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THE EFFECT OF MATERIAL STRENGTH ON THE BEHAVIOUR OF CONCRETE-FILLED STEEL ELEMENTS

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Abstract. Composite columns and beams are a combination of concrete and steel elements realising the advantages of both types of materials. According to codes for concrete-filled column, the plastic resistance of the cross-section is given as a sum of the components and taking into account the effect of confinement in case of circular sections. In this study the stress state in composite column is determined taking into account non-linear relationship of the modulus of elasticity and Poisson's ratio on the stress level in the concrete core. It has been determined that the effect of confinement occurs at a high stress level when structural steel acts in tension and concrete works in lateral compression. The stress state and load bearing capacity of section in bending is determined taking into account non-linear dependence on position of neutral axis. Because the ultimate limit state of material is not attained for all the parts simultaneously, to improve the stress state of a composite element and to prevent the possibility of a failure the appropriate strength of concrete and steel should be used. The safety of high-stressed composite structures can be achieved by using ultra-high-performance concrete (UHPC).

Keywords: composite columns and beams, stress state, non-linear deformation, ultra-high-performance concrete.

1. Introduction

Composite columns and beams, formed as hollow steel sections filled with concrete, have advantages in different architectural and structural solutions. The composite structure characterises with higher ductility than the concrete column and advantages of steel may be successfully used in connections. Concrete filling and reinforcing bars increase rigidity and load-bearing capacity of the hollow steel section with no changing in external dimensions of column. Filling improves the fire resistance as well. Composite column with a corresponding content of reinforcement can provide at least 90-minutes of fire-resistance rating [1].

According to EC4 [2], for determining the resistance of a section against bending moment, a full plastic stress distribution in the section has been assumed. The internal bending moment resulting from the stresses is the resistance of the section against acting moment. In order to increase the plastic bending capacity reinforcement can be included.

EC4 gives a simplified ultimate limit state design method for the composite structures, which is applicable for practical purposes. The method is based on consumption that ultimate strength of material is attained simultaneously in all parts of the section.

In previous decades investigations regarding interaction between steel shell and concrete core has been performed [3–7]. In general, it is concluded that for service load practically no bond exists and that the core and shell act as two independent materials. In the early stages of loading, the Poisson's ratio for concrete is lower than that for steel and the steel tube has no restraining effect on the concrete core. As the longitudinal strain increases, Poisson's ratio of concrete can remarkably increase. Therefore, the lateral expansion of unconstrained concrete gradually becomes greater than that of steel. A radial pressure develops at the steel-concrete interface thereby restraining the concrete core and setting up a hoop tension in the tube. At this stage, the concrete core is stressed triaxially and the steel tube biaxially. Interaction between steel tube and concrete is the key issue to understand the behaviour of this kind of column. It is shown [8] that concrete compaction affects the properties of the core and also may influence the interaction between the steel tube and its concrete core, and thus influences the behaviour of the composite columns.

The original contribution lies mainly in the use of concrete, especially UHPC, in the form of slender composite elements for bridges rather than massive elements more commonly associated with concrete structures [9–11].

In this study stress analysis of the composite column and beam is performed with the purpose to obtain the maximum value of load-bearing capacity and enhance the safety of the structure using components with the appropriate strength properties and taking into account the composite action. The effect of UHPC on the stress state and load-carrying capacity of composite elements is analysed.

2. Analytical model for stress analysis

The section presents a consideration on behaviour of short straight concrete-filled steel column under axial loading. The term "short column" refers to a compression member that can attain its ultimate capacity without overall buckling. Disregarding the local effects at the ends, the stress state in the cross-section of the middle part of the column is analysed. A reinforced concrete core with radius R_0 is included in steel tube of thickness t (Fig 1). The reinforcement consists of symmetrically arranged longitudinal steel bars and circumferential reinforcing wire located at radius R_s from axis z.

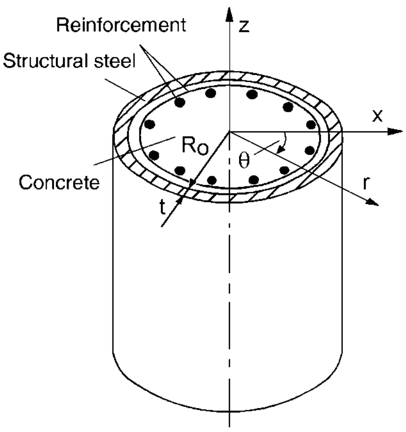

Fig 1. Concrete-filled hollow column with reinforcement

For radial displacements w and circumferential displacements u on the contact surface concrete-structural steel ($r = R_0$) and concrete-reinforcement surface ($r = R_s$) the following equalities are valid:

$$w^a = w^c; u^a = u^c; \quad (1)$$

$$w^s = w^c; \; u^s = u^c. \quad (2)$$

Here and below, the indices a, c and s refer to the steel tube, concrete and reinforcement. In the axial direction of the column, the cross-section area of the reinforcement is A_z^s, but in the circumferential direction - A_θ^s, placed with the given step. It is taken into account that the reinforcement is "spread" throughout the cross-sectional area and, as a result, an anisotropic column is formed.

The following relationships should be written assuming throughout the steel tube an in-plane membrane stressed state:

$$\frac{\partial \sigma_z^a}{\partial z} = 0 ; \quad (3)$$

$$\sigma_\theta^a = \frac{p_c R_o}{t} ; \quad (4)$$

$$\varepsilon_z^a = \frac{\partial w^a}{\partial z} = \text{const}; \; \varepsilon_\theta^a = \frac{u^a}{r} , \quad (5)$$

where p_c – the contact pressure on the boundary surface of steel-tube concrete, σ_z^a, σ_θ^a, ε_z^a and ε_θ^a – stresses and strains of the steel tube in the axial and circumferential direction. The existing equality of stresses in the radial and circumferential direction ($\sigma_r^c = \sigma_\theta^c$) in concrete core is assumed.

Taking into account the compatibility relationships (1) and (2), four equations can be written:

$$\frac{1}{E^a}\sigma_z^a - \frac{\nu^a}{E^a}\sigma_\theta^a = \frac{1}{E_z^c}\sigma_z^c - \frac{\nu_{zr}^c}{E_r^c}\sigma_r^c - \frac{\nu_{z\theta}^c}{E_\theta^c}\sigma_\theta^c ; \quad (6)$$

$$\frac{\nu^a}{E^a}\sigma_z^a - \frac{1}{E^a}\sigma_\theta^a = \frac{\nu_{zr}^c}{E_z^c}\sigma_z^c - \frac{1}{E_r^c}\sigma_r^c + \frac{\nu_{z\theta}^c}{E_\theta^c}\sigma_\theta^c ; \quad (7)$$

$$\frac{1}{E_z^s}\sigma_z^s = \frac{1}{E_z^c}\sigma_z^c - \frac{\nu_{zr}^c}{E_r^c}\sigma_r^c - \frac{\nu_{z\theta}^c}{E_\theta^c}\sigma_\theta^c ; \quad (8)$$

$$\frac{1}{E_\theta^s}\sigma_\theta^s = \frac{1}{E_\theta^c}\sigma_\theta^c - \frac{\nu_{r\theta}^c}{E_r^c}\sigma_r^c - \frac{\nu_{z\theta}^c}{E_z^c}\sigma_z^c , \quad (9)$$

were ν^a – is Poisson's ratio of the steel tube, ν_{ij}^c – Poisson's ratio values of the reinforced concrete ($i, j = r, z, \theta$; $i \neq j$). Equations (6) – (9) have been written taking into account the transverse deformation. The equilibrium equations in the z and θ directions are as follows

$$\frac{A_\theta^s}{R_s}\sigma_\theta^s + \frac{t}{R_0}\sigma_\theta^a + \sigma_\theta^c = 0 ; \quad (10)$$

$$A^c\sigma_z^c + A^a\sigma_z^a + A_z^s + q(A^a + A^c) = 0 , \quad (11)$$

where A^c and A^a – the cross-sectional area of the concrete and structural steel accordingly, E^a – modulus of elasticity of the steel tube, E_z^c, E_r^c and E_θ^c – moduli of elasticity of the reinforced anisotropic concrete core in the directions z, r and θ, E_z^s and E_θ^s – moduli of elasticity of the reinforcement in directions z and θ, q – a uniformly distributed axial load. It is assumed the load is applied to the entire section.

The load-carrying capacity of composite elements in bending can be characterised by plastic deformation of structural steel in tension, by cracking of concrete in compression or by both. At the same time, the stresses in reinforcement can be more or less of the yield limit of steel. The stress analysis of composite elements in bending has been performed with certain assumptions: 1) the hypothesis of flat sections has been used; 2) the concrete lying in the tension zone of the section is assumed to be cracked and is therefore neglected; 3) the conditions of deformation continuity of structural steel and concrete fulfil in the compression zone; 4) relationship for the stress distribution in the compression zone of concrete has been assumed in the following non-linear form:

$$\sigma^c{}_z = \sigma^c{}_f\,(z/x)^{n_c}\,, \qquad (12)$$

where $\sigma^c{}_z$ – stresses in concrete at distance z from neutral axis; $\sigma^c{}_f$ – stresses in concrete in more compressed fibre; x – depth of the compressed zone of concrete; n_c – characteristic of the stress diagram form. The cross-section of reinforced composite beam and the stress-strain distribution diagrams are shown in Fig 2.

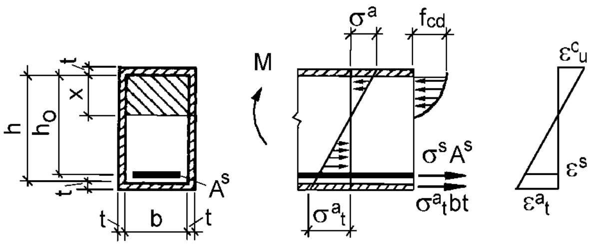

Fig 2. Stress and strain distribution in a composite beam

The characteristic n_c can be determined in the following way

$$n_c = 1 - \left(1 - \frac{E_u^c}{E^c}\right)\left(\frac{\sigma_f^c}{f_c}\right)^m, \qquad (13)$$

where experimental factor $m \cong 5{,}7$; f_c – concrete strength.

Based on the equilibrium conditions of the internal forces and taking into account the hypothesis of flat sections, the position of neutral axis can be determined by using non-linear equation:

$$\begin{aligned} & x^2\left[\frac{f_{cd}b}{(1+E^c{}_u/E^c)\varepsilon^c{}_u} - 2t(E^a + E^a{}_t)\right] + \\ & x\left[E^a(-bt+2ht-2t^2) + E^a{}_t(bt+4ht+2t^2) + E^sA^s\right] + \\ & E^a\left(hbt+2ht^2\right) - E^a{}_t\left(hbt+2h^2t+2ht^2\right) - \\ & E^sA^sh_0 = 0, \end{aligned} \qquad (14)$$

where $E^c{}_u$ and E^c – tangent and initial modulus of the concrete, $E^a{}_t$ – modulus of constructional steel in tension, $\varepsilon^c{}_u$ – ultimate deformation of concrete in compression. The ultimate bending moment M_u was determined by using the condition of equality of the moments in section of the beam

$$\begin{aligned} & M_u = b\int_0^x \sigma_f^c (z/x)^{n_c} z dz + \sigma^a btx + \\ & \frac{2}{3}\left[\sigma^a btx(x+t) + \sigma_t^a t(h-x)(h+t-x)\right] + \\ & \sigma^s A^s(h_0 - x) + \sigma_t^a bt(h-x). \end{aligned} \qquad (15)$$

where b, h and h_0 – width, height and effective height of concrete section.

3. Numerical solution and analysis

In order to perform the stress analysis in a composite column the system of six linear equations (6) – (11) was solved. In matrix form it is written as

$$[A]\,\mathbf{X} = \mathbf{B}. \qquad (16)$$

The components of the unknown vector **X** are stresses in the steel tube, concrete, longitudinal and spiral reinforcement, ie,

$$\mathbf{X} = \left[\sigma_z^a, \sigma_\theta^a, \sigma_z^c, \sigma_\theta^c, \sigma_z^s, \sigma_\theta^s\right]^{\mathrm{T}}. \qquad (17)$$

The matrix of system [A] and vector of constants **B** is determined from the stress-strain relationships and equilibrium equations as follows:

$$A = \begin{bmatrix} \frac{1}{E^a} & -\frac{\nu^a}{E^a} & -\frac{1}{E_z^c} & \frac{\nu_{zr}^c}{E_r^c} + \frac{\nu_{z\theta}^c}{E_\theta^c} & 0 & 0 \\ \frac{\nu^a}{E^a} & -\frac{1}{E^a} & -\frac{\nu_{rz}^c}{E_z^c} & \frac{1}{E_r^c} - \frac{\nu_{r\theta}^c}{E_\theta^c} & 0 & 0 \\ A^a & 0 & A^c & 0 & A_z^s & 0 \\ 0 & 0 & \frac{1}{E_z^c} & -\left(\frac{\nu_{zr}^c}{E_r^c} + \frac{\nu_{r\theta}^c}{E_\theta^c}\right) & -\frac{1}{E_z^s} & 0 \\ 0 & \frac{h}{R_0} & 0 & 1 & 0 & \frac{A_\theta^s}{R_s} \\ 0 & 0 & -\frac{\nu_{z\theta}^c}{E_z^c} & -\frac{\nu_{r\theta}^c}{E_r^c} + \frac{1}{E_\theta^c} & 0 & -\frac{1}{E_\theta^s} \end{bmatrix}; \qquad (18)$$

$$\mathbf{B} = \left[0, 0, -q(A^a + A^s), 0, 0, 0\right]^{\mathrm{T}}. \qquad (19)$$

The following geometrical parameters were chosen for the analysis: diameter of circular hollow section $d = 40{,}6$ cm, thickness of the steel tube section $t = 0{,}88$ cm, reinforcement content μ = 4 % of the concrete section, distance from the central axis to the reinforcement bar axis R_s = 15,5 cm, cross-sectional area of the longitudinal reinforcement A_z^s = 49 cm^2 and secondary reinforcement A_θ^s = 0,1 cm^2 (per 1 cm of the column length). The modulus of elasticity for the reinforcement is E^s = 200 000 MPa, for structural steel – E^a = 210 000 MPa and Poisson's ratio ν^a = 0,25.

Concrete is a heterogeneous material because of its composite structure. Nevertheless, concrete is assumed homogeneous and the mechanical behaviour can be expressed in the form of stress-strain relationship. This relation is non-linear. However, with increasing compressive strength, the initial slope corresponding to the modulus of elasticity increases and the linear part extends to higher stress levels. Furthermore, the ultimate strain, ie the strain at maximum stress, increases with increasing compressive strength [9].

In order to determine real mechanical characteristics of concrete under compression stresses, prisms were tested for uniaxial compression. The development of axial strain Σ_a and lateral strain ε_l fixed in the test is shown in Fig 3. In the first part of the loading phase, the stress-strain relation shows an almost linear response. Due to the concrete heterogeneity, a uniform uniaxial stress applied to a concrete specimen obviously results in local non-uniformity. At a stress, corresponding approximately to 40 % of the peak stress due to the difference in lateral deformation of material constituents, the stress-strain relation exhibits a visible non-linear response. Approximately at 85 % of the peak stress, the response is marked non-linear.

In the initial stage of composite column loading, the Poisson's ratio of concrete core is lower than that of steel. Therefore, the steel tube expands faster in the radial direction compared with the concrete core; hence, the steel tube does not restrain the concrete core. As a result, columns centric loaded on the entire section are affected by the difference between the values of Poisson's ratio of the steel tube, ν^a , and the concrete core, ν^c.

In analysis the initial tangent modulus and tangent modulus at the given stress level have been used as the modulus of elasticity of the concrete. Based on experimental results, the Poisson's ratio of the concrete changes in accordance to the stress level, starting with ν^c = 0,15 in the initial stage [12]. The moduli of elasticity of a reinforced anisotropic concrete core E_z^c, E_r^c, and E_θ^c as well as Poisson's ratio values ν_{ij}^c were found on the basis of the reinforcement theory [13] taking into account the content of the reinforcement μ and mechanical properties of the constituents.

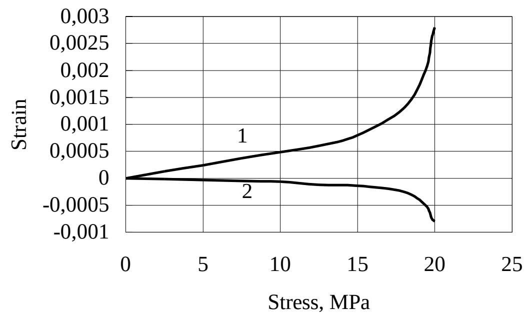

Fig 3. Stress-strain relation for concrete prism in uniaxial compression: 1 – Σ_a; 2 – Σ_l

The stresses in material components with applied force N for conventional concrete and UHPC are given in Table 1. In the case of conventional concrete, stresses in steel section, concrete and reinforcement exceed the strength of materials. By using UHPC as column filling the load-carrying capacity keeps enough and stress level is satisfactory.

According to EC4, for the concrete-filled circular hollow sections, the load-bearing capacity of the concrete is increased due to the prevention of transverse strain. This effect is shown in Fig 4. The transverse compression of the concrete (σ_r^c) at high stress levels leads to three-dimensional effects, which promotes the increase of the column resistance. At the same time, the circular tensile stresses (σ_θ^c) arise on the cylindrical surfaces reducing its normal stress capacity.

Note that when axial stress in concrete is $\sigma_z^c \cong 0{,}85\ f_{ck}$, the Poisson's ratio of concrete ν^c increases and becomes higher than that of the steel. In the region before fracture, the Poisson's ratio is approximately 0,3 [12]. Circular tensile stresses σ_θ^c in the concrete core of the column change into compression stresses due to the steel tube behaviour and the above-mentioned features of concrete deformation process.

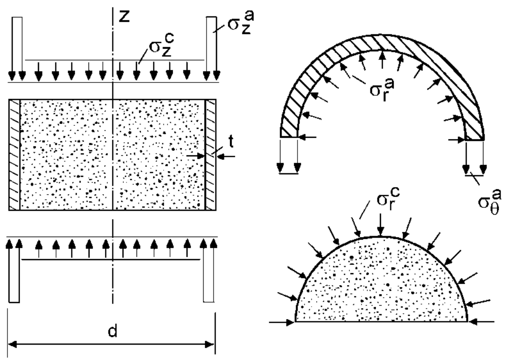

Fig 4. Stress distribution in the section of column

According to [2], the plastic resistance of the cross-section of a composite column with axial loading is given as the sum of the components:

$$N_{pl.Rd} = A^a f_{yd} + A^c f_{cd} + A_z^s f_{sd}\ , \qquad (20)$$

where A^a, A^c and A_z^s are the cross-sectional areas of the structural steel, concrete and reinforcement in the axial direction, and f_{yd}, f_{cd} and f_{sd} are design strength values of the materials mentioned above.

Table 1. Stress values (MPa) in constituents of composite column for ratio d/t = 44

Strength of concrete, MPa	Axial force N, kN	σ_z^a	σ_θ^a	σ_z^c	σ_z^c	σ_z^s	σ_θ^s
30	3200	– 332	– 4,3	– 47	– 0,1	– 316	94
180	3200	– 255	– 2,0	– 58	– 0,2	– 243	73

By using the above-mentioned geometrical and mechanical properties of materials, steel grade Fe275 with characteristic yield strength f_y = 275 MPa, concrete of class C30/37 and relative slenderness ratio of column 0,15 the load-bearing capacity is $N_{pl.Rd}$ = 7162 kN. Taking into account the effect of confinement for circular hollow section, the load-bearing capacity increases and becomes $N_{pl.Rd}$ = 7935 kN. Here the analysis is performed using the secant modulus of concrete according to the strength class of the concrete. According to the results of stress analysis, the axial stresses in composite column core in case of conventional concrete are $\sigma_z^c > f_{ck}$, where f_{ck} is the characteristic strength of concrete for the given strength class.

The analysis of the load-bearing capacity of the composite column is performed taking into account the design strength of steel and concrete as well as limit ratios of the circular hollow sections. Fig 5 shows that the first load limiting factors are concrete design strength f_{cd} and diameter thickness ratio *d/t*. Using concrete of the strength class C35/45 and steel of grade Fe235 the load-bearing capacity of the composite column increases by 18 % in comparison with concrete class C30/37. For a thin wall hollow section (*d/t* = 90) instead of section with *d/t* = 46 the steel economy is 50 %.

The results of the stress analysis of the composite beam are shown in Fig 6. By using non-linear approach the position of neutral axis of reinforced concrete element depends on ultimate deformation of concrete in compression ε^c_u. Taking into account this peculiarity, the stresses in structural steel σ^a in tension zone approximately are equal to the yield limit f_y (steel grade Fe235) when $\varepsilon^c_u < 1\cdot 10^{-3}$. The ultimate bending moment M_u in this case is by 40 % less than plastic moment $M_{pl.Rd}$ determined according to EC4. The comparison of composite beam load-bearing capacity in the case of conventional concrete and UHPC depending on ultimate compressive strain of concrete and compressive depth is given in Table 2.

Concrete filling leads to the increase of the load-bearing capacity that is much higher than that of steel elements and promotes the fire resistance as well. The concrete is contained within a circular steel profile and cannot split away, even if the ultimate strength of concrete is reached. Nevertheless, in order to prevent the possibility of a failure, especially, in the case of small

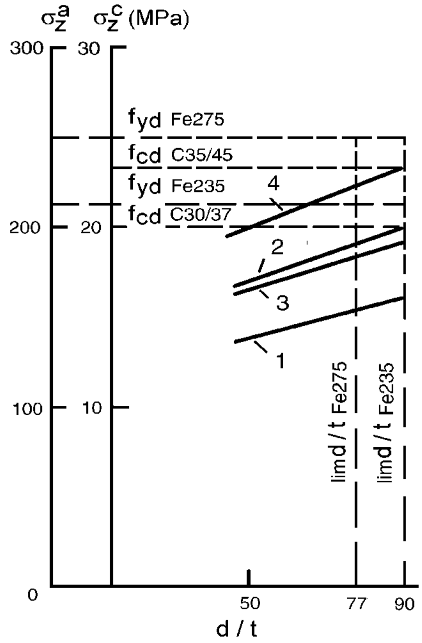

Fig 5. Variation of stresses in structural steel and concrete in relationship of ratio *d/t*: 1, 2 – σ_z^a and σ_z^c for axial force N = 4000 kN; 3, 4 – σ_z^a and σ_z^c for N = 5000 kN

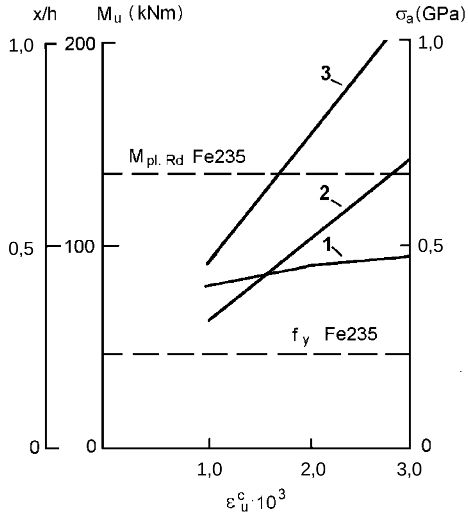

Fig 6. Influence of ultimate concrete deformation in compression on the position of neutral axis, stresses in structural steel and ultimate bending moment: 1 – *x/h*, 2 – σ^a, 3 – M_u

Table 2. Comparison of composite beams with conventional concrete and UHPC

Ultimate concrete deformation, $\sum_{cu}\cdot 10^3$	Compressive strength of concrete, MPa			
	30		180	
	Relative compressive depth, *x*/*h*	Ultimate moment M_u, kNm	Relative compressive depth, *x*/*h*	Ultimate moment M_u, kNm
1	0,42	92,7	0,27	186,8
2	0,47	155,4	0,34	271,8
3	0,49	248,0	0,39	383,4

thickness of the structural steel and fire, the appropriate strength classes of concrete and steel have to be used. The effect depends on the type of concrete because the strength of core affects the point at which confinement can take place.

4. Conclusions

1. On the basis of constitutive relationships for material components, the stress state in a composite column is determined, taking into account the dependence of the modulus of elasticity and Poisson's ratio on the stress level in the concrete. It has been proved that the effect of confinement acts at a high stress level when the structural steel behaves in tension and the concrete in compression. The main load limiting factors are concrete design strength f_{cd} and d/t ratio. In the case of higher concrete strength class and steel grade Fe235 the load-bearing capacity of the composite column increases by 18 %. In the case of a thin hollow section (d/t = 90), the steel economy is for 50 %.

2. By using non-linear approach, the position of neutral axis of composite beam depends on the ultimate deformation of concrete in compression ε^c_u. When $\varepsilon^c_u < 1 \cdot 10^{-3}$ the ultimate bending moment M_u is 40 % less than plastic moment $M_{pl.Rd}$ determined according to EC4 and the stresses in structural steel σ^a_t in tension zone are equal to yield limit f_y (steel grade Fe235).

3. The optimization of working conditions and cross-section area of the composite structure as well as the prevention of the possibility of a failure in the case of small thickness of structural steel and fire can be realized by using appropriate strength of concrete and steel. By using UHPC as steel element filling the increase of load carrying capacity can be significant.

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