BUCKLING RESISTANCE OF STEEL-CONCRETE COLUMNS COMPOSED OF HIGH-STRENGTH MATERIALS

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Abstract*. The paper deals with some problems of the actual behaviour and buckling resistance of steel-concrete compression members composed of high-strength steel and high-performance concrete. Especially, this paper is directed towards the steel-concrete composite columns of the dimensions used usually for columns of multi-storey buildings. High-strength materials (steel and concrete, too) can effectively increase the load-carrying capacity of members in the case of simple compression, because of their high strengths. In the case of the buckling the load-carrying capacity is significantly influenced by the stiffness (through the slenderness) dependent on the modulus of elasticity. For steel the modulus of elasticity is the same regardless of steel grade, in the case of high-strength concrete the modulus of elasticity increases slowly compared with the strength increasing, so that can influence the buckling resistance negatively. This paper presents brief information and some results of experimental and theoretical analysis oriented to the usage of high-strength steel and concrete in composite columns, from the viewpoint of the buckling resistance, for the particular examples of typical steel-concrete sections - open H-sections encased between flanges and circular tubes filled by concrete.*

1 INTRODUCTION

In the last period in civil engineering constructions the usage of modern progressive materials is frequent. In the reliable and efficient structures the combinations of steel and concrete is effective to reach the high load-carrying capacity commonly with the low self-weight and costs. For this reason the utilization of high-strength steels (HSS) and high-performance concretes (HPC) can be advanced in the case of steel-concrete columns, too. However, because of the buckling resistance increasing it is necessary to find the available cross-section type with the suitable relation of both section parts from the viewpoint of the cross-section form and the contribution of steel and concrete to the member resistance.

On the workplace of the Division of Metal and Timber Structures of the Faculty of Civil Engineering at the Brno University of Technology (Brno, Czech Republic) the intensive attention is oriented to the problems of the buckling resistance of steel-concrete composite members (including composite columns) in the period of last several years. Especially, the behaviour of the structural members composed of high-quality steels and concretes is investigated [2], [8]. These research activities are directed to the problems of the behaviour and resistance of steel-concrete compression members composed of HSS and HPC and cover various forms of the analysis, mainly for example: (i) experimental verification of the actual behaviour and the objective ultimate resistance with the utilization of the test results ([6], [7], [8]) for the design resistance determination (philosophy of the design assisted by testing), (ii) the buckling resistance analysis based on the general theoretical principles in comparison with the normative rules approaches (see [1], [4]), (iii) numerical analysis using the static modelling aimed to their verification and calibration helping the test results [1], (iv) parametric studies aimed to the finding of the optimal configuration of the cross-sections respecting the economic viewpoints [5], (v) statistical and probabilistic evaluation [9] aimed to the reaching of the guaranteed reliability level with the structural design economy, (vi) sensitivity analysis [3] directed to the influence of particular geometrical and physic-mechanical parameters and their importance to the result buckling resistance, and other methods.

2 THEORETICAL ANALYSIS OF DESIGN BUCKLING RESISTANCE

In accordance with the experimental research of the behaviour and load-carrying capacity of the steel-concrete columns (see below) the design buckling resistances were calculated using the normative rules given in [10], [11], to select available of cross-section types from the viewpoint of the optimal proportions of steel-to-concrete areas and strengths (also within the context of subsequent numerical modelling). The design buckling resistances have been calculated for two selected cross-section types – circular tube filled by concrete and HEA section encased by concrete between flanges. In this theoretical calculation the members of cross-sections and critical lengths typical for the usual building columns (in practice) were investigated. Hence for the calculation of design resistance steel-concrete compression members of the following cross-sections were utilized: (i) circular tube TR Ø152/4.5 and (ii) cross-section HE 140A, in both cases for the actual length L_{cr} = 3 000 mm. For the theoretical analysis of the design buckling resistance respecting the effect of material properties various steels and concretes were considered – steel grades were in the range from S 235 to S 690 (for more detail see [15]), concrete classes were in the range from C 20/25 to C 80/95 (for more detail see [16]). For the possibility of the comparison, cross-section areas of both parts (steel and concrete) were the same approximately. Design values of the buckling resistance were calculated according to relevant European Standards [10], [11], in the case of steel-concrete columns using the simplified method for the buckling resistance determination.

The design buckling resistance of steel and steel-concrete compression member is given as

$$N_{b,Rd} = \chi \cdot A \cdot \frac{f_y}{\gamma_{M1}}, \qquad \chi \cdot N_{pl,Rd} \tag{1}$$

where for the steel member A is the cross-section area, f_y is the nominal steel yield strength, γ_{M1} is the material partial safety factor and χ is the buckling reduction factor (see below) depending on the non-dimensional slenderness $\bar{\lambda}$; the full-plastic resistance for the steel-concrete member is given as

$$N_{pl,Rd} = A_a \cdot \frac{f_y}{\gamma_a} + (0.85) \cdot A_c \cdot \frac{f_{ck}}{\gamma_c}, \tag{2}$$

where A_a, A_c are the cross-section areas of steel and concrete section parts, f_y is the nominal steel yield strength, f_{ck} is the characteristic concrete cylindrical strength, γ_a, γ_c are the material partial safety factors for steel and concrete (see [10], [11]). The coefficient 0.85 is used for the open steel cross-section only, not for hollow sections. The non-dimensional slenderness $\bar{\lambda}$ can be obtained from the formula

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}}, \tag{3}$$

where the full-plastic section resistance (without buckling) $N_{pl,Rk}$ for steel cross-section and for steel-concrete cross-section respectively, is given in the forms of

$$N_{pl,Rk} = A \cdot f_y, \qquad N_{pl,Rk} = A_a \cdot f_y + A_c \cdot f_{ck} \tag{4}$$

and the critical force N_{cr} for steel and steel-concrete cross-section respectively, is given by the formats

$$N_{cr} = \pi^2 \cdot \frac{EI}{L_{cr}^2}, \qquad N_{cr} = \pi^2 \cdot \frac{(EI)_{eff}}{L_{cr}^2} \tag{5}$$

with the flexural stiffness EI or the effective flexural stiffness $(EI)_{eff}$ (for steel-concrete members). Then the buckling reduction factor χ is given generally (for steel or steel-concrete columns, too) in the form of

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \overline{\lambda}^2}}, \tag{6}$$

where
$$\phi = 0.5 \cdot \left[1 + \alpha_1 \cdot \left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right]. \tag{7}$$

The imperfection factors α_1 were considered as follows: (i) for circular tubes – in all cases (steel and also steel-concrete members) the imperfection factor $\alpha_1 = 0.21$ (buckling curve "*a*"), but for steel grades S 460 and higher $\alpha_1 = 0.13$ (buckling curve "a_0"); (ii) for hot-rolled HEA sections buckled to weak axis – the imperfection factor $\alpha_1 = 0.49$ (curve "*c*"), for steel grades S 460 and higher $\alpha_1 = 0.21$ (curve "*a*").

To show the influence of the steel yield strength and cylindrical concrete strength to the load-carrying capacity of steel-concrete columns the values of the design buckling resistance $\chi\ N_{pl,Rd}$ in comparison with the design full plastic resistance $N_{pl,Rd}$ (in the case of the simple compression) and with the critical force N_{cr} are depicted in graphs on Figure 1. These values were calculated for the configurations and dimensions of cross-sections and for material parameters in the range mentioned in the description above. Within the context of the experimental verification, in Figure 1 also the values of $\chi\ N_{pl,Rd}$, $N_{pl,Rd}$ and N_{cr} are drawn for the steel grade S 275 (in the case of circular tubes), respectively S 355 (HEA sections), in the combinations with concrete class C 20/25 and C 80/95, which were assumed for the test specimens.

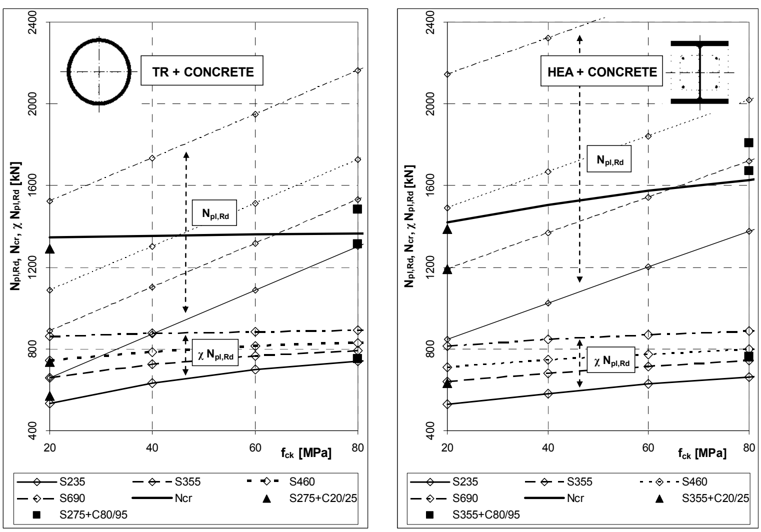

Figure 1: Steel-concrete columns: design buckling resistance $\chi\ N_{pl,Rd}$ in comparison with the full plastic resistance $N_{pl,Rd}$ and critical force N_{cr} – influence of steel yield strength and cylindrical concrete strength.

3 EXPERIMENTAL VERIFICATION OF BUCKLING RESISTANCE

3.1 Test specimens, test arrangement, test realization

Within the framework of the experimental verification 18 specimens have been tested, from that 9 specimens with the tube TR ∅ 152/4.5 and 9 specimens with HEA section HE 140A. In both groups the following types of specimens were tested: 3 steel specimens and 6 steel-concrete specimens filled or encased by concrete, from that 3 test specimens with normal concrete (NC) and 3 test specimens with high-performance concrete (HPC) – the specification and description of tested specimens (including the symbols used in graphs below), from the viewpoint of cross-section types and material parameters, which were measured, is presented in Table 1. Realized tests aimed at the comparison of actual results with theoretical analyses. Some illustrations of the test realization are shown in Figure 2.

Table 1: Specification of test specimens – cross-section configuration and material parameters

cross-section type	symbol and description of cross-section type			
	steel – circular tube		steel – HEA cross-section	
steel	TR	TR ∅ 152/4.5	HEA	HE 140 A
steel + concrete	TR + NC	TR ∅ 152/4.5 + normal concrete	HEA + NC	HE 140 A + normal concrete
	TR + HPC	TR ∅ 152/4.5 + high-performance concrete	HEA + HPC	HE 140 A + high-performance concrete
material	measured physical-mechanical parameters – mean values			
steel	TR	yield strength f_{ym} = 354 MPa	HEA	yield strength f_{ym} = 456 MPa
concrete	NC	cube strength f_{ccm} = 34 MPa, cylindrical strength f_{cm} = 27 MPa, E_{cm} = 32 GPa		
	HPC	cube strength f_{ccm} = 102 MPa, cylindrical strength f_{cm} = 87 MPa E_{cm} = 49 GPa		

Figure 2: Illustration of test specimens and loading tests – HEA sections with concrete

3.2 Test results evaluation

From the realized tests the basic results were obtained, especially the ultimate objective load-carrying capacity reached in the moment of the specimen failure, which was used as the most important result for the evaluation of the actual buckling parameters. These test results were utilized i.a. for the derivation of the actual buckling length and slenderness respectively, and for the determination of the actual initial imperfections, which can influenced the buckling resistance very significantly.

3.2.1 Actual buckling length and slenderness

During loading tests the actual supporting of member ends was investigated. Theoretically the hinges on both member ends were supposed, if the member length between supports was L = 3 070 mm. It is evident, that due to real structural detailing of the ends supporting and other influences (e.g. the loading force is not centric, the member axis is not parallel with the force direction etc.) the assumption of ends hinges is more or less inexact. These effects will influence the actual member slenderness, which can be derived from the test results and related to the slenderness supposed theoretically.

If N_{test} is the force corresponding the member buckling and N_{cr} is supposed critical force calculated for the hinges on both ends ($L_{cr} = \beta\, L$, where L_{cr} is the buckling length, L is the member length, β is the buckling length factor, here for the theoretical assumptions $\beta = \beta_{th} = 1$), then for the non-dimensional slenderness $\bar{\lambda}_{test}$ from the tests and for the theoretically calculated slenderness $\bar{\lambda}_{th}$ it can be written

$$\bar{\lambda}_{test} = \sqrt{\frac{N_{pl}}{N_{test}}} = \frac{L_{cr,test}}{\pi}\sqrt{\frac{N_{pl}}{E \cdot I}} = \frac{\beta_{test} \cdot L}{\pi}\sqrt{\frac{N_{pl}}{E \cdot I}}\,, \tag{8}$$

$$\bar{\lambda}_{th} = \sqrt{\frac{N_{pl}}{N_{cr,th}}} = \frac{L_{cr,th}}{\pi}\sqrt{\frac{N_{pl}}{E \cdot I}} = \frac{\beta_{th} \cdot L}{\pi}\sqrt{\frac{N_{pl}}{E \cdot I}}\,. \tag{9}$$

From equations (8), (9) the actual buckling length follow as $L_{cr,test} = \beta_{test}\, L$ and the actual buckling length factor β_{test} can be written (if $\beta_{th} = 1$) in the format of

$$\frac{\bar{\lambda}_{test}}{\bar{\lambda}_{th}} = \frac{L_{cr,test}}{L_{cr,th}} = \frac{\beta_{test} \cdot L}{\beta_{th} \cdot L} = \frac{\beta_{test}}{\beta_{th}} = \beta_{test}\,. \tag{10}$$

Table 2: Actual buckling length and non-dimensional slenderness

cross-section type		N_{cr} [kN]	N_{test} [kN]	β_{test}	$L_{cr,test}$ [mm]	$\bar{\lambda}$
circular tube	TR	1 248.2	667.7	1.367	4 197	1.051
			640.0	1.397	4 287	1.074
			602.5	1.439	4 419	1.107
	TR + NC	1 660.9	1 000.6	1.288	3 955	1.082
			904.6	1.355	4 160	1.138
			929.3	1.337	4 104	1.123
	TR + HPC	1 880.2	1 498.8	1.120	3 438	1.194
HEA cross-section	HEA	855.4	881.8	0.985	3 024	1.274
			931.2	0.958	2 942	1.240
			835.2	1.012	3 107	1.309
	HEA + NC	1 388.7	1 239.5	1.058	3 250	1.223
			1 321.5	1.025	3 147	1.184
			1 078.7	1.135	3 483	1.311
	HEA + HPC	1 672.0	1 707.3	0.990	3 038	1.278
			1 603.4	1.021	3 135	1.319
			1 541.4	1.042	3 197	1.345

The actual buckling length factors, buckling lengths, respectively non-dimensional slenderness calculated using the equations (8), (9), (10) are written in Table 2. The graphic expressions of the actual and theoretically calculated values of the buckling resistance in dependence on the non-dimensional slenderness are seen in Figures 3 (for circular tubes) and 4 (for HEA sections). In Figures 3 and 4 these values are shown for all 3 types of specimens (TR, TR+NC, TR+HPC, respectively HEA, HEA+NC, HEA+HPC), to show the influence of the section configuration and material properties on the buckling resistance. For the comparison the test values are drawn for the actual and also for the theoretically assumed slenderness, respectively buckling lengths. It is seen, that the buckling resistances obtained from the tests are practically equal to the Euler critical force, in the case of the test specimens with the HEA cross-sections especially. It is mainly due to the very small imperfections, which were observed in the range from L_{cr} / 1 500 to L_{cr} / 2 000 (see Figure 6) in this particular specific case, where quite randomly the conditions were suitable to allow the buckling resistance reaching the Euler critical force.

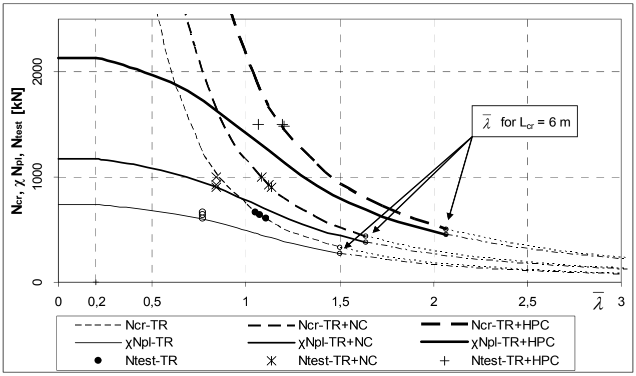

Figure 3: Buckling resistance in dependence on the non-dimensional slenderness – circular tubes

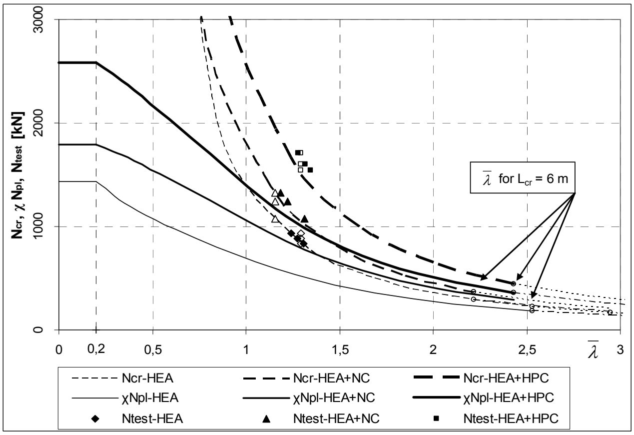

Figure 4: Buckling resistance in dependence on the non-dimensional slenderness – HEA sections

In Figure 5 the ratios of buckling resistances and critical forces to full plastic resistances related to the non-dimensional slenderness, are drawn for all cross-section types. For the comparison the same ratios for the test results are added to the graphs. The ratio of the buckling resistance to the full plastic resistance is the reduction factor χ corresponding to the buckling curve "*a*" (circular tubes), respectively "*c*" (HEA sections). The ratio of the critical force to the full plastic resistance, using (3) is given as

$$\frac{N_{cr}}{N_{pl}} = \frac{1}{\bar{\lambda}^2}, \tag{11}$$

so that in dependence on $\bar{\lambda}$ this relationship is the same for all types of cross-sections regardless of the cross-section configuration and mechanical properties.

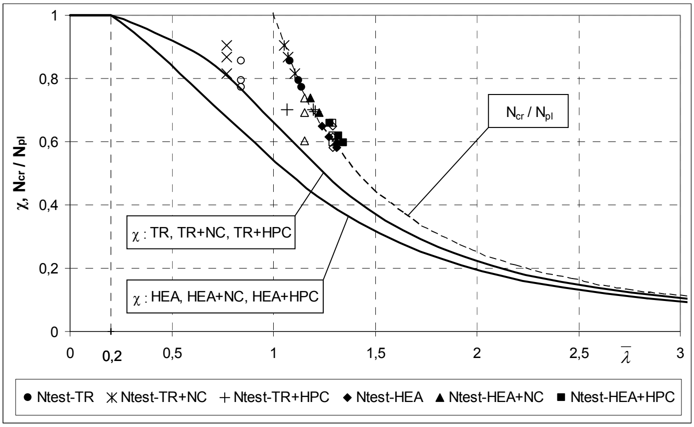

Figure 5: Ratios of the buckling resistances and critical forces to the full plastic resistances in dependence on the non-dimensional slenderness – comparison with test results

3.2.2 Actual initial imperfections

Information on the influence of the initial imperfections the Southwell line can give (it expresses the linear relationship between the values w / N and w), helping the initial imperfection e_0 (for its definition see Figure 6), which in the form of the relative initial imperfection $m_0 = e_0 / j$ (where $j = W / A$) is used in the procedure of the derivation of the reduction buckling factor χ. In Figure 6 the Southwell lines and corresponding actual initial imperfections are depicted for the test results of all test specimen types.

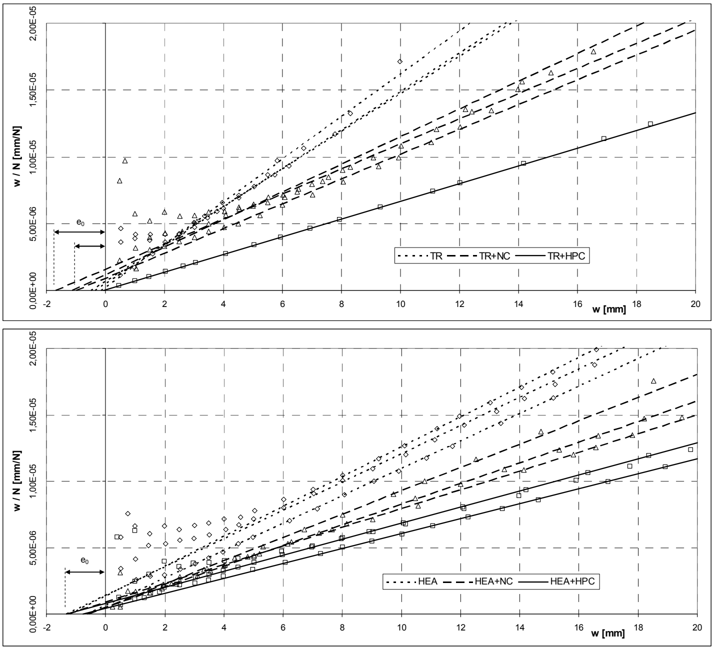

Figure 6: Soutwell lines and actual initial imperfections

CONCLUSIONS

Summarizing the results of the design analysis and experimental programme particular concluding remarks can be mentioned: (i) the actual objective buckling resistances were higher than the calculated ultimate resistances, so that they were much higher (in average by 30 %) than design resistances; (ii) the actual supporting of the member ends significantly influences the actual buckling length – for circular tubes the actual buckling lengths were even by 40 % higher than theoretical buckling lengths considered for the theoretical assumptions; (iii) actually obtained initial imperfections were very low (even several times) in comparison with the imperfections given in [10], [11]. Ascertained information can influence the test evaluation not only positively, but in some cases negatively, too.

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