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# THEORETICAL AND EXPERIMENTAL ANALYSES OF COMPOSITE COLUMNS WITH THE USE OF HIGH STRENGTH CONCRETE

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## ABSTRACT

*This paper presents some results of theoretical and experimental analyses of steel-reinforced concrete composite columns with the use of high strength concrete. Columns of high-rise buildings must resist the high values of normal forces. A higher degree of resistance can be obtained with the use of high-strength concrete. The theoretical analysis was made with respect to the current applicable European standards, which were compared with the experimental results of the columns tested and a non-linear analysis using Atena software.*

## KEY WORDS

- *column*
- *composite*
- *steel*
- *concrete*
- *high-strength concrete*

## INTRODUCTION

Composite steel-reinforced concrete (SRC) columns are a very important application of composite structures, and they have wide application in high-rise buildings. Composite columns can be designed according to STN EN 1994-1-1 Design of composite steel and concrete structures, part 1.1, General rules and rules for buildings. According to this standard, only columns and compression members can be designed which are from a normal weight concrete of the strength classes C20/25 to C50/60 and from the steel grades S235 to S460. If high-strength concrete (HSC) is used in a composite column, the resistance will be greater than the resistance of the column with the use of normal strength concrete; respectively, we will achieve a smaller size of the cross-section. In slender composite columns it is necessary to take into account an increase in bending moments according to the second-order theory.

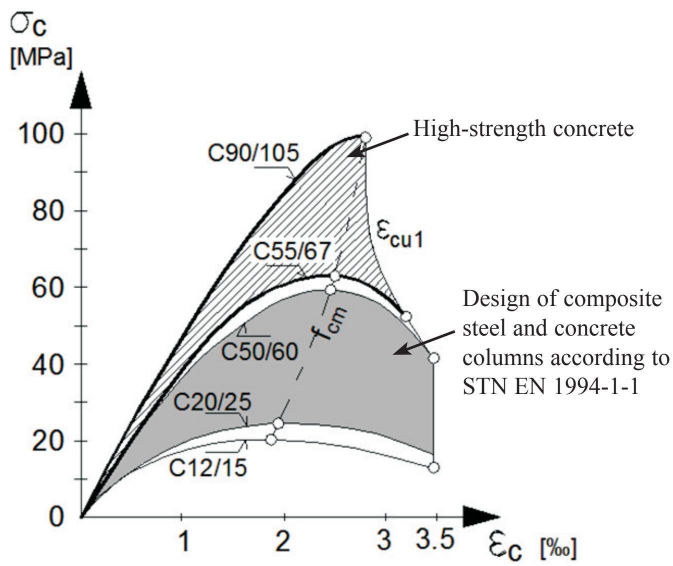


**Fig. 1** *Stress-strain diagram of concrete strength classes C12/15 to C90/105.*

## THEORETICAL ANALYSIS

Composite columns are mainly subjected to compression or to compression and bending. A simple program was created

SLOVAK JOURNAL OF CIVIL ENGINEERING

2009/4 PAGES 31 — 40

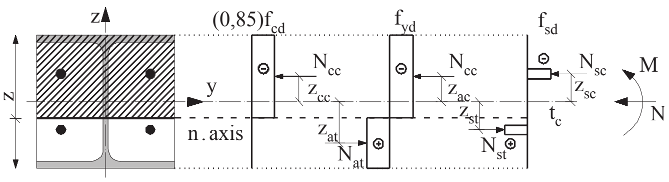


**Fig. 2** *Determination of individual points of interaction diagram.*

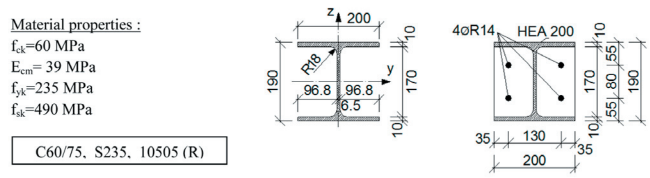


**Fig. 4** *Geometry of the cross-section of composite columns and material properties used.*

for the theoretical analyses to generate interaction diagrams for composite columns with partially concrete-encased steel cross-sections. The interaction diagrams were generated to take into account the second-order theory and imperfections. Cross-sections of partially concrete-encased steel sections are often used in bearing structures and were also chosen for this experimental study.

The plastic interaction diagram (curve) is generated by the definition of the neutral axis position (sectional division of the specified number of parts), and the values of each interaction point N and M result from its position (*Fig. 2*). The resulting moment forces are related to the plastic axis of the cross section, which is identical to the elastic axis in a cross-section with two axis of symmetry. Interaction diagrams are generated in both directions (directions *z* and *y*). Direction *z* (web direction) is generated to take into account the second-order theory and imperfections according to STN EN 1994-1-1.

The generated interaction diagrams are based on these assumptions:

- only the interaction diagram is generated in the region of the normal compressive forces and positive (+) bending moments,
- full interaction between the concrete and steel (full shear connection).

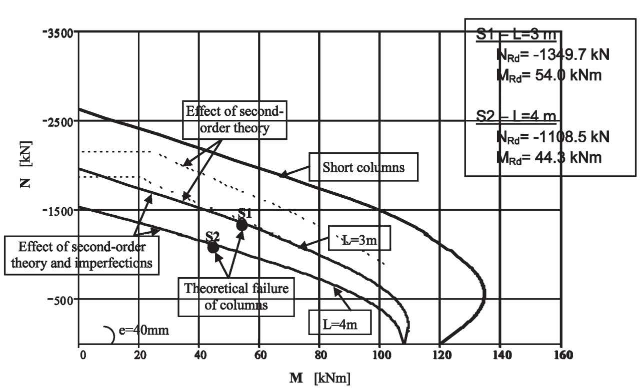


**Fig. 3** *Interaction diagrams with the effect of second-order theory and imperfections for the design values of material properties.*

Fig. 3 presents the interaction diagrams of research taken under on the column two series: S1 and S2. The steps of the load were specified from these interaction diagrams (direction of bending – direction z). The step of load n=1.0 was specified with the effect of the second-order theory and imperfections.

## TEST OF COLUMNS

For short-term laboratory tests of composite steel-reinforced concrete columns with the use of HSC, a partially encased steel-reinforced concrete cross-section with a steel HEA profile was chosen (*fig. 5*). A total of 6 columns in two series were tested. In the first series, 3 columns with a length of 3m were tested (relative slenderness $\bar{\lambda} = 0.58$ - type S1). In the second series, 3 columns of a length of 4m were tested (relative slenderness $\bar{\lambda} = 0.78$ - type S2). The eccentricities of normal compression forces were the same for all the types of the columns (S1 and S2): e = 40mm. The end boundary conditions were joint couplings on both sides of the columns. The relative strains were measured using deformeters (accuracy of 0.001mm) and tensometric strain gauges in the middle of the column. The relative strains and the horizontal deflection up to the failure were measured for each column using:

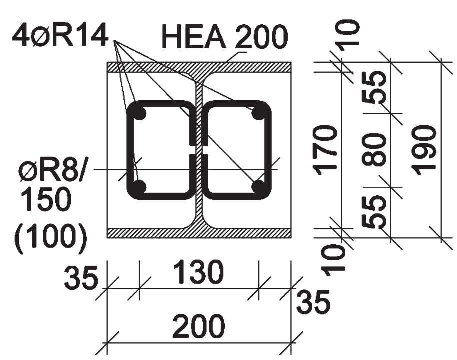


**Fig. 5** *Cross section of composite columns.*

SLOVAK JOURNAL
OF
CIVIL ENGINEERING

2009/4 PAGES 31 — 40

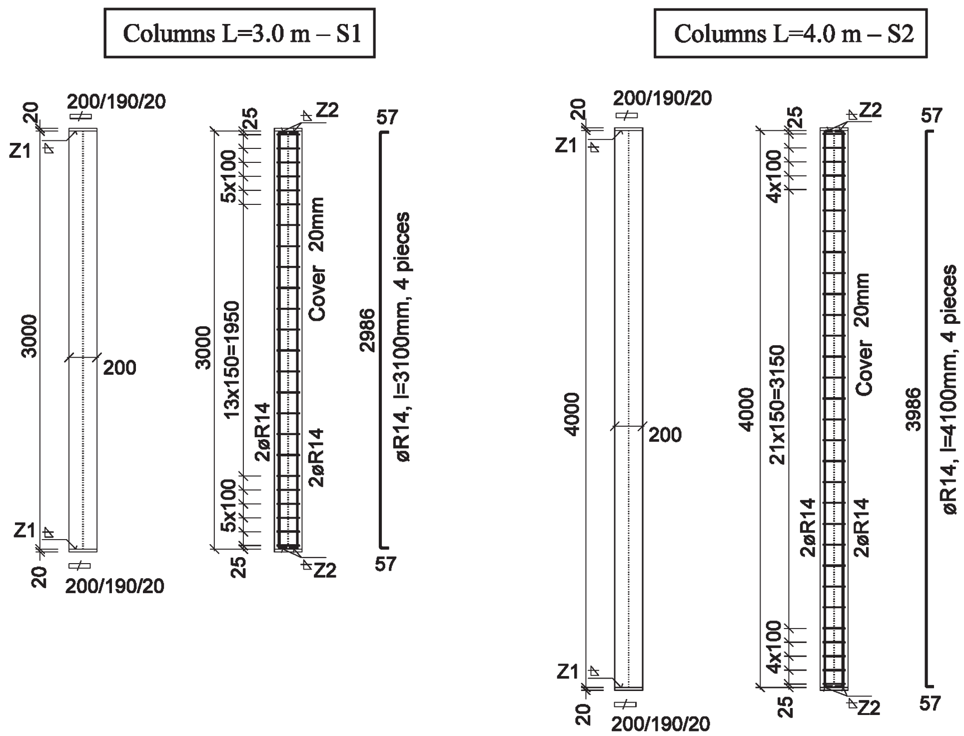


**Fig. 6** *Reinforcement of composite columns for S1 and S2 experimental research series.*

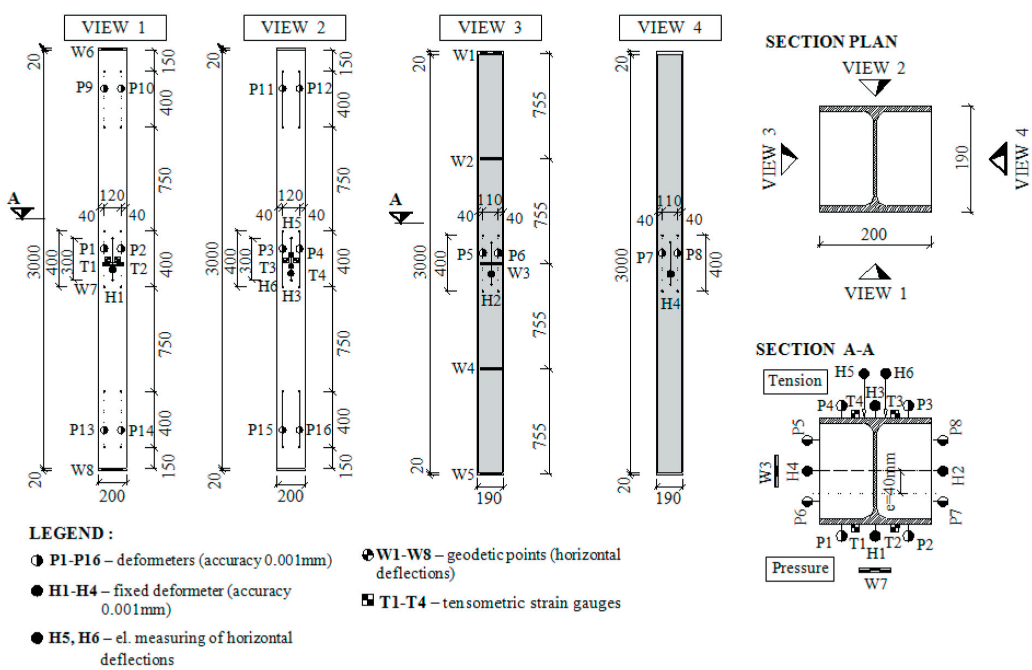


**Fig. 7** *Arrangement of the measured apparatus for the columns of the S1 series.*

SLOVAK JOURNAL OF CIVIL ENGINEERING

2009/4 PAGES 31 — 40

**Tab. 1** *Max. values of horizontal deflections of geodetic surveying in the middle of column lenght –direction Z, direction Y .*

| No. | Series | Step of load in failure (n) | Measured horiz. deflection [mm] | | | | "zero" horiz. deflection [mm] | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | $w_z$ | $w_{pz}$ | $w_y$ | $w_{py}$ | $w_{0z}$ | $w_{p0z}$ | $w_{0y}$ | $w_{p0y}$ |
| 1 | S1-1 | 1.7 | 19.9 | 4.1 | 0.0 | 0.0 | 18.2 | 1.0 | 0.3 | 0.9 |
| 2 | S1-2 | 1.7 | 20.3 | 0.2 | 0.3 | 2.7 | 18.2 | 1.4 | 0.5 | 1.5 |
| 3 | S1-3 | 1.7 | 20.5 | 5.3 | 1.4 | 3.1 | 18.4 | 0.8 | 0.4 | 0.2 |
| 4 | S2-1 | 1.8 | 29.6 | 0.3 | 2.0 | 1.0 | 27.8 | 1.7 | 1.6 | 2.0 |
| 5 | S2-2 | 1.8 | 34.3 | 0.8 | 5.5 | 2.0 | 31.3 | 0.2 | 4.8 | 0.2 |
| 6 | S2-3 | 1.8 | 29.6 | 1.4 | 1.2 | 2.3 | 28.2 | 1.3 | 0.8 | 0.5 |
| Mean value S1 | | | 20.2 | 3.2 | 0.6 | 1.9 | 18.3 | 1.1 | 0.4 | 0.9 |
| Mean value S2 | | | 31.2 | 0.8 | 2.9 | 1.8 | 29.1 | 1.1 | 2.4 | 0.9 |

- (P1 – P16) – deformeters with 400mm bases in three vertical levels,
- (H1 – H4) – fixed deformeters with 300mm bases in the middle of the columns,
- (T1 – T4) – uniaxial tensometric strain gauges in the middle of the steel flange; 2 on the side of the pressure and 2 on the side of the tension. These were directly connected to the computer, and the relative strains in the defined steps were measured and saved.

The horizontal deflections were measured in both directions – direction of bending (direction of the web – direction *z*) and direction perpendicular to the direction of bending (direction of the flange – direction *y*). They were measured using:

- (H5) (H6) – electronic measuring in the middle of the column`s length,
- (W1 – W8) – theodolities in the direction of the web (W1-W5) and in the direction of the flange (W6-W8) of the steel HEA profile.

The arrangement of the measured apparatus for the composite columns of the S1 series is shown in F*ig. 7.* The S2 series was measured with the same arrangement of the apparatus.

### Horizontal deflection of the columns

For each step of the load (n), two types of horizontal deflection were determined from the geodetic surveying – the measured and "zero" horizontal deflections. The measured horizontal deflection represents the values directly measured with a theodolite, while the "zero" horizontal deflection represents the deviation of the vertical axis from the specified vertical plane of the column. The maximum values of the measured ($w_z$, $w_y$) and "zero" horizontal deflections ($w_{0z}$, $w_{0y}$) are in *Table 1*. These maximum values are always in the middle of columns where the failure of the cross-section occurs. There are also the values of the initial horizontal deflections ($w_{pz}$, $w_{py}$). The horizontal deflections increased in the *z* direction (the direction of bending); in the second direction *y* (perpendicular to the direction of bending), there was almost no deflection, respectively, only a very small increase in the horizontal deflection. The mean value of the measured horizontal deflection for the step of the load before failure in the direction *z* was 20.2mm (S1 series) and 31.2mm (S2 series). The mean value of the "zero" horizontal deflection was 18.3mm (S1 series) and 29.1mm (S2 series).

The measured and "zero" horizontal deflections in direction *z* at the measured points are shown in *Fig. 8* for the S1-1 series in the step of the load before failure (n=1.6), and they are compared with the initial horizontal deflection.

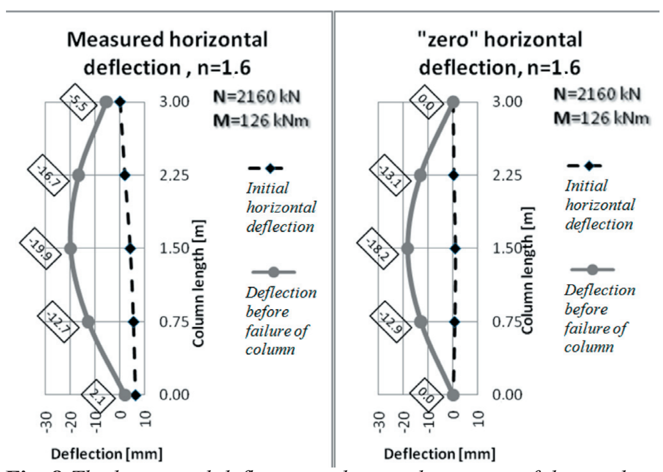


**Fig. 8** *The horizontal deflection values in direction z of the geodetic surveying for the S1-1 series before failure.*

SLOVAK JOURNAL
OF
CIVIL ENGINEERING

2009/4 PAGES 31 — 40

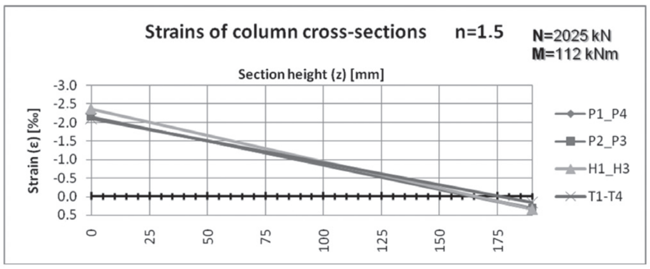


**Fig. 9** *Strains of column cross-section in the middle of the length (S1-2).*

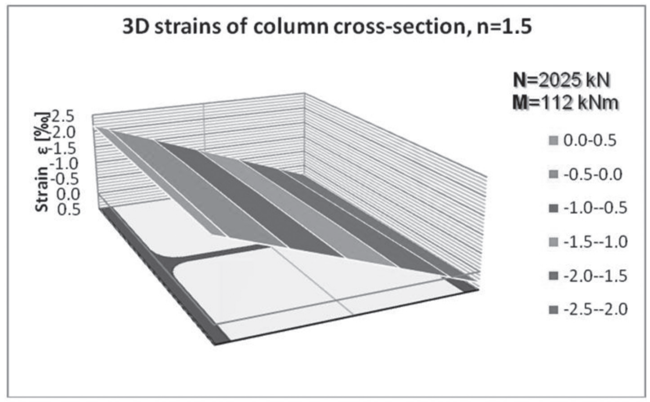


**Fig. 10** *3D strains of column cross-section in the middle of the length (S1-2).*

### Strains of column cross-sections

The strains of the column`s cross-section in the middle of the length in direction *z* for the S1-2 series are shown in *Fig. 9*. The values are measured at the step of the load n=1.5. The results show a good match of the measured values with the use of all the measuring devices. The higher values were measured only with fixed H1-H3 deformeters compared with the tensometric strain gauges and other deformeters.

The 3D strains of the column`s cross-section in the middle of the length are shown in *Fig. 10*. These 3D strains are based on the individual measured values of the P1-P4 deformeters.

It can be concluded that the plane section remained a plane and that the full composite behaviour up to failure was between the steel and concrete components of the member.

### Resistance of columns

The failure of all the columns occurred as was expected in the direction of bending (direction *z*), so the analysis of the resistance was executed in this direction. The measured resistances of the columns were compared to the calculated resistances. They were calculated with the design values of the material properties and also with the real measured values of the material properties (*Table 2*). Resistance according to the "$N_{Rd}M_{Rd}$" method was used for the comparison. The assumption was that the failure occurs at the same value of the eccentricity. The resistance was set as a cross of the

**Tab. 2** *Comparison of column resistances with the design values of the material properties and the real measured values of the material properties.*

| No. | Series | Final eccentricity | Measured resistance | | Resistance $N_{Rd}$, $M_{Rd}$ | | Ratio | Resistance $N_{Rd}$, $M_{Rd}$ | | Ratio |
|---|---|---|---|---|---|---|---|---|---|---|
| | | e [mm] | N [kN] | M [kNm] | $N_{Rd}$ [kN] | $M_{Rd}$ [kNm] | [%] | $N_{Rd}$ [kN] | $M_{Rd}$ [kNm] | [%] |
| 1 | S1-1 | 67.85 | 2287 | 155 | 1324 | 90 | 173 | 2022 | 137 | 113 |
| 2 | S1-2 | 68.87 | 2296 | 158 | 1314 | 90 | 175 | 2004 | 138 | 115 |
| 3 | S1-3 | 69.50 | 2281 | 159 | 1307 | 91 | 174 | 1993 | 139 | 114 |
| 4 | S2-1 | 88.75 | 1988 | 176 | 1128 | 100 | 176 | 1714 | 152 | 116 |
| 5 | S2-2 | 83.32 | 1986 | 165 | 1174 | 98 | 169 | 1788 | 149 | 111 |
| 6 | S2-3 | 89.85 | 2049 | 184 | 1119 | 101 | 183 | 1699 | 153 | 120 |
| Mean value S1 | | | 2288 | 157 | 1315 | 90 | 174 | 2006 | 138 | 114 |
| Mean value S2 | | | 2008 | 175 | 1140 | 100 | 176 | 1734 | 151 | 116 |
| | | | | | design values of material properties | | | real measured values of material properties | | |

SLOVAK JOURNAL OF CIVIL ENGINEERING

2009/4 PAGES 31 — 40

interaction diagram with a line through the "zero" point, which represents the angle of eccentricity.

Conclusions

- The interaction diagram with the real measured values of the material properties is equal to approximately 1.5 times the interaction diagram with the design values of the material properties (in accordance with European standards).
- The resistance of the tested columns was about 1.75 times greater than the resistance of the columns with the design values of the material properties (in accordance with European standards, $\alpha_M$=0.9 and 0.85.$f_{cd}$).
- The resistance of the tested columns was about 1.15 times greater than the resistance of the columns with the real measured values of the material properties ($\alpha_M$=0.9 and 0.85.$f_{cm}$).
- The interaction diagram with the real measured values of the material properties is most suitable for real measured column resistances ($\alpha_M$=1.0 and 1.0.$f_{cm}$).

### Analysis of buckling effect

The theoretical assumptions of the second-order theory and imperfections were determined according to STN EN 1994-1-1. To compare the measured and calculated values of the "k" factor, the following input parameters were used:

- values of eccentricities from the geodetic surveying,
- effective bending stiffness was calculated according to STN EN 1994-1-1,
- factor β was determined from the initial values of the eccentricities.

A comparison of the measured and calculated values of the "k" factor is set at the same value of the pressure`s normal force, i.e., at the maximum value of the compression force (column failure). The results are given in *Table 3*, and their graphic interpretation is given in *Fig. 12*.

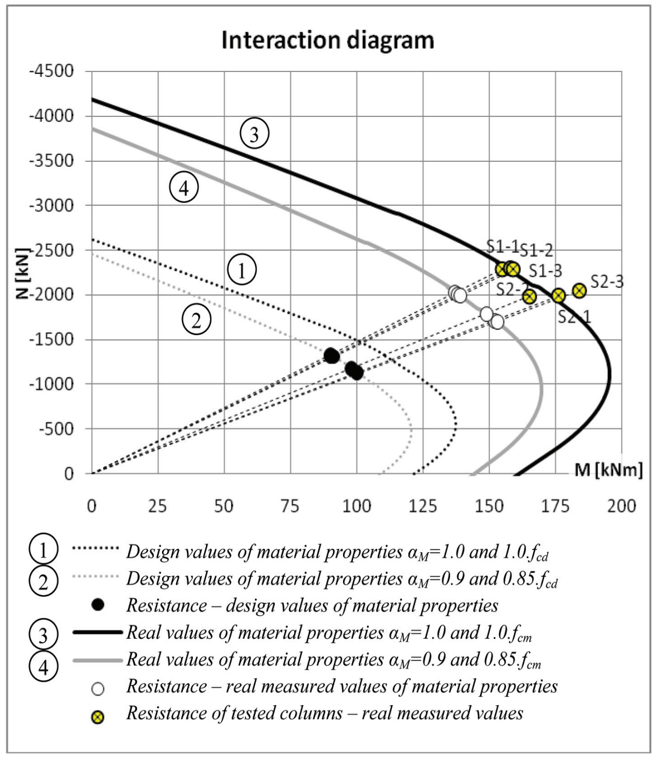


**Fig. 11** *Resistances of composite columns of S1 and S2 series –"$N_{Rd}$, $M_{Rd}$"type.*

When comparing the experimentally measured values of the "k" factor and the calculated values, there is a gap of about 15%. A possible solution is to modify the relationship for calculating the "k" factor, so that the moment of the second-order theory`s effect corresponds to the real deflections of the columns. Based on the tested columns and results, there are some solutions to the modification of the "k" factor relationship (*Fig. 13*). This relationship should be

**Tab. 3** *Comparison of measured and calculated values of "k"factor.*

| No. | Series | N [kN] | $N_{cr,eff}$ [kN] | $N/N_{cr,eff}$ [-] | r [-] | β [-] | Calculated | | Measured | | Gap |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | k [-] | k/β | k [-] | k/β | [%] |
| 1 | S1-1 | 2287 | 9499 | 0.241 | 0.87 | 1.04 | 1.371 | 1.317 | 1.612 | 1.548 | 14.9 |
| 2 | S1-2 | 2296 | 9499 | 0.242 | 0.95 | 1.08 | 1.419 | 1.319 | 1.620 | 1.506 | 12.4 |
| 3 | S1-3 | 2281 | 9499 | 0.240 | 0.82 | 1.02 | 1.341 | 1.316 | 1.590 | 1.561 | 15.7 |
| 4 | S2-1 | 1988 | 5343 | 0.372 | 0.94 | 1.07 | 1.707 | 1.593 | 2.064 | 1.926 | 17.3 |
| 5 | S2-2 | 1986 | 5343 | 0.372 | 0.95 | 1.08 | 1.716 | 1.592 | 2.017 | 1.871 | 15.0 |
| 6 | S2-3 | 2049 | 5343 | 0.383 | 0.88 | 1.05 | 1.699 | 1.622 | 2.042 | 1.949 | 16.8 |
| | | | | | | | | | | Mean value S1 | 14.3 |
| | | | | | | | | | | Mean value S2 | 16.3 |

SLOVAK JOURNAL
OF
CIVIL ENGINEERING

2009/4 PAGES 31 — 40

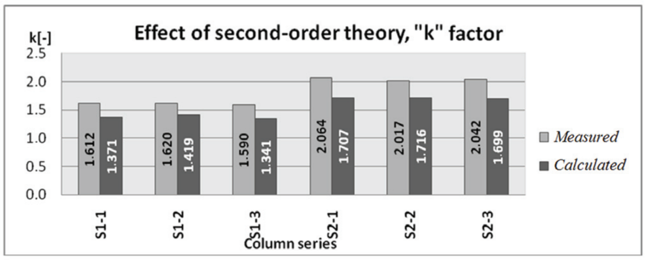


**Fig. 12** *Graphic interpretation of measured and calculated values of "k" factor.*

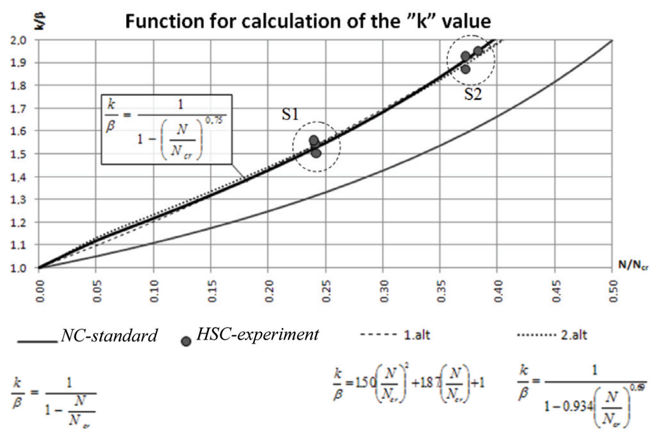


**Fig. 13** *Possible solutions for the modification of the "k" factor relationship for HSC composite columns with the use of HSC.*

verified with further testing of the composite columns with the use of HSC. We recommend the following relationship from the solutions:

$$\frac{k}{\beta} = \frac{1}{1 - \left(\frac{N}{N_{cr}}\right)^{0.75}} \qquad (1.1)$$

## NONLINEAR ANALYSIS WITH THE ATENA 3D SOFTWARE

The same type of columns as the tested composite columns were also analyzed with the model using ATENA 3D. With this accurate modeling it is possible to determine the real resistance of the members, the width of cracks, the deformation and crushing of concrete or the creep of the reinforcement. The program is designed

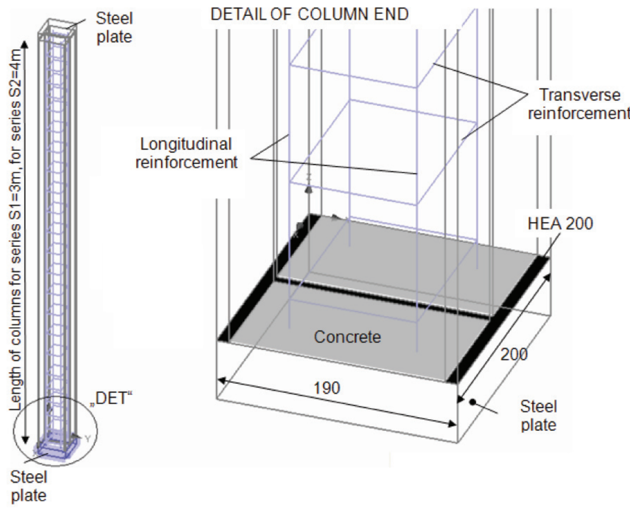


**Fig. 14** *Model of the column and the detail of the column end.*

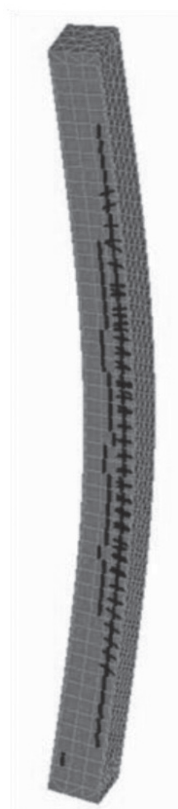

**Fig. 15** *Deformed shape of S1 series column.*

2009/4 PAGES 31 — 40

**Tab. 4** *Comparison of non-linear analyses and experimental measurements.*

| | Resistance | | | | | | Horizontal deflection | | |
|---|---|---|---|---|---|---|---|---|---|
| | Experiment | | Non-lin. analysis | | Gap | | Experiment. [mm] | Non-lin. analysis [mm] | Gap [%] |
| | N [kN] | M [kNm] | NRd [kN] | MRd [kNm] | [%] | | | | |
| Series S1 | 2288 | 157 | 2290 | 165 | 2.6 | | 25.97 | 32.22 | 19.4 |
| Series S2 | 2008 | 175 | 1935 | 185 | 0.7 | | 44.54 | 55.39 | 19.6 |

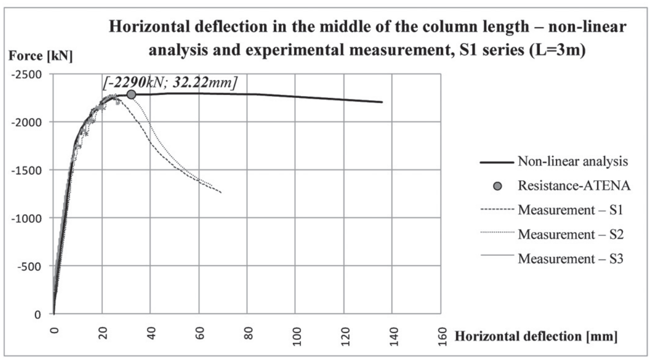


**Fig. 16** *Force - deflection relationships of composite S1 series columns.*

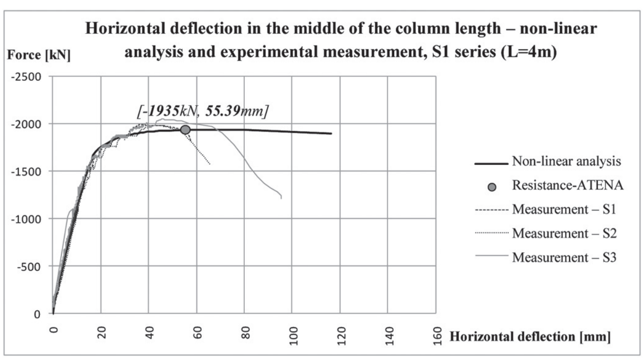


**Fig. 17** *Force - deflection relationships of composite S2 series columns.*

for non-linear analyses of structures and members based on the finite elements method.

## S1 column series

The maximum column resistance of the S1 series columns was 2.290 MN (*Fig. 16*). This resistance was determined as the maximum value of force, where the horizontal deflection was increasing without the force increasing. This value represents a very good match with the experimental results, where the mean value was 2.288 MN (T*able 2*). The horizontal deflection of this maximum force was greater than the experimental value, which was 32.22 mm. It is about 6 mm greater than the average measured value. The force – deflection relationship of the S1 series columns obtained from the nonlinear analysis and the measured horizontal deflections are in *Fig. 16*. It shows a very good match of the non-linear analysis of ATENA with the experimental measurements.

## S2 series column

The maximum column resistance of the S2 series columns was 1.935 MN (*Fig. 17*). This value was lower than the mean value of the experimental results, which was 2.008 MN (T*able 2*). The horizontal deflection at the maximum force was greater than the experimental value and was 55.39 mm. The force - deflection relationships of the S2 series columns obtained from the nonlinear analysis and the measured horizontal deflections are in *Fig. 17*. A very good match of the non-linear analysis of ATENA with the experimental measurements is shown.
The previous calculations and comparisons in T*able 4* provided the following findings:

- the values of the non-linear analysis of the composite columns show a very good match with the tested columns; the non-linear analysis gives greater bending moments in the failures of the columns,
- the horizontal deflections in the middle of the column`s length measured in the failure of the columns are greater in the non-linear analysis than in the experimental measured deflections (approximately a 1.25 times greater value),
- the force – deflection relationship showed a very good match of the non-linear analysis of the ATENA with the experimental measurements.

## CONCLUSIONS

The theoretical and experimental analyses of the composite steel-concrete columns with the use of high-strength concrete provides the following conclusions:

SLOVAK JOURNAL
OF
CIVIL ENGINEERING

2009/4 PAGES 31 — 40

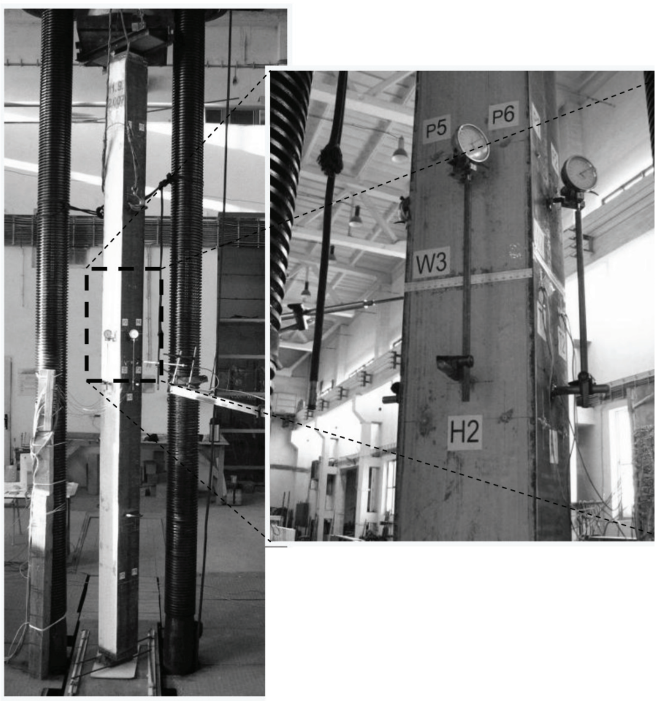


**Fig. 18** *Tested column and detail of measurement in the middle of the length of the column.*

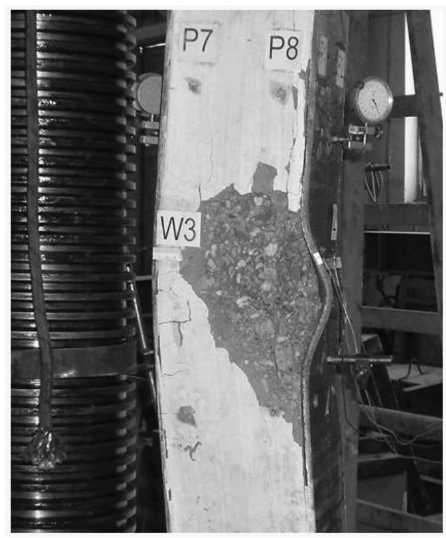


**Fig. 19** *Failure of the tested column in the middle of the length of the column.*

- the value of the experimental measured bending moment according to the effect of the second-order theory was greater than the value calculated according to code STN EN 1994-1-1, i.e. the calculated value of "k" factor was less than the values of the tested columns. We recommend the relationship (1.1),
- the resistance of the tested columns was about 1.75 times greater than the resistance of the columns with the design values of the material properties (in accordance with the codes, $\alpha_M$=0.9 and 0.85.$f_{cd}$),
- a very good match of the resistances of the tested composite columns and the values of the resistances calculated in accordance with the code was found,
- the values of the non-linear analysis of the composite columns with the use of the real measured material properties indicate a very good match with the tested columns,
- the experimental results can be used for further research of composite steel-concrete columns.

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SLOVAK JOURNAL
OF
CIVIL ENGINEERING

2009/4 PAGES 31 — 40

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