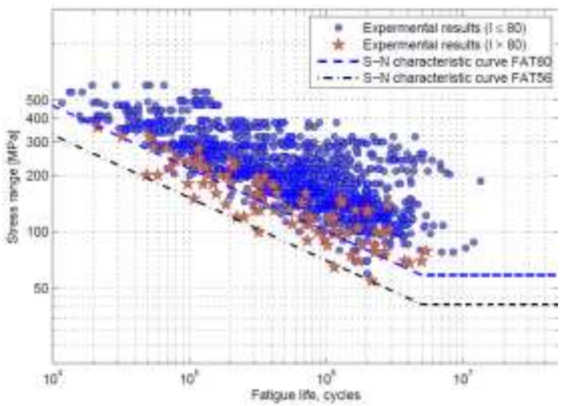


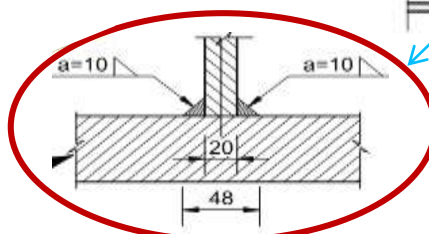


$\Delta s_{E,2}$=78 MPa (Design B S690)

FAT 80

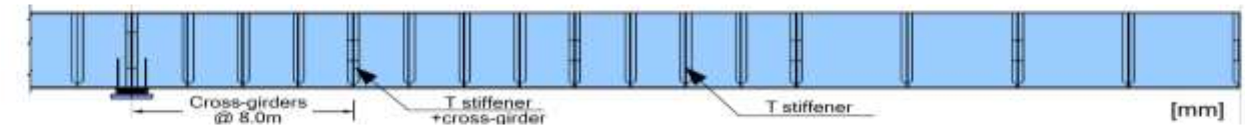
SN curve for transversal attachments





Solution: Detail FAT 80 is unavoidable !

OVERVIEW OF FAT CHECK

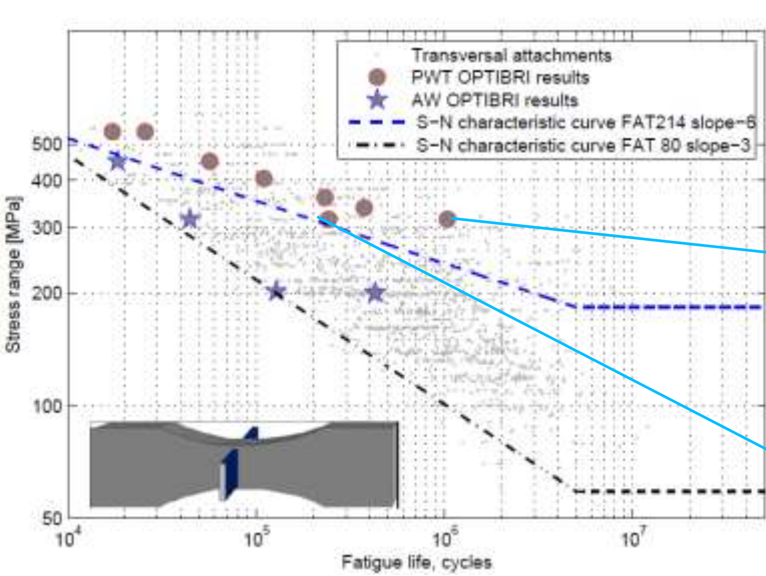


Design A											
Section (X)	1 (0.0)	2 (4.0)	3 (8.0)	4 (12.0)	5 (16.0)	6 (20.0)	7 (24.0)	8 (28.0)	9 (32.0)	10 (36.0)	11 (40.0)
t_f (mm)	120	120	120/80	80	80	80/50	50	50	50	50	50
$\gamma_{Mf}\ \gamma_{Ff}\ \Delta\sigma_{E,2}$	25.5	21.7	32.1	36.5	29.9	47.6	51.0	53.0	56.9	57.4	56.6
FAT	56	80	80	80	80	80	80	80	80	80	80
Design B											
t_f (mm)	70	70	70/60	60	60	60/45	45	45	45	45	45
$\gamma_{Mf}\ \gamma_{Ff}\ \Delta\sigma_{E,2}$	49.8	42.7	57.9	65.8	55.2	66.9	71.9	70.7	76.5	77.8	77.1
FAT	56	80	80	80	80	80	80	80	80	80	80

- HSS allows for 34% overall steel reduction in main beams
- Fatigue becomes the leading ULS check at span sections

CRITICAL FAT DETAILS (DESIGN A to C)

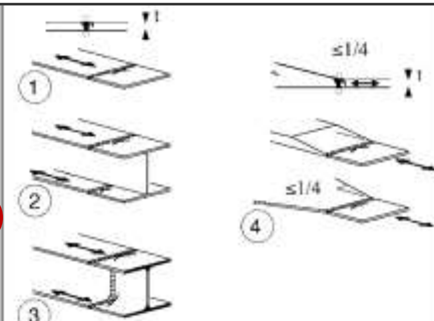
PWT Transversal attachment (beams and plates):



CRITICAL FAT DETAILS (DESIGN A to C)

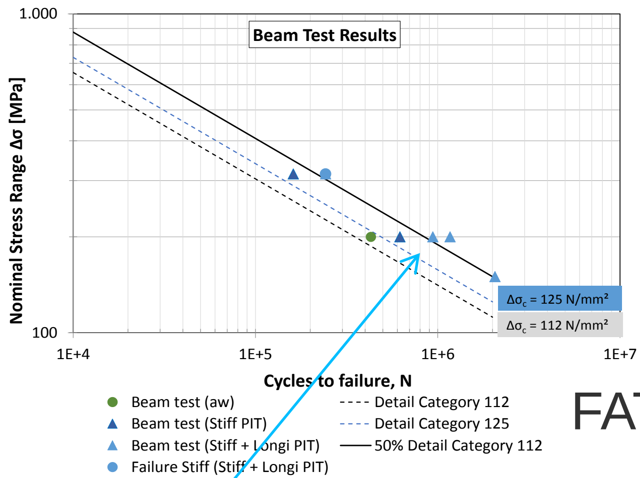
Butt welds on bottom flange of main beams:

FAT 112

112	size effect for t>25mm: $k_s=(25/t)^{0.2}$		<u>Without backing bar:</u> 1) Transverse splices in plates and flats. 2) Flange and web splices in plate girders before assembly. 3) Full cross-section butt welds of rolled sections without cope holes. 4) Transverse splices in plates or flats tapered in width or in thickness, with a slope ≤ ¼.	- All welds ground flush to plate surface parallel to direction of the arrow. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT. <u>Detail 3):</u> Applies only to joints of rolled sections, cut and rewelded.

CRITICAL FAT DETAILS (DESIGN A to C)

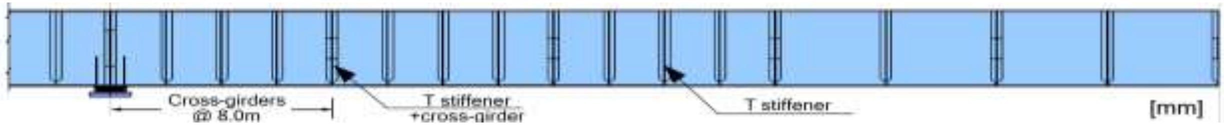
Web-to-flange longitudinal weld:



FAT 125

Detail category	Constructional detail	Description	Requirements
125	1 2	Continuous longitudinal welds: 1) Automatic butt welds carried out from both sides. 2) Automatic fillet welds. Cover plate ends to be checked using detail 6) or 7) in Table 8.5.	Details 1) and 2): No stop/start position is permitted except when the repair is performed by a specialist and inspection is carried out to verify the proper execution of the repair.
112	3 4	3) Automatic fillet or butt weld carried out from both sides but containing stop/start positions. 4) Automatic butt welds made from one side only, with a continuous backing bar, but without stop/start positions.	4) When this detail contains stop/start positions category 100 to be used.

OVERVIEW OF FAT CHECK



Design B											
Section (X)	1 (0.0)	2 (4.0)	3 (8.0)	4 (12.0)	5 (16.0)	6 (20.0)	7 (24.0)	8 (28.0)	9 (32.0)	10 (36.0)	11 (40.0)
t_f (mm)	70	70	70/60	60	60	60/45	45	45	45	45	45
$\gamma_{Mf}\,\gamma_{Ff}\,\Delta\sigma_{E,2}$	49.8	42.7	57.9	65.8	55.2	66.9	71.9	70.7	76.5	77.8	77.1
FAT	56	80	80	80	80	80	80	80	80	80	80
Design C											
t_f (mm)	70	70	70/45	45	45	45/30	30	30	30	30	30
$\gamma_{Mf}\,\gamma_{Ff}\,\Delta\sigma_{E,2}$	49.8	42.7	88.7	100.3	83.4	90.5	96.9	100.8	109.1	110.3	102.6
Detail			112			112		112			112
Size effect			0.89			0.96		0.96			0.96
FAT	56	125	100	125	125	108	125	108	125	125	108

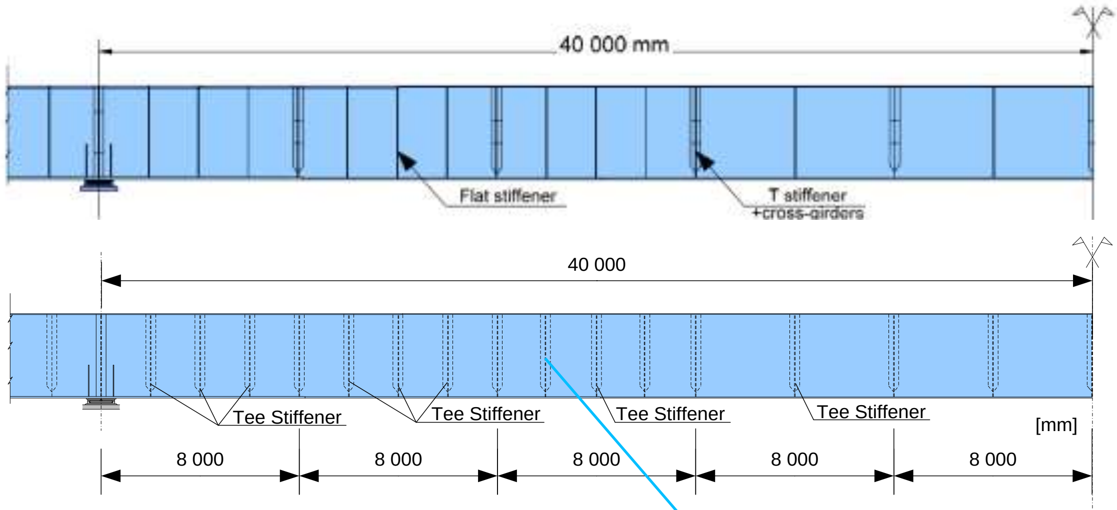
- PWT allows for 7% overall steel reduction

TRANSVERSAL STIFFENERS

DESIGN A
S355

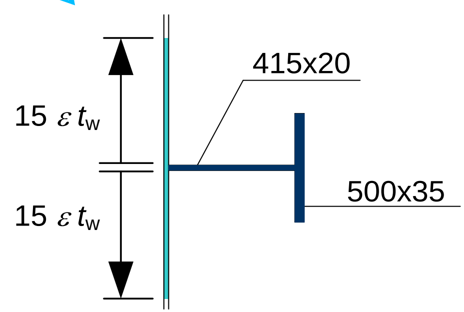
DESIGN C
S690

DESIGN B
S690



Design Requirements:

- ✓ Safety to torsional buckling
- ✓ Minimum stiffness requirement for shear verification of the webs
- ✓ Resistance to tension field action



TRANSVERSAL STIFFENERS

✓ **Safety to torsional buckling**

$$\sigma_{cr} \geq \theta f_y$$

σ_{cr} = elastic critical stress for torsional buckling of the stiffener

θ = 6 for T stiffeners or θ = 2 for flat stiffeners

f_y = is taken as the maximum stress σ_{max} and not the yielding stress

For DESIGN B:

$$\sigma_{cr} = 2969\text{MPa} \approx 6\,\sigma_{max\,.Ed} = 6 \times 472.78\text{ MPa} \quad >> \quad 6\sigma_{max\,,Ed}/\sigma_{cr} = 0.96$$

Often the critical criterion to design the transversal stiffeners

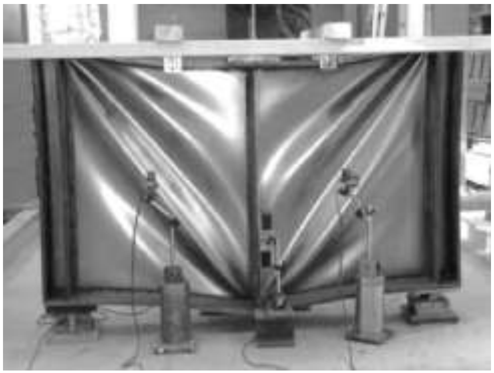
✓ **Minimum stiffness required to the stiffeners to act as rigid supports for shear verification web panels**

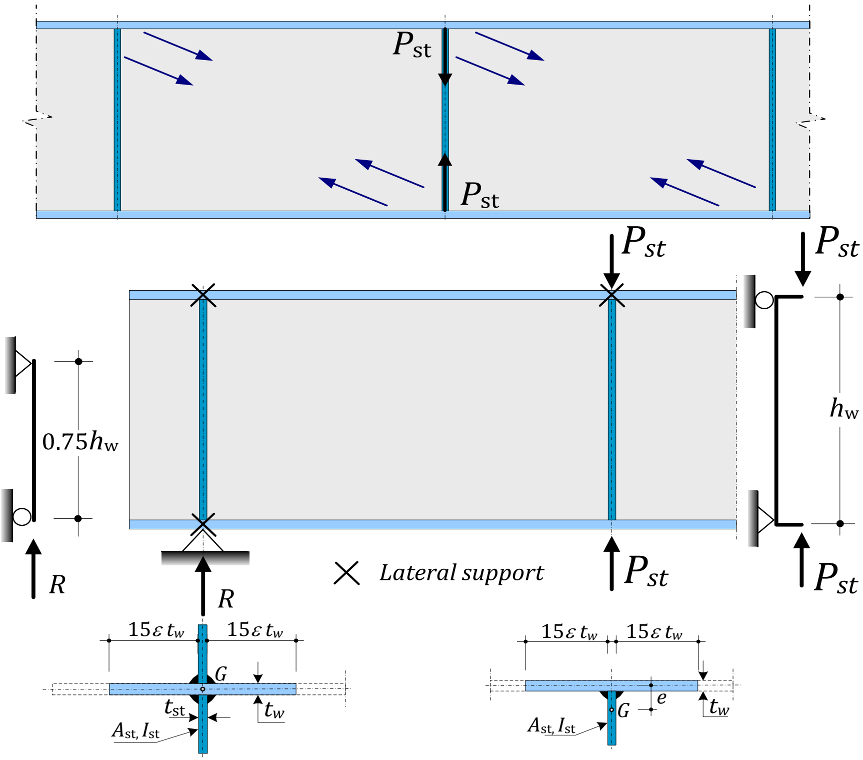
Usually verified by a large margin

$$\begin{aligned} I_{st} &\geq 1{,}5\, h_w^3 \cdot t_w^3/a^2 && \text{if} \quad a/h_w < \sqrt{2} \\ I_{st} &\geq 0{,}75\, h_w \cdot t_w^3 && \text{if} \quad a/h_w \geq \sqrt{2} \end{aligned}$$

TRANSVERSAL STIFFENERS

✓ **Resistance of the stiffener to tension field action**





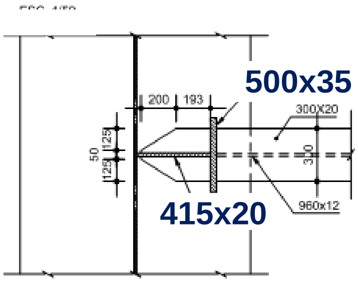
$$P_{st} = V_{Ed} - \frac{1}{\bar{\lambda}_w^2} \frac{f_{yw}\, h_w\, t}{\sqrt{3}\,\gamma_{M1}}$$

$$= V_{Ed} - V_{cr,w}$$

Tests show $P_{exp} \leq 56\% P_{st}$ [Sinur & Beg, 2012]

TRANSVERSAL STIFFENERS

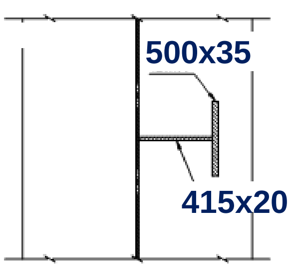
SECTION A-A - TEE STIFFENERS



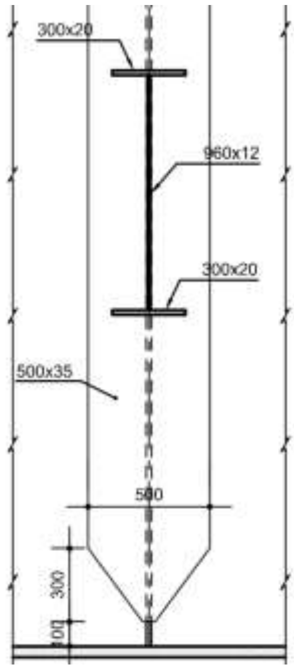
Design Case B - S690

SECTION C-C - TEE STIFFENERS

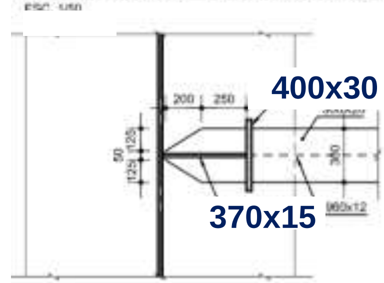
ESC. 1/50



SECTION B-B - CROSS-GIRDERS



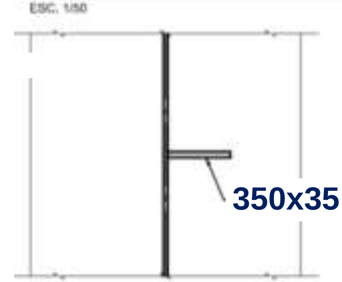
SECTION A-A - TEE STIFFENERS



Design Case C – S690

SECTION C-C -FLAT STIFFENERS

ESC. 1/50



- Re-design of stiffeners allows for 7% overall steel reduction

CONCLUSIONS

Advantages:

- The use of S690 HSS instead of S355 enables a reduction up to
  - 25% (Design B)
  - 35% (Design C)

Comparative analysis (structural steel weight ratios [kg/m²])	Steel in the deck	Reduction (%)
Design A – S355	219 kg/m^2	---
Design B – S690	165 kg/m^2	-25%
Design C – S690	**143 kg/m^2**	**(-14%) -35%**

CONCLUSIONS

Advantages:

- The use of S690 HSS instead of S355 enables a reduction up to:
  - 25% (Design B)
  - 35% (Design C)
- Aesthetics of bridge is improved by increased deck slenderness
- S690 allows the use of thinner plates which:
  - Reduces the full penetration weld volume more than 65%
  - Reduces the brittle fracture problems
  - Reduces the size effect and thus increases fatigue resistance
- PWT details are effective and useful to take full advantage of the HSS steel

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Thank you for your attention!

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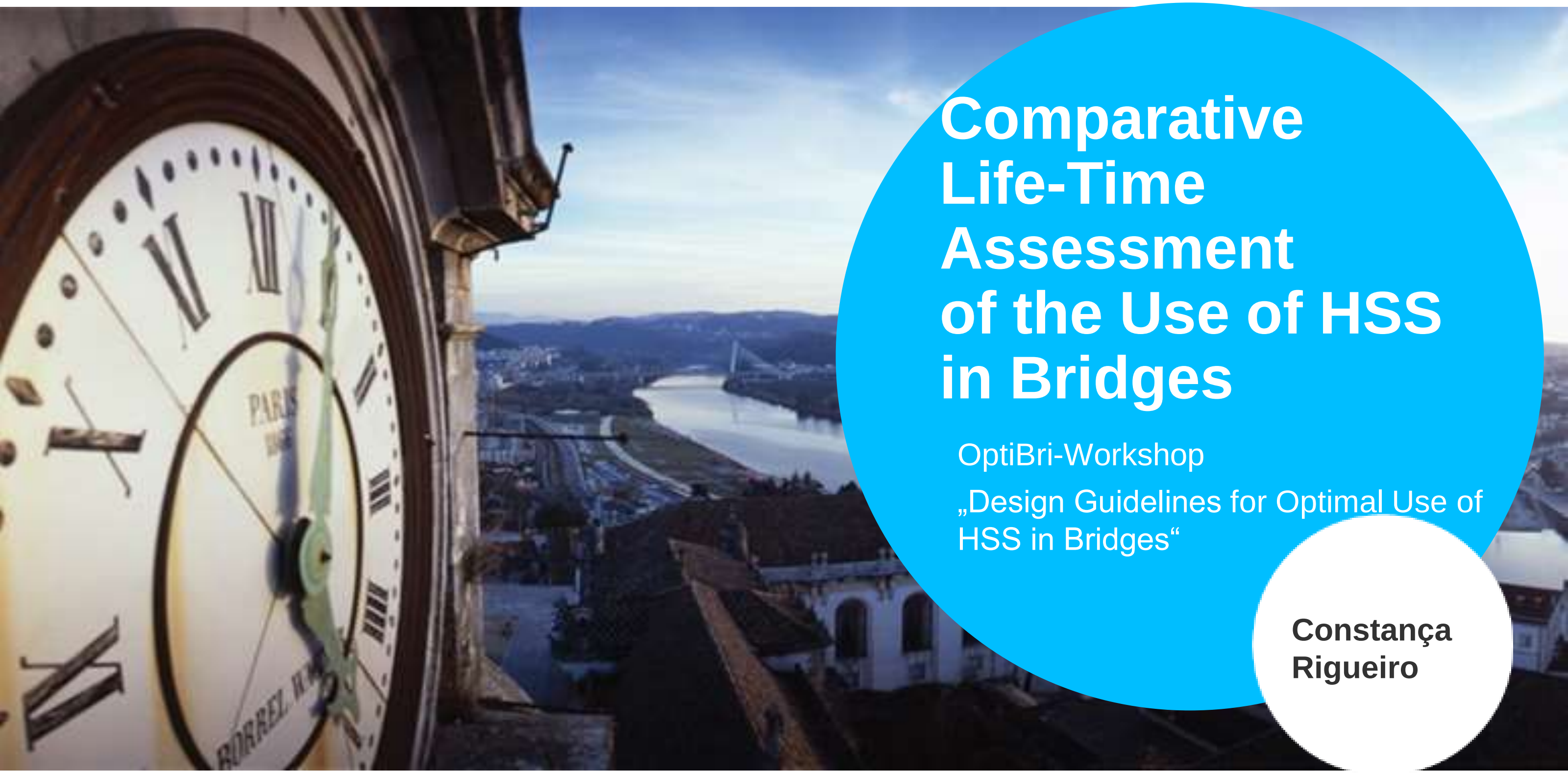
FCTUC FACULDADE DE CIÊNCIAS E TECNOLOGIA UNIVERSIDADE DE COIMBRA

Research Fund for Coal & Steel



Comparative Life-Time Assessment of the Use of HSS in Bridges

OptiBri-Workshop

„Design Guidelines for Optimal Use of HSS in Bridges“

Constança Rigueiro

Life Cycle Analysis

ISO STANDARDS 14040/14044

Goal and scope definition ⇄ **Interpretation**

⇅

Inventory analysis ⇄ **Interpretation**

⇅

Impact assessment ⇄ **Interpretation**

Mandatory elements

Selection of indicators

Classification

Characterization

$$IA_{jk} = \sum_{i=1}^{n} I_{ij} \times IAfactor_i$$

Optional elements

Normalization

Weighting

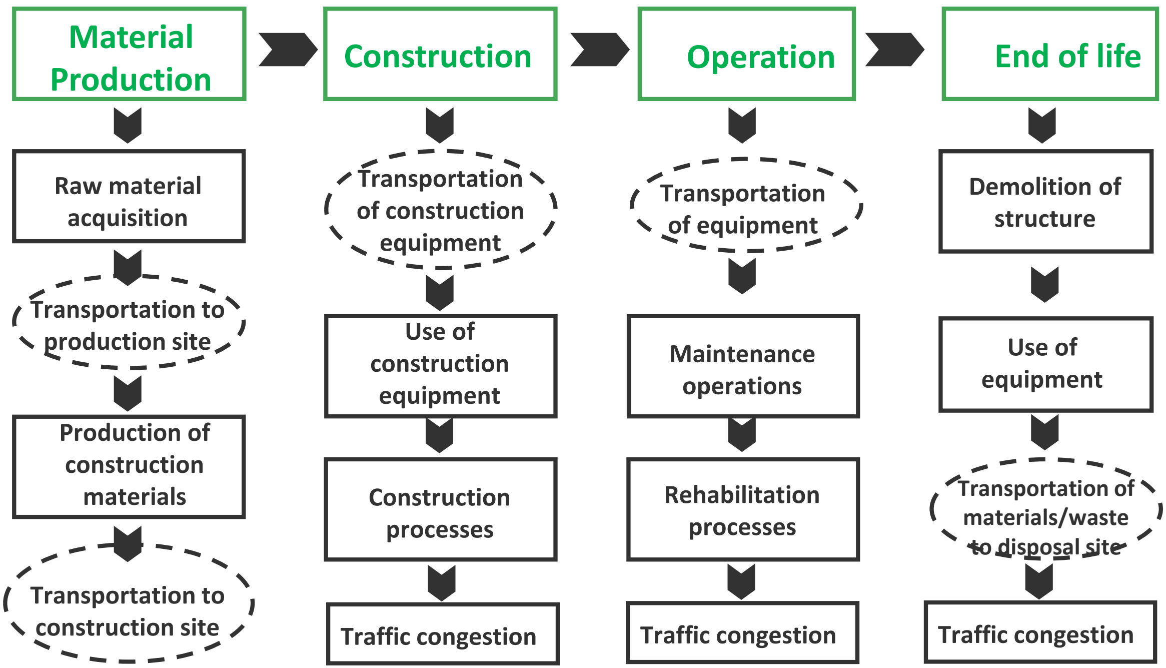
$$IAScore_{jk} = \frac{IA_{jk} \times IVwt_k}{Norm_k}$$

<u>Other relevant standards </u>(CEN TC350):

EN15643 & EN 15978 – Sustainability of construction works

Life-time assessment of bridges

SCOPE OF THE ANALYSIS



Departamento de Engenharia Civil – Faculdade de Ciências e Tecnologia – Universidade de Coimbra

Life-time assessment of bridges

SCOPE OF THE ANALYSIS ACCORDING TO EN 15978

PRODUCT stage	CONSTRUCTION PROCESS stage	USE stage	END-OF-LIFE stage	Benefits and loads beyond the system boundary
A1 Raw material supply	A4 Transport	B1 Use	C1 Deconstruction demolition	D Reuse- Recovery- Recycling- potential
A2 Transport	A5 Construction – installation process	B2 Maintenance	C2 Transport	
A3 Manufacturing		B3 Repair	C3 Waste processing	
		B4 Replacement	C4 Disposal	
		B5 Refurbishment		
		B6 Operational energy use		
		B7 Operational water use		

Mandatory (A1–A3)
EPD cradle-to-gate

Mandatory (A1–A3) ... *Optional* (A4–D)
EPD cradle-to-gate with option

Mandatory (A1–C4) ... *Optional* (D)
EPD cradle-to-grave

Life-time assessment of bridges

INVENTORY ANALYSIS

✓ ***Environmental data for different steel grades***

2. Product

2.1 Product description

This EPD applies to 1 t of structural steel (sections and plates). It covers steel products of the grades S235 to S960 rolled out to structural sections, merchant bars and heavy plates.

5. LCA: Results

DESCRIPTION OF THE SYSTEM BOUNDARY (X = INCLUDED IN LCA; MND = MODULE NOT DECLARED)

PRODUCT STAGE			CONSTRUCTION PROCESS STAGE		USE STAGE							END OF LIFE STAGE				BENEFITS AND LOADS BEYOND THE SYSTEM BOUNDARYS
Raw material supply	Transport	Manufacturing	Transport from the gate to the site	Assembly	Use	Maintenance	Repair	Replacement[1]	Refurbishment[1]	Operational energy use	Operational water use	De-construction demolition	Transport	Waste processing	Disposal	Reuse-Recovery-Recycling-potential
A1	A2	A3	A4	A5	B1	B2	B3	B4	B5	B6	B7	C1	C2	C3	C4	D
X	X	X	MND	MND	MND	MND	MND	MND	MND	MND	MND	MND	MND	MND	MND	X

RESULTS OF THE LCA - ENVIRONMENTAL IMPACT: 1 tone structural steel

Parameter	Unit	A1 - A3	D
Global warming potential	[kg CO_2-Eq.]	1735	-959
Depletion potential of the stratospheric ozone layer	[kg CFC11-Eq.]	1.39E-7	6.29E-9
Acidification potential of land and water	[kg SO_2-Eq.]	3.52	-1.32
Eutrophication potential	[kg $(PO_4)^{3-}$-Eq.]	3.7E-1	-1.29E-1
Formation potential of tropospheric ozone photochemical oxidants	[kg Ethen Eq.]	6.98E-1	-4.14E-1
Abiotic depletion potential for non fossil resources	[kg Sb Eq.]	2.85E-4	-1.11E-4
Abiotic depletion potential for fossil resources	[MJ]	17000	-7450

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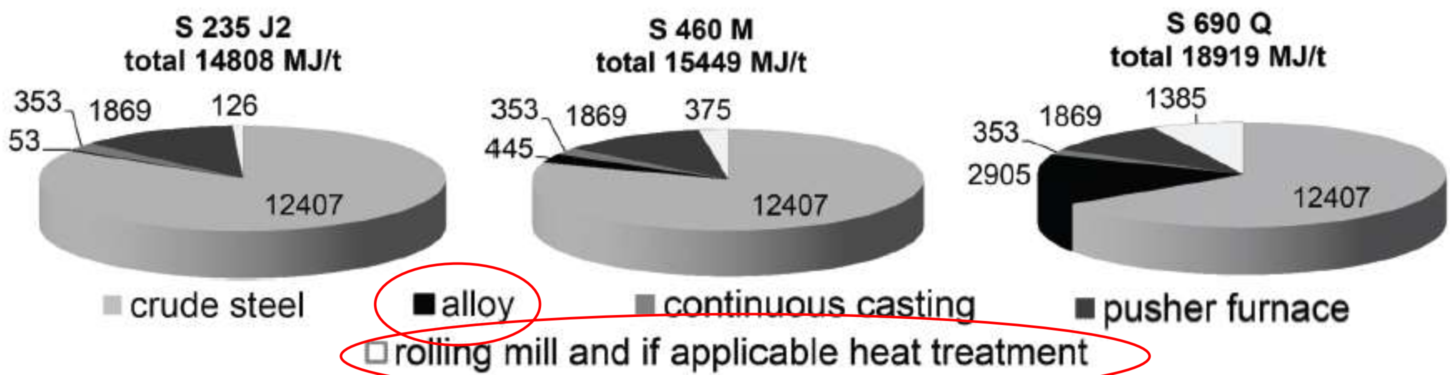
Life-time assessment of bridges

INVENTORY ANALYSIS

✓ ***Environmental data for different steel grades***



Cumulated energy demand (CED) for heavy plates (closed-loop-approach) made of various steel grades

Source: Stroetmann, R. HSS for improvement of sustainability. Eurosteel 2011.

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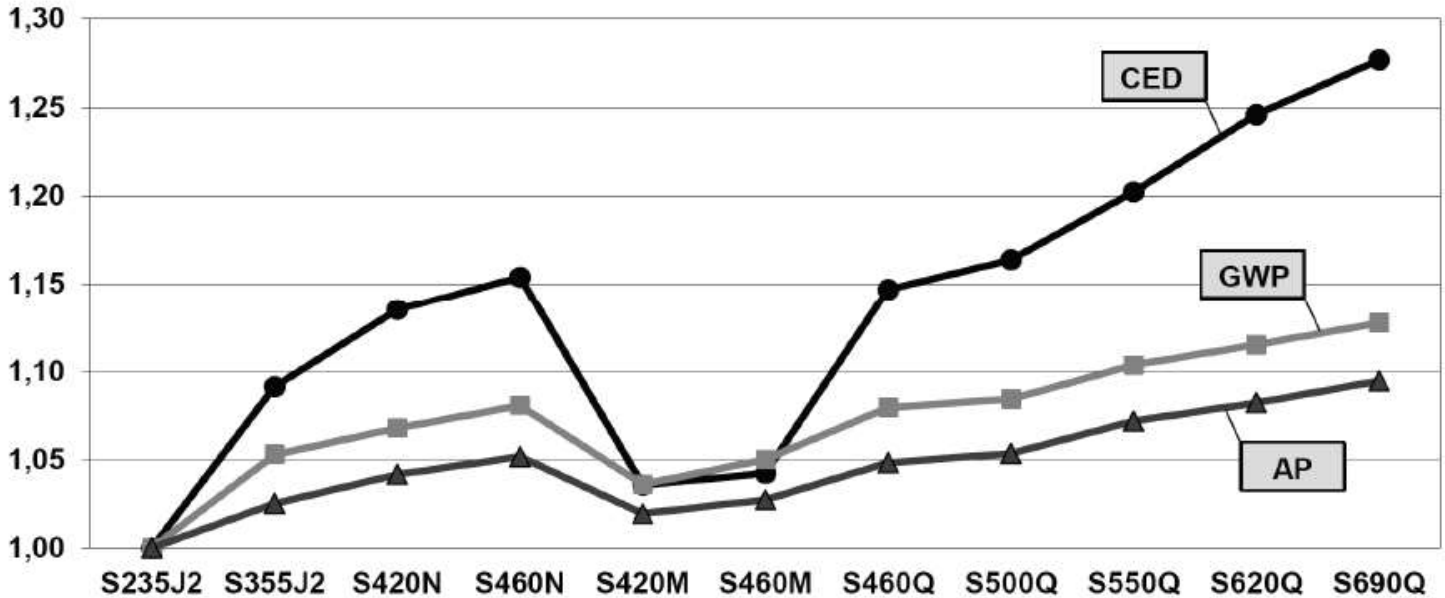
Life-time assessment of bridges

INVENTORY ANALYSIS

✓ ***Environmental data for different steel grades referring to S235J2***



Relation of CED, GWP_{100} and AP of heavy plates
for various steel grades referring to S235J2

Source: Stroetmann, R. HSS for improvement of sustainability. Eurosteel 2011.

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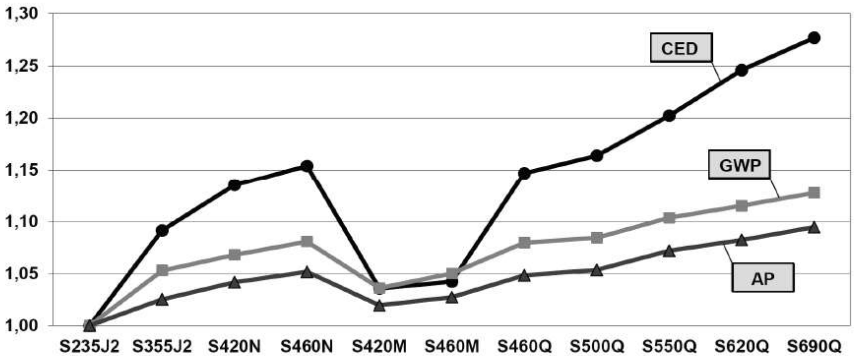
Life-time assessment of bridges

INVENTORY ANALYSIS

✓ ***Environmental data for different steel grades referring to S235J2***



Relation of CED, GWP_{100} and AP of heavy plates
for various steel grades referring to S235J2

	S355J2	S420N	S460N	S420M	S460M	S460Q	S500Q	S550Q	S620Q	S690Q
heavy plates	6,6	9,3	10,6	3,3	4,1	10,2	11,1	13,4	15,5	17,1
rolled sections	6,7	8,8	9,9	3,4	4,2	10,0	10,7	12,6	14,2	15,6

Required weight saving ΔG in [%] compared to steel grade S235J2

Source: Stroetmann, R. HSS for improvement of sustainability. Eurosteel 2011.

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INVENTORY ANALYSIS

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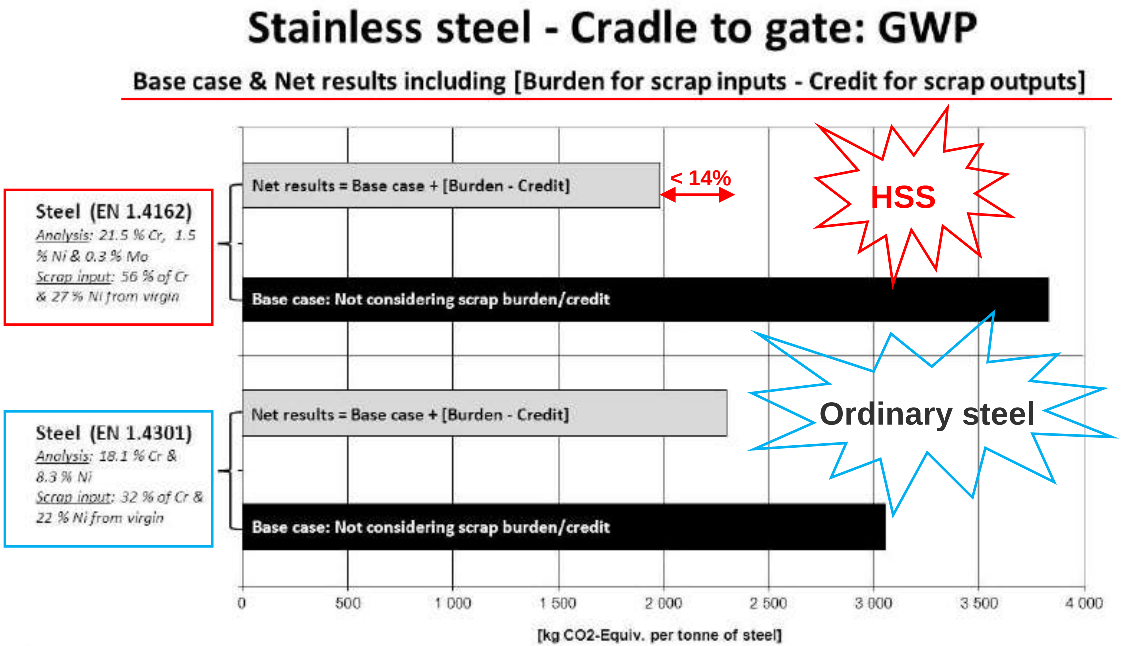


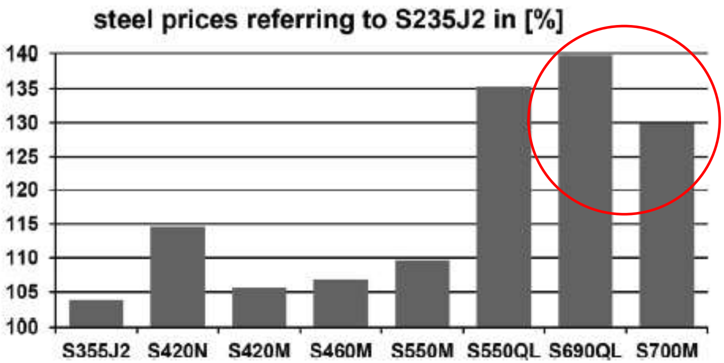
Cradle-to-gate results of two stainless steel grades.

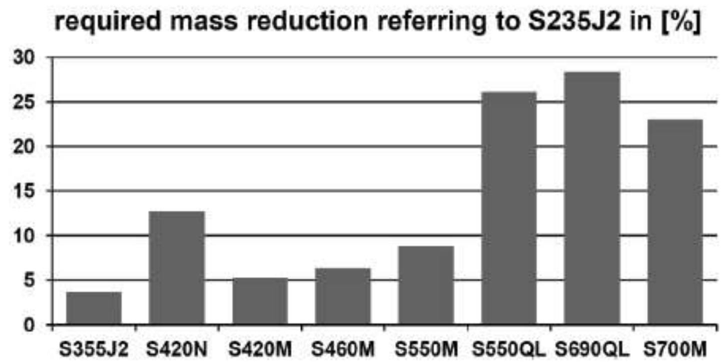
Source: Hallberg, L. & Sperle, J. Assessing the environmental advantages of HSS. The Steel Eco-Cycle, Environmental Research Programme for the Swedish Steel Industry, 2004 – 2012.

Life-time assessment of bridges

INVENTORY ANALYSIS

✓ ***Cost of different steel grades (production and fabrication)***





Economic efficiency - Relative price comparison for heavy plates of various steel grades

A moderate increase in price that may be compensated by appropriate weight savings.

Source: Stroetmann, R. HSS for improvement of sustainability. Eurosteel 2011.

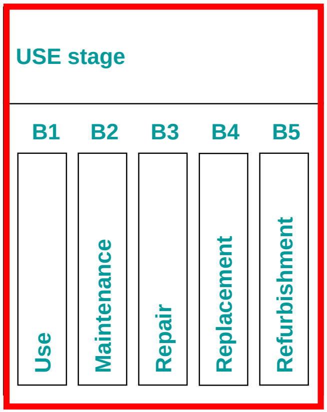
Life-time assessment of bridges

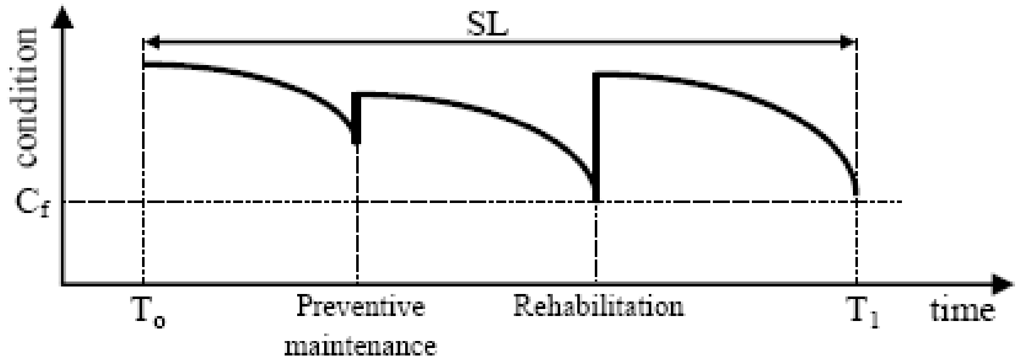
IMPACT ASSESSMENT

Indicator	Brief description	Unit
Abiotic depletion	Depletion of natural resources	kg of antimony (Sb) eq.
Acidification	Atmospheric pollution arising from anthropogenically derived sulphur (S) and nitrogen (N), which enhances the rates of acidification of soils and may then exceed its natural neutralising capacity	kg SO_2 eq.
Eutrophication	The gradual increase and enrichment of ecosystems by nutrients such as nitrogen (N) and/or phosphorus (P)	kg PO_4 eq.
Global warming	The potential contribution of a substance to the greenhouse effect.	kg CO_2 eq.
Ozone layer depletion	Defines ozone depletion potential of different gasses	kg CFC-11 eq.
Photochemical oxidation	Formation of reactive substances (mainly ozone) which are injurious to human health and ecosystems	kg of ethylene (C_2H_4) eq.

Life-time assessment of bridges

Life cycle performance - Analysis of use stage (modules B1-B5)





Focus on fatigue assessment

Life-time assessment of bridges

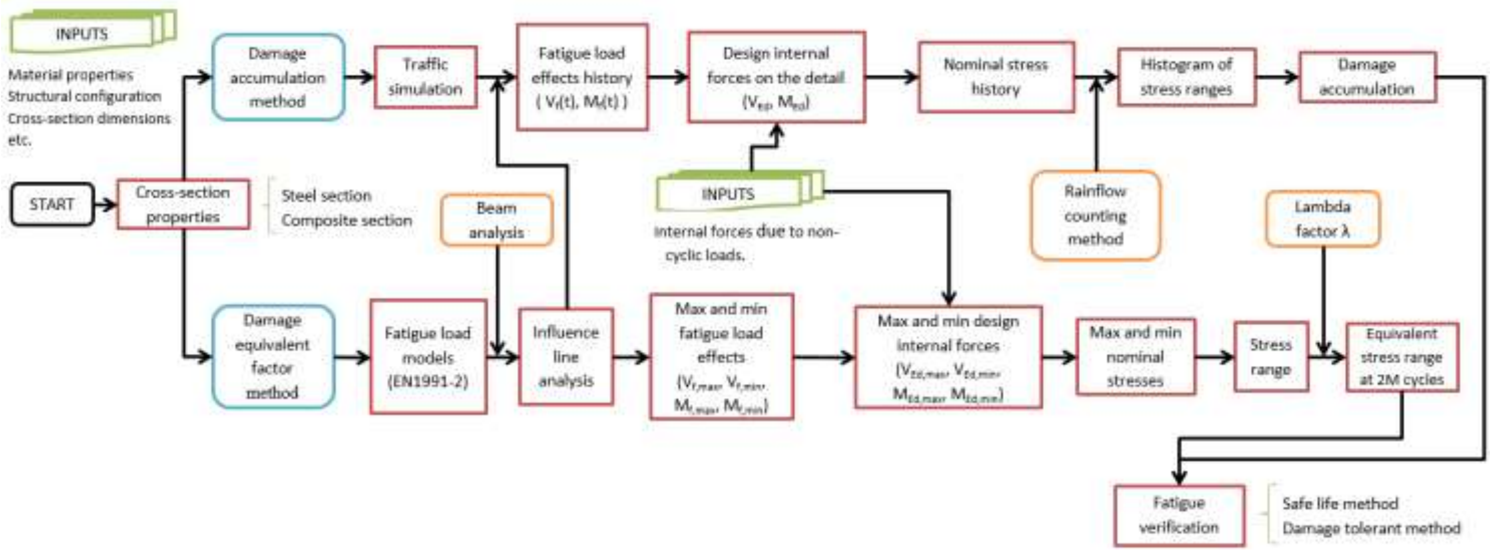
PROGRAM DEVELOPMENT

- **Scope: Composite girder-bridge with numerous spans.**
- **Program developed in PYTHON 2.7.12.**
- **Organised in 4 Main Modules**
  1. **Beam Analysis**
  2. **Influence line and FLM3**
  3. **Traffic simulation**
  4. **Cross-section and detail verification to fatigue**
- **Databases using SQLite**

Life-time assessment of bridges

PROGRAM DEVELOPMENT

FLOWCHART



Life-time assessment of bridges

PROGRAM DEVELOPMENT

- **Scope: Composite girder-bridge with numerous spans.**
- **Program developed in PYTHON 2.7.12.**
- **Organised in 4 Main Modules**

1. **Beam Analysis**

   This module aims to get the load effects on the main girders (shear and bending moment).

2. **Influence line and FLM3**
3. **Traffic simulation**
4. **Cross-section and detail verification to fatigue**

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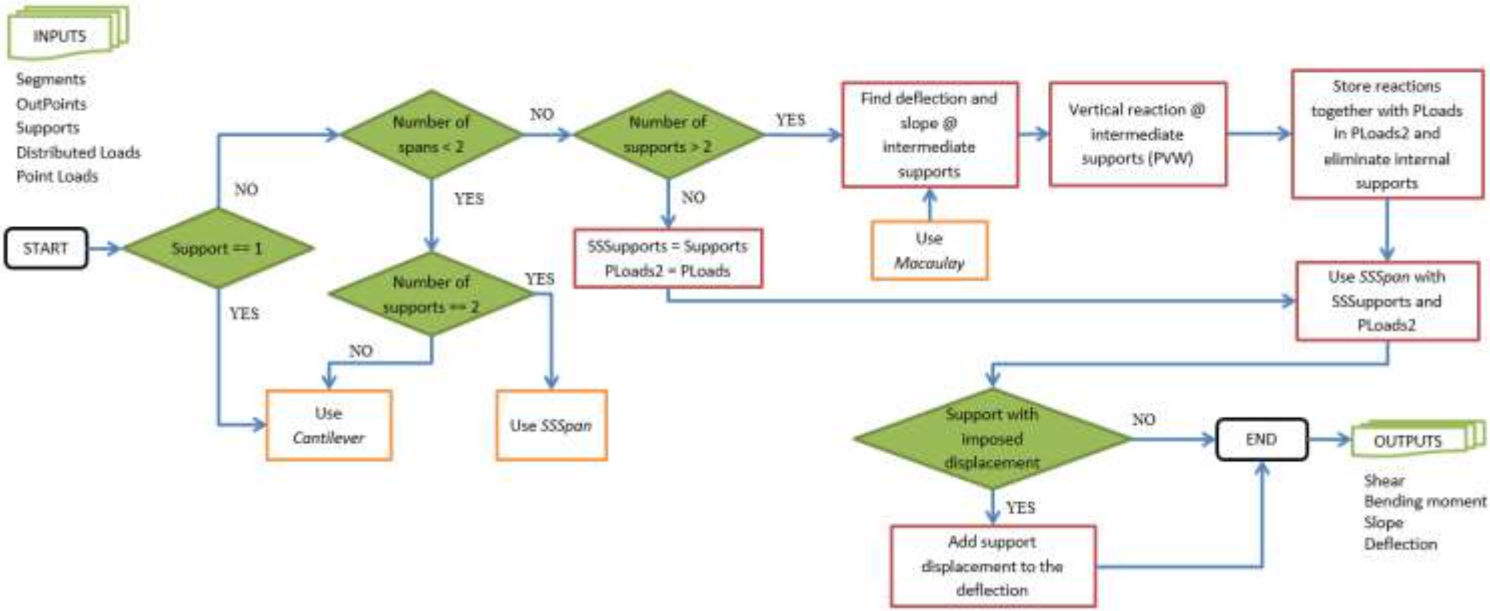
Life-time assessment of bridges

PROGRAM DEVELOPMENT

BEAM ANALYSIS



Life-time assessment of bridges

PROGRAM DEVELOPMENT

- **Scope: Composite girder-bridge with numerous spans.**
- **Program developed in PYTHON 2.7.12.**
- **Organised in 4 Main Modules**

1. **Beam Analysis**
2. **Influence line and FLM3**

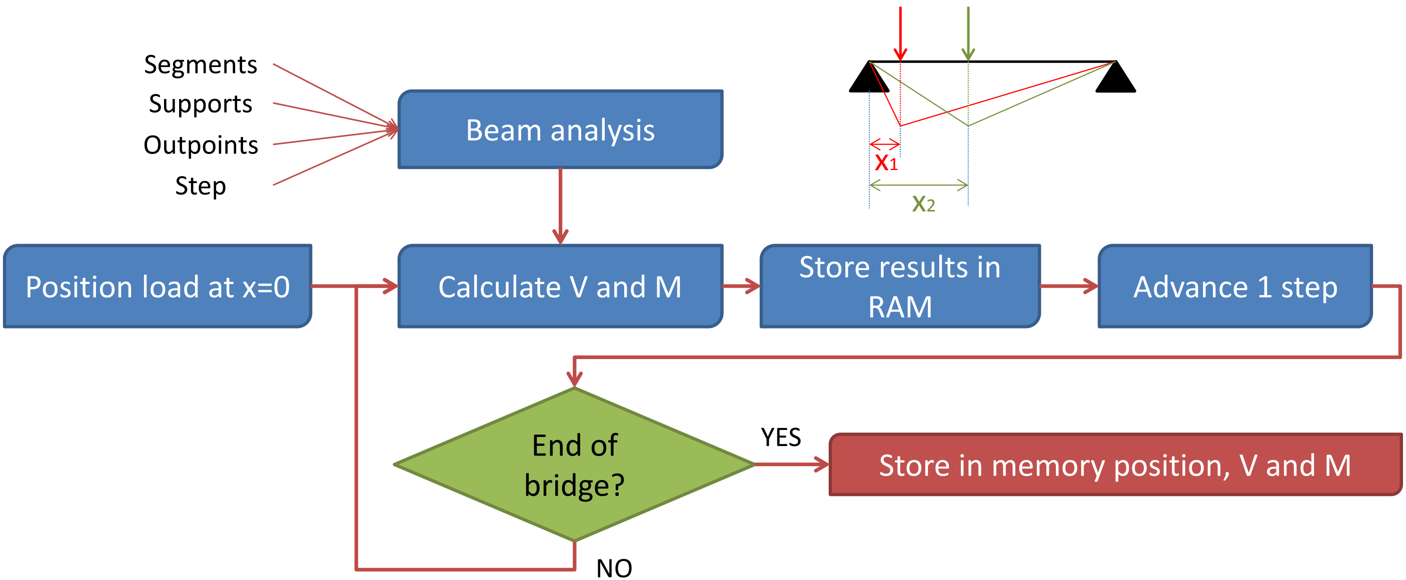
Calculates the shear and moment influence lines for a particular cross-section and applies the FLM3 in order to get the absolute maximum load effects for that section.

3. **Traffic simulation**
4. **Cross-section and detail verification to fatigue**

Life-time assessment of bridges

PROGRAM DEVELOPMENT

INFLUENCE LINE



This module allow us to study where should the loads be positioned in order to get maximum and minimum load effects in the cross-section where the detail under study is located.

Life-time assessment of bridges

PROGRAM DEVELOPMENT

FLM3

Inputs:
- Position of detail
- FLM3
- Transverse position
- Influence line

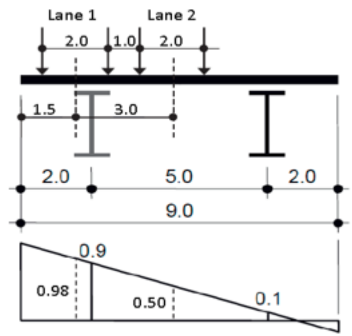
Reaction on girder (transversal position of loads)

↓

Position loads where max and min load effect (iterative)

↓

Return position, max and min load effect



The Eurocode proposes a load model - FLM3 - for fatigue design and verification when considering a finite life of the structure, which is most commonly used in practice along with the simplified damage equivalent factor method

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Life-time assessment of bridges

PROGRAM DEVELOPMENT

- **Scope: Composite girder-bridge with numerous spans.**
- **Program developed in PYTHON 2.7.12.**
- **Organised in 4 Main Modules**

1. **Beam Analysis**
2. **Influence line and FLM3**
3. **Traffic simulation**

   Generates a random stream of heavy load traffic and evaluates its action effects on the structure
4. **Cross-section and detail verification to fatigue**

Life-time assessment of bridges

PROGRAM DEVELOPMENT

TRAFFIC SIMULATION

Generate a stream of truck traffic in one lane

↓

Calculate the bridge load effects for a stream of truck traffic

INPUTS

- Min and max truck speed [km/h]
- Min gap between vehicles [sec] (safety)
- Period of time [hr]
- Start of day period [hr]
- End of day period [hr]
- Min and max flow rate during day period [truck/h]
- Min and max flow rate during day night [truck/h]
- Time step [sec]

Life-time assessment of bridges

PROGRAM DEVELOPMENT

- **Scope: Composite girder-bridge with numerous spans.**
- **Program developed in PYTHON 2.7.12.**
- **Organised in 4 Main Modules**

1. **Beam Analysis**
2. **Influence line and FLM3**
3. **Traffic simulation**
4. **Cross-section and detail verification to fatigue**

Calculates the cross-section properties and checks the verification of the detail under fatigue using both damage equivalent and damage accumulation methods.

Life-time assessment of bridges

PROGRAM DEVELOPMENT

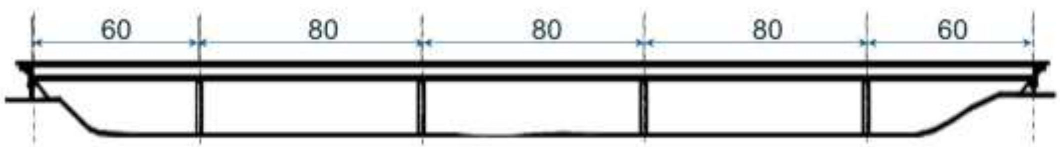
CROSS SECTION

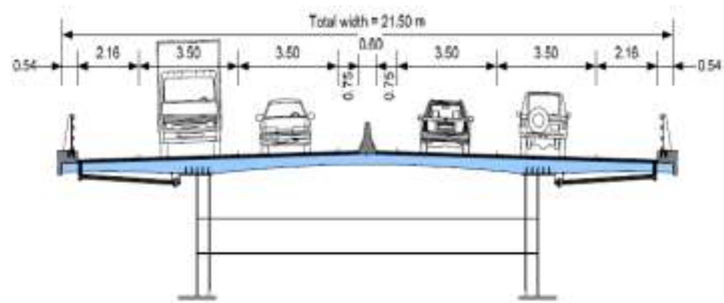
- Steel cross-section
- Effective properties (local buckling)
- Shear lag (effective width)
- Concrete cracking
- Creep (modular ratios)

Case study

GENERAL DESCRIPTION

- Composite steel-concrete girder bridge with a continuous multiple-span configuration
- Steel grades - **CASE A: S355** and **CASE B: S690** (HSS).
- Concrete C35/45. Reinforcement steel B500B. Head stud connectors S235.
- 2 lanes of traffic per direction.
- 1 slow lane per direction.





Case study

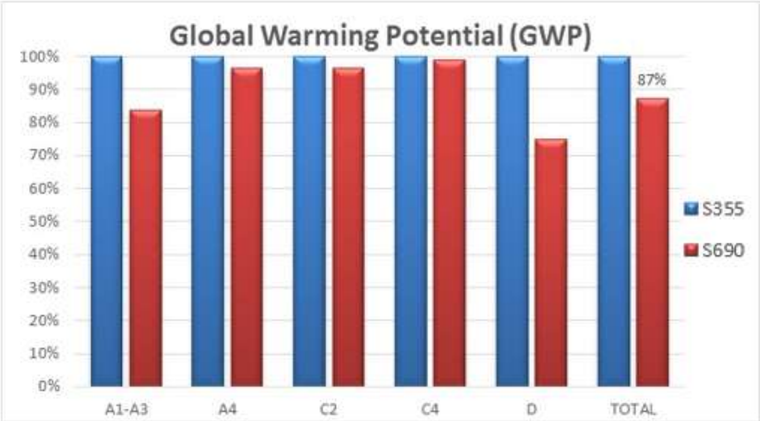
Bill of main materials (case A vs. case B)

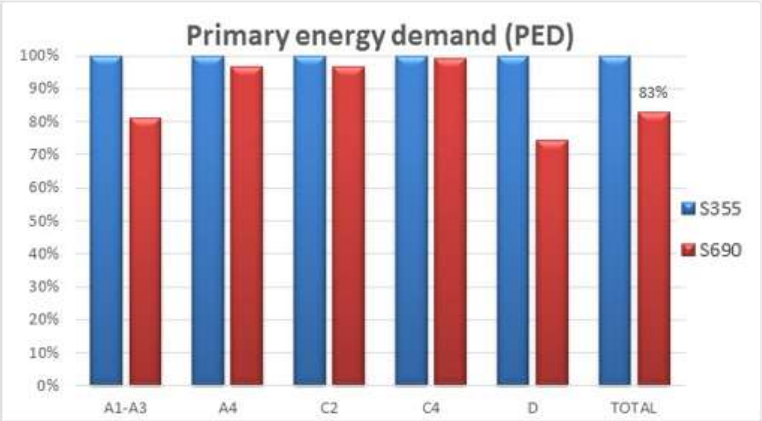
	Case A	Case B
Concrete slab (kg)	1373100	1367810
Steel girders (kg)	159021	110097
Connectors (kg)	790	790
Stiffners (kg)	15084	14028
Reinforcement (kg)	67521	67261

≈- 30%

Case study

Comparison of results - case A vs. case B

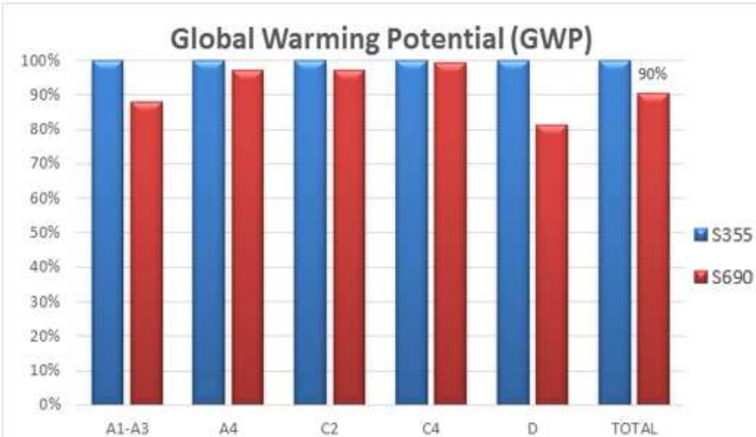


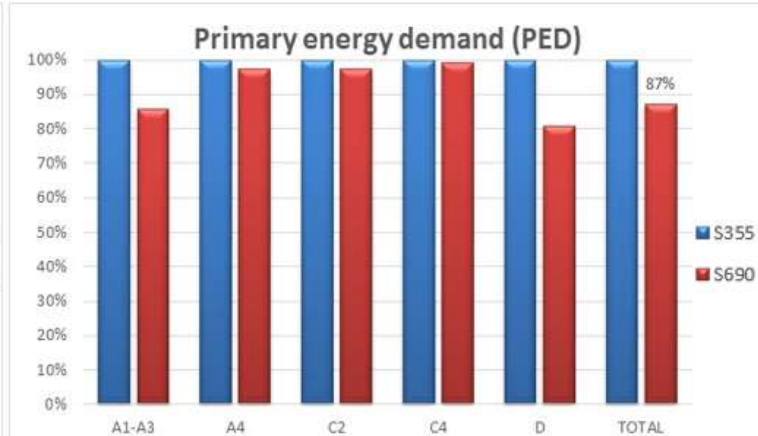


Case study

Comparison of results - case A vs. case B

Sensitivity analysis: assuming +10% for HSS





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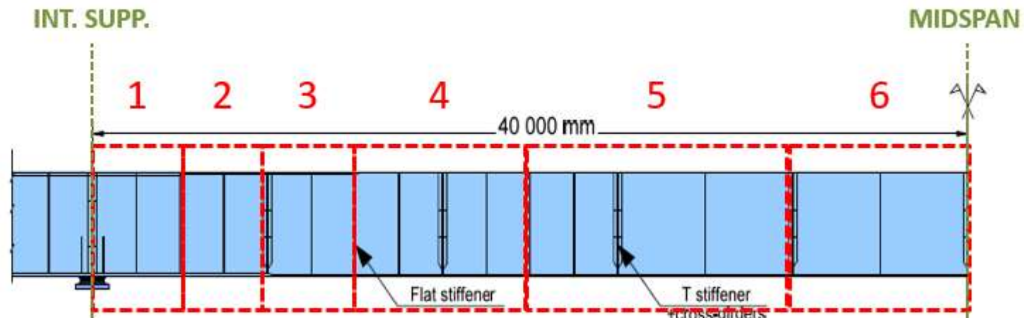
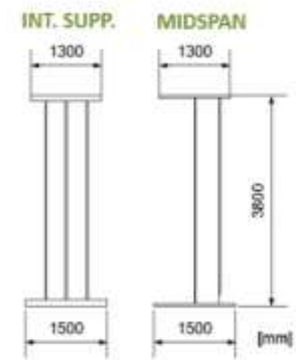
Case study

Comparison of results - case A vs. case B

STRUCTURAL STEEL DISTRIBUTION



Top Flanges (mm)	1300 x 100 / 4000	1300 x 70 / 8000	1300 x 35 / 28000
Web thickness (mm)	26 / 12000	22 / 20000	18 / 8000
Bottom Flanges (mm)	1500 x120 / 8000	1500 x 80 / 12000	1500 x 50 / 20000
Head Stud Connectors	Stud Connectors 5 ϕ22// 300	Stud Connectors 5 ϕ22// 400	Stud Connectors 3 ϕ22// 400
Longitudinal Reinforcement	2.0% / 12000	1.5% / 12000	1.0% / 16000



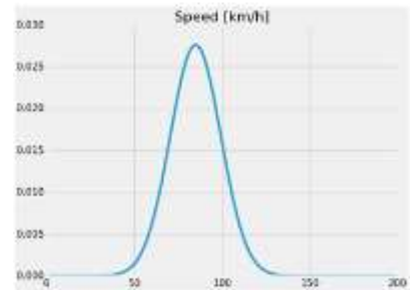
Case study

Comparison of results - case A vs. case B

CUMULATIVE DAMAGE METHOD

- **Initial conditions:**

Days of analysis	min speed	max speed	min gap	time analysis	min flow day	max flow day	min flow night	max flow night	start time day	end time day
	[km/h]	[km/h]	[sec]	[hrs]	[tr/hr]	[tr/hr]	[tr/hr]	[tr/hr]	[hr]	[hr]
28	60	110	1.5	720	100	200	10	100	6	22



	Mean	St. dev.
Truck speed [km/hr]	84.9	14.44

- **Maximum stress range:**

$\Delta\sigma_{max}$ [MPa]	S355	S690
At intermediate support	6.247	14.321
At mid-span	24.505	30.667

$$\frac{\Delta\sigma_L}{\gamma_{Mf}} = \frac{32.4}{1.35} = 24\ \text{MPa}$$

- **Damage:**

Cumulative damage	1 month		50 years		100 years	
	S355	S690	S355	S690	S355	S690
At intermediate support	0	0	0	0	0	0
At mid-span	8.524 E-4	9.946 E-4	0.511	0.597	1.023	1.194

D > 1 !

✓ ***Thus, minor repairs are expected to occur in both cases;***

✓ ***However, as there is not traffic under the bridge, no significant differences are estimated for the environmental performance of the bridges over their service lives.***

Conclusions

- ✓ ***The use of HSS enables to reduce the amount of steel used in the structural system of bridges;***
- ✓ ***This reduction leads to improvements in the life cycle environmental performance of the bridge as resources are saved and emissions are reduced;***
- ✓ ***Steel structures made by HSS may be more vulnerable to fatigue problems;***
- ✓ ***The use of post-welding treatments may enable to reduce this vulnerability (this will be assessed in the near future).***

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Thank you for your attention!

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