COST Action C26

Urban habitat constructions under catastrophic events

Prague, Czech Republic, 30-31 March 2007

WG1- Fire resistance

Collected contributions

Structural Members Behaviour

Nuno Lopes

B. Uppfeldt
The Sweden Institute of Steel Construction, Sweden

M. Veljkovic
Luleå University of Technology, Sweden

Nuno Lopes, Paulo Vila Real
University of Aveiro, Portugal

Luís Silva
University of Coimbra, Portugal

Jean-Marc Franssen
University of Liege, Belgium

Viktor Gribniak, Darius Bačinskas, Gintaris Kaklauskas
Vilnius Gediminas Technical University, Lithuania

E. Nigro, G. Cefarelli
Department of Structural Engineering, University of Naples “Federico II”, Italy

Contributions from four papers:

- Class 4 stainless steel box columns in fire
- Stainless steel structural elements in case of fire
- Non-linear modelling of reinforced concrete beams subjected to fire
- Some remarks on the simplified design methods for steel and concrete composite beams

Class 4 Stainless Steel Box Columns in Fire

B. Uppfeldt
The Swedish Institute of Steel Construction, Stockholm, Sweden

M. Veljkovic
Luleå University of Technology, Sweden

Introduction

- A study of **stainless steel cold-rolled box columns** at elevated temperatures is presented, which is a part of an on-going RFCS project ”Stainless Steel in Fire”.

- Experimental results of six, **class 4**, stub columns at elevated temperature, were used to evaluate the FE model.

- The FE analysis obtained shows that the critical temperature **was closely predicted.**

- A parametric study was the basis to check the quality of prediction of a **newly proposed improvement for design rules** of class 4 cross-sections in fire according to Part1-4 and Part 1-2 of EC3 (stainless steel and fire design part respectively).

Experiments

- Four cold rolled stainless steel stub columns, $\lambda < 0.1$, with cross-section class 4 were tested at the **ambient temperature**, Ala-Outinen (2005). Four strain-gauges were used to measure stresses at mid column.

- The geometry of the columns and local imperfections were measured. The material used in the columns was **EN 1.4301**. Fully restrained ends were achieved in experiments.

- Six unprotected columns were tested at **elevated temperatures**. The test set-up was equivalent to the ambient temperature tests.

- The temperatures were measured with twelve chromel-alumel thermocouples and the axial deformation was measured using transducers.

- The transient procedure was applied, meaning that the axial load was kept constant and the furnace temperature was raised in a controlled way, at the rate of 10° C/min.

FE-Model

Elements

A general-purpose **shell element,** called S4R, within Abaqus/Standard was used.

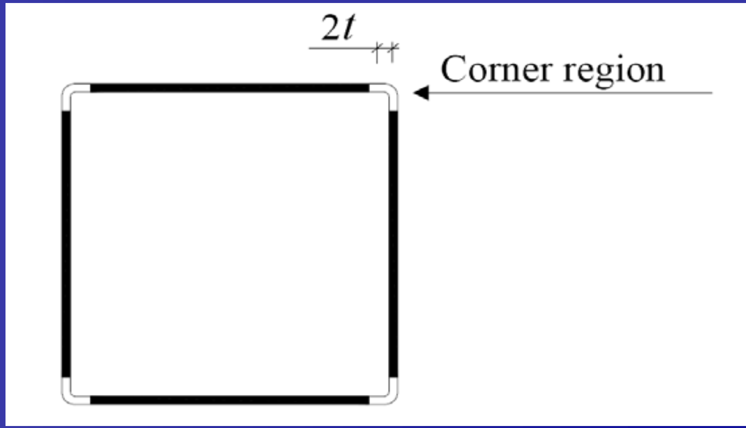
Imperfections

The two types of geometrical imperfections that have to be considered are, **global imperfections** and **local imperfections**.

For the modelling of the tested stub columns the measured local imperfections were used and no global imperfections were introduced.

Material

It is well established that the mechanical properties of stainless steel are strongly influenced by the level of cold-work.



FE-Model

Residual stresses

No residual stresses were introduced in the modelling of the tested columns.

Validation

No. specimen Cross-section	Experiment Temp. °C	FEA Temp. °C	$Temp_{FEA}/Temp_{exp}$
1. 150x150x3	676	716	1.06
2. 150x150x3	720	758	1.05
3. 150x150x3	588	593	1.01
4. 200x200x5	609	482	0.79
5. 200x200x5	685	645	0.94
6. 200x200x5	764	732	0.96

It is concluded that the FE-model predicts the failure temperatures with **good accuracy** for all tests but specimen No. 4 and the general conclusion is that the model is reliable for parametric study.

Development of improved design model for class 4 cross-sections

$$\chi = \frac{1}{\phi + \left(\phi^2 - \bar{\lambda}^2\right)^{0.5}} \leq 1$$

$$\varepsilon_\theta = \varepsilon \left[\frac{k_{E,\theta}}{k_{0.2p,\theta}}\right]^{0,5}$$

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - \bar{\lambda}_0\right) + \bar{\lambda}\right]$$

$$\bar{\lambda}_\theta = \bar{\lambda}\left[\frac{k_{0.2p,\theta}}{k_{E,\theta}}\right]^{0,5}$$

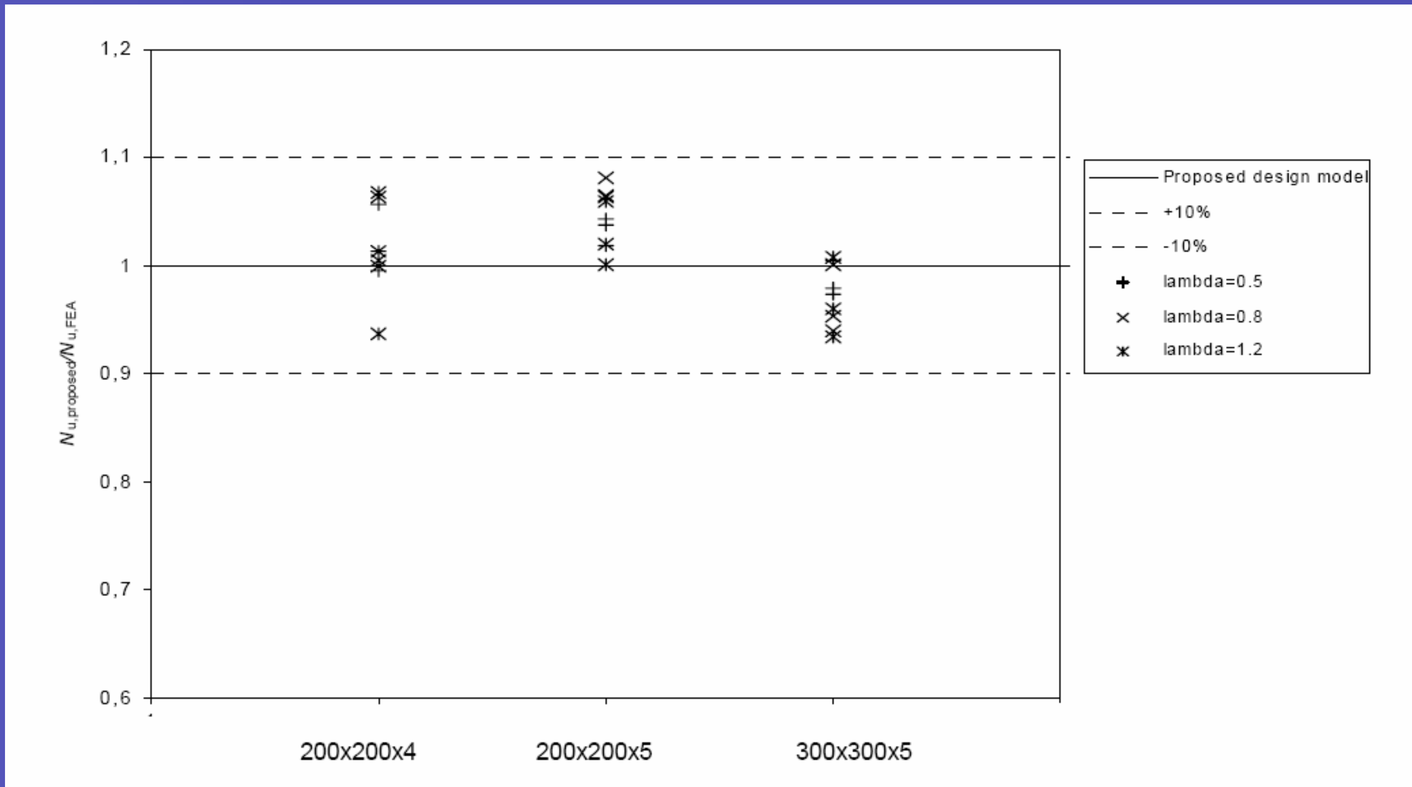
$$\bar{\lambda}_{p,\theta} = \frac{\bar{b}/t}{28,4\varepsilon_\theta\sqrt{k_\sigma}}$$

$$\bar{\lambda}_0 = 0,4$$

$$\alpha = 0,49$$

$$\bar{\lambda}_{0,\theta} = \bar{\lambda}_0\left[\frac{k_{0.2,p,\theta}}{k_{E,\theta}}\right]^{0,5}$$

Development of improved design model for class 4 cross-sections



Conclusions

- Comparison between experiments at the elevated temperature and results obtained from FEA indicates that
  - \- assumptions made for the influence of the **material properties** in the **corners** are **realistic**;
  - \- assumptions for the shape and level of the local buckling, b/200, and global imperfections, L/1000, are consistent with assumptions established at ambient temperature.

- In this work it was made an test that showned that it is possible to use **unprotected** stainless steel columns and fulfil requirement for resistance, **R30**.

- Design recommendations for class 4 cross sections made of austenitic stainless steel presented are **coherent** with part1-2 and part1-4 of EC3.

- The proposed design model is an **improvement** compared to the design model on EN 1993-1-2.

Stainless steel structural elements in case of fire

Nuno Lopes
University of Aveiro, Portugal

Paulo Vila Real
University of Aveiro, Portugal

Luis Silva
University of Coimbra, Portugal

Jean-Marc Franssen
University of Liege, Belgium

Introduction

- The part 1-4 of Eurocode 3 (EC 3) “Supplementary rules for stainless steels” gives design rules for stainless steel structural elements at room temperature and only mentions its **fire resistance** by doing reference to the fire part of EC 3 (**EN 1993-1-2**).

- Carbon and stainless steel exhibit **different stress-strain relationships**.

- This study was made using the program **SAFIR** that has been adapted, according to the material properties defined in prEN 1993-1-4 and EN 1993-1-2, to model the behaviour of stainless steel structures.

- In this work the accuracy and safety of the currently prescribed **column** and **beam-column** formulae are evaluated.

Case study

- **Axially loaded column with:**
  - Welded cross-sections: equivalent HEA 200 and HEB 280 sections of the stainless steel grades 1.4301 and 1.4401
  - The temperatures chosen were 400, 500, 600 and 700 ºC

- **Beam-columns with combined axial compression and uni-axial major and minor uniform moment with:**
  - Welded equivalent to a HEA 200 cross-sections of the stainless steel grade 1.4301
  - The temperature chosen was 600 ºC

- **For both types of elements**
  - Buckling around the strong and around the weak axis
  - A lateral geometric imperfection was considered given by
  - An initial rotation around the beam axis with a maximum value of *l*/1000 radians at mid span was also considered
  - It were adopted residual stresses

EC3

New Proposal

$$N_{fi,b,Rd} = \chi_{fi} A f_{y,\theta} \frac{1}{\gamma_{M,fi}}$$

$$\chi_{fi} = \frac{1}{\phi_\theta + \sqrt{\phi_\theta^2 - \beta \bar{\lambda}_\theta^2}} \leq 1$$

$$\alpha = 0.65\sqrt{235 / f_y}$$

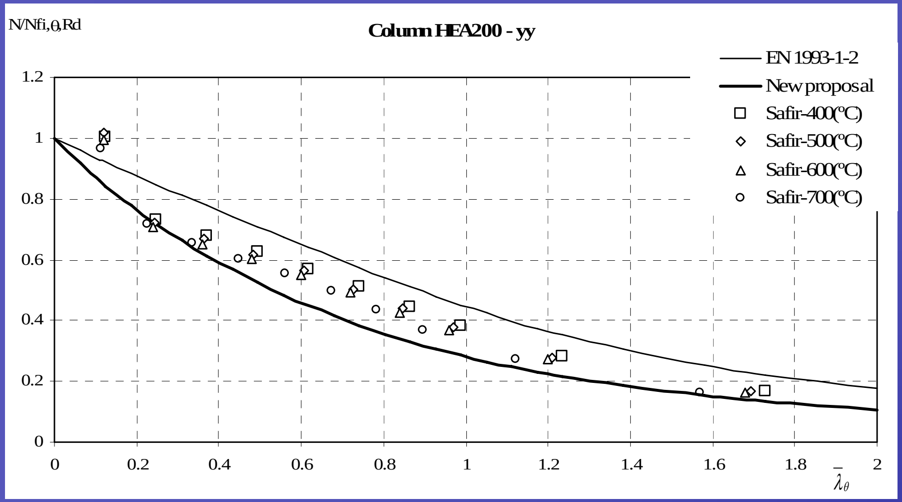
$$\beta = 1.5$$

$$\phi_\theta = \frac{1}{2}\left[1 + \alpha \bar{\lambda}_\theta + \beta \bar{\lambda}_\theta^2\right]$$

$$\alpha = 1.4\sqrt{235 / f_y}$$

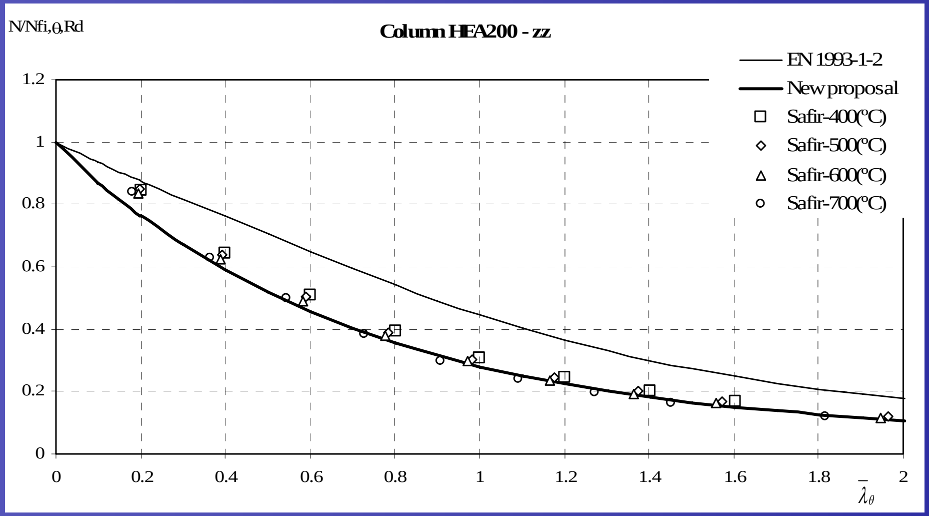
Member stability

Column buckling at high temperatures



HE 200A
grade 1.4301

The new proposal is more safe



➢**EC 3 proposal** *"EN1993-1-2"*

$$\frac{N_{fi,Ed}}{N_{b,i,fi,Rd}} + k_i \frac{M_{i,fi,Ed}}{W_{pl,i} \frac{f_{y,\theta}}{\gamma_{M,fi}}} \leq 1 \qquad k_i = 1 - \frac{\mu_i N_{fi,Ed}}{\chi_{i,fi} A k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} \leq 3$$

$$\mu_y = (1.2\beta_{M,y} - 3)\bar{\lambda}_{y,\theta} + 0.44\beta_{M,y} - 0.29 \leq 0.8$$

$$\mu_z = (2\beta_{M,z} - 5)\bar{\lambda}_{z,\theta} + 0.44\beta_{M,z} - 0.29 \leq 0.8$$

$$\text{with} \quad \bar{\lambda}_{z,\theta} \leq 1.1$$

➢With the new proposal for columns *"EN1993-1-2NP"*

Beam columns

➢**EC 3 proposal for stainless steel interaction curves at room temperature adapted for fire situation** *"prEN1993-1-4fi"*

$$\frac{N_{Ed}}{N_{b,i,Rd}} + k_i \frac{M_{i,Ed}}{W_{pl,i}\dfrac{f_y}{\gamma_{M1}}} \leq 1$$

$$k_i = 1.0 + 2\left(\bar{\lambda}_i - 0.5\right)\frac{N_{Ed}}{N_{b,Rd,i}}$$

$$\text{but} \quad \cancel{1.2} \leq k_i \leq 1.2 + 2\frac{N_{Ed}}{N_{b,Rd,i}}$$

➢With the new proposal for columns *"prEN1993-1-4fiNP"*

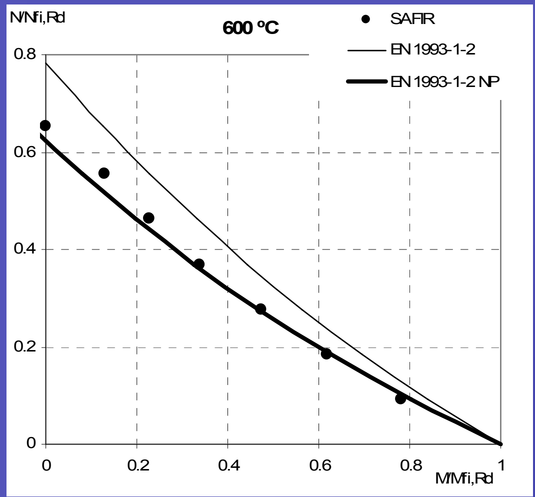
➢Without the minimum limiting value of 1.2 for *ki "prEN1993-1-4fiNP+NK"*

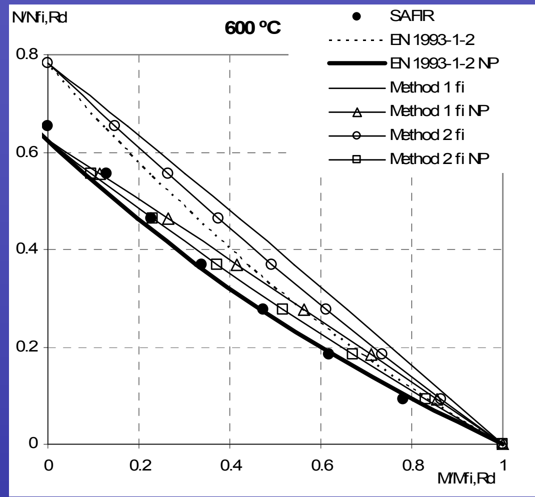
➢**EC 3 proposal for carbon steel interaction curves at room temperature adapted for fire situation and for stainless steel**

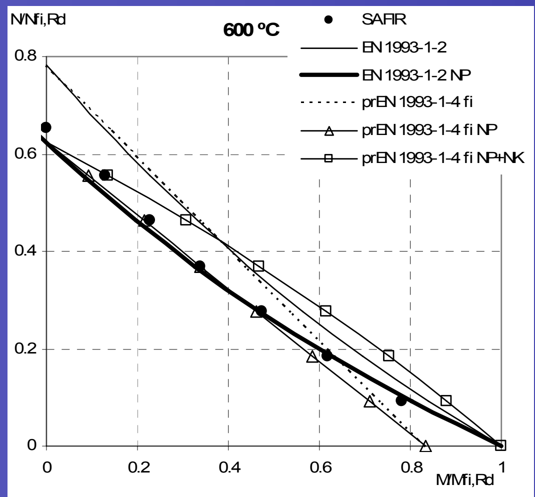
➢Two alternative proposals were adopted for the carbon steel interaction formulae at room temperature. Here a same approach was adopted using the expressions for stainless steel columns with the interaction formulae from Part 1.1 of EC3 *“Method 1fi”* and *“Method 2fi”*

➢ With the new proposal for columns *“Method1fiNP”* and *“Method2fiNP”*

Parametric Study



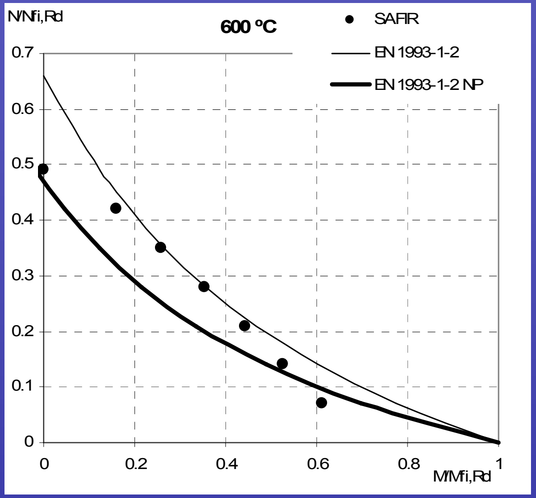


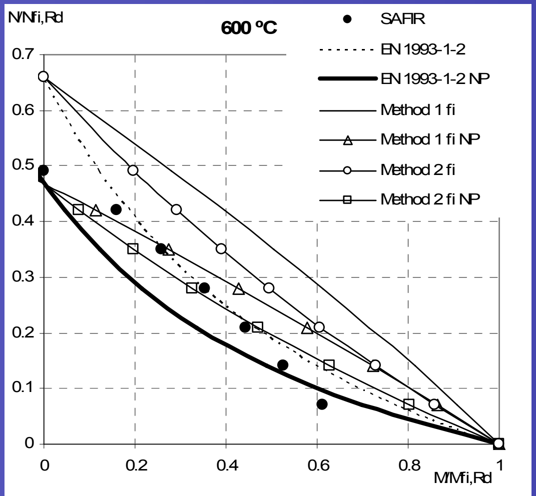


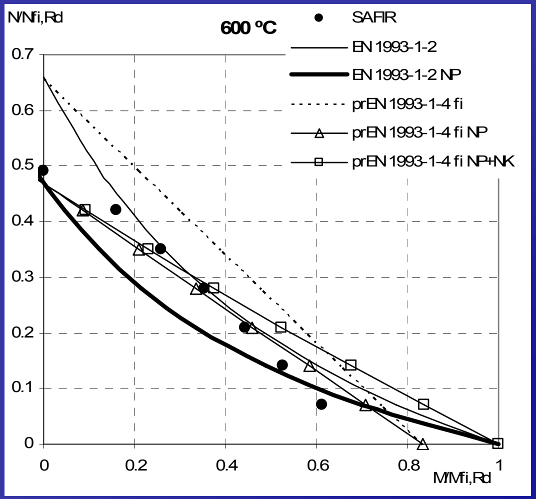
N+My
buckling around
the y-y-axis
L=3000mm;
$\bar{\lambda}_{y,\theta}$ =0.360

Parametric Study

N+Mz
buckling around the z-z-axis
L=3000mm;
$\bar{\lambda}_{z,\theta}$=0.584







Conclusions

➢It is shown that the **new proposal** for the design buckling resistance of stainless steel **compression members** at high temperatures is in **good agreement** with the numerical results obtained with the program SAFIR, in opposition to the results obtained with the formulae of the Part 1-2 of EC 3, which are not on the safe side.

➢For beam-columns with bending in the strong axis and buckling around the yy-axis, the curves obtained with the **new proposal for columns** shows a **better approximation** to the numerical results. The method that approximates more closely the real behaviour of stainless steel beam-columns under fire conditions is "EN 1993-1-2 NP". However, for the case of bending around the weak axis there is not a curve that provides a good approximation to the numerical results, which means that new interaction factors should be developed. Nevertheless **"EN 1993-1-2 NP"** still remains the **best method**.

➢The results presented in the paper show that **EC 3 formulae** for the evaluation of the fire resistance of columns and beam-columns **need to be improved**.

Non-linear modelling of reinforced concrete beams subjected to fire

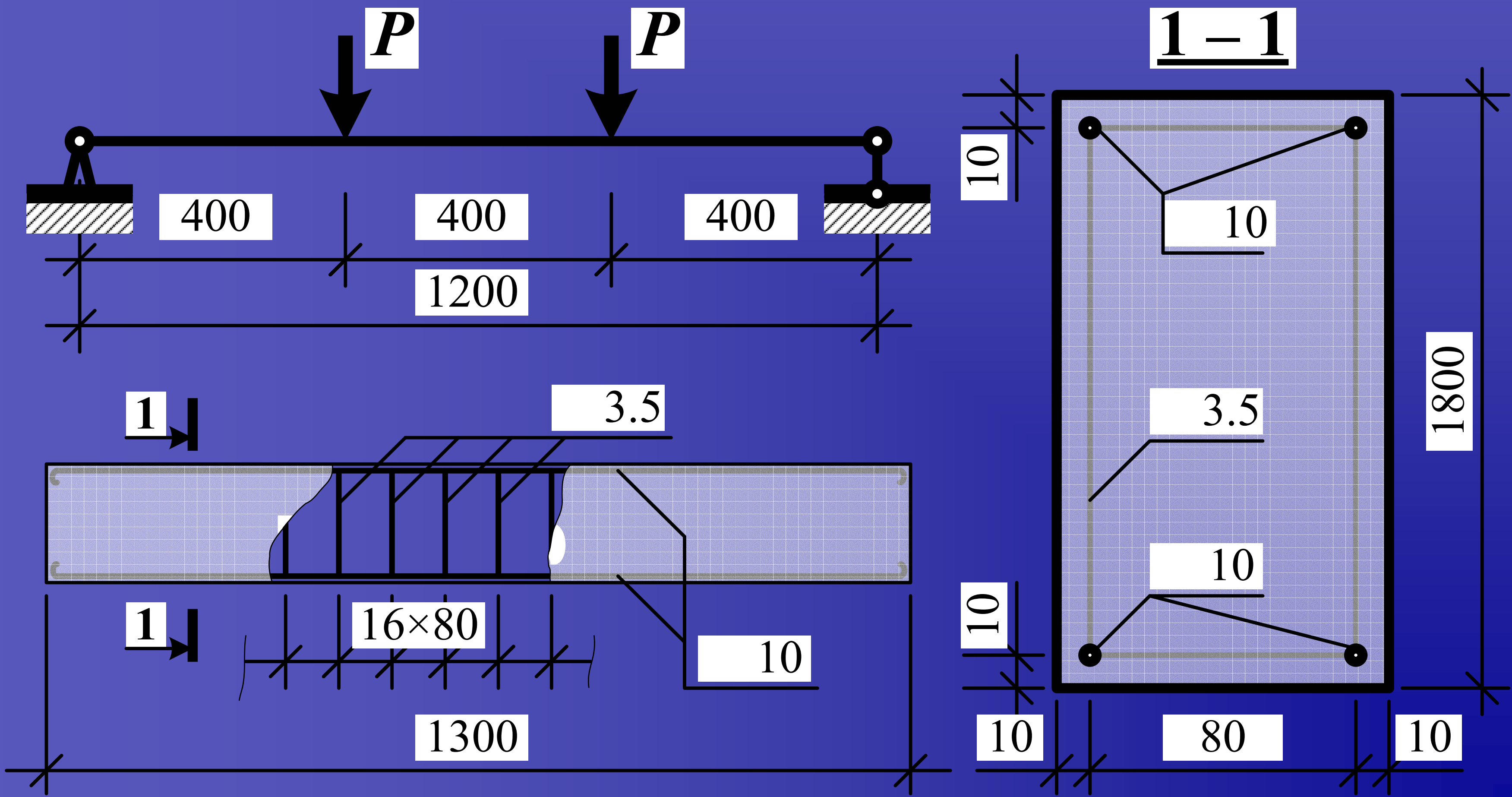
Viktor Gribniak
Vilnius Gediminas Technical University, Lithuania

Darius Bačinskas
Vilnius Gediminas Technical University, Lithuania

Gintaris Kaklauskas
Vilnius Gediminas Technical University, Lithuania

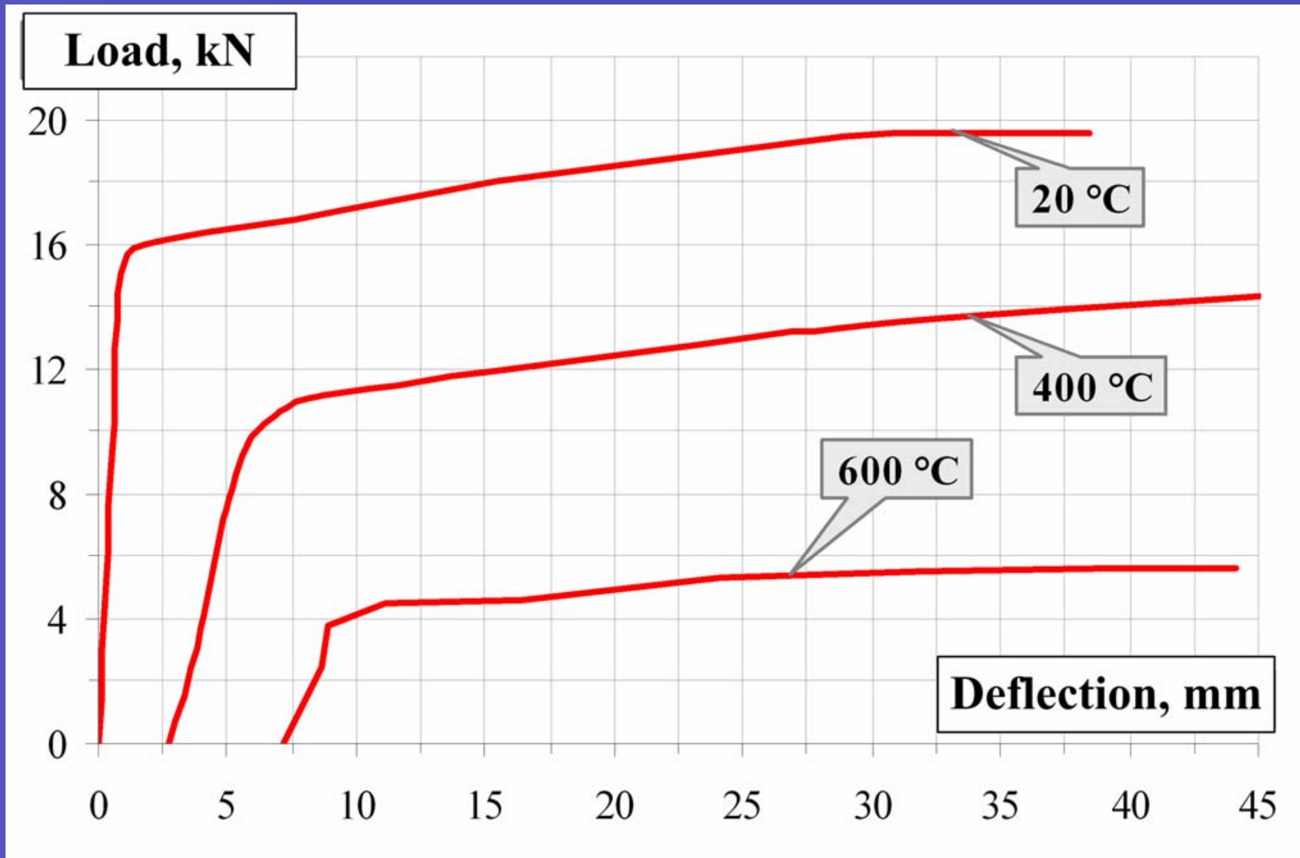
Experimental specimens

(Shi, X., Tan, T.-H., Tan, K.-H., Guo, Z.)

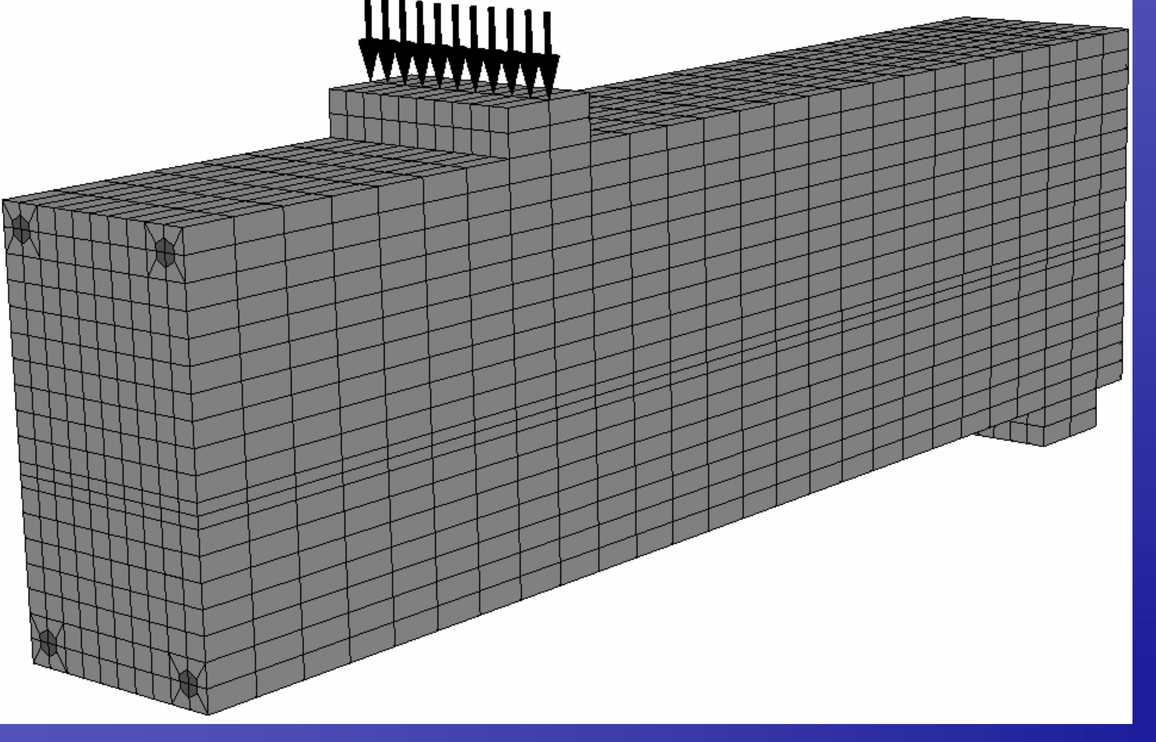


Test load-deflection Diagrams

(Shi, X., Tan, T.-H., Tan, K.-H., Guo, Z.)



Finite element model

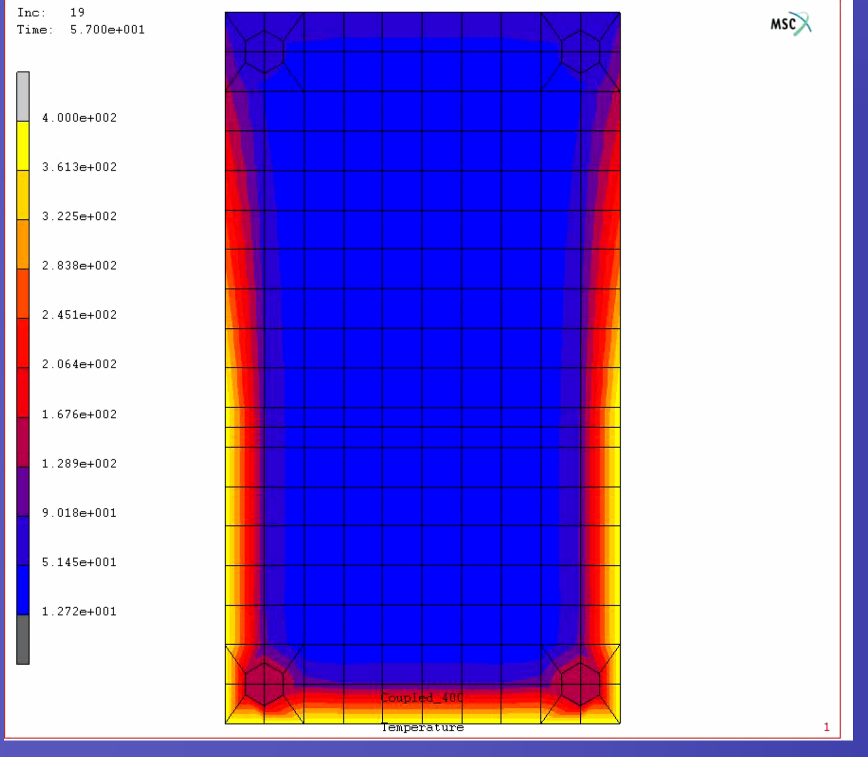


MSC.MARC – MSC.Software Co.

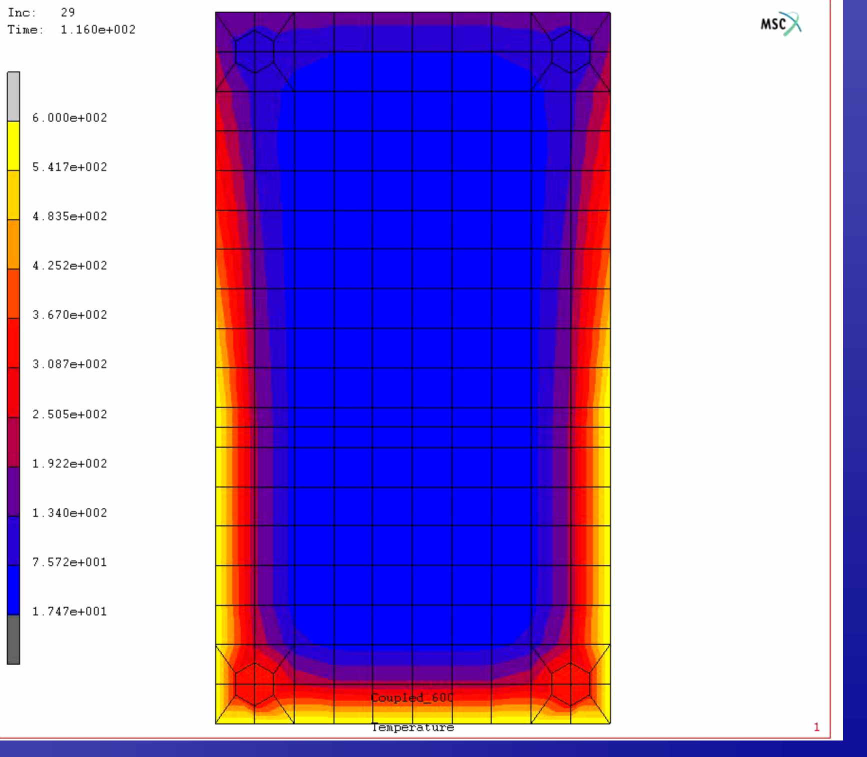
Results of numerical analysis

TEMPERATURE DISTRIBUTION IN THE SECTION

400 °C

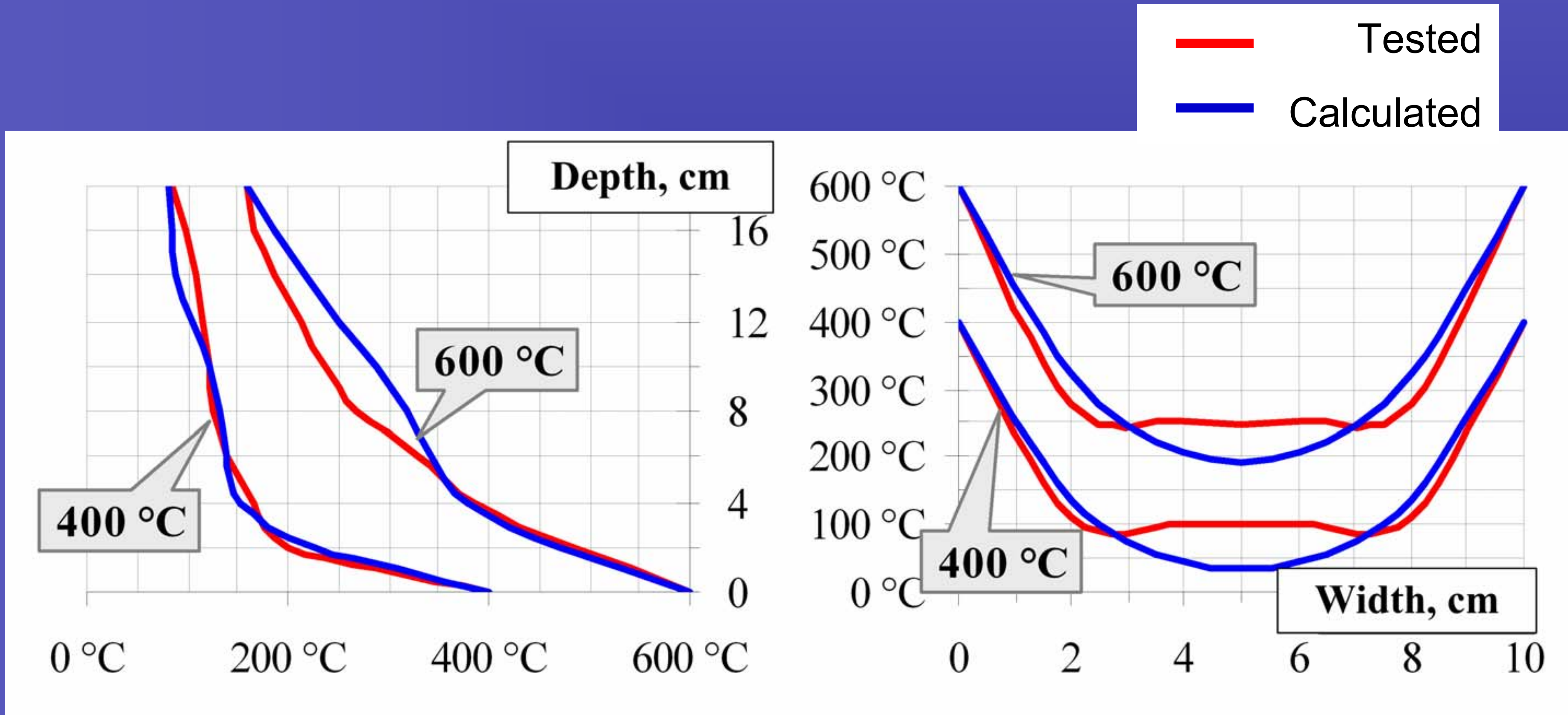


600 °C



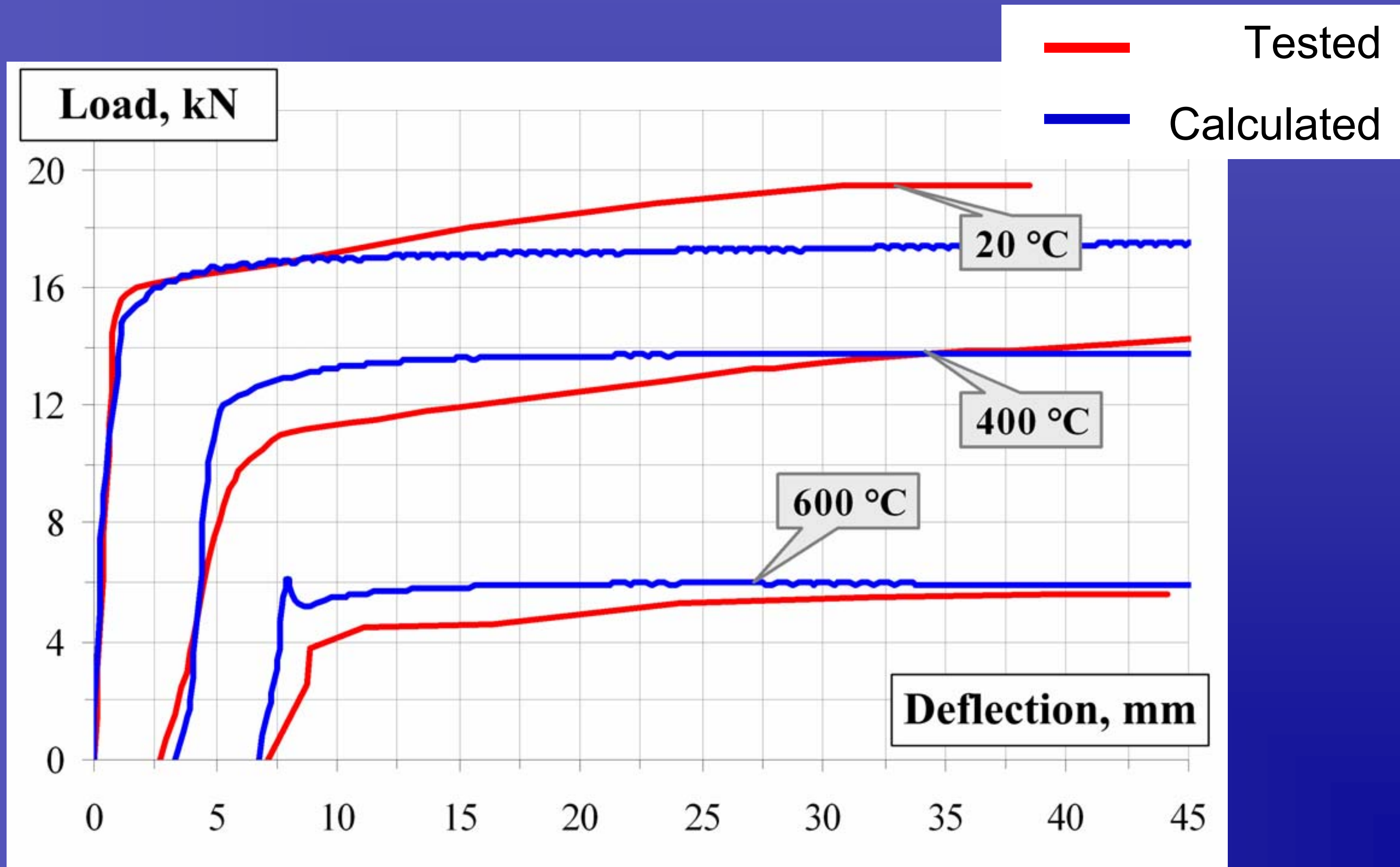
Results of numerical analysis

COMPARISON OF CALCULATED AND TEST TEMPERATURES



Results of numerical analysis

COMPARISON OF CALCULATED AND TEST DEFLECTIONS



Concluding remarks

- Comparison of the experimental and modelling results has shown that MSC.MARC has **satisfactorily** captured the **load-deflection behaviour** of the beams

Objectives of future research

- Developing a simplified *layer* (grid) *model* for non-linear thermo-mechanical analysis of reinforced concrete members

- Verification of the *layer model* using commercial FE software (*DIANA*, *ATENA*, *MSC.MARC*)

Some Remarks on the Simplified Design Methods for Steel and Concrete Composite Beams

E. Nigro
Department of Structural Engineering, University of Naples "Federico II", Italy

G. Cefarelli
Department of Structural Engineering, University of Naples "Federico II", Italy

Abstract

The present paper recalls the main characteristics of a general numerical approach to assess the ultimate bearing capacity of steel and concrete composite beams in fire conditions. The behaviour of the composite beams during a **standard fire exposure** is investigated.

- It is shown the **comparison** of resistance between **steel beam, composite beam and composite beam with partial concrete encasement**.

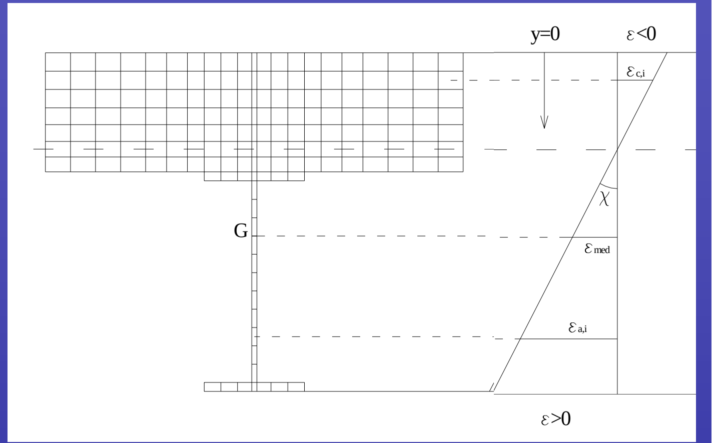
- The following features affecting the resistance of the composite beam with partial concrete encasement are firstly investigated: influence of the **beam dimensions** and **effectiveness of the reinforcing bars** in concrete encasement.

- Moreover, it is shown a **comparison** between the general **numerical approach** and the simplified method proposed in **EN 1994-1-2** for evaluating the sagging moment resistance of the composite beam with partial concrete encasement.

- Finally, it is **proposed a simplified plastic method** for evaluating the sagging moment resistance of the composite beam with partial concrete encasement in fire conditions.

Description of an accurate procedure to assess bearing capacity of composite beams

The procedure for evaluating the moment-curvature diagram, for each time of fire exposure, is based on the following steps:

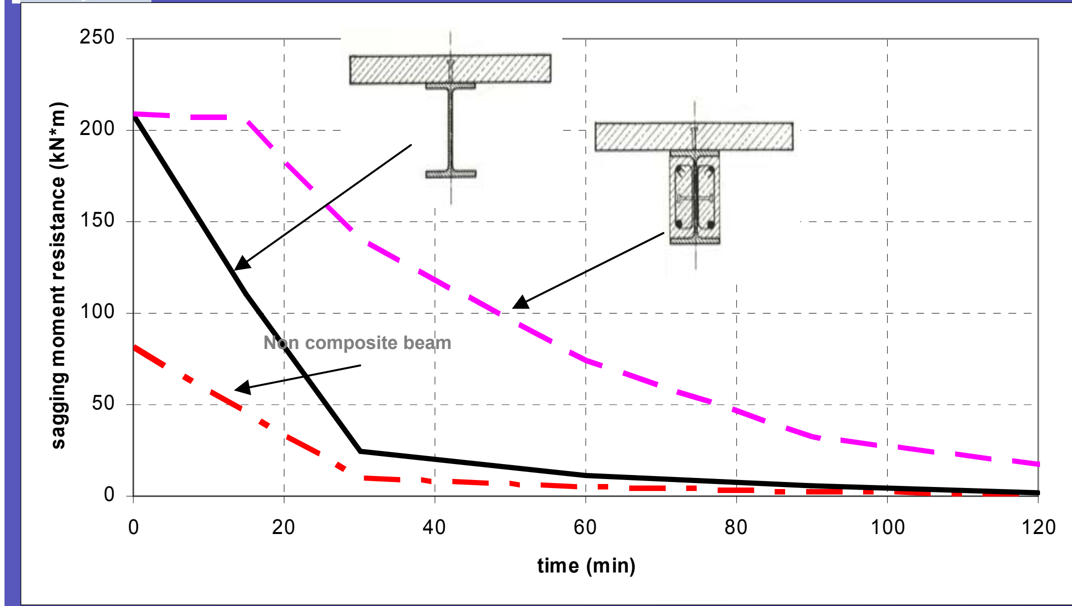


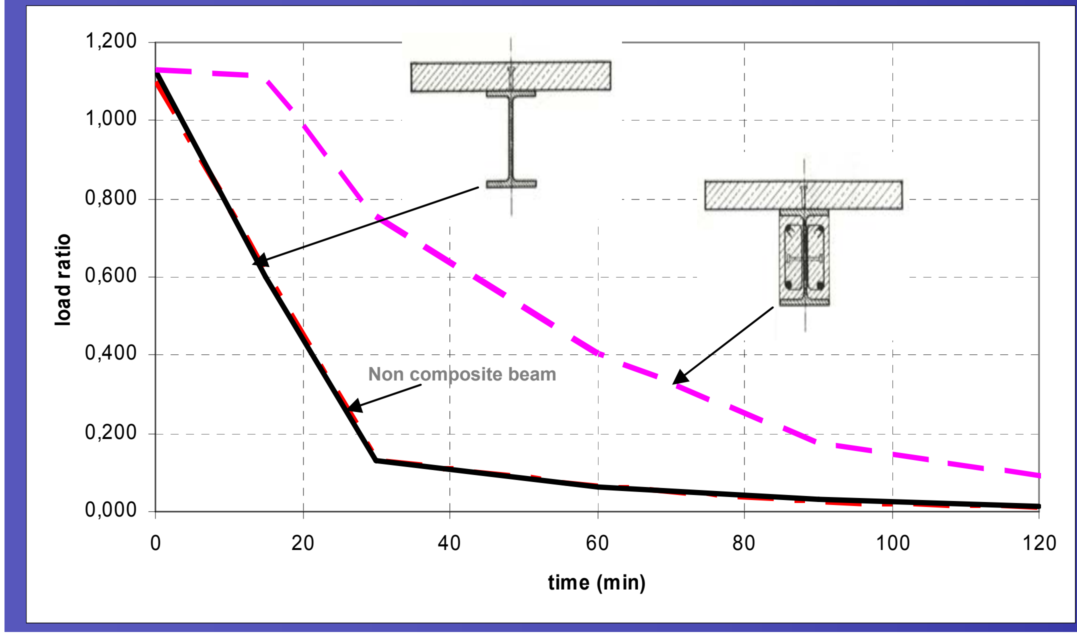
- The finite elements technique must be used;
- The external axial force N_{ext} (N_{ext} = 0 in pure bending) and the distribution of the temperatures Ti(t) within the section, related to the assigned exposure time t, are known and fixed for each moment-curvature diagram (M-c ; Next ; t);
- For an assigned curvature χ_j , a tentative value for the average strain ε_{med} of the cross-section is initially assumed and the corresponding distributions of strain ei and stress $\sigma_i = \sigma(\varepsilon_i)$ within the section are determined on the basis of the temperature-dependent stress-strain laws;
- The internal axial force N_{int} is then evaluated starting from the stress distribution;

$$\sigma_i = f(\varepsilon_i) \longrightarrow N_{\text{int}} = \sum_i A_i \cdot \sigma_i$$

- Iterations varying the average strain ε_{med} of the section need up to satisfying the longitudinal equilibrium equation within a suitable error: $(N_{\text{int}} - N_{est}) \leq \delta$
- Then the bending moment M_j corresponding to the assigned curvature χ_j may be determined.

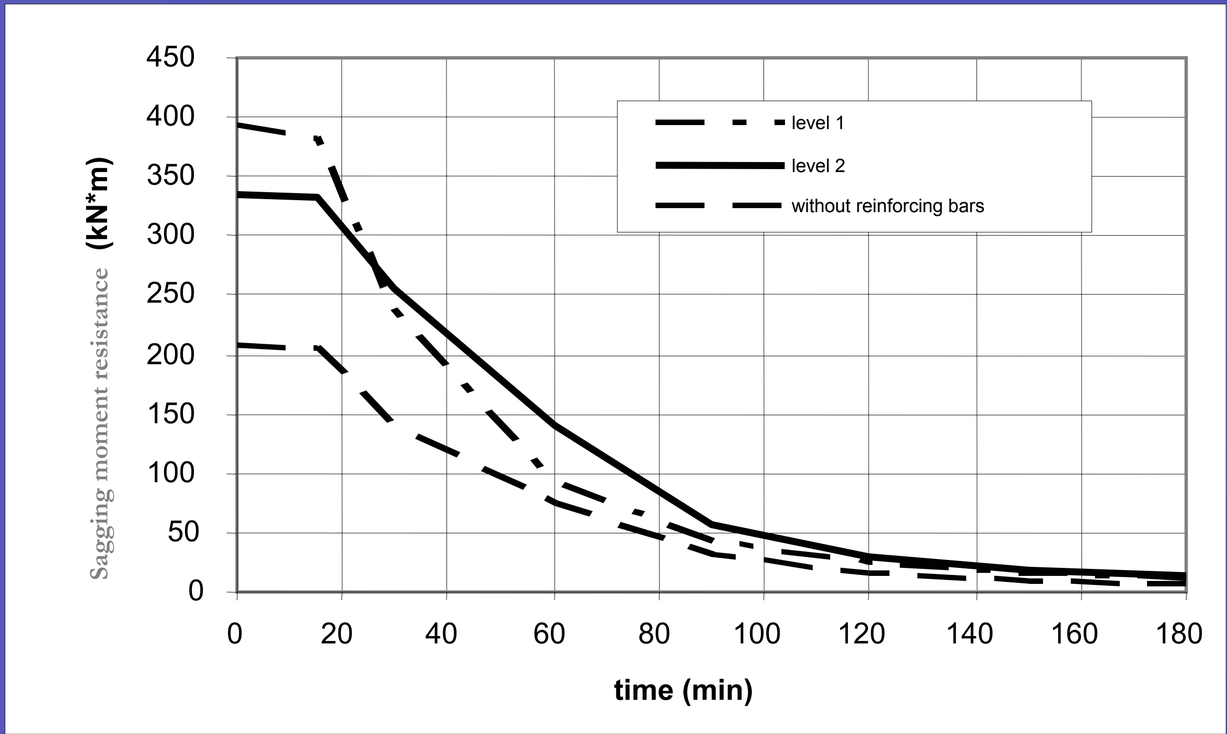
Comparison between various types of beams



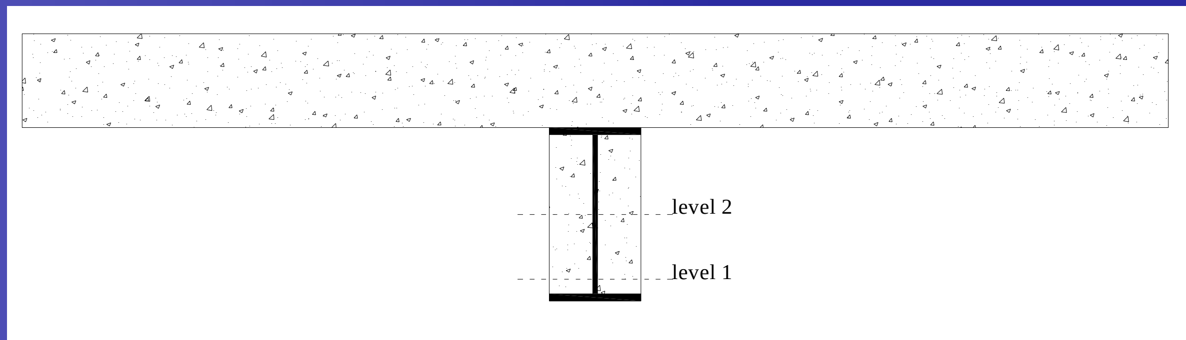


The comparison in resistance field shows the better behaviour of composite members; however, in load ratio field, the composite beam without concrete encasement shows a similar behaviour to non-composite beam. This is due to, both in the composite beam without concrete encasement and non-composite beam, the moment capacity depends on **loss of strength of metallic part** exposed to fire. In the case of composite beam with partial concrete encasement, it is shown a quite better behaviour, thanks to lower temperature values in the steel beam.

Effectiveness of the reinforcing bars



It is clear how the reinforcement to level 1 provides a better performance in ambient condition, but it provides a worth performance in fire condition, compared to the case of reinforcement placed to level 2.

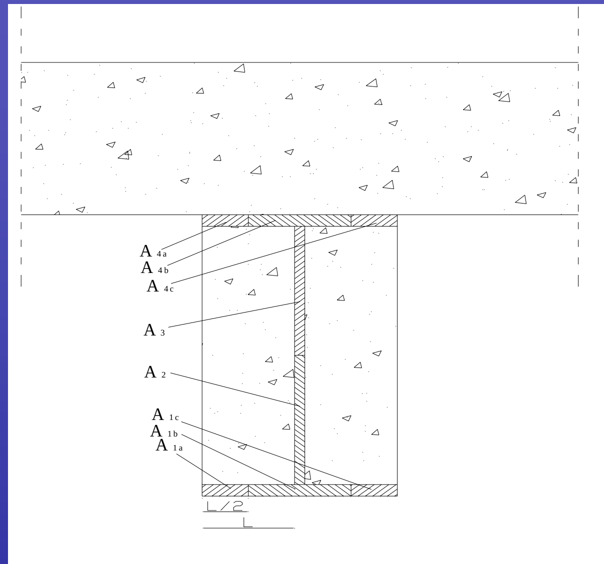


Proposed simplified plastic method

The cross-section of the beam is divided in **5 main parts**:

- concrete slab;
- upper flange of steel beam;
- upper half of the web steel beam;
- bottom half of the web steel beam;
- bottom flange of the steel beam.

Moreover, the flanges are further divided in 3 parts.



A **uniform temperature**, equal to average temperature of the same part, is assigned to each of this parts. The average temperature is evaluated from the thermal analysis.

Concrete with temperatures in excess of 500°C is assumed not to contribute to the load bearing capacity of the member, whilst the residual concrete cross-section retains its initial values of strength.

time min	Ultimate Resistant Moment - general procedure - KN*m	Ultimate Resistant Moment - proposed simplified moment - KN*m	M_{simpl}/M_{gen}
0	208,33	208,279	1,00
15	205,35	208,279	1,01
30	139,31	130,256	0,93
60	73,75	73,313	0,99
90	31,69	31,406	0,99
120	16,79	16,856	1,00
150	9,98	10,024	1,00
180	7,16	7,510	1,05

Thank you for your attention