Fire Engineering Research - Key Issues for the Future II

Naples, Italy, 6 – 9 June 2013

decivil universidade de aveiro departamento de engenharia civil

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PhD: SHEAR BUCKLING IN STEEL MEMBERS SUBJECT TO FIRE

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Introduction

SHEAR BUCKLING IN STEEL MEMBERS SUBJECT TO FIRE

Motivation:

- I steel profiles with class 4 cross-section are frequently used
- The utilisation of beams with slender webs has increased in the last few years
- The shear buckling phenomena at high temperatures have not been widely studied

Introduction

SHEAR BUCKLING IN STEEL MEMBERS SUBJECT TO FIRE

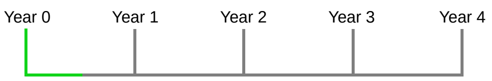
Objectives:

- Study the shear buckling phenomenon of steel members, with welded or hot-rolled H or I shape, subjected to high temperatures
- Develop simple design rules for fire design of these steel elements, as close as possible to the principles of the design rules implemented by Eurocode 3 at normal temperatures

Point of Situation

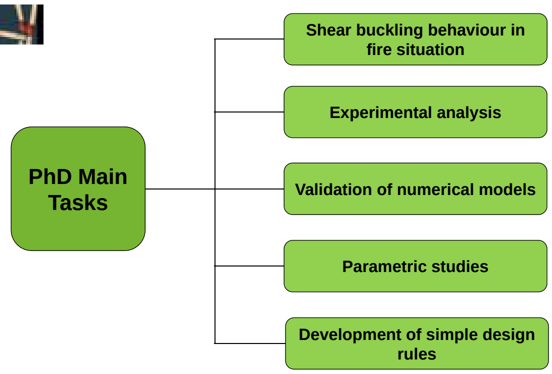
Schedule:

My PhD started in **January 2013.**



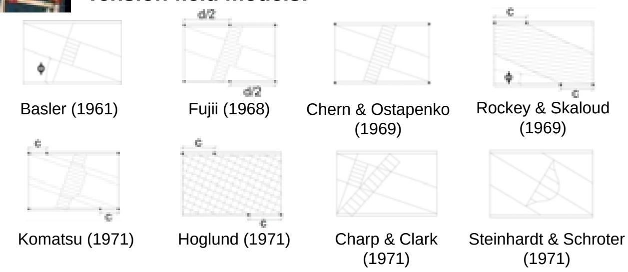
Presently, I am in the middle of the first year.

PhD working programme



Resistance to shear

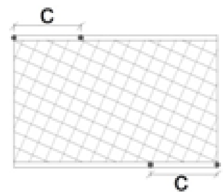
Tension field models:



- These theories mainly assume superposition of buckling and post-buckling shear strength
- They differ regarding the definition of tension field action

Resistance to shear

Tension field models:

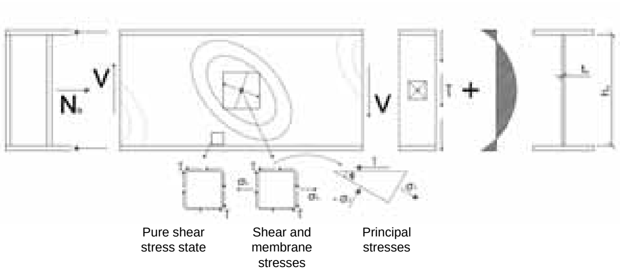


Hoglund (1971)

- EN 1993-1-5 implemented the method known as "Rotated Stress Field" developed by Hoglund
- This method was firstly developed for girders with web stiffeners at supports only, because other existing methods were very conservative in this case

Resistance to shear

Mechanical model of the Rotated Stress Field method:



Design rules according to EN 1993-1-5

Is shear buckling verification necessary?

For unstiffened webs:

$\frac{h_w}{t_w} > 72\,\frac{\varepsilon}{\eta}$?

For stiffened webs:

$\frac{h_w}{t_w} > 31\,\frac{\varepsilon}{\eta}\sqrt{k_\tau}$?

False ⟹ Verification is not necessary!

True ⟹ Verification is necessary!

Design rules according to EN 1993-1-5

Design shear resistance is taken as:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta\, f_{yw}}{\sqrt{3}\,\gamma_{M1}} h_w\, t_w$$

- $V_{bw,Rd}$: Resistance from the web
- $V_{bf,Rd}$: Resistance from the flanges
- η: Coefficient that depends on the steel grade

For $f_{yw} \leq 460$ MPa → $\boldsymbol{\eta = 1.2}$

For $f_{yw} > 460$ MPa → $\boldsymbol{\eta = 1.0}$

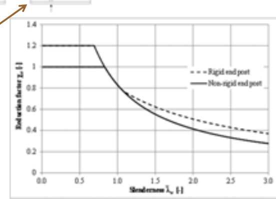
Design rules according to EN 1993-1-5

Contribution from the web

$$V_{bw,Rd} = \chi_w\, h_w\, t_w \frac{f_{yw}}{\sqrt{3}\,\gamma_{M1}}$$

	χ_w Rigid end post	Non rigid end post
$\bar{\lambda}_w < 0.83/\eta$	η	η
$0.83/\eta \leq \bar{\lambda}_w < 1.08$	$0.83/\bar{\lambda}_w$	$0.83/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1.08$	$1.37/(0.7+\bar{\lambda}_w)$	$0.83/\bar{\lambda}_w$

$$\bar{\lambda}_w = \sqrt{\frac{f_{yw}/\sqrt{3}}{\tau_{cr}}} = 0.76\sqrt{\frac{f_{yw}}{\tau_{cr}}}$$



where

$\tau_{cr} = k_\tau\, \sigma_E$ (Annex A.3 of EN 1993-1-5)

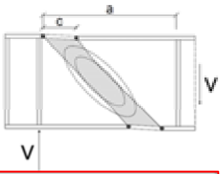
$$\sigma_E = \frac{\pi^2\, E\, t_w^2}{12\,(1-\nu^2)\, d^2}$$

Design rules according to EN 1993-1-5

Contribution from the flanges

The contribution from the flanges is given by the Equation below, which assumes the formation of four plastic hinges in the flanges at the distance c.

$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c \, \gamma_{M1}}\left[1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right]$$



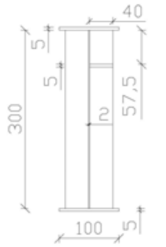
Usually the contribution of flanges is small and can be neglected.

Experimental tests at normal temperature from literature

- Some characteristics of performed experimental tests:
  - Steel grade S275 (f_y = 274 MPa)
  - Young modulus (E): 206 GPa
  - Beam length: 1.8 m

Structural system of the girder

  - Web depth (h_w): 300 mm
  - Web thickness (t_w): 2 mm
  - Flange width (b): 100 mm
  - Flange thickness (t_f): 5 mm



Cross-section of the tested beams

C. Gomes, P. Cruz, and L. Silva - "Experimental evaluation of the shear behaviour of slender steel beams", University of Minho, Civil Engineering Department, 2000.

Experimental tests at normal temperature from literature

- Beams with transverse stiffeners:

- Transverse stiffeners thickness (t_{ts}): 5 mm
- Space between transverse stiffeners (a): 300, 600 and 900 mm

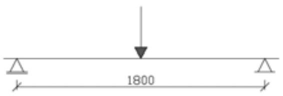
Experimental tests at normal temperature from literature

- Beams with transverse and longitudinal stiffeners:

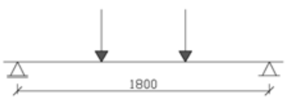
- Longitudinal stiffener thickness (t_{ls}): 5 mm
- The longitudinal stiffener was placed 60 mm below the lower surface of the upper flange

Experimental tests at normal temperature from literature

- Loading:
  - VT1, VT2, VTL1 and VTL2 (1 load point)

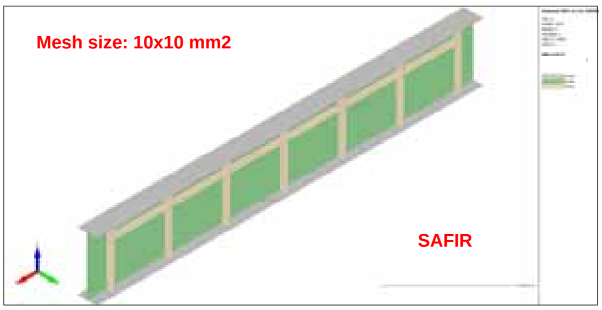


  - VT3 and VTL3 (2 load points)



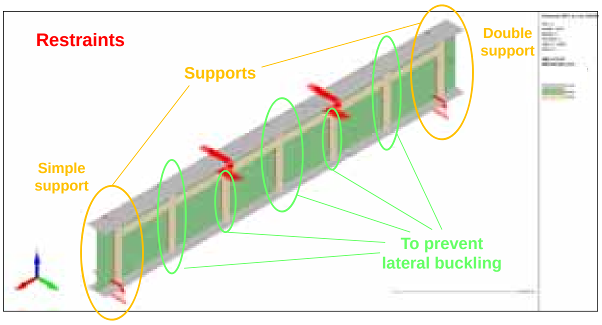
Shell finite element model

Model 1:



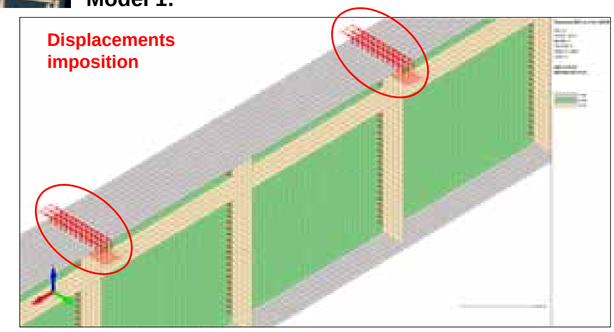
Shell finite element model

Model 1:



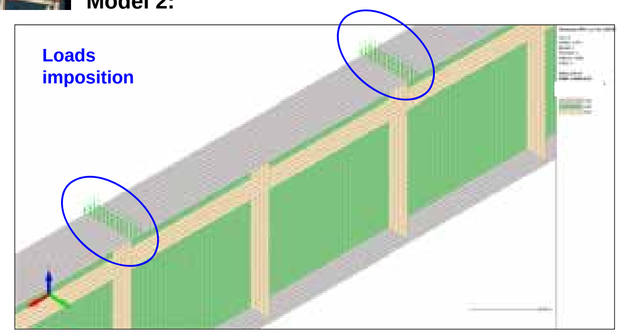
Shell finite element model

Model 1:



Shell finite element model

Model 2:

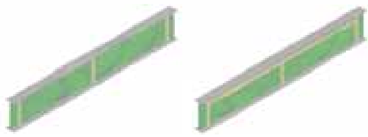


Local Imperfections – buckling modes

Maximum imperfection amplitude:

CAST3M + Ruby

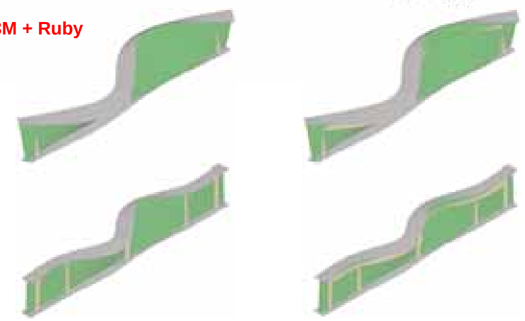
- Flange: $0.8\frac{b}{100}$
- Web: $0.8\frac{h_w}{100}$



Global Imperfections – buckling modes

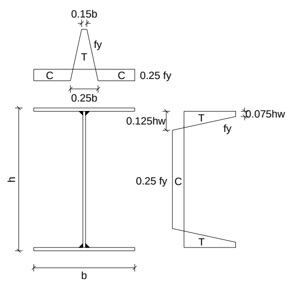
Maximum imperfection amplitude: $0.8\frac{L}{750}\approx\frac{L}{1000}$

CAST3M + Ruby



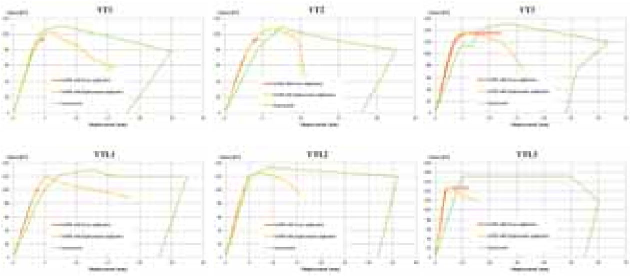
Residual Stresses

- Pattern of the residual stresses considered in numerical models (welded profile):



Results at normal temperature

Experimental vs. numerical analyses



Results at normal temperature

Experimental vs. numerical tests

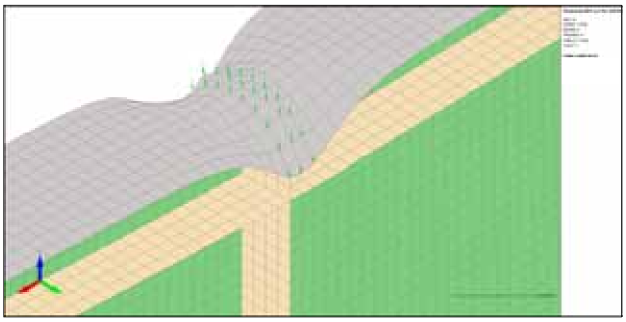
- Ultimate shear resistance → **Numerical model 1 adopted**

Test	Experimental [kN]	Numerical model 1 (with imposition of displacements) * [kN]	Numerical model 2 (with imposition of forces) * [kN]	Comparison between experimental test and numerical model 1 (%)
VT1	110	103.92	93.48	5.52
VT2	110	106.56	93.26	3.13
VT3	150	135.37	134.96	9.75
VTL1	130	120.54	100.73	7.28
VTL2	133	124.59	100.44	6.32
VTL3	172	146.23	146.85	14.98

* results obtained taking account the global imperfections, the local imperfections and the residual stresses

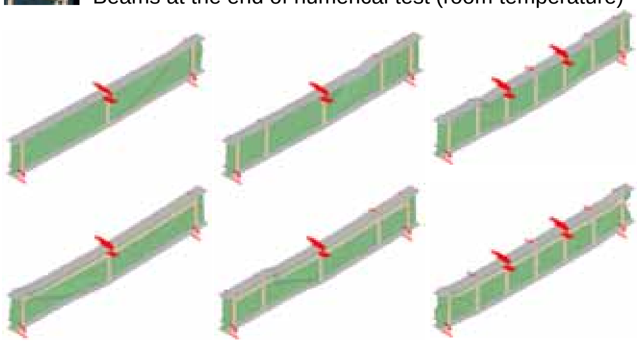
Results at normal temperature

Problems with model 2



Results at normal temperature

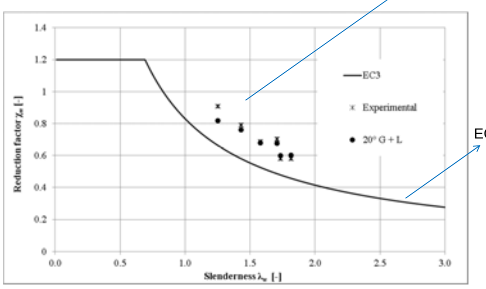
Beams at the end of numerical test (room temperature)



Results at normal temperature

Experimental vs. Numerical with Global & Local Imp.

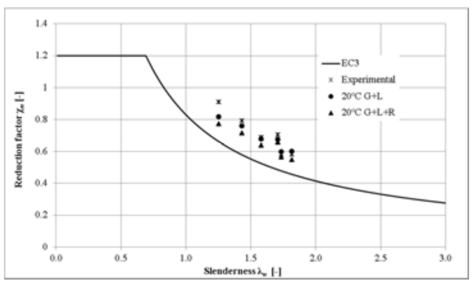
$$\chi_w = \frac{V_{SAFIR}}{h_w\ t_w\ f_{yw}/\sqrt{3}}$$



EC3 Part 1-5

Results at normal temperature

Experimental vs. Numerical with Global & Local Imp. vs. Numerical with Global Imp. & Local Imp. & Residual Stresses



Design rules at high temperatures

No explicit rules are given in Part 1-2 of EC3 for shear buckling verification at high temperatures

Therefore, we used the design rules for design shear resistance at normal temperature **with the reduction factors for stress-strain relationship of carbon steel at elevated temperatures**

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq h_w\, t_w \frac{\eta\, f_{yw}}{\sqrt{3}\,\gamma_{M1}} \quad + \quad \boxed{k_{E,\theta}} \;\&\; \boxed{k_{y,\theta}}$$

Design rules at high temperatures

- Room temperature:

$$\varepsilon_{20^\circ C} = \sqrt{\frac{235}{f_{yw}}}$$

$f_{yw} = 274$ MPa
$E = 206$ GPa

- High temperature (**500ºC**):

$$\varepsilon_{500^\circ C} = \sqrt{\frac{k_{E,500^\circ C}}{k_{y,500^\circ C}}} \sqrt{\frac{235}{f_{yw}}} \sqrt{\frac{E}{210000}}$$

$$\sqrt{\frac{k_{E,500^\circ C}}{k_{y,500^\circ C}}} = \sqrt{\frac{0.600}{0.780}} = 0.877$$

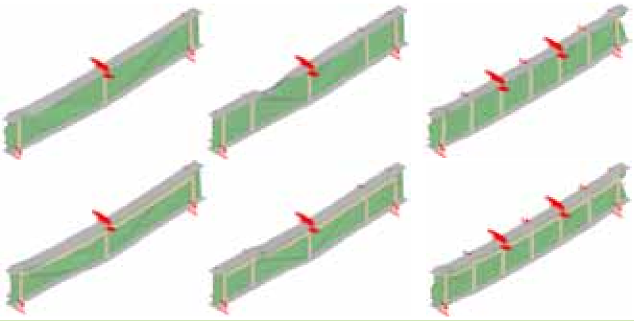
$$\varepsilon_{500^\circ C} = 0.877\, \varepsilon_{20^\circ C}$$

$$\varepsilon_{\theta} = \sqrt{\frac{235}{f_{yw,\theta}}} \sqrt{\frac{E_\theta}{210000}}$$

Test	Slenderness ($\bar{\lambda}_w$)	Slenderness ($\bar{\lambda}_{w,500^\circ C}$)
VT1	1.818	2.073
VT2	1.737	1.980
VT3	1.431	1.631
VTL1	1.707	1.947
VTL2	1.579	1.800
VTL3	1.253	1.429

Results at high temperature

Beams at the end of numerical test (high temperature)

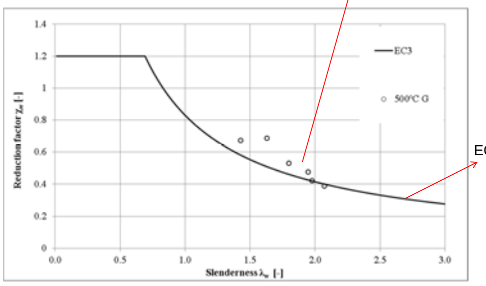


Results at high temperature

Results at **500ºC**:

Numerical with Global Imp.

$$\chi_w = \frac{V_{SAFIR,hot}}{h_w\, t_w\, f_{yw}\, k_{y,500^\circ C}/\sqrt{3}}$$

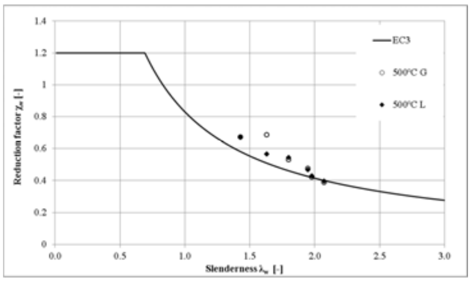


EC3 Part 1-5

Results at high temperature

Results at **500ºC**:

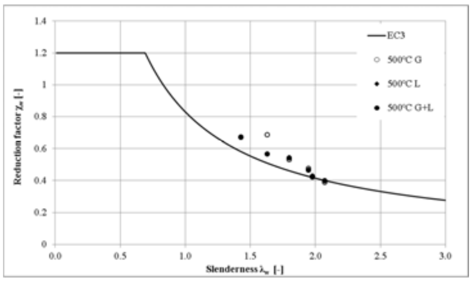
Numerical with Global Imp. vs. Numerical with Local Imp.



Results at high temperature

Results at **500ºC**:

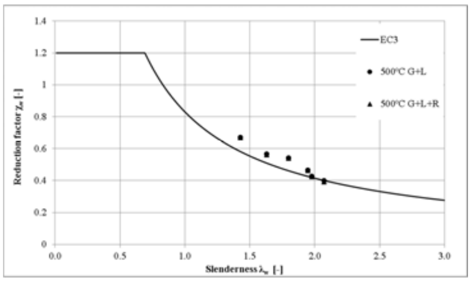
Numerical with Global Imp. vs. Numerical with Local Imp. vs. Numerical with Global & Local Imp.



Results at high temperature

Results at **500ºC**:

Numerical with Global & Local Imp. vs. Numerical with Global Imp. & Local Imp. & Residual Stresses



Conclusions

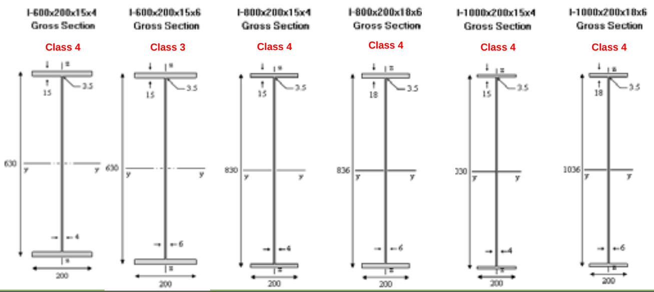
Conclusions:

- Understand the physical behaviour of this phenomenon
- Analyse the EC3 prescriptions to numerically model elements with shear buckling failure:
  - normal temperature
  - fire situation
- Specific concluding remarks regarding numerical modelling:
  - Residual stresses are important at normal temperature but not so much at higher temperatures
  - The geometric imperfections should always be considered
  - EC3 has shown conservative

Future work

Parameters to be analysed in shear buckling study

- Different cross-sections



Future work

Parameters to be analysed in shear buckling study

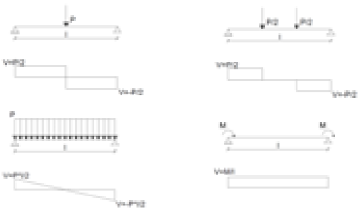
$0 \leq \bar{\lambda}_w \leq 3$

- Different **span lengths**: 1200, 1800 and 2400 mm
- **Transverse stiffeners** with several distances between stiffeners (1200, 900, 600, 450, 300 and 150 mm)
- **Longitudinal stiffeners**
- Different **steel grades**: S275, S355, S460 and >S460
- **Rigid** and **non-rigid end posts**
- **Welded** and **hot-rolled** cross-sections
- Different **steady-state temperatures**: 20ºC, 350ºC, 500ºC, 600ºC and 700ºC

Future work

Parameters to be analysed in shear buckling study

- Different **shear effort diagrams**:



- Contribution from flanges to shear buckling
- Interaction between shear and bending moment

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Thank you!

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