High strength steel at elevated temperature

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Presentation overview

- Introduction
- Material properties at elevated temperature
- Buckling behaviour at elevated temperature
- Future work

High strength steels (HSS)?

Name	Yield Strength (MPa)
Mild/Normal Strength Steel	<460
High Strength Steel	**460 - 700**
Very High Strength Steel	700 - 1100

The industrial need

Sustainable Structural Design → High Performance Materials → High Strength Steel?

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Challenges?

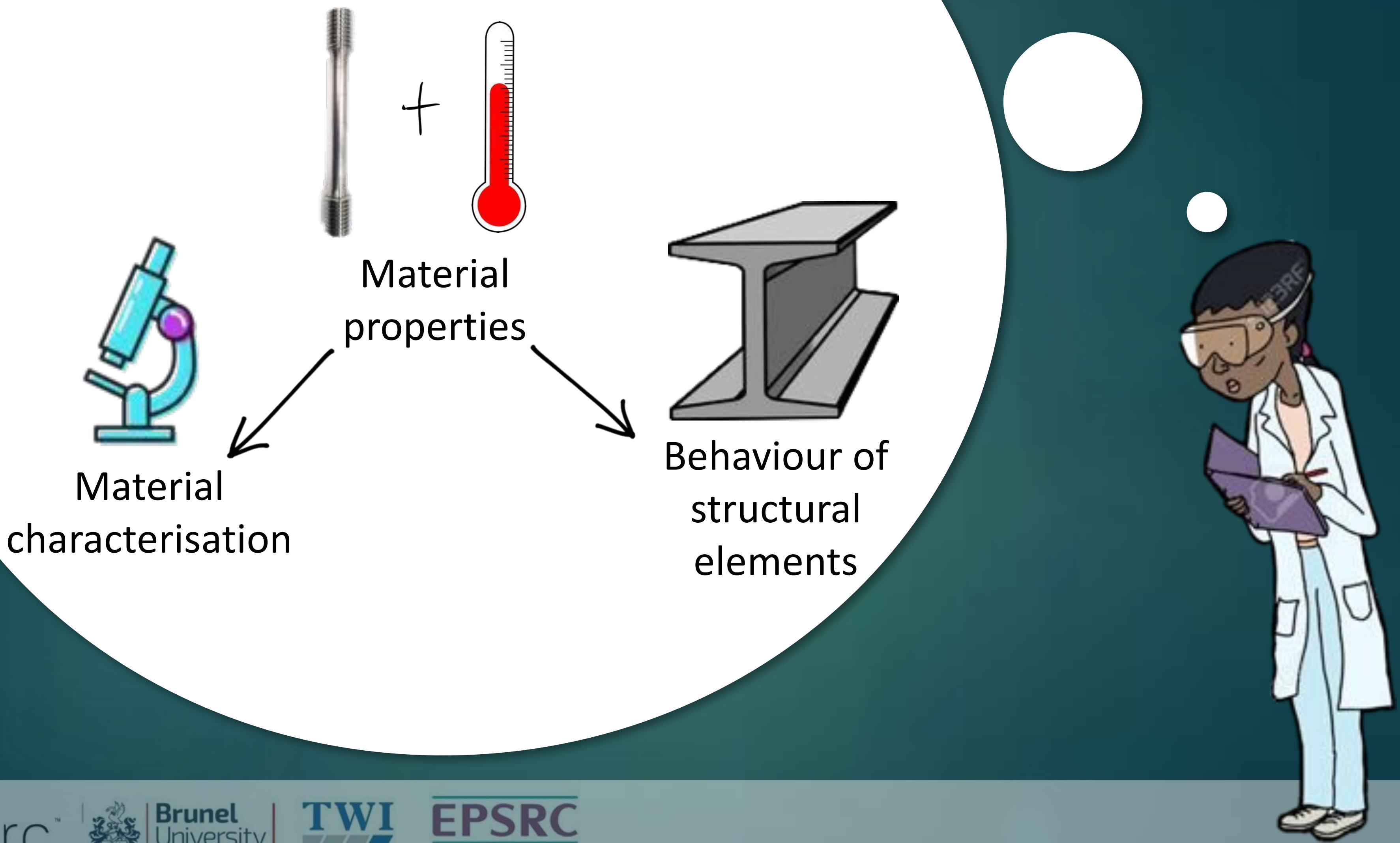
Cost

Instability & Serviceability

Welding

Lack of design guidelines

PhD:
HSS in fire conditions



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Commercial steels

	Grade	σ_y (N/mm²)	Plate Thickness (mm)	Manufacturing Process
Steel A	S690QL	690	16	Quenched and Tempered
Steel B	S700MC	700	12	TMCP + Cold-Formed

PhD: HSS in fire conditions

Material
properties

Material
characterisation

Behaviour of
structural elements

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Test method:

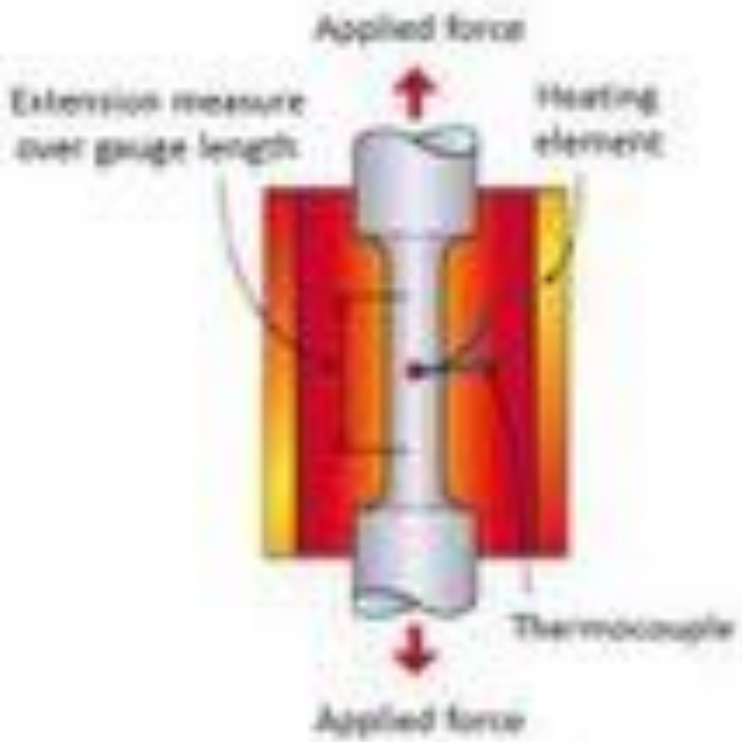
Tensile tests at elevated temperature

1. Isothermal conditions

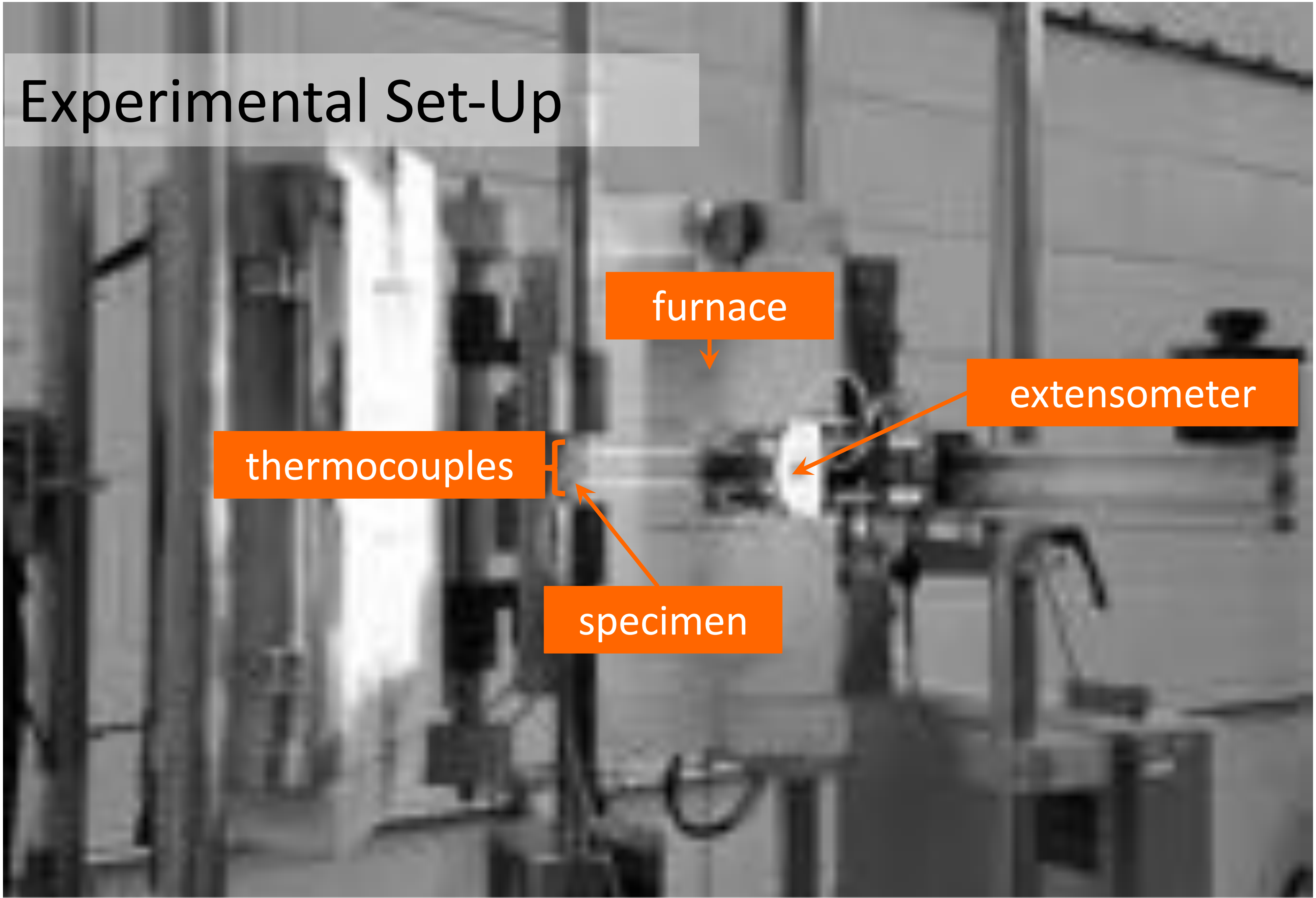
- Constant temperature (e.g. 100°C intervals)
- Vary strain or load at a constant rate (e.g. 0.005/mm)

2. Anisothermal conditions

- Constant load (e.g. 80 N/mm^2 intervals)
- Vary temperature (e.g. 10°C/min)



Experimental Set-Up



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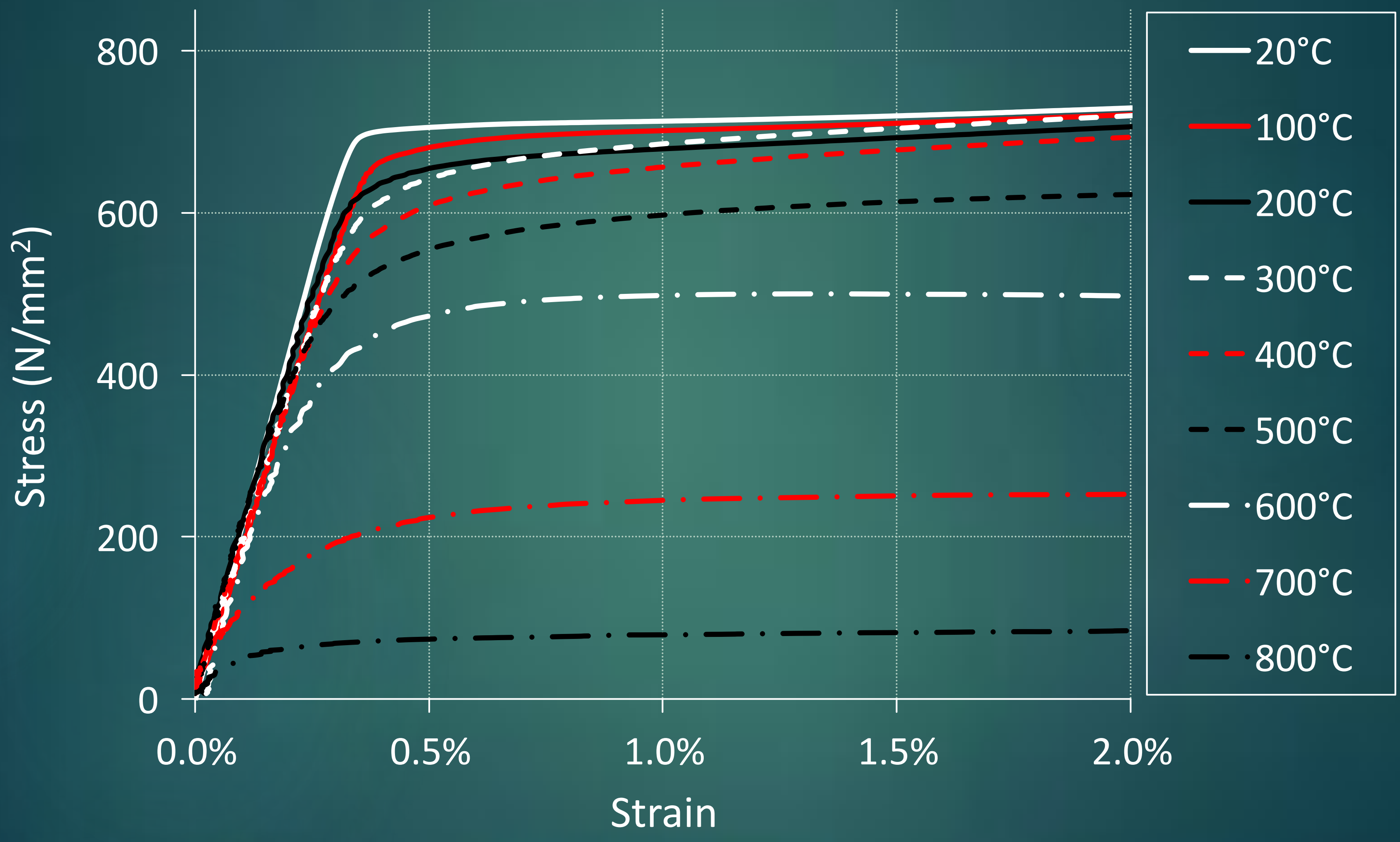
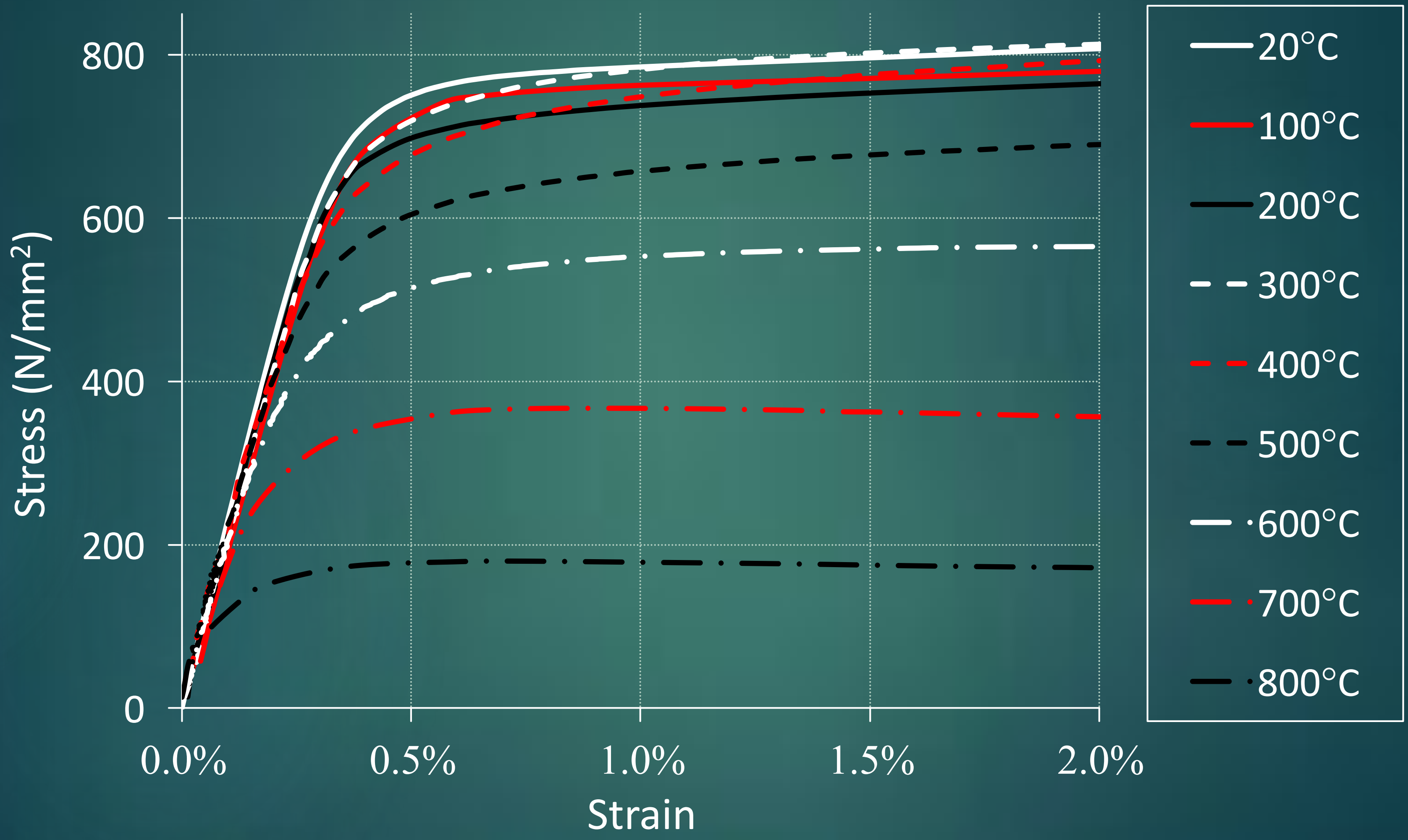
Isothermal tests - Steel A (S690QL)



Isothermal tests - Steel B (S700MC)



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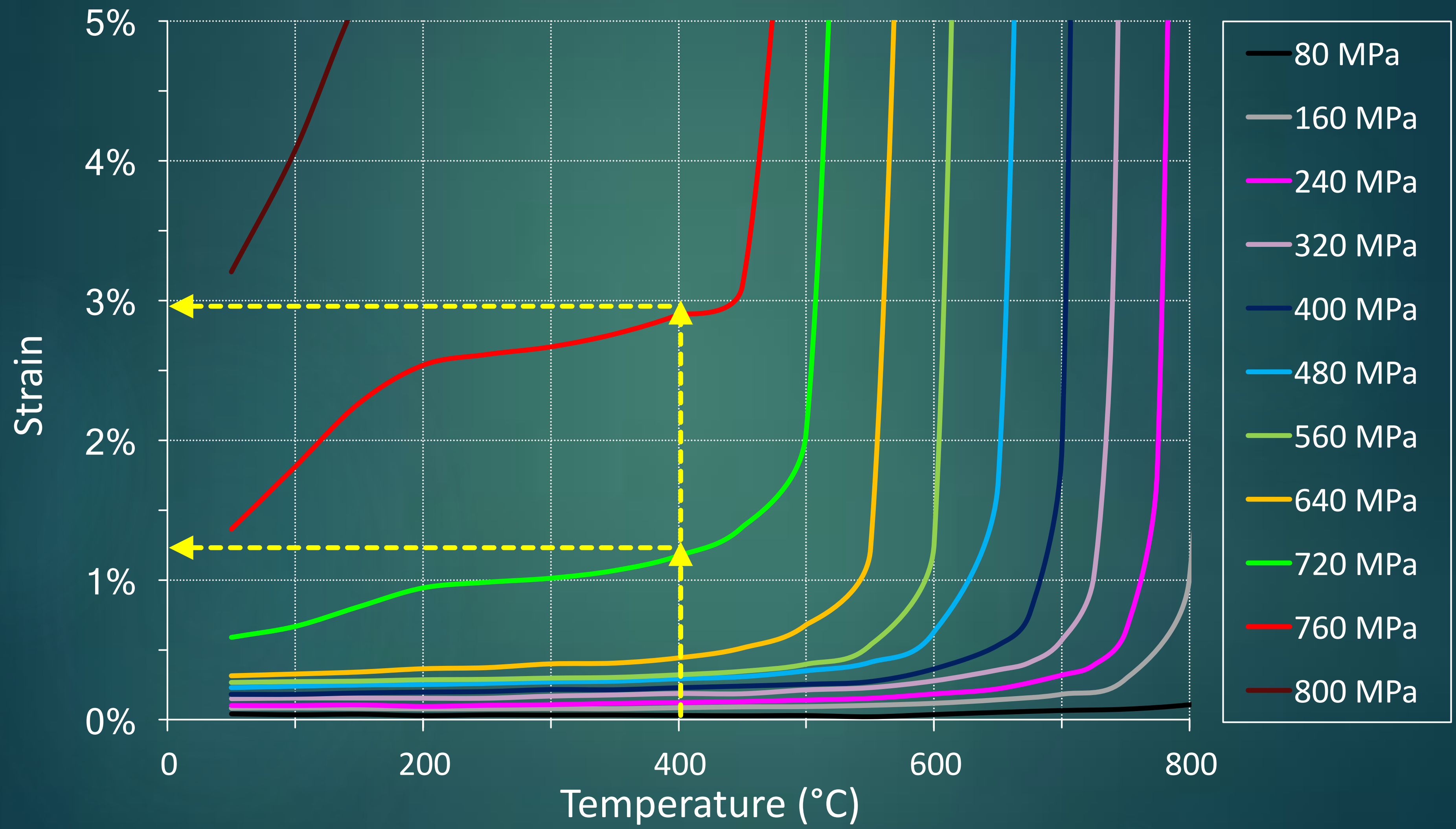
Anisothermal results Steel B (S700MC)



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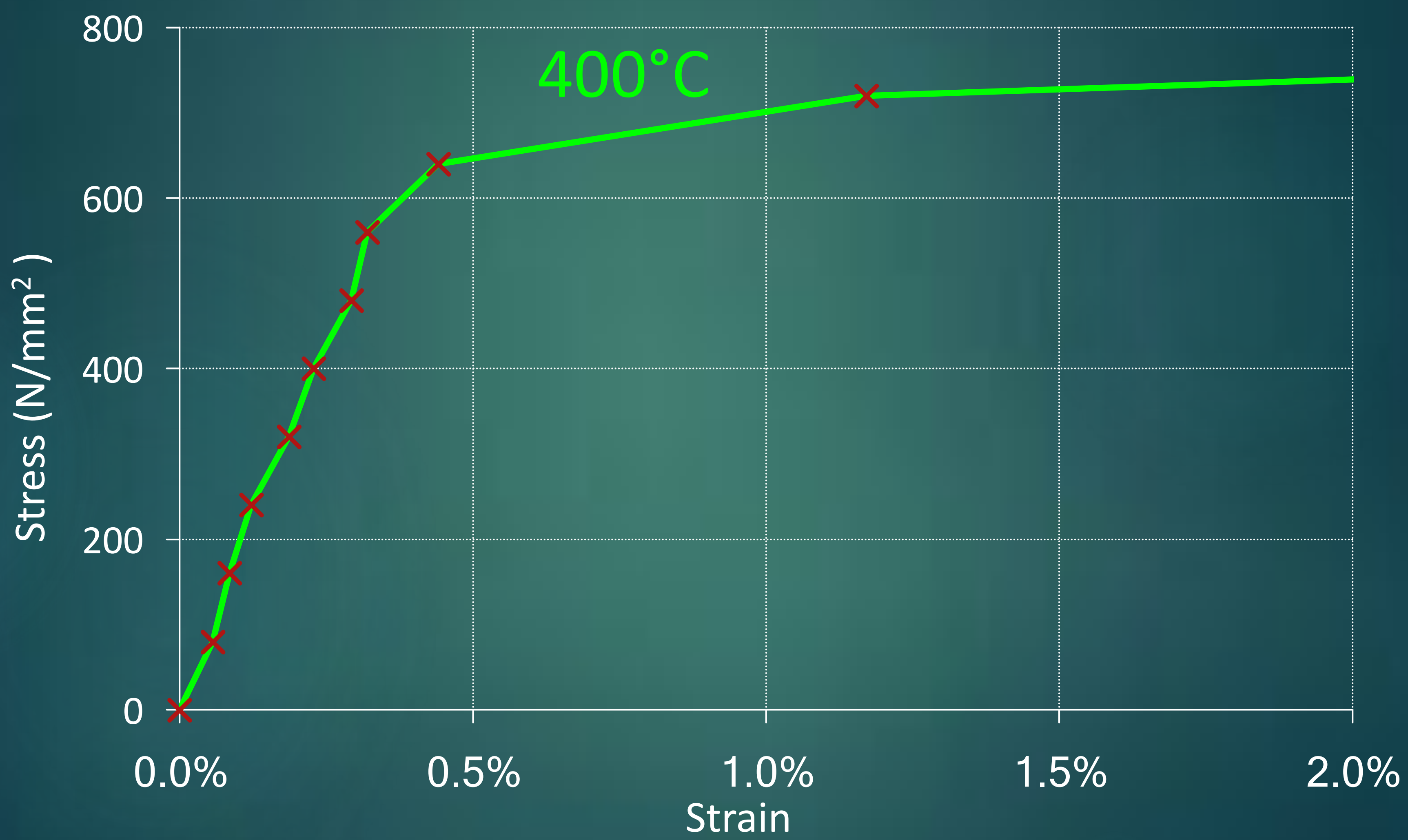
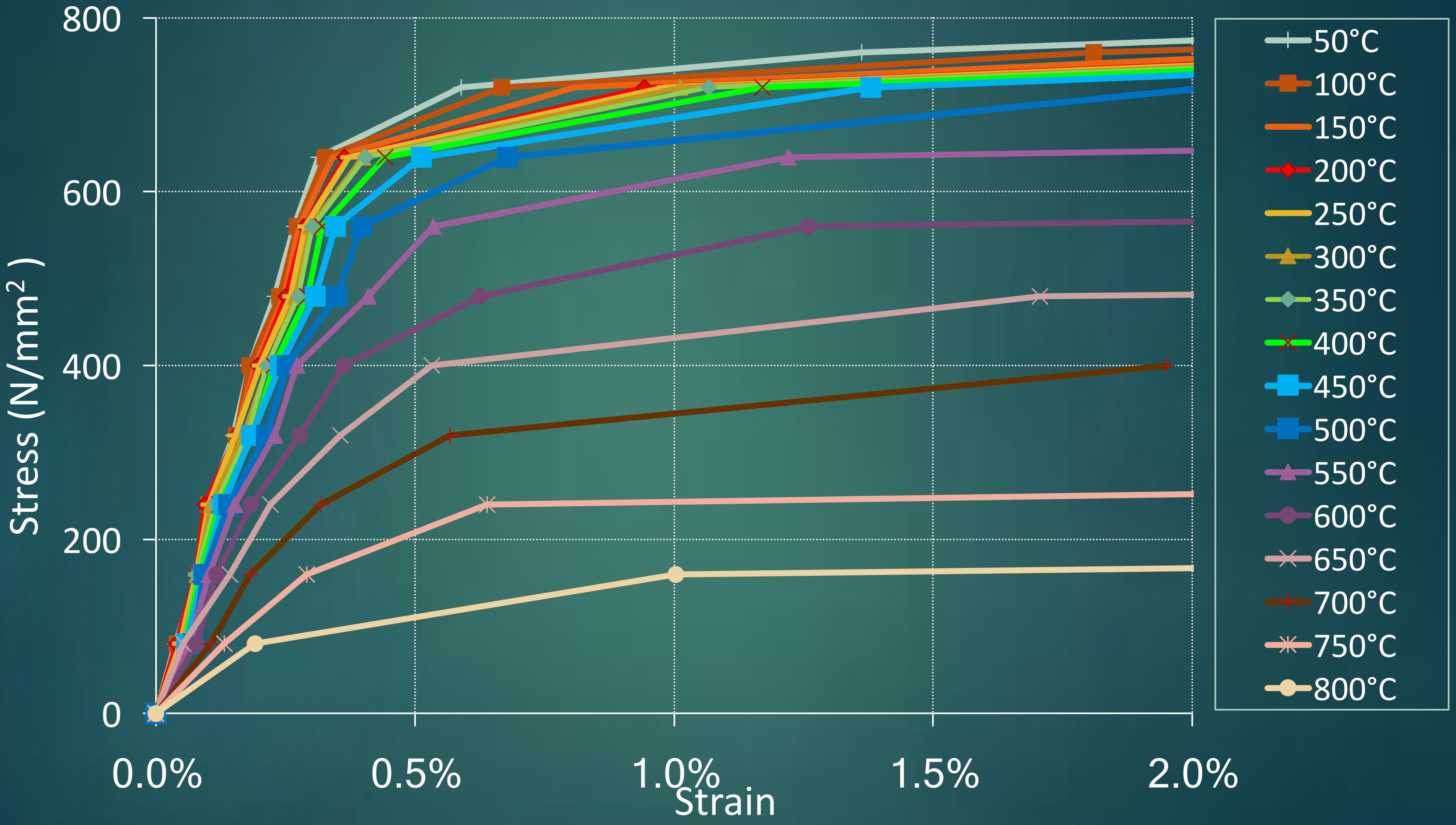
Anisothermal results Steel B (S700MC)



Anisothermal results Steel B (S700MC)



Eurocode approach

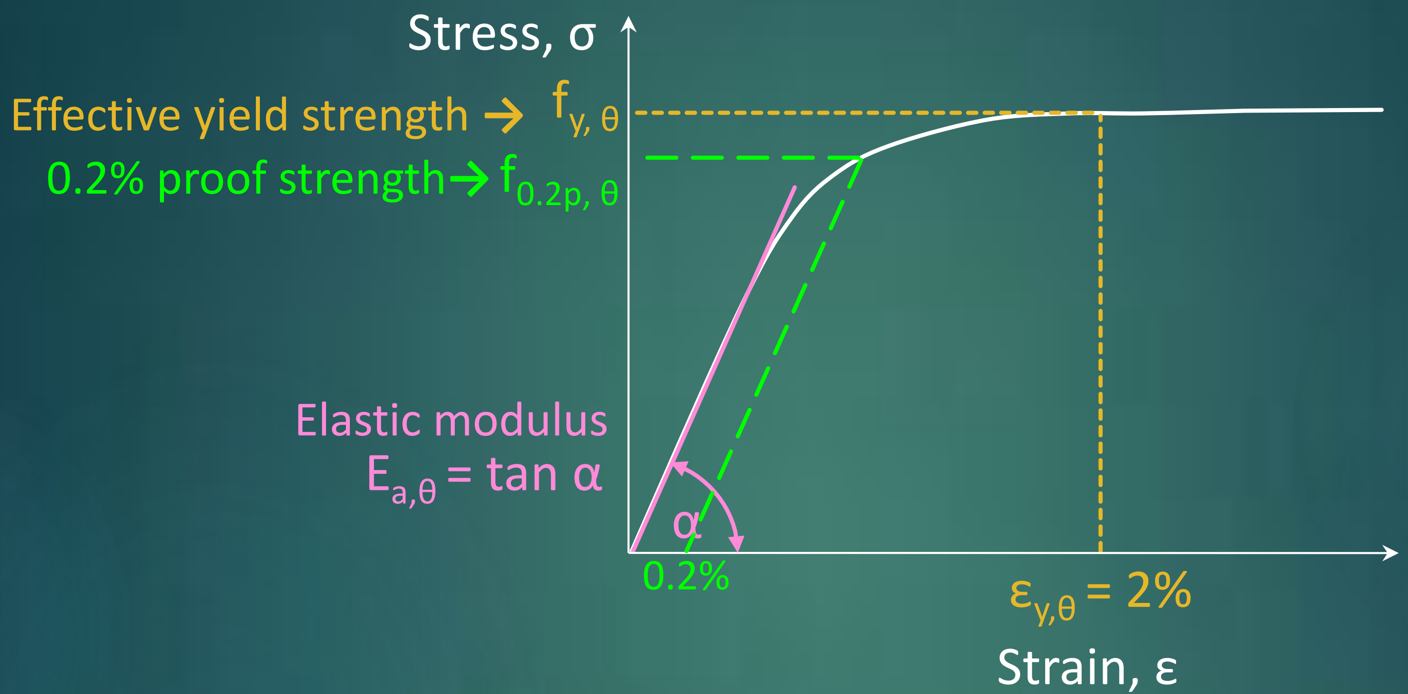


Table. Average mechanical properties at ambient temperature

	$f_{0.2p,20}$ (N/mm²)	$f_{y,20}$ (N/mm²)	$E_{a,20}$ (GPa)
Steel A (S690QL)	706.3	739.3	199.3
Steel B (S700MC)	749.3	800.7	224.7

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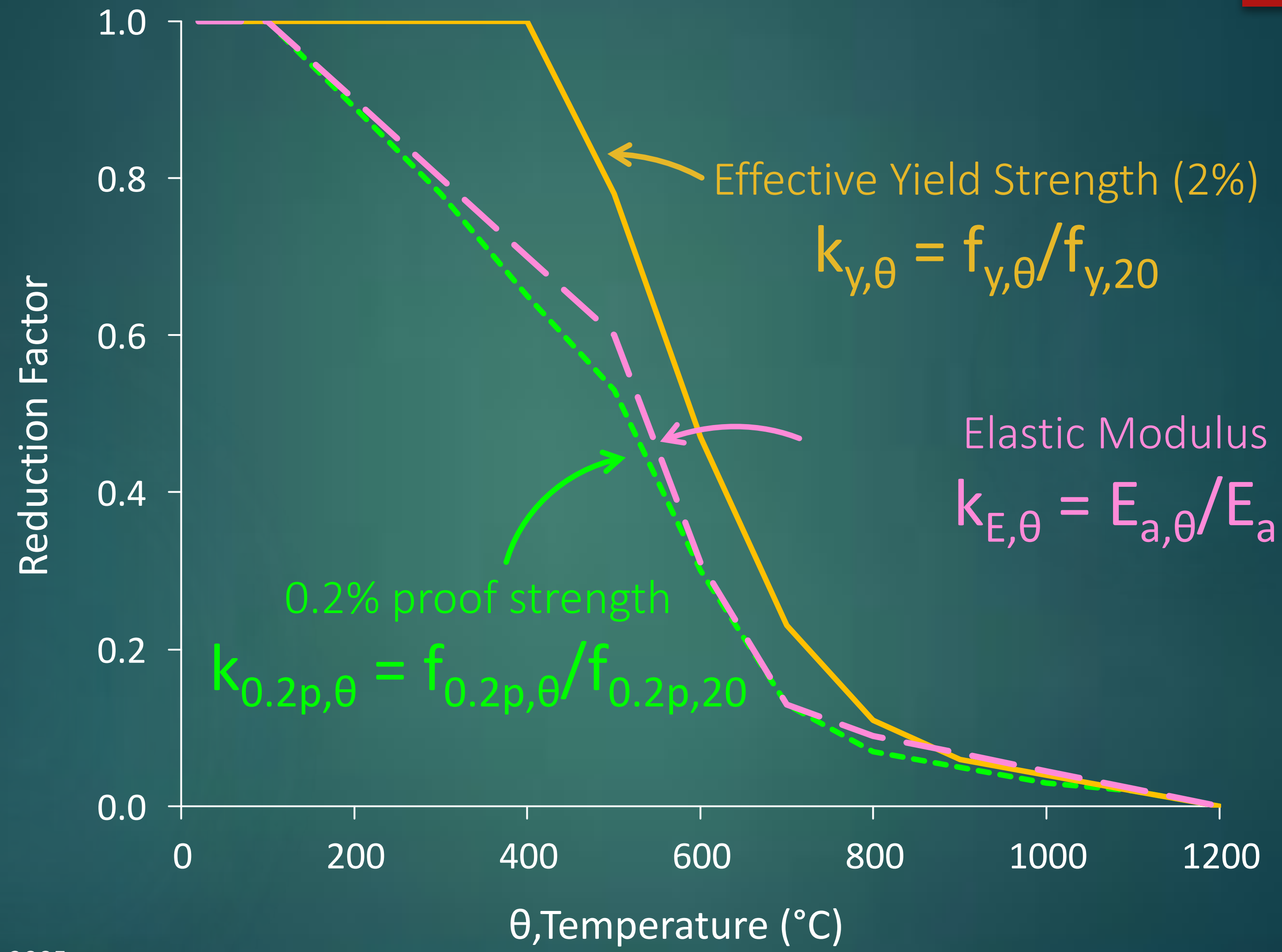
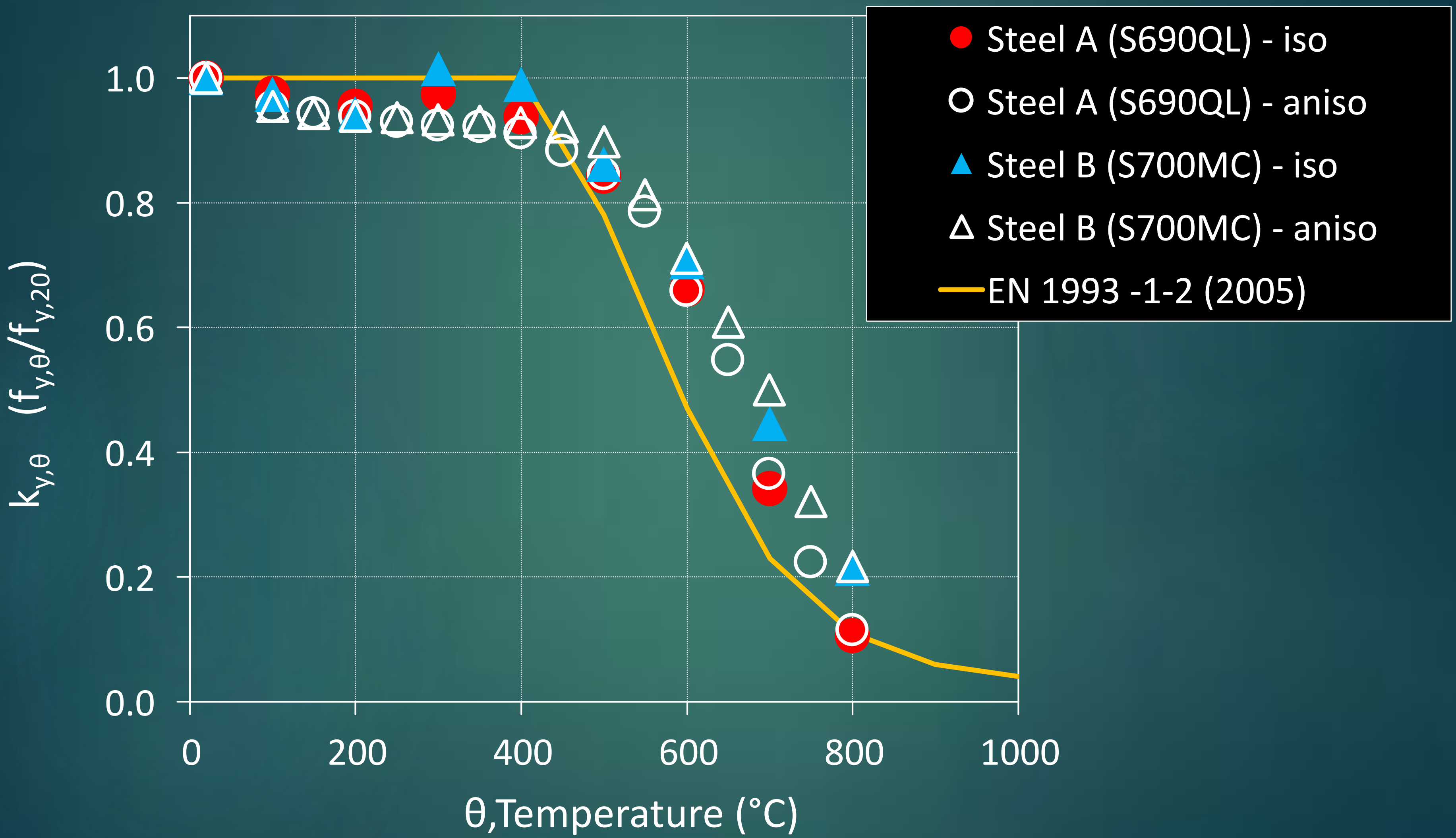
Reduction Factors



Source: EN 1993-1-2: 2005

Effective yield strength, $f_{y,\theta}$ (2%)



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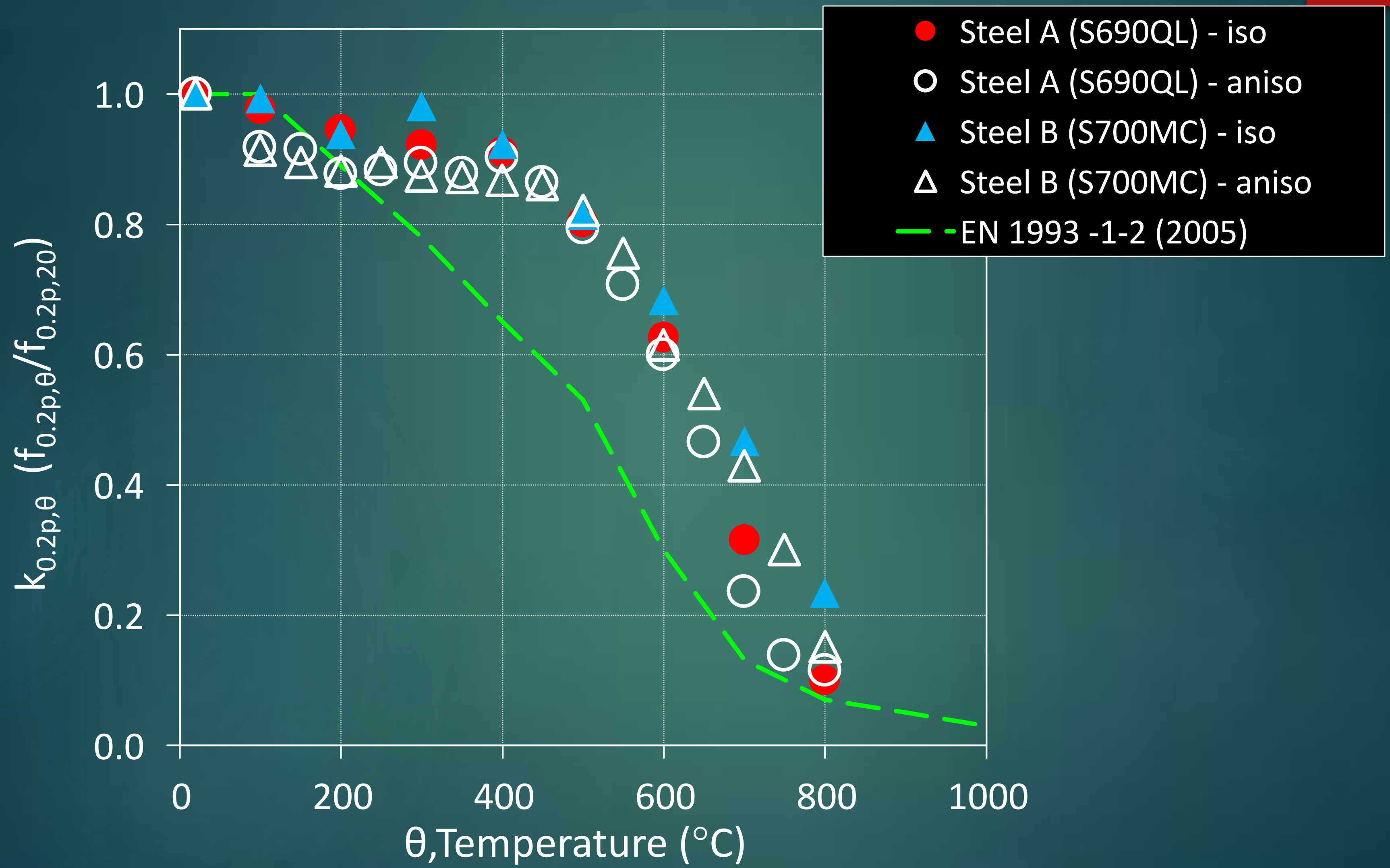
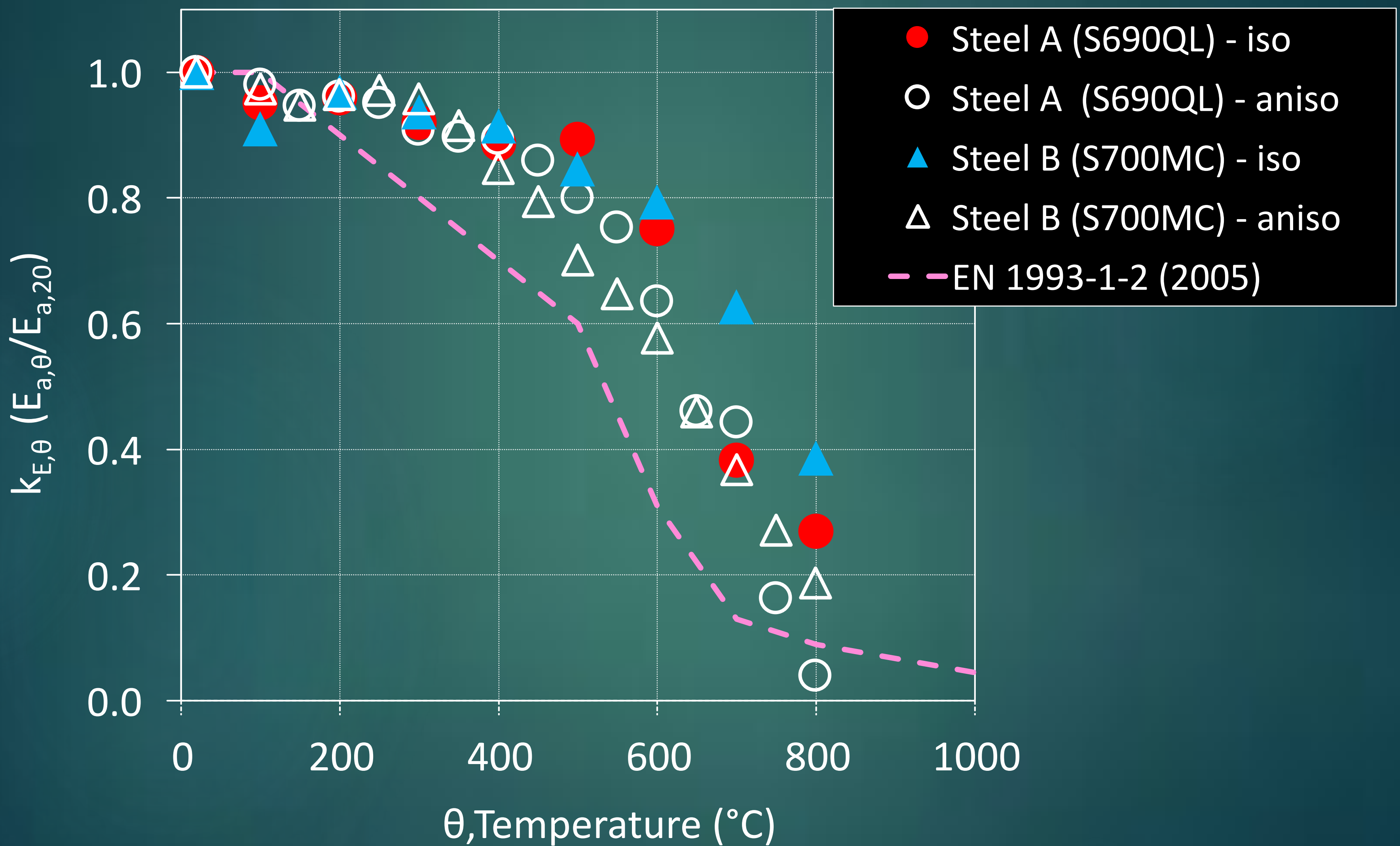
0.2% proof strength, $f_{0.2p,\theta}$



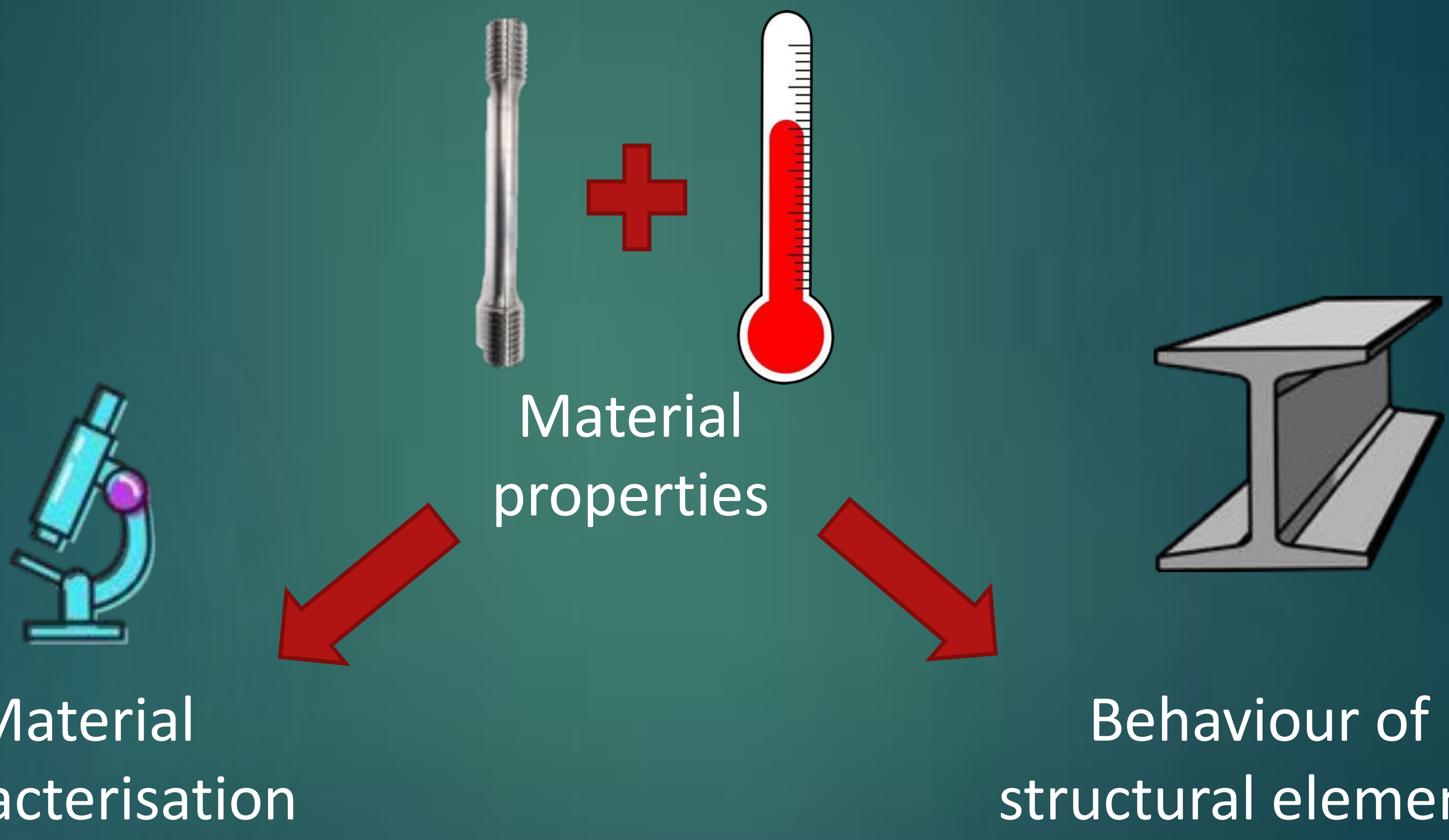
Elastic modulus, $E_{a,\theta}$



Material properties - summary

- $f_{0.2p,\theta}$, $f_{y,\theta}$ (2%) and $E_{a,\theta}$ obtained from isothermal and anisothermal tensile tests, 20 – 800°C
- Steel B (S700MC) had better strength retention properties than Steel A (S690QL) at temperatures up to 800°C
- Does the Eurocodes safely predict the strength and stiffness reduction factors for Steel A and B?
  - No – it can be unconservative and over predict the strength and stiffness at elevated temperatures when normalised by the average property at ambient temperature

PhD: HSS in fire conditions



Behaviour of steel columns

buckling

Windsor tower, Madrid (2005)

Critical buckling load, N_{cr}:

elastic modulus

$$N_{cr} = \frac{\pi^2 EI}{L^2}$$

area moment of inertia of the cross-section

unsupported length of the column

Validation model

#1 Wang & Gardner (2017)

Steel Grade	Section size	Length (mm)
S460N	100 x 100 x 5	858 - 2949
S690Q	50 x 50 x 5 100 x 100 x 5.6	426 – 2150 858 - 2950

- Ambient temperature
- End conditions: pinned

#2 Pauli *et al.* (2012)

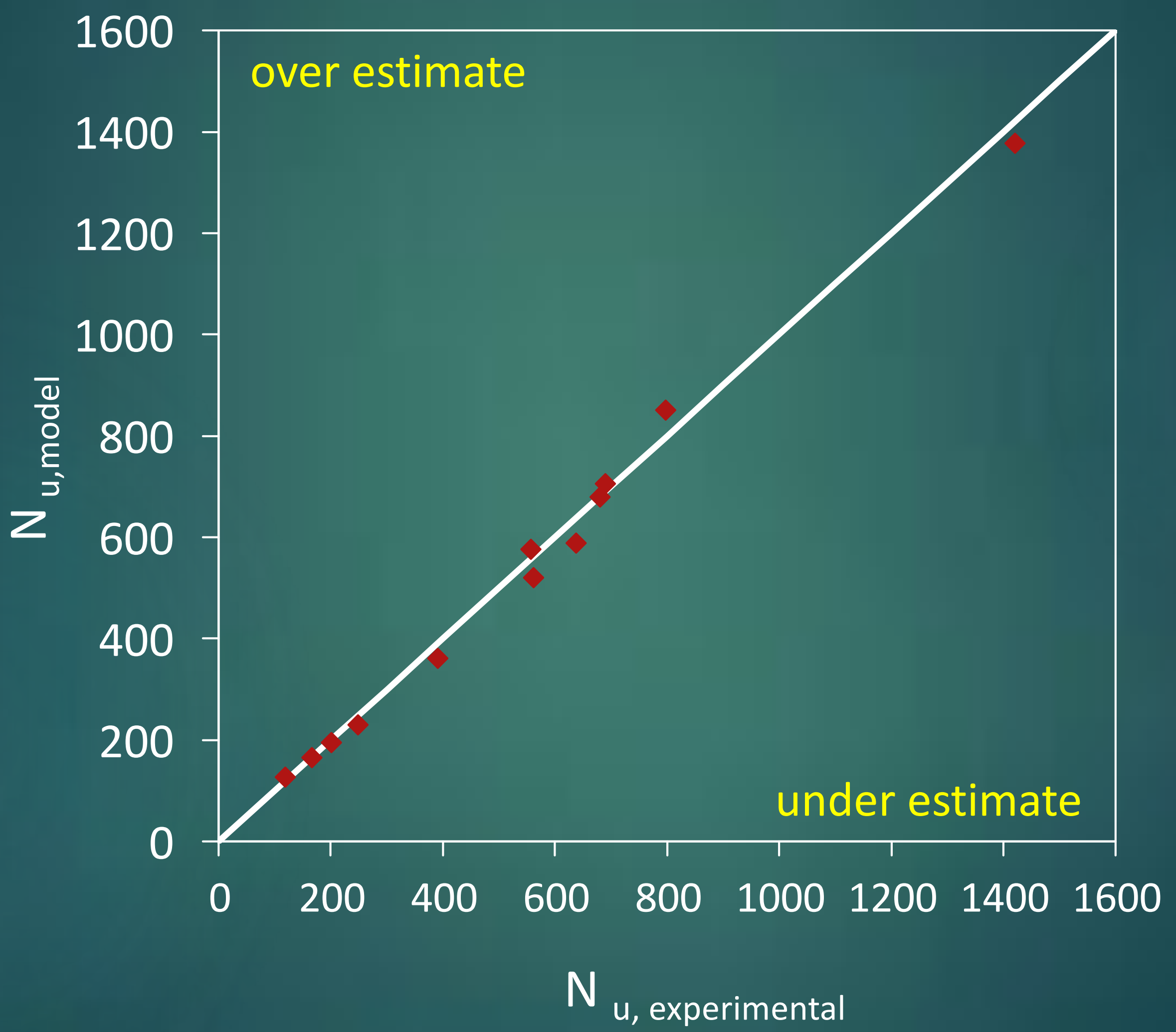
Steel Grade	Section size	Length (mm)
S355	160 x 160 x 5 60 x 120 x 3.6	480, 1840 360, 1840

- Isothermal conditions: 400, 550, 700°C
- End conditions:
  - Stub columns: fixed
  - Long columns: pinned

Description of numerical model

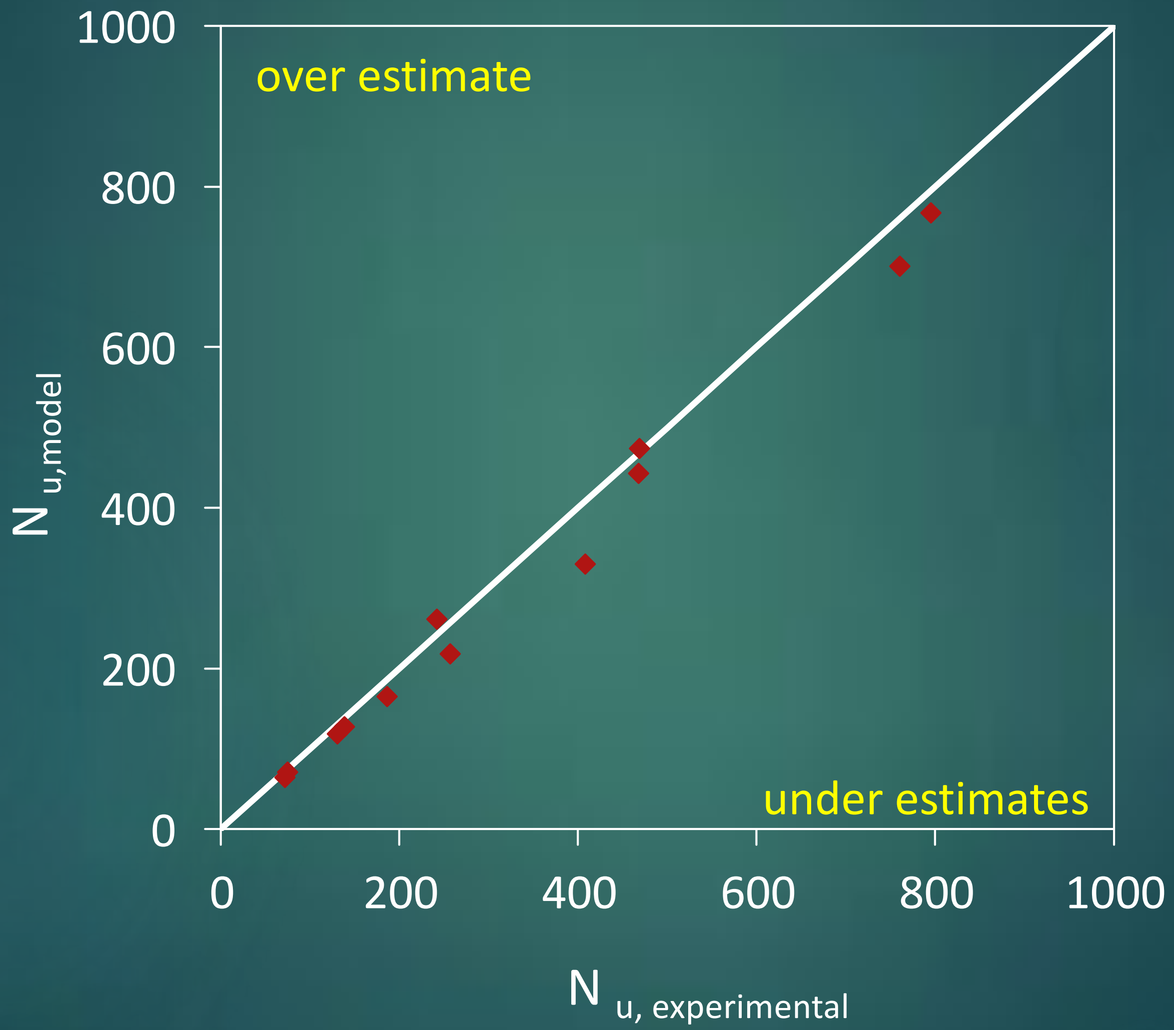
- Software: ABAQUS
- Element type: S4R - four-node doubly curved general-purpose shell element with reduced integration
- Mesh size: equal to the thickness of the cross-section area
- Boundary conditions test boundary conditions were replicated by restraining suitable displacement and rotation degrees of freedom of the column ends.
- Stress-strain response: measured stress-strain curves at ambient and elevated temperatures were incorporated in the FE models in terms of the true stress and plastic strain
- Load response: modified Riks method was used to trace the load-deformation response and determine the ultimate test load
- Residual stress: was not considered due to low measured amplitudes and minimal influence on similar fabricated columns reported by Wang *et al.* (2017)

Validation model results: Wang *et al.* (2017)



Mean: 1.01
COV: 0.05

Validation model results: Pauli *et al.* (2012)

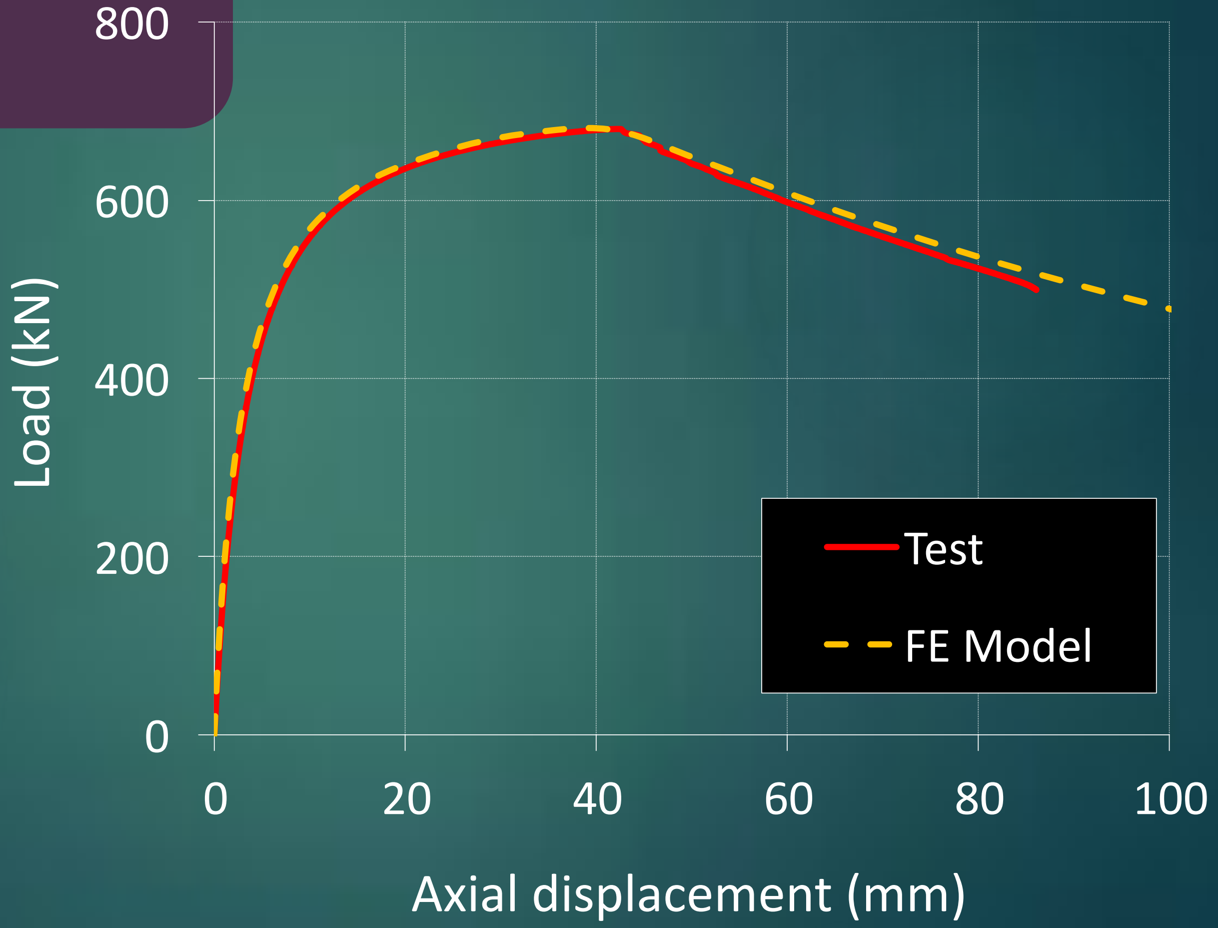


Mean: 1.09

COV: 0.083

Load – deflection response

Example: Wang & Gardner(2017)
Material: S690QL
Section size: 50 x 50 x 5
Length: 1700 mm



Parametric studies:

Buckling behaviour of HSS columns at elevated temperature

Steel Grade	Section size	Cross-section classification	Temperature (°C)
Steel A (S690QL)	100 x 100 x 5.6	Class 1	20, 100, 200, 300, 400, 500, 600, 700, 800
	100 x 50 x 4.0	Class 1	
	100 x 100 x 5.6	Class 3	
	100 x 50 x 4.0	Class 3	
Steel B (S700MC)	100 x 100 x 5.6	Class 1	20, 100, 200, 300, 400, 500, 600, 700, 800
	100 x 50 x 4.0	Class 1	
	100 x 100 x 5.6	Class 3	
	100 x 50 x 4.0	Class 3	

- Stress strain response (isothermal tests) → true stress and plastic strain
- Global imperfection: L/1000
- Geometric imperfection based on Dawson and Walker model
- RHS: major and minor axis
- End conditions: Pinned

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Eurocode approach

The buckling design resistance ($N_{b,fi,t,Rd}$) for Class 1-3 members:

$$N_{b,fi,t,Rd} = \frac{\chi_{fi} A k_{y,\theta} f_{y,20}}{\gamma_{M,fi}} \Rightarrow \chi_{fi} = \frac{N_{b,fi,t,Rd}}{A k_{y,\theta} f_{y,20}}$$

where

χ_{fi} is the reduction factor for flexural buckling in the fire design situation,

A is the gross area cross-section of the structural member,

$f_{y,20}$ is the effective yield strength, taken as the stress at 2% total strain at ambient temperature

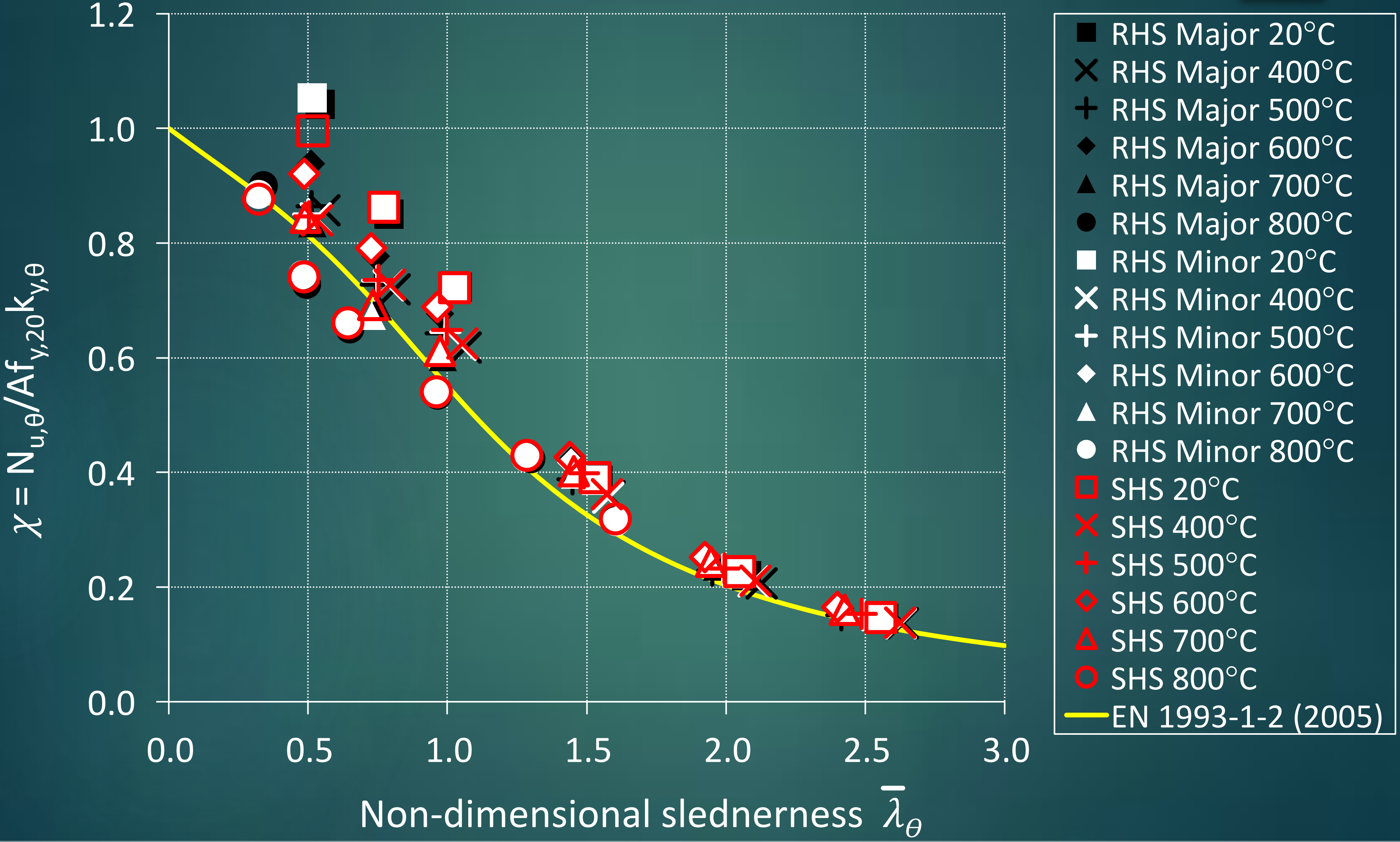
$k_{y,\theta}$ is the reduction factor for the effective yield strength (i.e. $k_{y,\theta} = f_{y,\theta}/f_{y,20}$)

$\gamma_{M,fi}$ is the partial factor for fire situation which is taken as 1.0 in this study

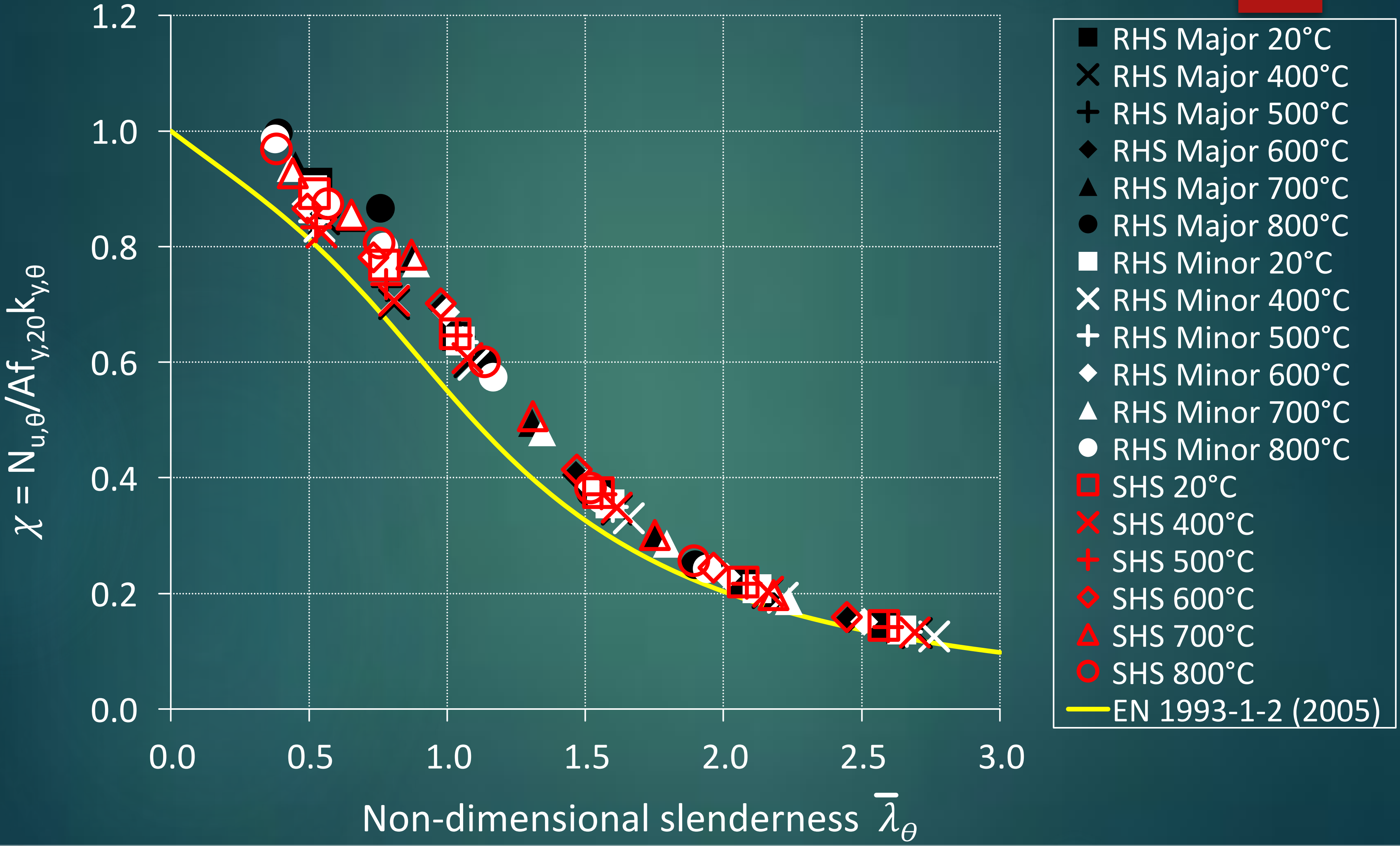
The non-dimensional slendrness ($\bar{\lambda}_\theta$) for Class 1-3 members:

$$\bar{\lambda}_\theta = \bar{\lambda}\,(k_{y,\theta} / k_{Ea,\theta})^{0.5}$$

Buckling coefficients – Steel A (S690QL)



Buckling coefficients - Steel B (S700MC)



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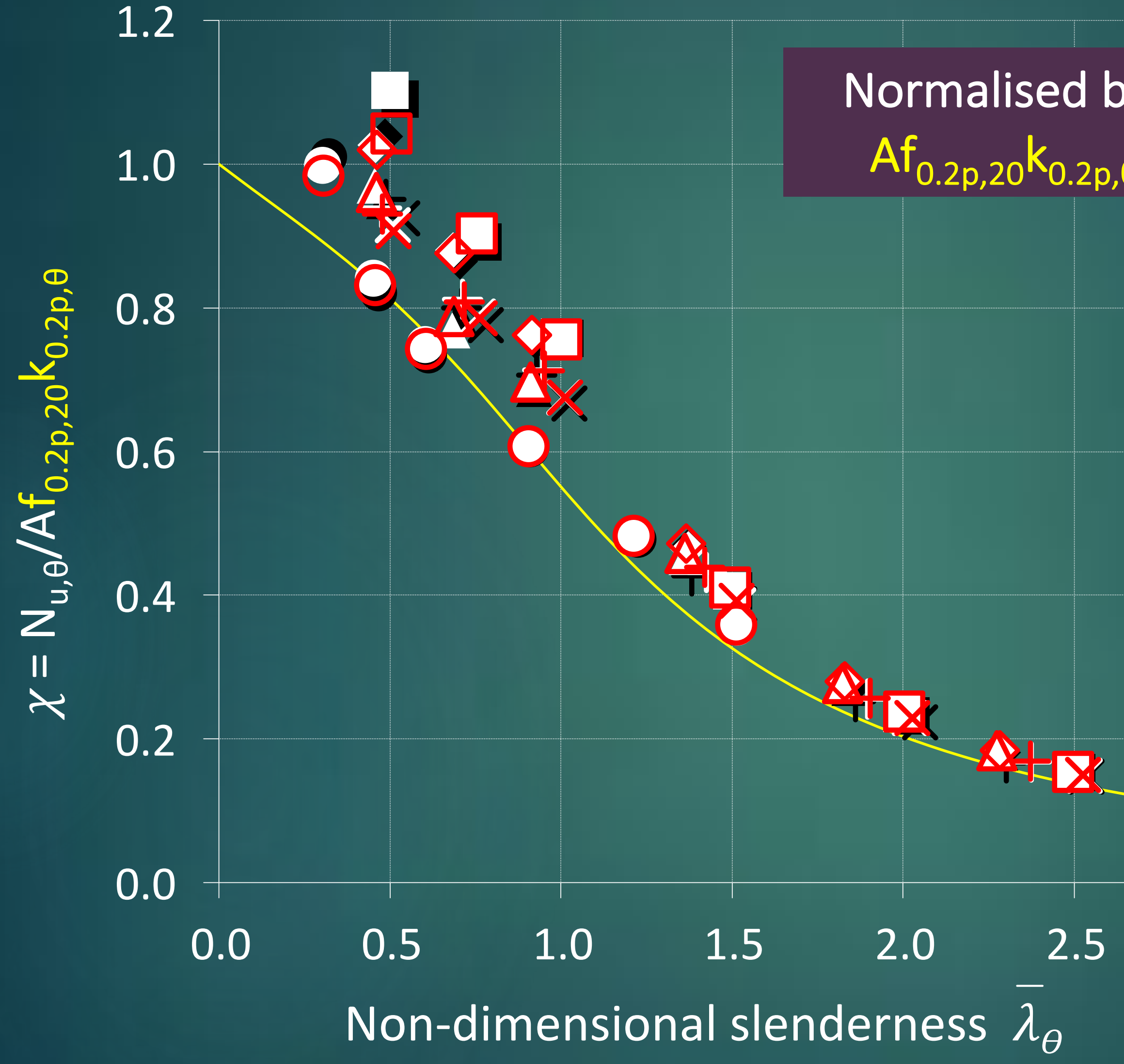
What happens if $f_{0.2p,\theta}$ is used instead of $f_{y,\theta}$ (2.0%)?

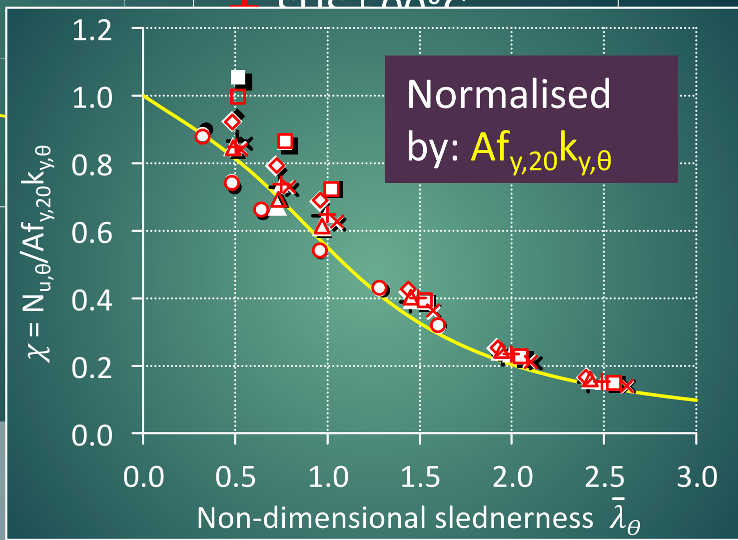
$$\chi_{fi} = \frac{N_{b,fi,t,Rd}}{Ak_{y,\theta} f_{y,20}} \quad \text{against} \quad \bar{\lambda}_{\theta} = \bar{\lambda}\,(k_{y,\theta} \,/\, k_{Ea,\theta})^{0.5}$$

What happens if $f_{0.2p,\theta}$ is used instead of $f_{y,\theta}$ (2.0%)?

$$\chi_{fi} = \frac{N_{b,fi,t,Rd}}{A k_{0.2p,\theta} f_{0.2p,20}} \quad \text{against} \quad \bar{\lambda}_\theta = \bar{\lambda}\,(k_{0.2p,\theta} / k_{Ea,\theta})^{0.5}$$

Buckling coefficients – Steel A (S690QL)





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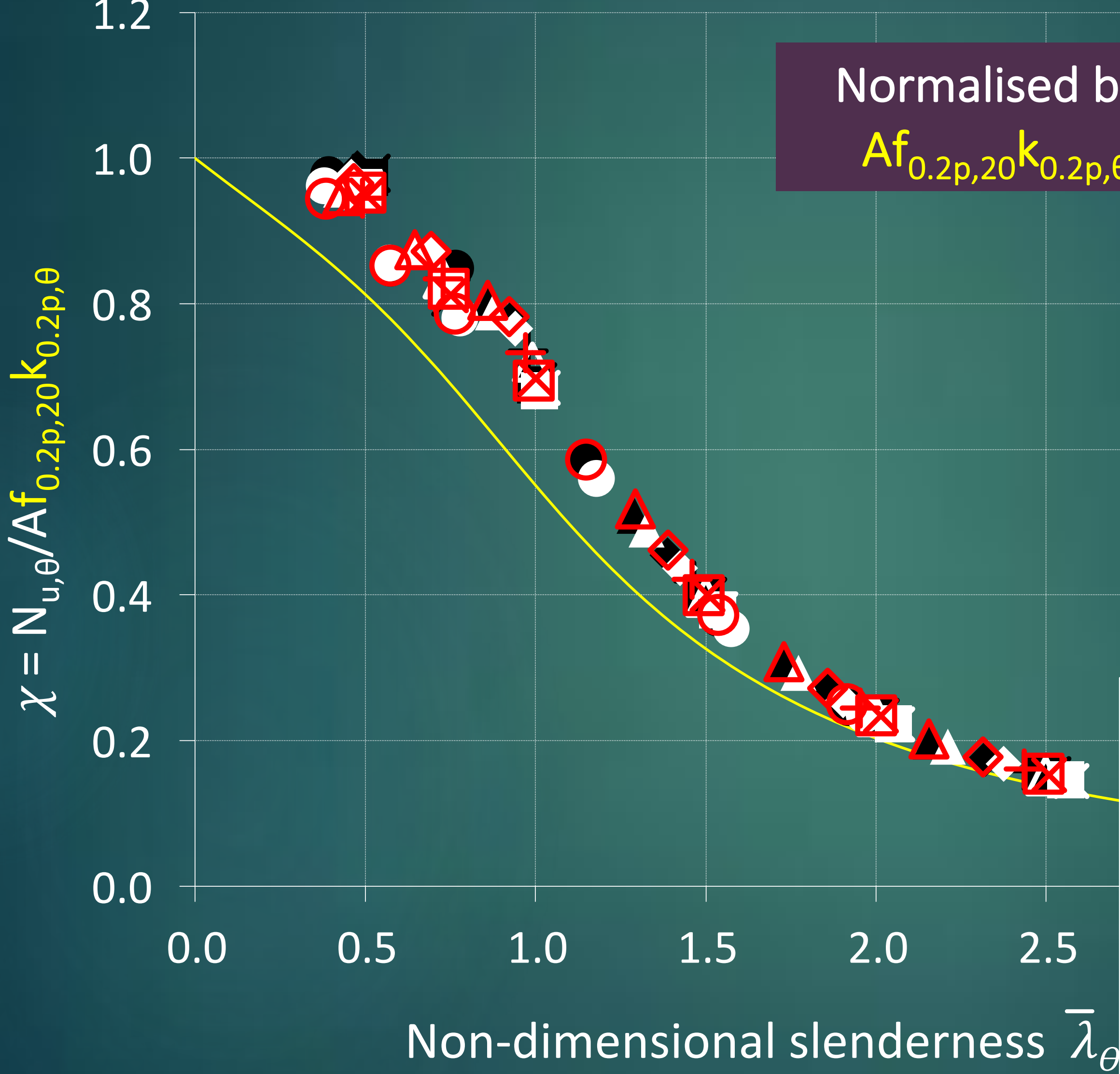

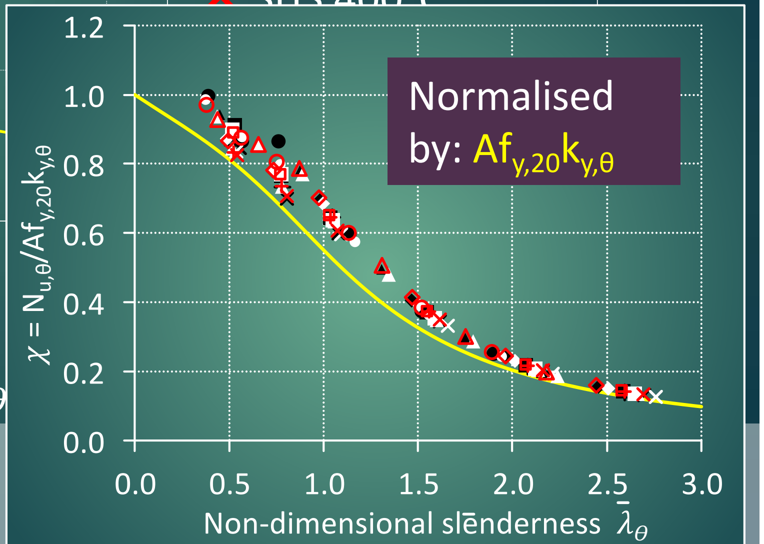

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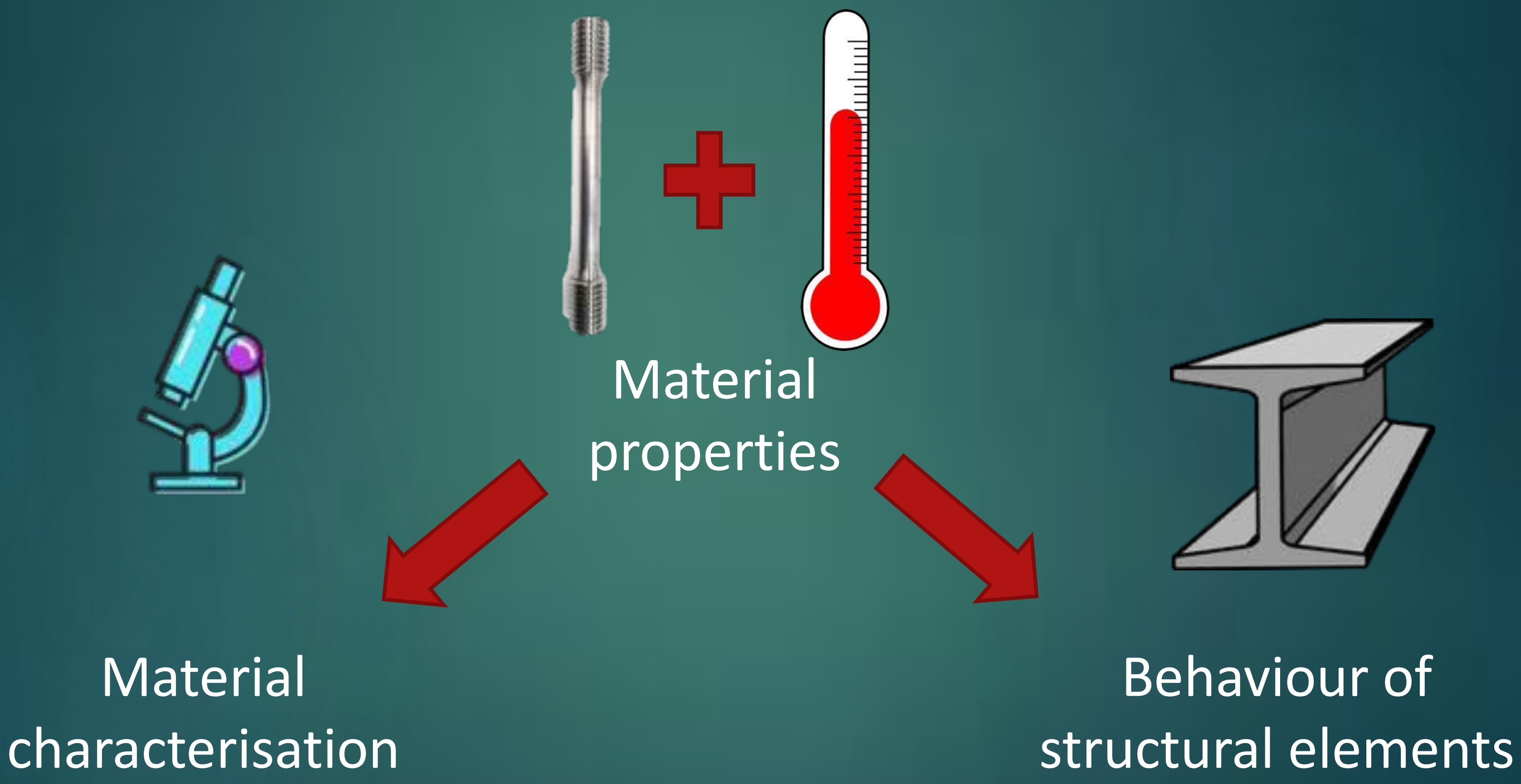
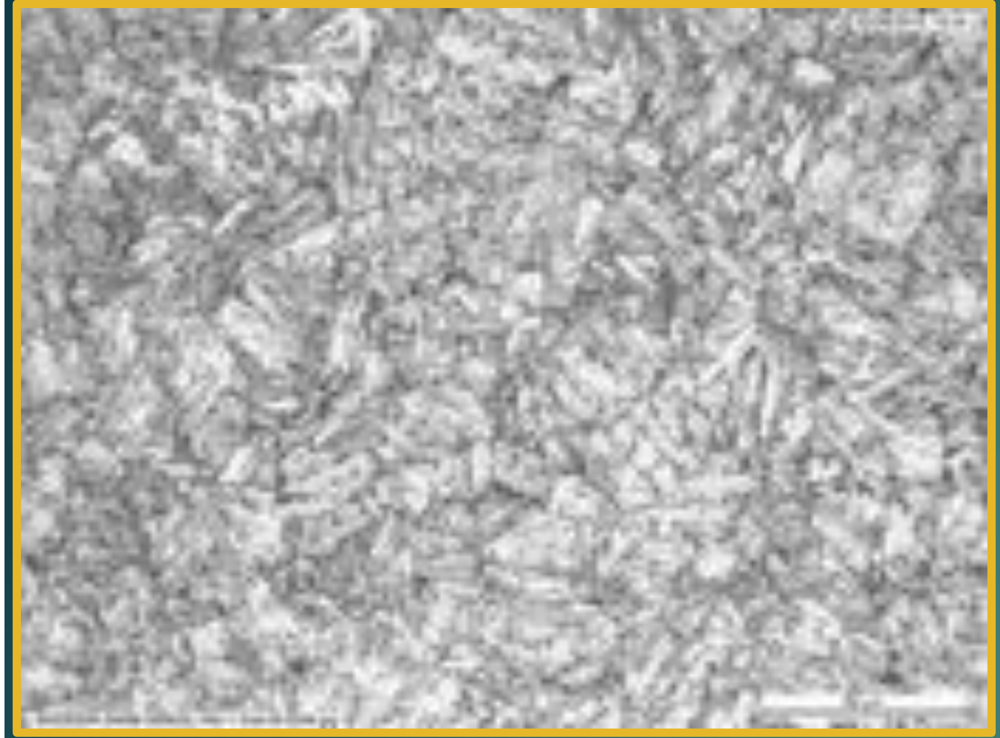
Buckling behaviour - summary

- Numerical study on the flexural behaviour HSS columns (S700MC and S690QL)
- The production route (QT or TMCP) influences the buckling behaviour at ambient and elevated temperatures
- The Eurocode is generally conservative with respect to the buckling coefficients and safely predicts the buckling resistance of steel B (S700MC), while a lower buckling curve may be needed for steel A (S690QL)
- The $f_{0.2p,\theta}$ is better parameter than $f_{y,\theta}$ (2.0%) for deriving buckling fire curves

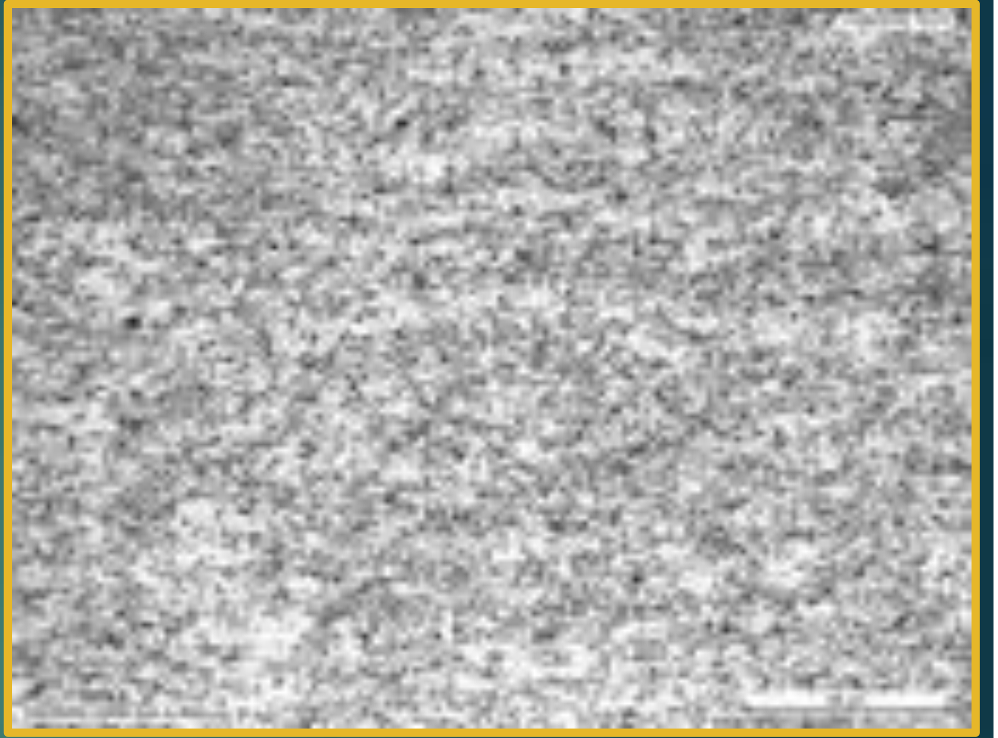
Future work


Material
properties
Material
characterisation
Behaviour of
structural elements

Steel A (S690QL)



Steel B (S700MC)



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Steel B (S700MC)	749.3	800.7


Oresund Bridge – Malmö, Sweden (personal photo)

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