Focussed on Modelling in Mechanics

Nonlinear analysis of reinforced and composite columns in fire

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ABSTRACT. The paper describes a customer project, the objective of which was to develop a software program for the calculation of reinforced concrete and steel/concrete composite columns subjected to various loadings and exposed to normal temperature or to fire conditions. Such analysis required the use of two Finite Element calculation cores (i) for thermal analysis and (ii) for geometrical and material non-linearity. Calculation cores were connected to graphical user interface via open interface – IDEA Open Model.

KEYWORDS. Column; Concrete; Composite; Analysis; Non-linear; Fire.

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INTRODUCTION

No single software tool exists so far for complete design of prefabricated concrete and prestressed concrete members. Specialized programs are currently used in combination with Excel design sheets. Due to the absence or unreliability of the links between the programs, the workflow is interrupted and manual data transfer is needed at the interfaces. This system is ineffective and highly vulnerable. That is why producers of prefabricated beams and columns tend to improve their pre-manufacturing processing and member design. Usually they prefer their own software tools customized for their specifics in the types of members, production, logistical and administrative processing. At the same time the pressure on cost-effectiveness of the structures is growing, and the tolerance for structural defects is zero. Therefore they need to improve the economy and the effectiveness of the design by using highly sophisticated analysis methods, which they cannot develop on their own.

This situation resulted in a collaboration of F.J. Aschwanden, Swiss prefab company with high technological competence and innovation, and IDEA RS, Czech software company, which develops software for structural analysis and design of civil engineering structures and their members. The objective of common development project was to develop a software program for the calculation of reinforced concrete, concrete/concrete and steel/concrete composite columns subjected to various loadings and exposed to normal temperature or to fire conditions. Graphical user interface was developed by F.J. Aschwanden together with HOST module, which controls the sequence of individual analyses, and which mediates data transfer between the input and calculation cores for thermal analysis (TA) and for geometrical and material non-linearity (NLA). Both calculation cores are part of IDEA StatiCa software and were developed by IDEA RS. IDEA Open Model (IOM) has been created as an open interface to guarantee an effective interaction between HOST and IDEA StatiCa.

THERMAL ANALYSIS

Nonlinear 2-dimensional steady state and transient analysis for heat transfer across column cross-section has been developed. Quadrilateral and triangular finite elements are used. Each finite element may have different nonlinear material properties dependent on the temperature. Initial temperature is defined in each node of mesh. Ambient temperature is given as the function of the time on boundary of cross-section. Convection and radiation coefficients are constant.
No humidity transfer is taken into account, and no spalling of the surface is considered. The results are the temperatures across cross-section (in nodes of mesh) in selected time steps.

GEOMETRICAL AND MATERIAL NON-LINEARITY

Prismatic beam element with general eccentricity is used - Euler formulation (without shear deformations included). The load can be applied in nodes or may be uniformly distributed. Cross section of general shape can have material nonlinear properties dependent on current temperature in each point. Initial strain is considered across the cross-section for any beam element (strain, curvature). Nonlinear stress across the element is solved in two Gauss integration points. Reinforcement bars are modeled as the points for stress analysis, therefore different mesh appears for TA and NLA.
Corotational beam formulation is used. Deformation of beam element is split into the local deformation of beam and global deformation beam as rigid body. Multi load step iteration solver uses Finite Element Analysis in combination with Newton iteration, which can determine load carrying capacity without the possibility of post-critical behavior. This is not detrimental to the main objective – the column resistance.

VERIFICATION

The methods used for both TA and NLA have been verified in a number of benchmarks. Selected benchmark examples are analyzed and documented in this report.

Reinforced concrete column

First example is reinforced concrete column according to [2], which was also chosen by the RILEM Technical Committee TC 114 as one of the benchmark problems for testing the computational models and computer programs for reinforced concrete structures. The column of rectangular cross-section 150 mm × 200 mm and length 2.25 m is loaded by eccentric compressive axial force, see Fig. 1. The eccentricity of axial force F is e = 15 mm, reinforcement 4 × ϕ 12 mm (A_s = 4 × 113 mm^2). Position of bars (measured from the centre) x_i = ± 55 mm, y_i = ± 80 mm. Material properties of concrete is described by the stress-strain relationship for non-linear analysis given by EN 1992-1-1 [4], provision 3.1.5 with following parameters: f_{cm} = 38.3 MPa, E_{cm} = 33.6 GPa, ε_{c1} = 2.3 ‰, ε_{cu1} = 3.5 ‰. Resulting stress-strain diagram for concrete used for the analysis is illustrated in Fig. 2. It is worth noticing that horizontal branches are added to code stress-strain diagrams in order to maintain numerical stability of the calculation in IDEA software.

For reinforcing steel, we assume stress-strain diagram with a horizontal top branch given by EN 1992-1-1 [4], provision 3.2.7, with following parameters: f_y = 465 MPa, E_s = 200 GPa, ε_{su} = 20 ‰. The resulting stress-strain diagram for reinforcing steel used for the analysis is illustrated in Fig. 3.

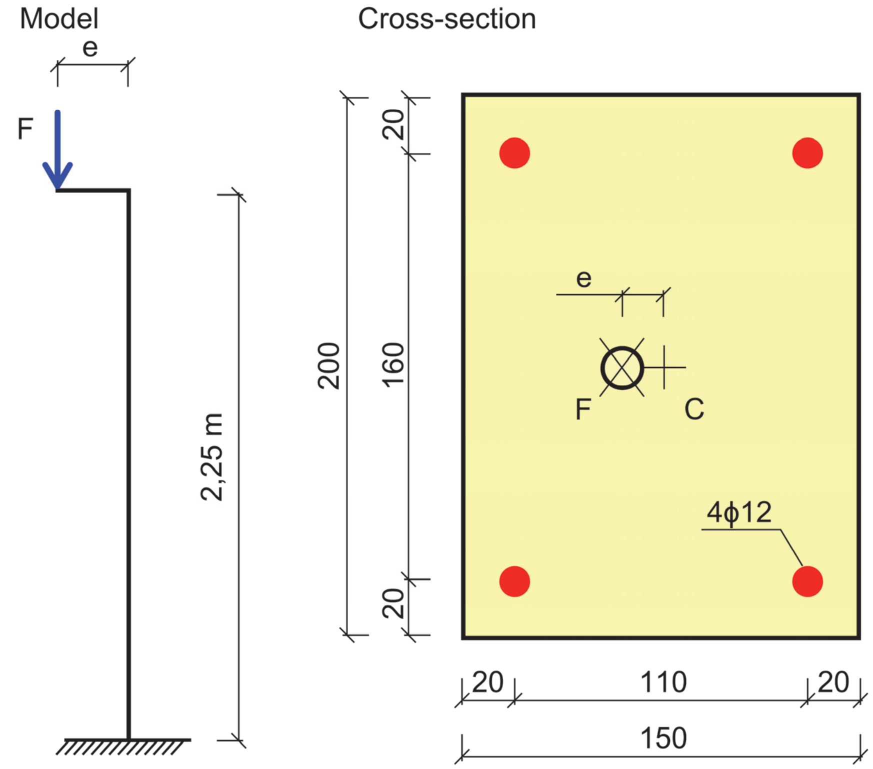

Figure 1: Geometry, cross-section, and the loading of RC column.

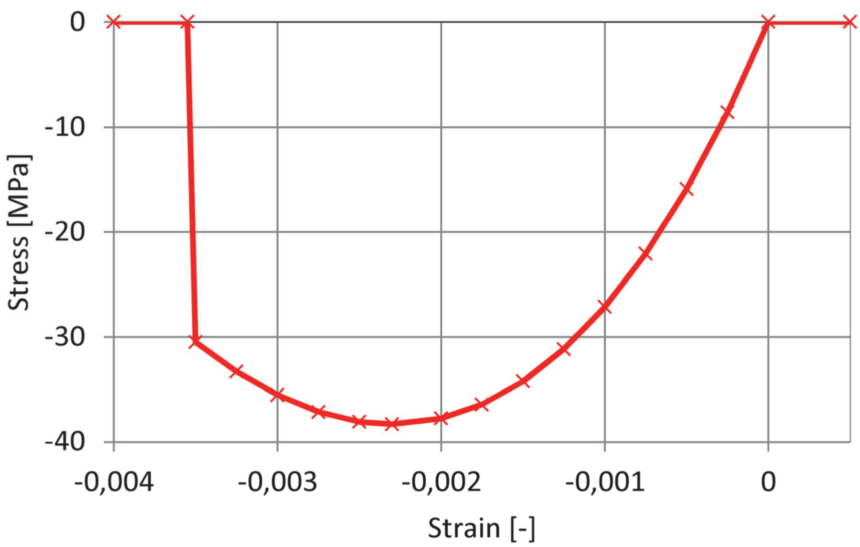

Figure 2: Stress-strain diagram of concrete used for the analysis.

Both the results of numerical analysis and experimental data are presented in [2] and they are compared with IDEA StatiCa results, see Fig. 4. Horizontal displacement at the top of the analysed column is investigated. Both material and geometrical nonlinearity is taken into account. Consistently with the assumptions no post-critical behavior was analyzed in case of NLA.

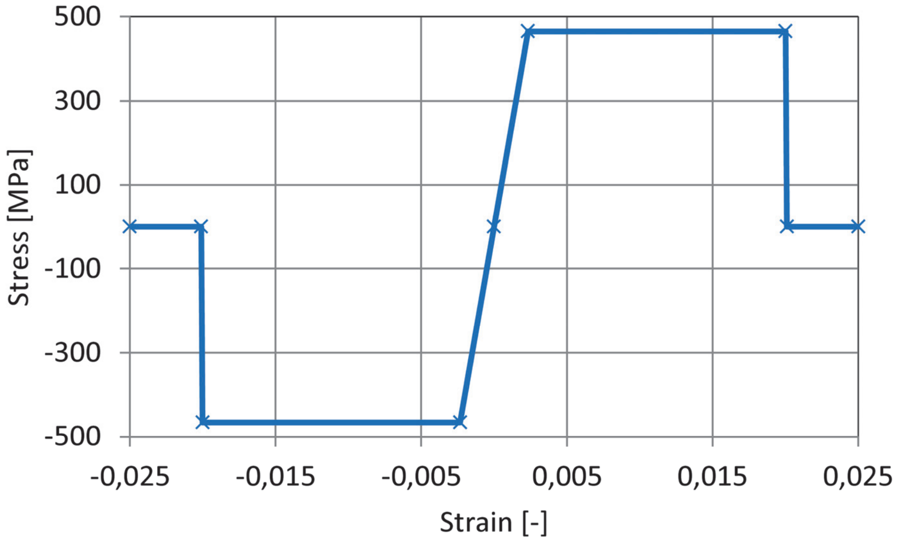

Figure 3: Stress-strain diagram of reinforcing steel used for the analysis.

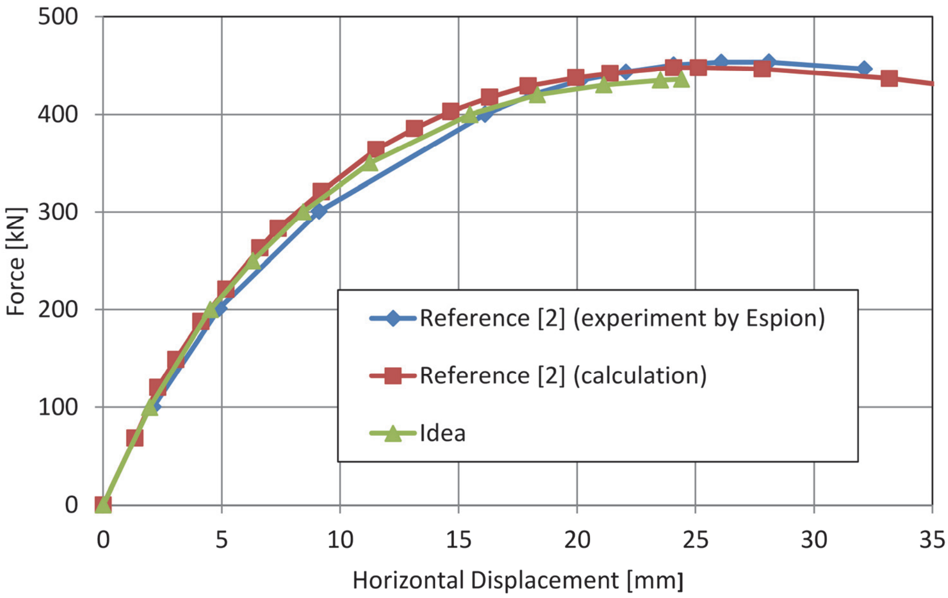

Figure 4: Normal force–horizontal displacement diagrams according to [2] vs. IDEA StatiCa.

	Ref. [2] experiment	Ref. [2] calculation	IDEA results	Difference experiment	Difference calculation
F_{max} [kN]	454	447	436	δF_{max} = – 3.96 %	δF_{max} = – 2.46 %
w_{Fmax} [mm]	26.1	25.1	24.4	δw_{Fmax}= – 6.51 %	δw_{Fmax}= – 2.79 %

Table 1: Comparison of results.

The values of ultimate resistance F_{max} and corresponding horizontal displacement w_{Fmax} obtained by the IDEA StatiCa solver are compared with the data stated in reference [2], see Tab. 1. Some improvements might be obtained by refining the model and increasing number of load steps and the requirements for the precision of iteration.

The results of non-linear analysis were tested against the moment-normal force interaction diagram, both calculated in IDEA StatiCa. Moment-normal force response of critical section of the column was calculated using both force and displacement control of non-linear analysis. Displacement control analysis is in excellent agreement with the ultimate limit state of critical cross-section, see Fig. 5.

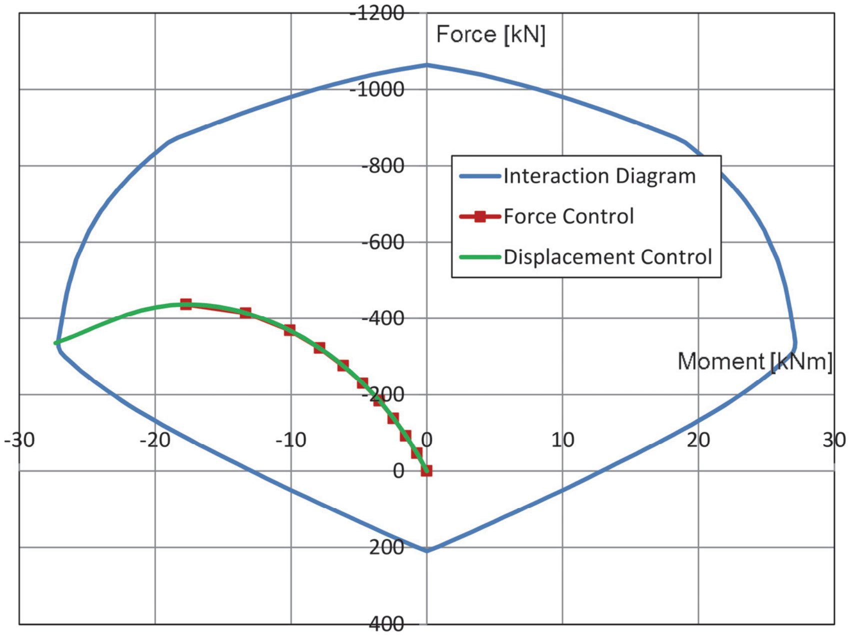

Figure 5: M-N interaction diagram and M-N response of the column in IDEA StatiCa.

Concrete-filled steel tubular column

Second example is concrete-filled steel tubular column according to [8], where the behavior of concrete-filled steel tubular columns of different lengths (2, 2.5, 3, 3.5 and 4 m) and under different loading conditions (axial compression, single curvature bending, double curvature bending) was examined experimentally.

The column is loaded by eccentric axial force F, with the eccentricities e_t and e_b at the top and the bottom of the column, respectively, see Fig. 6. Here, we assume the specimen denoted as *column 18* in the original paper [8]. For this column, following geometrical parameters are given: D = 159.8 mm, t = 5.02 mm, L = 2000 mm, $e_t = e_b = +$ 24 mm.

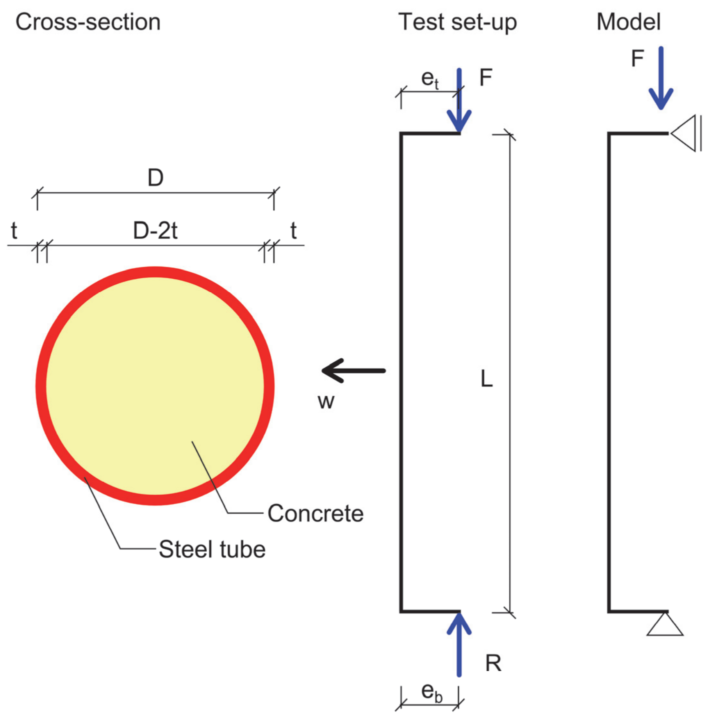

Figure 6: Geometry, cross-section, and the loading of concrete-filled steel tubular column.

For the steel tube, the yield strength of steel is f_y = 280 MPa, the elastic modulus can be set as E_s = 210 GPa, and corresponding stress-strain diagram (linear elastic perfectly plastic model) used for the analysis appears in Fig. 7 (assuming ε_{su} = 20 ‰).

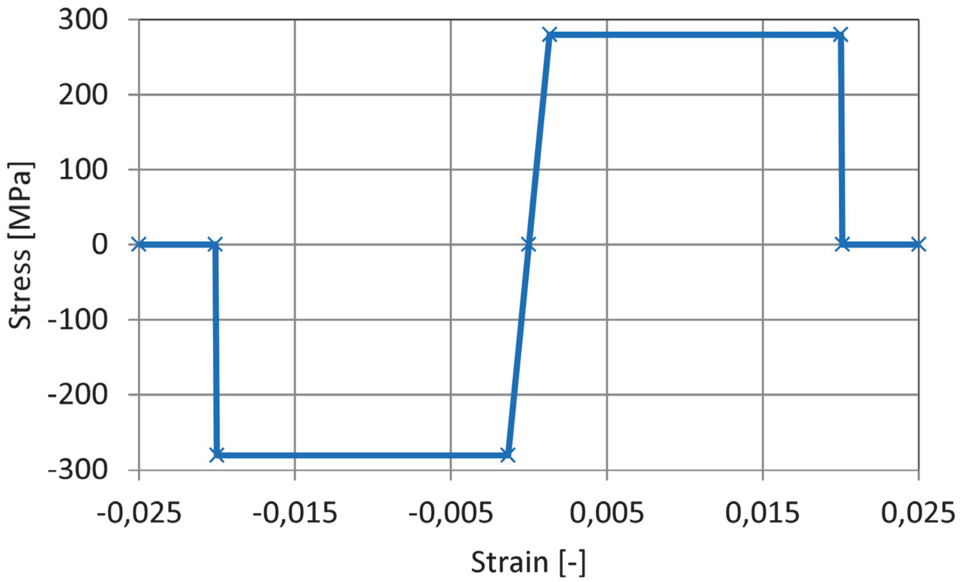

Figure 7: Stress-strain diagram of steel tube.

For concrete, the compressive strength and the elastic modulus are specified as f_c = 101 MPa and E_c = 45 GPa. We assume the stress-strain relationship for non-linear analysis given by EN 1992-1-1 [4], provision 3.1.5, with: f_{cm} = 101 MPa, E_{cm} = 45 GPa, ε_{c1} = 2.8 ‰, ε_{cu1} = 2.8 ‰. The resulting stress-strain diagram for concrete used for the analysis is illustrated in Fig. 8.

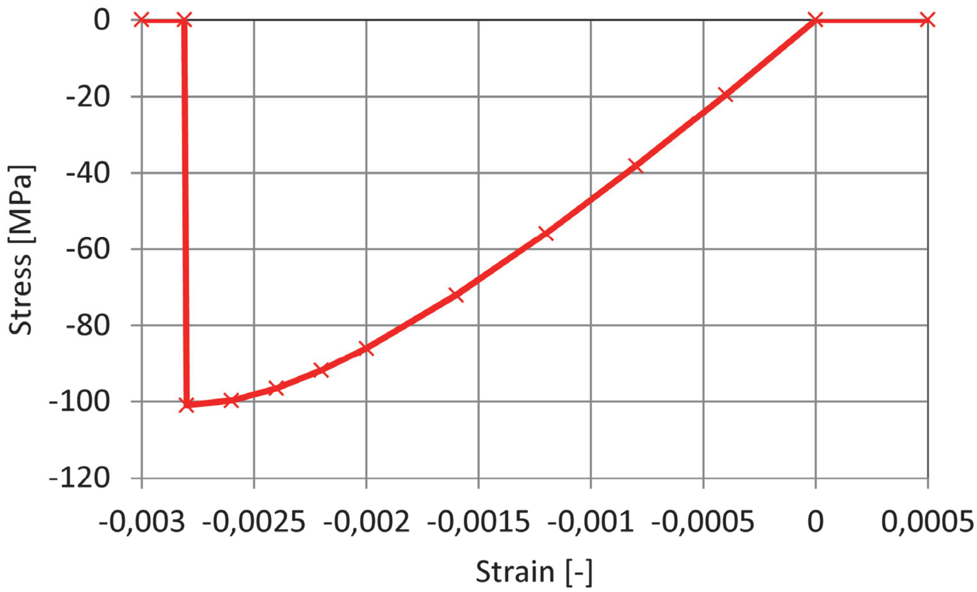

Figure 8: Stress-strain diagram of concrete.

Horizontal displacement at the mid-length of the analysed column is compared with IDEA StatiCa results, see Fig. 9. Both material and geometrical nonlinearity is taken into account with no analysis of post-critical behavior.

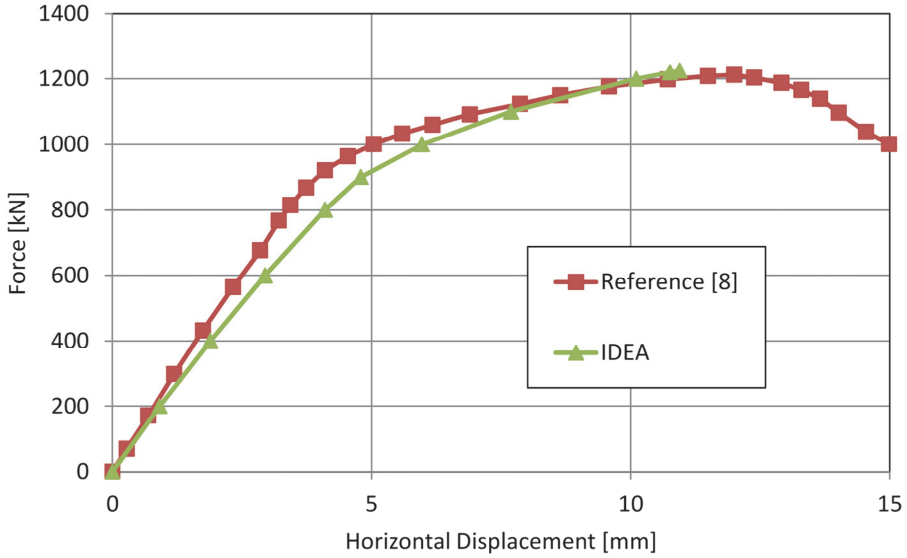

Figure 9: Normal force–horizontal displacement diagrams according to [8] vs. IDEA StatiCa.

The calculation was performed for several values of the normal force F (200 kN, 400 kN, 600 kN, 800 kN, 900 kN, 1000 kN, etc.) with the eccentricities at the top and the bottom of the column, respectively, $e_t = e_b = 0.024$ m until the collapse was reached (normal force of 1225 kN in this case). It is obvious that the results obtained by the IDEA StatiCa solver and data stated in reference [8] are closely similar.

The values of ultimate resistance F_{max} and corresponding horizontal displacement w_{Fmax} at mid-length of the column obtained by the IDEA StatiCa solver are compared with the data stated in reference [8], see Tab. 2.

	Ref. [8]	IDEA	Difference
F_{max} [kN]	1212	1225	$\delta F_{max} = -1.07$ %
w_{Fmax} [mm]	12.0	10.9	$\delta w_{Fmax} = -9.17$ %

Table 2: Comparison of results.

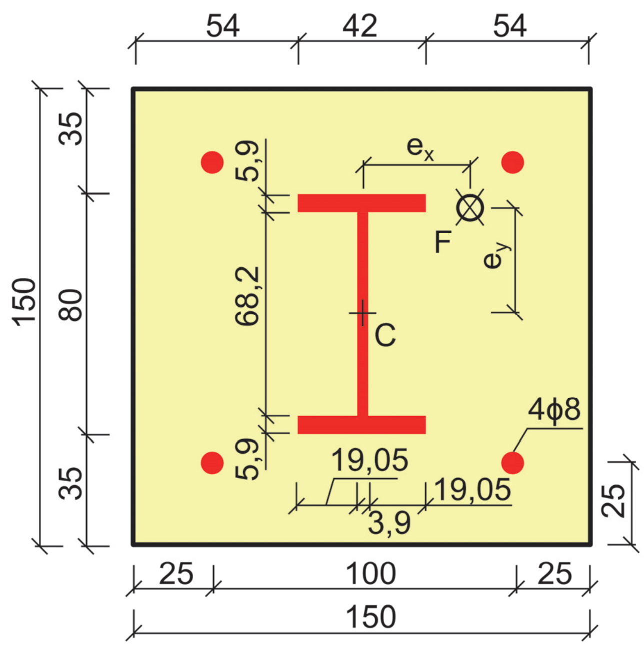

Figure 10: Cross-section of concrete-encased composite column.

Concrete-encased composite column 1

Third example is concrete-encased composite column according to [7], where the behavior of concrete-encased composite columns under biaxial loading is examined experimentally and also by numerical analysis.

The column is loaded by biaxial eccentric force F, with the eccentricities e_x and e_y, which are the same at the top and the bottom of the column. Here, we assume the specimen denoted as *column CC6* in the original paper [7]. For this column, the following geometrical parameters are given: column length L = 1300 mm, eccentricities $e_x = e_y = +$ 55 mm, the other parameters are given in Fig. 10.

For the steel I-section, the yield strength of steel is f_y = 235 MPa, the elastic modulus can be set as E_s = 200 x 10^3 MPa, and corresponding stress-strain diagram (linear elastic perfectly plastic model) used for the analysis appears in Fig. 11 (assuming ε_{su} = 20 ‰).

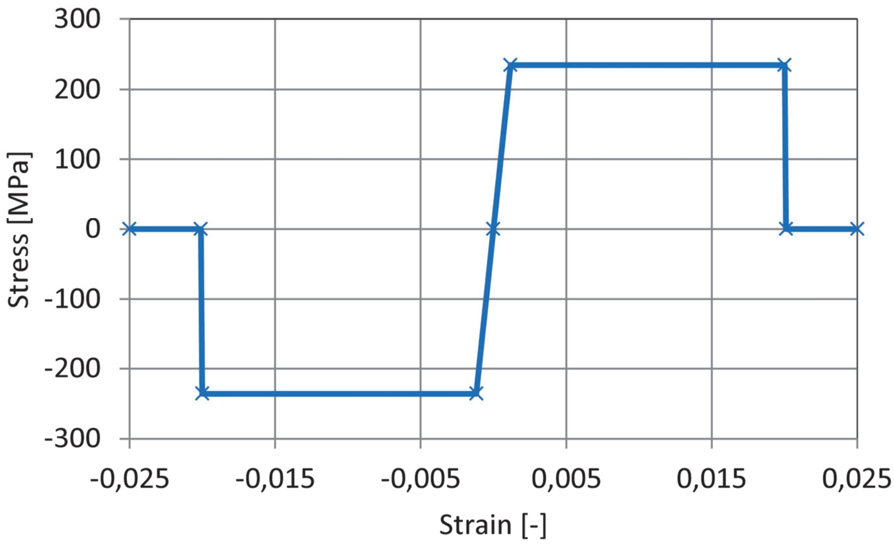

Figure 11: Stress-strain diagram of steel I-section.

For steel reinforcement, the yield strength is f_y = 500 MPa, the elastic modulus can be set as E_s = 200 x 10^3 MPa, and corresponding stress-strain diagram (linear elastic perfectly plastic model) used for the analysis appears in Fig. 12 (assuming ε_{su} = 20 ‰).

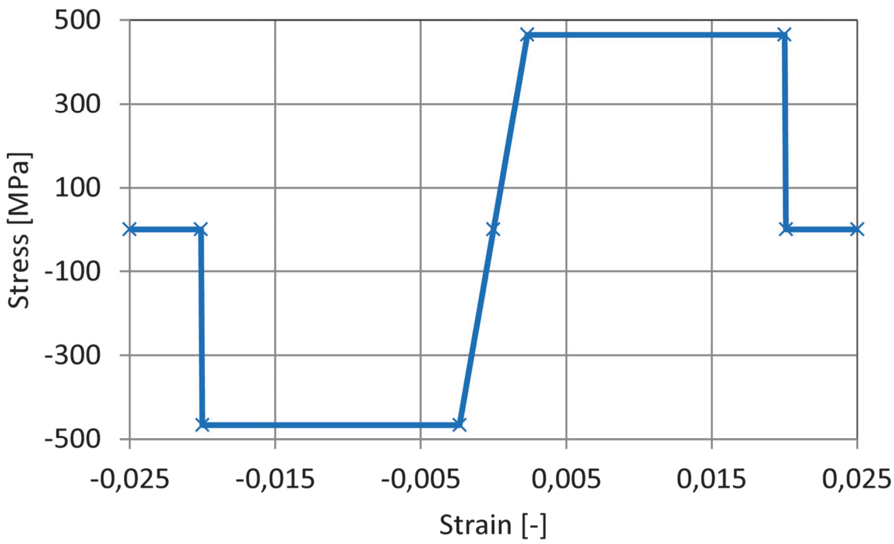

Figure 12: Stress-strain diagram of reinforcing steel.

For concrete, the compressive strength is specified as f_c = 33.99 MPa. We assume the stress-strain relation for non-linear analysis given by EN 1992-1-1 [4], provision 3.1.5, with: f_{cm} = 33.99 MPa, E_{cm} = 31 GPa, ε_{c1} = 2.1 ‰, ε_{cu1} = 3.5 ‰. The resulting stress-strain diagram for concrete used for the analysis is illustrated in Fig. 13.
In order to evaluate the results, we can compare the ultimate normal force determined by the IDEA StatiCa with the value stated in [7], see Tab. 3. It is obvious that the result obtained by the IDEA StatiCa solver and the value of the ultimate normal force stated in the reference are closely similar.

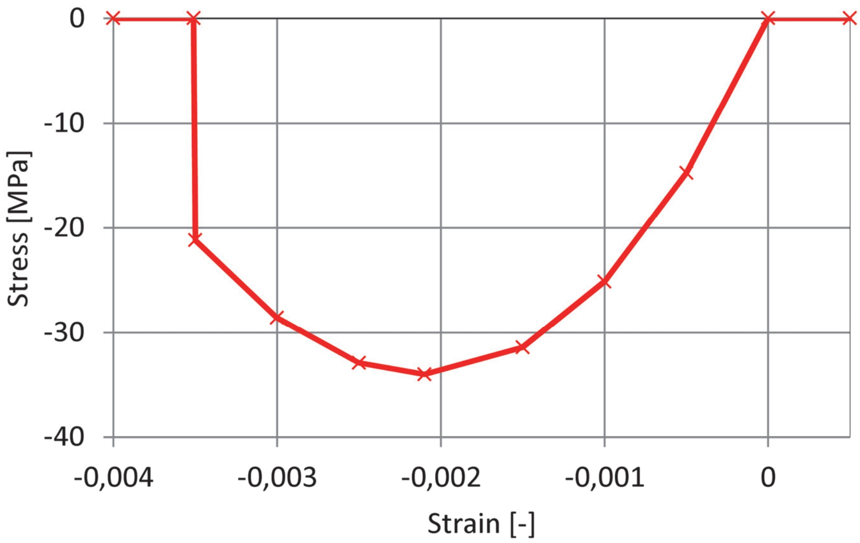

Figure 13: Stress-strain diagram of concrete.

	Ref. [7]	IDEA	Difference
F_{max} [kN]	208	196	δF_{max} = – 5.77 %

Table 3: Comparison of results.

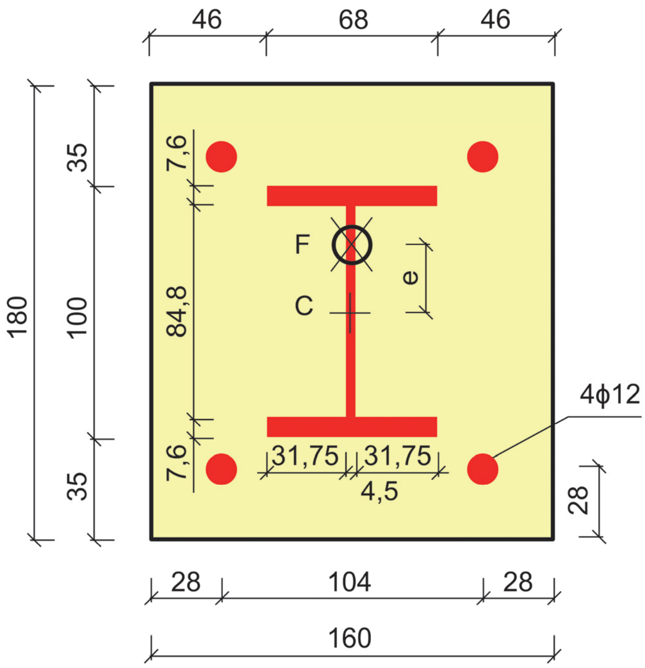

Figure 14: Cross-section and the load of concrete-encased composite column.

Concrete-encased composite column 2

Forth example is concrete-encased composite column according to [9], where the behavior of concrete-encased composite columns under eccentric loading is examined experimentally and also by numerical analysis.

The column is loaded by eccentric force F, with the eccentricity e (along the major axis), which is the same at the top and the bottom of the column. Here, we assume the specimens denoted as *SRC-E2* and *SRC-E8* in the original paper [9]. For these columns, the following geometrical parameters are given: column length L = 3200 mm, e = 30 mm (*SRC-E2*); e = 150 mm (*SRC-E8*), the other parameters are stated in Fig. 14.

For the steel I-section (height = 100 mm, width = 68 mm, web thickness 4.5 mm, flange thickness 7.6 mm), the yield strength and the elastic modulus of steel, respectively, are f_y = 379 MPa and E_s = 205.8 GPa. Corresponding stress-strain diagram (linear elastic perfectly plastic model) used for the analysis appears in Fig. 15 (assuming ε_{su} = 20 ‰).

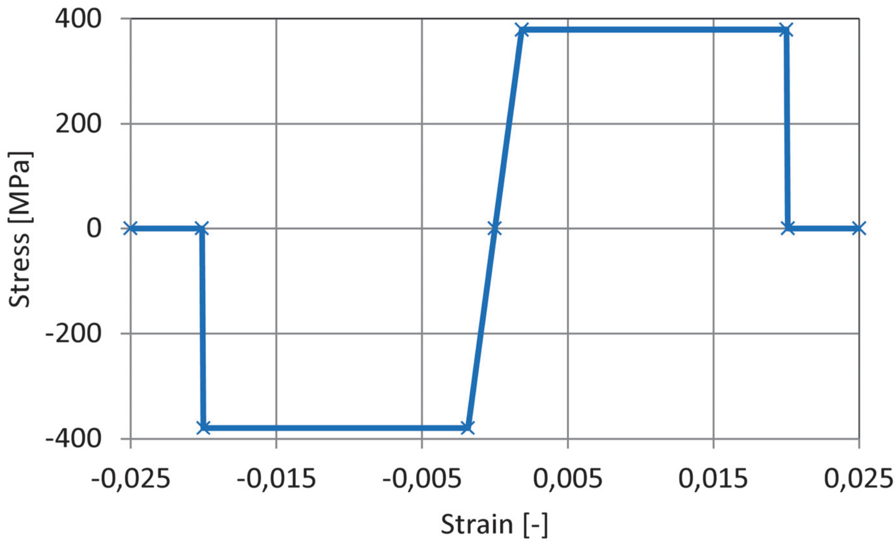

Figure 15: Stress-strain diagram of steel I-section.

For steel reinforcement, the yield strength and the elastic modulus respectively are f_y = 358 MPa and E_s = 224 GPa, and corresponding stress-strain diagram (linear elastic perfectly plastic model) used for the analysis appears in Fig. 16 (assuming ε_{su} = 20 ‰).

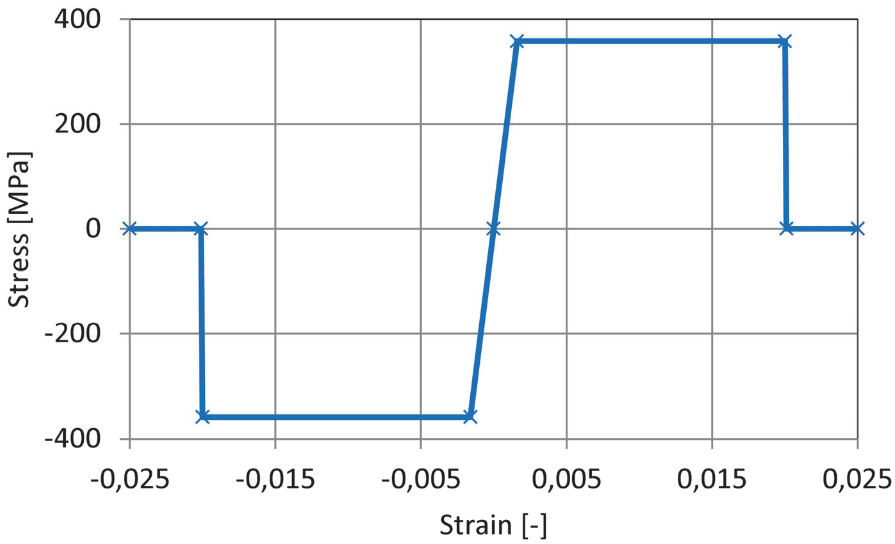

Figure 16: Stress-strain diagram of reinforcing steel.

For the *SRC-E2* specimen, the compressive strength of concrete is specified as f_c = 41.2 MPa. We assume the stress-strain relation for non-linear analysis given by EN 1992-1-1 [4], provision 3.1.5, with: f_{cm} = 41.2 MPa, E_{cm} = 34 GPa, ε_{c1} = 2.25 ‰, ε_{cu1} = 3.5 ‰. The resulting stress-strain diagram for concrete used for the analysis is illustrated in Fig. 17.

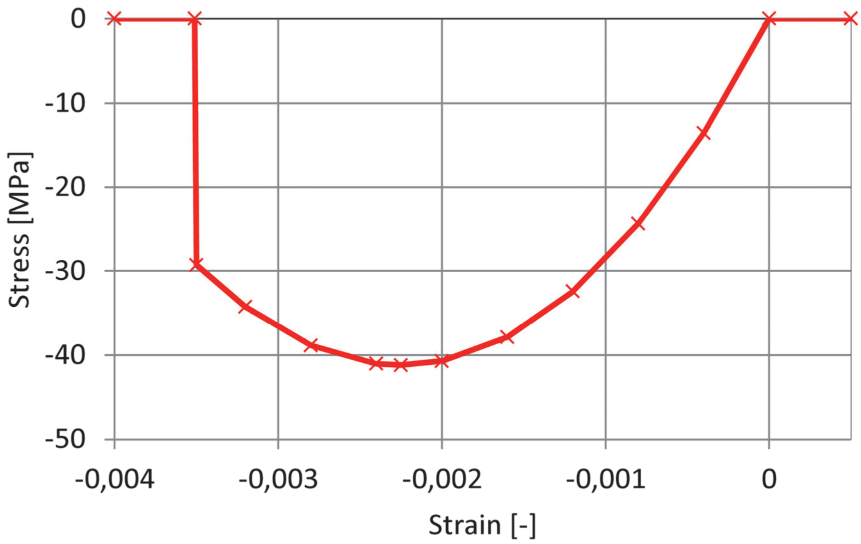

Figure 17: Stress-strain diagram of concrete of SRC-E2 specimen.

For the *SRC-E8* specimen, the compressive strength of concrete is specified as f_c = 61.9 MPa. We assume the stress-strain relation for non-linear analysis given by EN 1992-1-1 [4], provision 3.1.5, with: f_{cm} = 61.9 MPa, E_{cm} = 38 GPa, ε_{c1} = 2.5 ‰, ε_{cu1} = 3.2 ‰. The resulting stress-strain diagram for concrete used for the analysis is illustrated in Fig. 18.

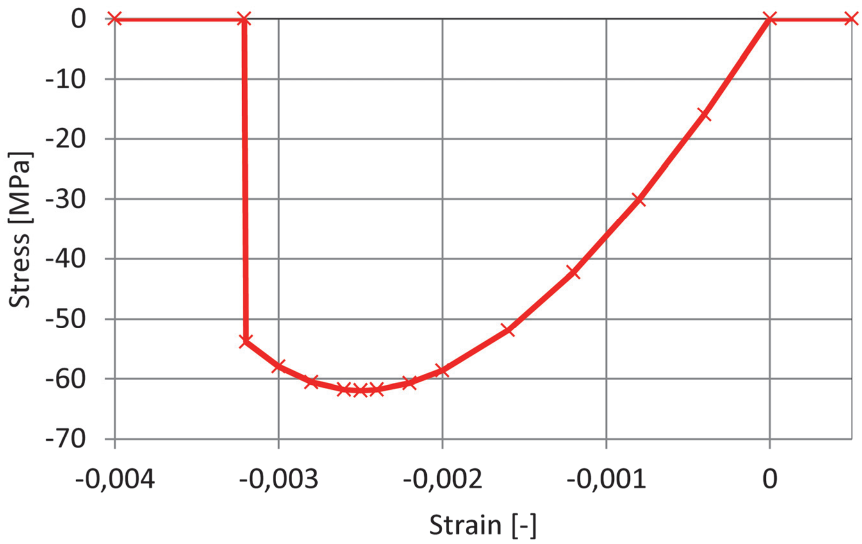

Figure 18: Stress-strain diagram of concrete of SRC-E8 specimen.

Horizontal displacement at the mid-length of the analysed column is compared with IDEA StatiCa results. Both material and geometrical nonlinearity is taken into account with no analysis of post-critical behavior. The calculation was performed for several values of the normal force F with the eccentricities at the top and the bottom of the column, respectively, $e_t = e_b$ = 0.03 m for column *SRC-E2* and $e_t = e_b$ = 0.15 m for column *SRC-E8* until the collapse is reached. The resulting comparison appears in Fig. 19 and Fig. 20. It is obvious that the results obtained by the IDEA StatiCa solver and the data stated in reference [9] are closely similar.

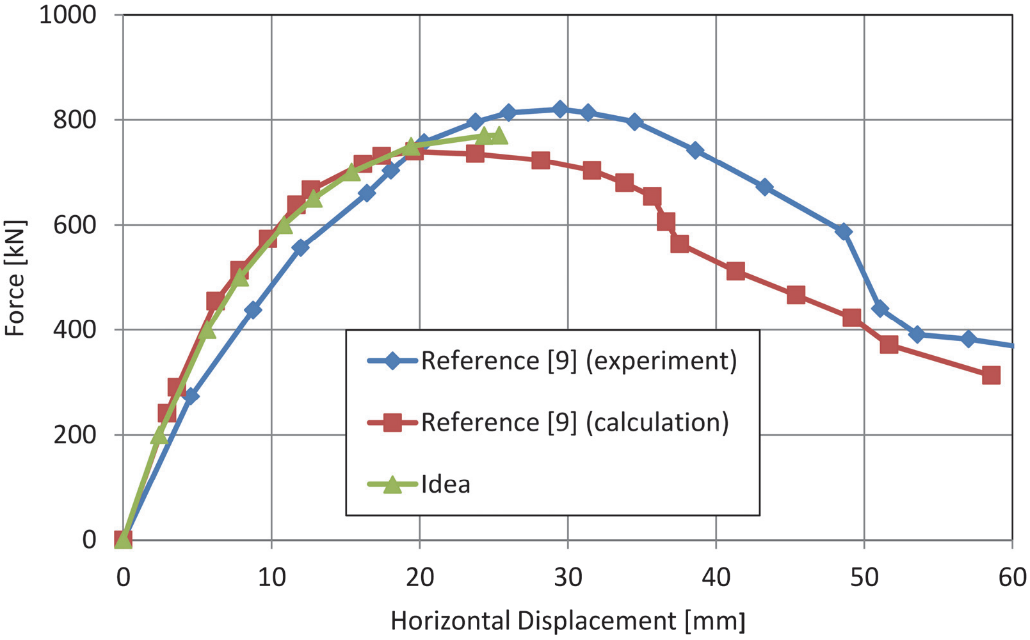

Figure 19: Normal force–horizontal displacement diagrams according to [9] vs. IDEA StatiCa, SRC-E2 column.

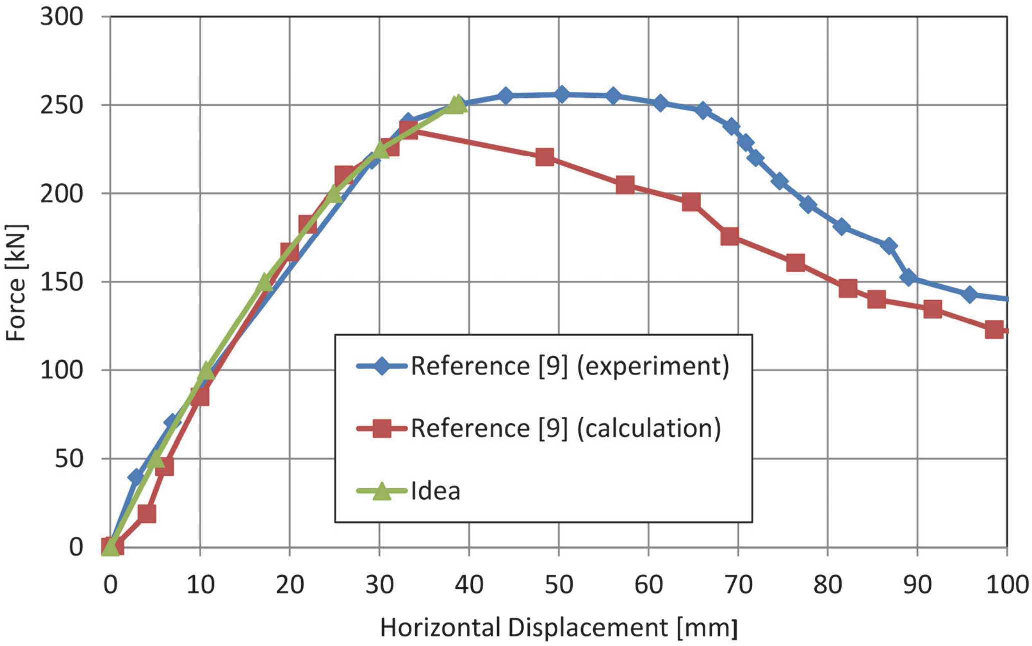

Figure 20: Normal force–horizontal displacement diagrams according to [9] vs. IDEA StatiCa, SRC-E8 column.

The values of ultimate resistance F_{max} and corresponding horizontal displacement w_{Fmax} at mid-length of the column obtained by the IDEA StatiCa solver are compared with the data stated in reference [9], see Tab. 4.

	Ref. [9] experiment	Ref. [9] calculation	IDEA results	Difference experiment	Difference calculation
F_{max} [kN] *SRC-E2*	803	744	**771**	δF_{max} = – 3.99 %	δF_{max} = + 3.63 %
F_{max} [kN] *SRC-E8*	249	237	**251**	δF_{max} = + 0.80 %	δF_{max} = + 5.91 %

Table 4: Comparison of results.

Reinforced concrete column exposed to fire

Final example is reinforced concrete column exposed to fire and loaded by compressive axial force according to [1] , see Fig. 21. The column C_1 (length of 4 m) is loaded by axial force $N_{Ed,fi}$ = 675 kN. Self-weight loading of 2.25 kN/m. Cross-section 300 mm × 300 mm; symmetric reinforcement 2 × 4 × ϕ 15.5 mm (A_s = 8 × 189 mm²) for the case of zero excentricity of the normal force (due to a numerical stability, we assume a small eccentricity of 0,1 mm in z-direction, see Fig. 21, cf. [1, p. 5724]), and 2 × 4 × ϕ 18.4 mm (A_s = 8 × 267 mm²) for the case of e_z = 15 mm. Position of bars – corner bars: x_i = ± 100 mm, y_i = ± 100 mm, inner bars x_i = ± 33 mm, y_i = ± 100 mm..

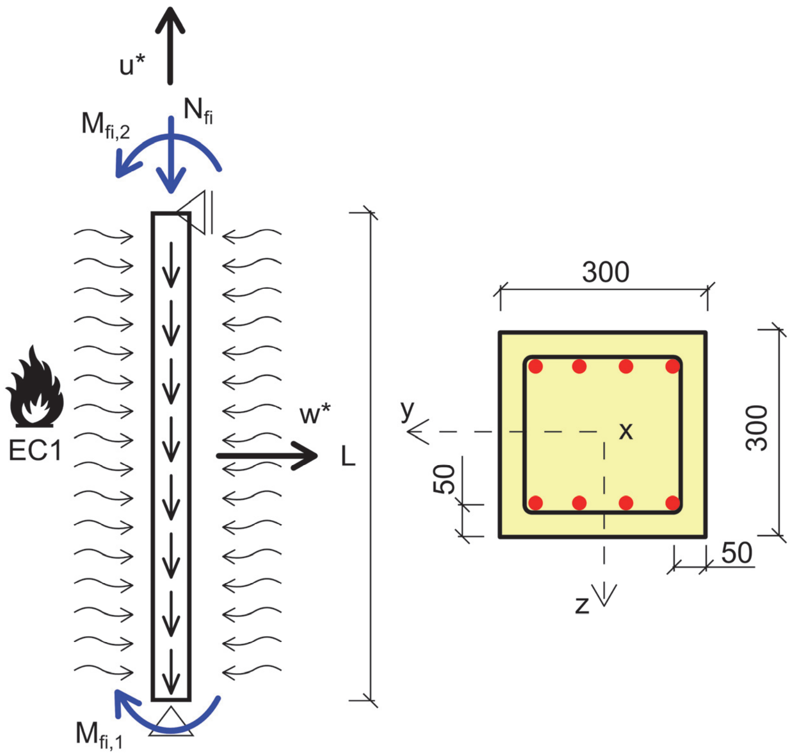

Figure 21: Geometry, cross-section, and the loading of RC columns.

The heating is governed by the standard ISO 834 fire curve, see [3], Eq. (3.4). The heat flux (on the whole boundary of the cross-section and for the whole length of the column) is assumed according to [3], Eqs. (3.1)–(3.3), with:

- α_c = 25 W m^{-2} K^{-1},
- Φ = 1.0,
- ε_m = 0.7,
- ε_f = 1.0,
- σ = 5.67 × 10^{-8} W m^{-2} K^{-4}.

For the calculation with the IDEA solver, the material properties of concrete are assumed as follows (cf. [1]):

- the thermal conductivity given by the lower limit (see [5], 3.3.3(2)),
- the specific heat according to [5], 3.3.2, with u = 1.5 % of concrete weight,
- the density set to the constant value ρ = 2500 kg m^{-3}.

Thermal material properties of steel are assumed according to EN 1992-1-2 [5]. The mechanical properties of steel and concrete and their temperature dependencies are assumed according to EN 1992-1-2 [5], with (cf. [1]):

- f_{ck} = 25 MPa, $E_{c,m}$ = 31.5 GPa,
- f_{yk} = 400 MPa.

First of all it is necessary to compare the temperatures within the analysed column. The temperature distributions within the cross-section are shown in referenced paper [1, Fig. 10] for three times (30, 60 and 120 minutes). Also the evolution of the temperature in steel bars is plotted. The comparison of these results with the data obtained by the IDEA StatiCa appears in Fig. 22.

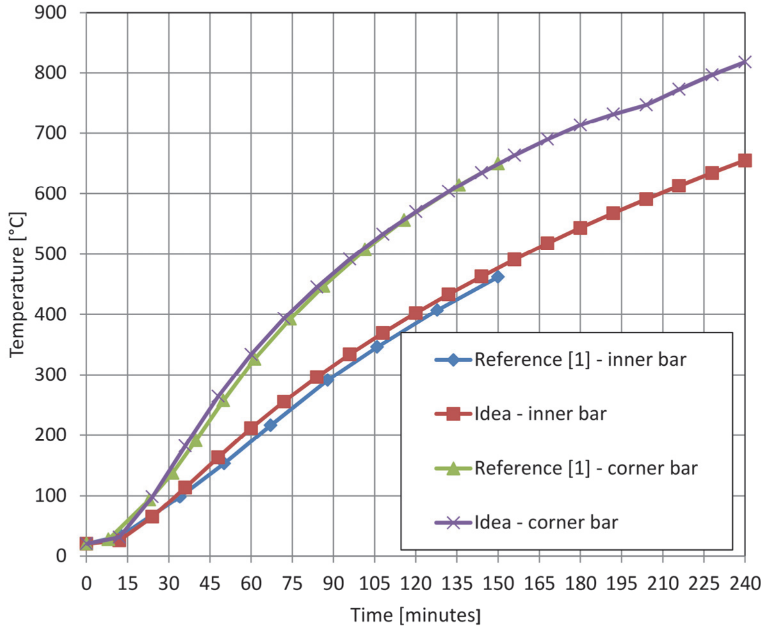

Figure 22: Comparison of temperature evolution in steel bars: [1, Fig. 10] vs. IDEA StatiCa.

In order to evaluate the resulting deformation of the analysed column, we use Fig. 23 to Fig. 26. The results are compared with the results obtained in [1] with the mechanical model, which includes mechanical and the thermal-strain components only (the creep strain, transient strain, etc. are disregarded).
Vertical and horizontal displacements of analysed column are investigated. Geometrical nonlinearity and the temperature dependency are taken into account. The calculation is performed for 10-minutes time periods (0 min, 10 min, 20 min, 30 min, etc.) until the collapse is reached. After that, the calculation is performed in one minute steps (101 min, 102 min, 103 min, etc.) until the exact time of collapse is determined - 107 minutes for the first case (zero excentricity) and 117 minutes for the second case (e_z = 15 mm).

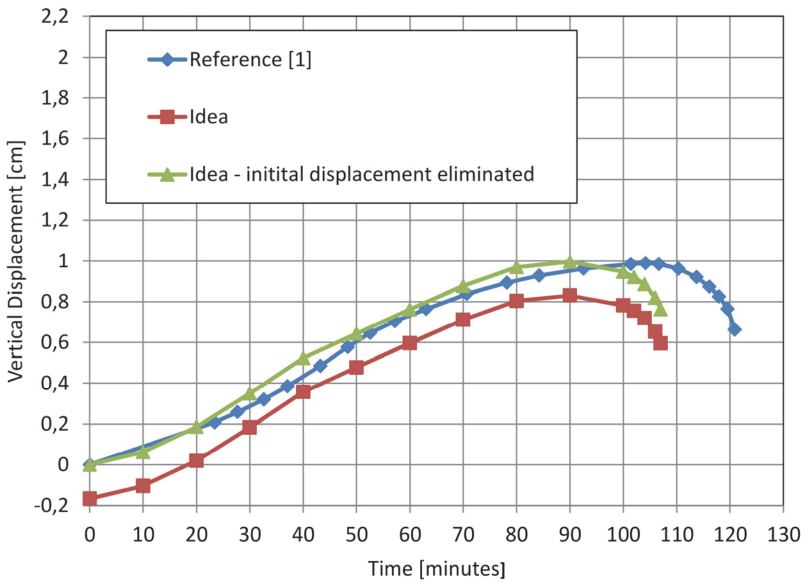

Figure 23: Comparison of vertical displacement for zero eccentricity.

It is obvious that the results stated in reference [1] describe the change between current and initial displacements (the initial displacement is caused by the application of mechanical force). Hence, for comparison of the result obtained by IDEA StatiCa (for which the displacement determined for time 0 includes the deformation due to the mechanical load), it is necessary to eliminate the initial displacement. The results obtained by the IDEA StatiCa and the data stated in [1] correspond each other.

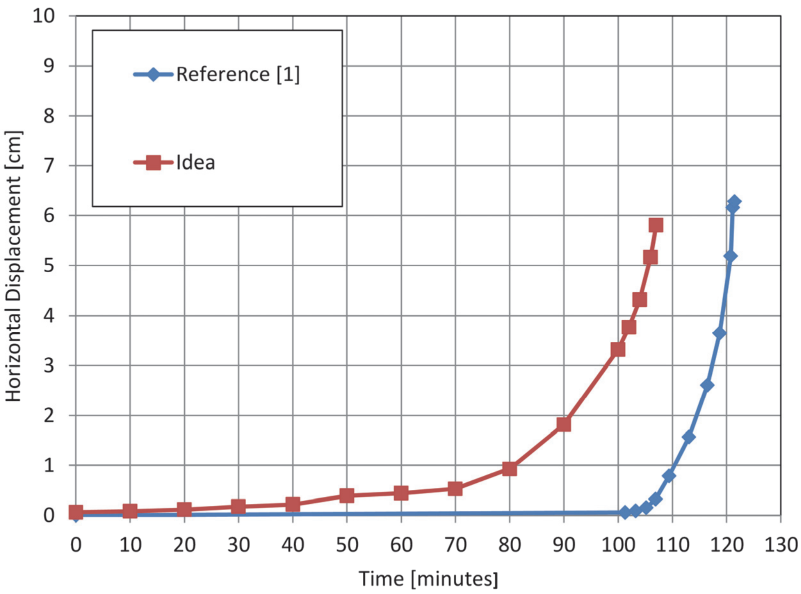

Figure 24: Comparison of horizontal displacement for zero eccentricity.

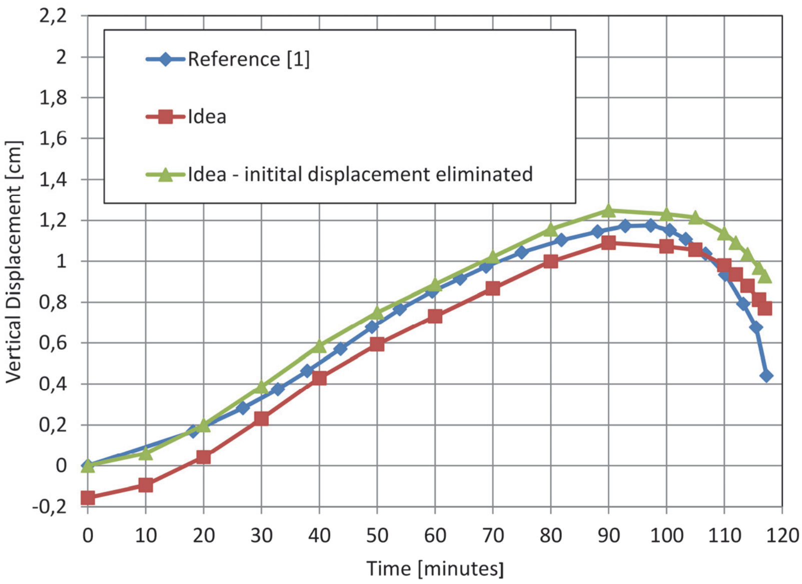

Figure 25: Comparison of vertical displacement for eccentricity 15 mm.

CONCLUSION

A software program for the calculation of reinforced concrete, concrete/concrete and steel/concrete composite columns subjected to various loadings and exposed to normal temperature or to fire conditions was developed inclusive of nonlinear transient analysis for heat transfer and for geometrical and material non-linearity. The

comparison with tests and referenced solutions proved excellent agreement for all types of analyses.
Both calculation cores are part of IDEA StatiCa software [6] and can be used as non-linear solver by third party with the possibility to define input data using public interface - IDEA Open Model.

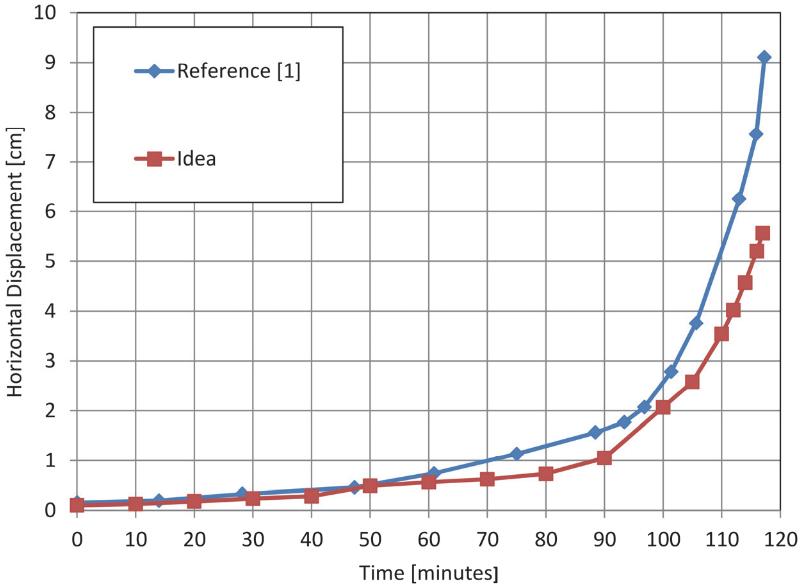

Figure 26: Comparison of horizontal displacement for eccentricity 15 mm.

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