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Tubular composite beam-columns of annular cross -sections and their design practice

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Abstract. The expediency of using tubular composite steel and concrete columns of annular cross-sections in construction is discussed. The new type space framework with tubular composite columns of multi-storey buildings and its rigid beam-column joints are demonstrated. The features of interaction between the circular steel tube and spun concrete stress-strain states during the concentrical and eccentrical loading of tubular composite members are considered. The modeling of the bearing capacity of beam-columns of composite annular cross-sections is based on the concepts of bending with a concentrical force and compression with a bending moment. The comparison of modeling results for the composite cross-sections of beam-columns is analysed. The expediency of using these concepts for the limit state verification of beam-columns in the methods of the partial safety factors design (PSFD) legitimated in Europe and the load and resistance factors design (LRFD) used in other countries is presented and illustrated by a numerical example.

Keywords: composite structures; composite frame; composite annular cross-section; beam-column; eccentric compression.

1. Introduction

Concrete-filled steel tubes are widely applied in construction practice. The research results obtained by Kvedaras, *et al*. (1983, 1996, 1998 and 2005) illustrated the great structural efficiency of tubular composite steel and concrete members of annular cross-sections. Circular steel tubes with spun concrete cores, characterized by self-regulating resistance, are economically and structurally rational composite columns for buildings. These columns of reduced weight display large energy absorption against lateral loads. The increased ductility of slender steel tubes of annular cross-sections may be obtained using not only concrete-filled double skin circular hollow sections as it recommended by Zhao, *et al*. (2002) but also annular spun concrete cores. However, the structural performance and reliability of a new type of hollow concrete-filled circular steel tubular members have not been sufficiently investigated.

Eurocode 4 (EN 1994-1-1) presents specific design directions and recommendations only for the members the circular steel tubes that are supplied with solid concrete cores. The examination of the design compressive resistance of composite columns under eccentric loads in the USA is based on the ACI Committee 318 (2005) and AISC (2005) recommendations and is presented also only for steel tubes fully

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filled with vibrated concrete.

The initial experimental data on slender differently loaded structures show their greater efficiency than that on stub members, although such results are in some contradiction with the limitation of Eurocode 4 (EN 1994-1-1) allowance. Such and other code features come to light in the design stage and safety assessment processes of new type composite structures. A lack of experimental and theoretical research results hampers the development of limit state analysis methods of tubular composite members. In some cases it can lead to groundless overestimation or underestimation of the reliability of designed and executed structures. Therefore, it is expedient to analyze their robustness and bearing capacity both by semi-probabilistic ultimate limit state and probabilistic safety methods.

The object of this paper is to enable structural engineers to assess the bearing capacity of tubular composite beam-columns of annular cross-sections using unsophisticated approaches and procedures of the limit state analysis based on the partial safety factors design (PSFD) and the load and resistance factors design (LRFD) used in Europe and the USA, respectively.

2. Composite frames with tubular columns of annular cross-sections

A new type space framework consisting of composite load bearing structures is designed and realized in multi-storey office construction. The presented frameworks allow complicated outline and layout, and a different number of storey and levels of floors of the buildings (Fig. 1).

In many cases, the composite members of hollow steel sections with spun concrete cores are more rational structurally and economically (Kvedaras, *et al*. 1996, 1998, 2005). The pre-cast composite columns of analyzed frameworks were designed and erected as hollow concrete-filled circular steel tubes that external diameter depends on column length and compressive force intensity. The thickness of steel tube and spun concrete core walls depends on the intensity of action effects of the column.

A space framework consists of perpendicular frames of two types. The headers of the main frames are designed as composite structural beams consisting of built-up twin-channel sections reinforced with inserted cages of steel reinforcement and concrete filled after the erection of floor slabs. The pre-cast multi-core concrete floor slabs are 5,860 mm in length and from 1,190 to 1,590 mm in width. The channel sections as continuous steel beams are capable of bearing the mass of pre-cast reinforced concrete floor slabs and site (erection) loads. The detailing of the composite girder-column joints is presented in Fig. 2.

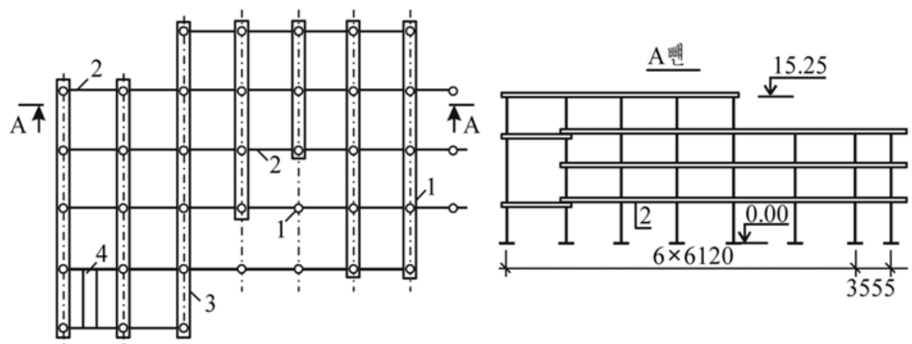

Fig. 1 The layout of structures and the elevation drawing of a space framework: 1-composite column, 2-composite beam, 3-cast-in-situ reinforced concrete slab, 4-pre-cast concrete slab

The cast-in-situ bracing reinforced concrete slabs as frame beams (headers) of the second-row frames are concreted together with the headers of the main frames. Therefore, the beams of the main and secondary frames are connected rigidly.

As one of the main merits of the presented frame headers is their increased resistance to longitudinal shear and lateral-torsion buckling. The longitudinal reinforcing bars in the upper and lower zones of the composite header withstand the bending moments caused by additional permanent loads, live floor or snow and wind velocity pressure actions. The composite steel and concrete header satisfies the basic design requirements for the ultimate limit state given in Eurocode 4 (EN 1994-1-1). However, the design course of buildings has convinced us that the rules of action combinations recommended by this code do not allow assessing the specific behaviour of composite structures during the execution and service periods of buildings. Therefore, these rules should be perfected.

The space framework of buildings is designed and constructed in such a way that, with appropriate degrees of reliability, it may withstand all actions and satisfy the relevant design requirements given in Eurocode 2 (EN 1992-1-1) and Eurocode 3 (EN 1993-1-1), dismissing the effects of shrinkage and creep of concrete.

It is of most importance that the steel surfaces of composite columns and headers allow one to repair and strengthen the framework quite easily and simply. The total savings of materials and manpower coming to approximately 25% substantiate the rationality of the structural members of recommended frameworks. Besides, the frameworks as sustainable structural systems allow one executing construction work at a rapid pace.

3. The performance of annular composite cross-sections

The self-regulating resistance property is characteristic of concrete-filled steel tubular members. The

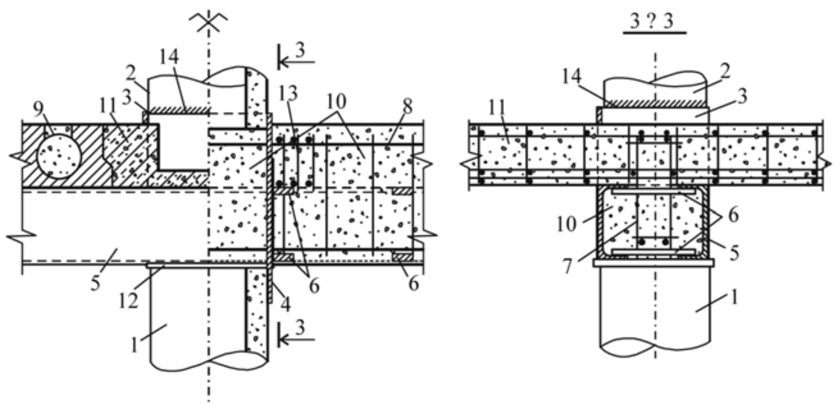

Fig. 2 The scheme of the rigid connection of a hollow concrete-filled circular steel tubular column and a continuous composite beam: 1-lower composite column; 2-upper composite column; 3-erection ring for upper column; 4-joining the external steel shells of lower and upper columns by welded reinforcing bars; 5-steel twin-channels of composite beam; 6-batten-plates of beams as partial shear connectors of concrete core; 7-cages of steel reinforcement of beams; 8-additional longitudinal steel reinforcing bars going through the connection and joining cages of beams; 9-pre-cast multi-core concrete slab; 10-cast-in-situ concrete; 11-cast-in-situ bracing concrete slabs; 12-steel head-plate of hollow concrete-filled column; 13-re-bar cage of bracing slabs; 14-site weld

interaction between a steel tube and a spun concrete core at their interface occurring during concentric compression causes an increase in the compressive strength of both components and in the robustness of whole such composite members (Fig. 3). However, the theoretical and experimental description of the interaction between these components is under discussion. Usually (an approach of) this description is based on the postulates of the mathematical theory of elasticity and of the theory of plasticity of small elastic-plastic deformations and it takes into account the different values of Poisson's ratio of component materials. A homologous definition of the strain criteria allows an exact definition of the robustness of concrete-filled steel tubular members evaluating their increase against the criteria determined by superposition of the resistances of these composite member components.

The structural behaviour of hollow concrete-filled circular steel tubular members under concentric compression is more complicated than that of composite members with solid concrete cores. The resistance analysis of an cross-section members may be based on the postulates of the theory of plasticity. From the generalized Hooke's law, the normal ultimate stresses of both media – external steel shell and internal concrete coreunder either concentric compression or tension have to be expressed by the formula:

$$\sigma_x = (4/3)E_{im}(\varepsilon_{x,y} + \nu\varepsilon_z) \tag{1}$$

where: ν – is Poisson's ratio, different values of which are observed for steel and concrete at different stages of loading; at ultimate state $\nu = 0.5$ for materials of both components of composite concrete-filled steel tubular members; E_{im} is the secant modulus of elasticity of the corresponding media; $\varepsilon_{xy} = 0.5(\varepsilon_z + \sqrt{3(\varepsilon_{iy}^2 - \varepsilon_z^2)})$ and $\varepsilon_z = (\sigma_z - 0.5\sigma_x)/E_a$ are the values of longitudinal and tangential strains of the external steel shell, respectively (Kvedaras 1988), where σ_z is the radial stress of both media (external steel shell and internal concrete core) interaction at their interface which in case of the annular concrete core is expressed as:

$$\sigma_z = 2(f_{cm}E_a - f_yE_c)/\{E_a(2\beta_i - 1)/(\beta_i - 1) - E_c\} \tag{2}$$

with $\beta_i = d_{ce}/d_{ci}$ taken as relational thickness of the annular concrete core.

Eq. (1) is not to be used to determine the core concrete strength in tension because of its comparatively small value in the strength analysis of composite steel and concrete members which by Eurocode 4 (EN 1994-1-1) is usually taken as negligible.

Because of the assumed equality of biaxial stress state in a steel shell and a concrete core (Kvedaras 1988), the ultimate value of the generalized strain $\varepsilon_{iy} = 1.5f_y/E_a$ will be the same for both materials. Thus,

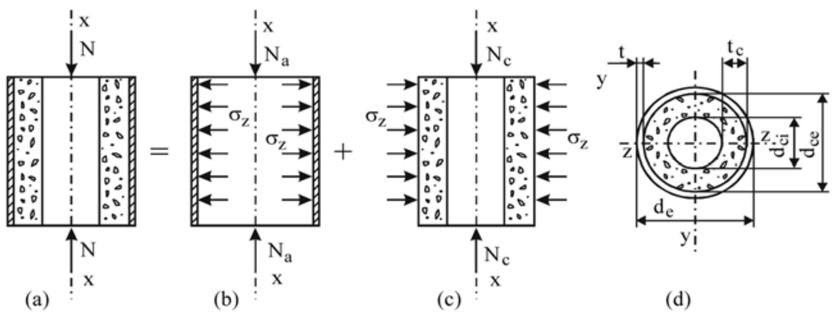

Fig. 3 Diagrammatic sketch of concentric actions on the hollow composite member (a), steel tube (b) and hollow concrete core (c); shape and dimensions of annular cross-section (d)

the ultimate normal stresses of steel σ_{ax} in tension and compression and of concrete σ_{cx} in compression defined from Eq. (1) represent the modified values of the steel and concrete strengths as follows:

$$f_a = \sigma_{ax} = \eta_{am} f_y \text{ and } f_c' = (\sigma_{cx} + f_c)/2 = \eta_{cm} f_c \tag{3}$$

where η_{am} and η_{cm} are the constraining factors as random variable values characterizing the interaction effect of the components of a composite member on its load-bearing capacity under compression; f_y and f_c are the values of steel yield and concrete specified compressive strengths, respectively.

The values of f_y and $\eta_a f_y$ on the yield strength of mild steel are presented in Fig. 4. When the thickness of a steel tube is less than its ultimate minimum, the critical steel strength f_{cr} has to be used instead of f_y. The value of the constraining factor for annual core concrete η_c is obtained taking into account the unequal distribution of the normal stresses through the wall thickness of the annular cross-section of the concrete core with its maximum value on the interface with the steel shell and the minimum one at the internal surface of its cavity. One has also to evaluate the necessary jump from the 3D and 2D stress states to the 2D and 1D stress states in the annular cross-section of the concrete core in the strength analysis due to the postulates of the theory of plasticity applied for a concrete-filled tubular steel member with components from materials with different Poisson's ratios was evaluated. The stress-strain relationships of concrete under different deformation laws are presented on Fig. 4(b).

Composite columns may be treated as concentrically compressive members for the eccentricity of applied force according to first-order theory does not exceeding 0.1 d, where d is the external diameter of a steel tube. The ultimate resistance of investigated stub composite members under concentric compression may be introduced by the expression

$$N_R = \kappa(\sigma_{ax}A_a + \sigma_{cx}A_c) = \kappa(f_a A_a + f_c' A_c) = \kappa(\eta_a f_y A_a + \eta_c f_c A_c) \tag{4}$$

where κ is the reduction factor for the flexural buckling mode (EN 1994-1-1); A_a and A_c are the cross-sectional areas of the steel shell and concrete annular cores, respectively. Thus, the resistance to compression of composite members should be calculated by adding the plastic resistances of their steel and concrete components.

According to the experimental investigations (Tables 1 and 2) carried out in the Department of Steel and Timber Structures of Vilnius Gediminas Technical University, the mean and standard deviation of the ratio

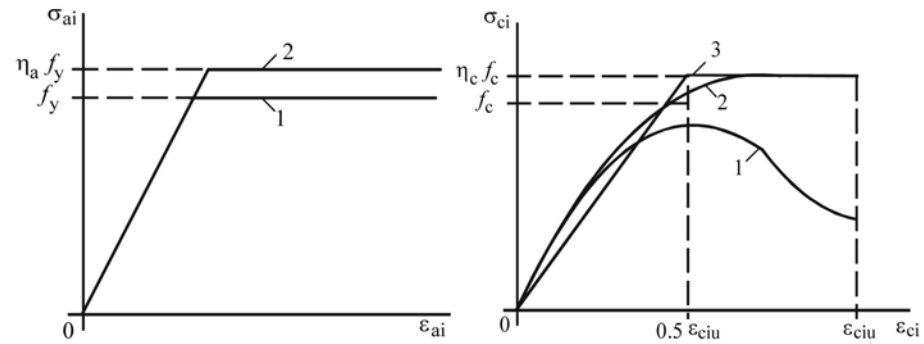

Fig. 4 Stress-strain relationship of the steel tube (a) and a concrete core (b) under unblocked (1), blocked typical (2) and bilinear blocked (3) deformations

of trial destroying forces and calculated resistances by using Eq.(4) for stub concentrically loaded members are: $\theta_{Rm} = N_{R,tr}/N_{R,m} = 1.005 \approx 1.0$ (Fig. 5(a)) and $\sigma\theta_R = 0.0485 \approx 0.05$. These values may be treated as the statistical parameters of additional random variables representing resistance model

Table 1 Specimen labels and material properties

No.	$d_e \times t_a \times t_c \times l$ [mm]	A_c [cm^2]	A_a [cm^2]	f_y [MPa]	f_c' [MPa]
1	218.0 × 4.5 × 32.5 × 500	180	30.2	296	37.9
2	219.0 × 4.5 × 29.5 × 500	165	30.3	282	49.0
3	217.5 × 4.5 × 32.5 × 500	180	30.2	273	47.0
4	219.0 × 4.5 × 30.5 × 500	172	30.3	282	48.8
5	219.0 × 4.5 × 30.5 × 500	172	30.3	282	41.0
6	218.0 × 4.5 × 24.5 × 500	142	30.2	296	26.7
7	159.8 × 4.85 × 22.0 × 400	85.4	23.6	349	20.8
8	159.8 × 4.85 × 31.0 × 400	116	23.6	349	40.8
9	159.8 × 4.85 × 30.0 × 400	113	23.6	349	41.1
10	159.8 × 4.85 × 29.0 × 400	110	23.6	349	39.0
11	159.8 × 4.85 × 28.5 × 400	108	23.6	349	31.6
12	159.8 × 4.85 × 31.5 × 400	118	23.6	349	30.0
13	219.0 × 4.5 × 29.0 × 500	168	30.2	286	37.4
14	219.0 × 4.5 × 30.5 × 500	172	30.2	286	23.9
15	219.0 × 4.5 × 31.0 × 500	174	30.2	279	38.8
16	219.0 × 4.5 × 30.5 × 500	172	30.2	279	37.1
17	218.0 × 4.5 × 26.5 × 500	152	30.2	296	32.0
18	217.8 × 4.5 × 30.5 × 500	171	30.1	273	22.0
19	219.0 × 4.5 × 29.5 × 500	165	30.2	282	40.0
20	152.0 × 4.8 × 29.9 × 400	106	22.2	287	33.7
21	152.0 × 4.8 × 32.0 × 400	111	22.2	287	33.7
22	152.0 × 4.8 × 30.7 × 400	108	22.2	287	38.7
23	152.0 × 4.8 × 30.8 × 400	108	22.2	287	38.7
24	152.0 × 4.8 × 29.2 × 400	104	22.2	287	38.7
25	152.0 × 4.8 × 29.1 × 400	103	22.2	287	38.7
26	152.0 × 4.8 × 30.2 × 400	106	22.2	287	38.7
27	152.0 × 4.8 × 29.3 × 400	105	22.2	287	38.7
28	152.0 × 4.8 × 29.2 × 400	104	22.2	287	38.7
29	152.0 × 4.8 × 29.5 × 400	105	22.2	287	38.7
30	152.0 × 4.8 × 29.8 × 400	105	22.2	287	33.7
31	152.0 × 4.8 × 31.2 × 400	109	22.2	287	33.7
32	152.0 × 4.8 × 31.6 × 400	110	22.2	287	40.5
33	152.0 × 4.8 × 32.6 × 400	113	22.2	287	40.5
34	152.0 × 4.8 × 32.2 × 400	112	22.2	287	38.7
35	152.0 × 4.8 × 32.7 × 400	113	22.2	287	38.7
36	152.0 × 4.8 × 32.4 × 400	112	22.2	287	38.7
37	152.0 × 4.8 × 31.2 × 400	109	22.2	287	38.7
38	152.0 × 4.8 × 29.5 × 400	104	22.2	287	38.7
39	152.0 × 4.8 × 30.0 × 400	106	22.2	287	38.7
40	152.0 × 4.8 × 29.2 × 400	104	22.2	287	38.7
41	152.0 × 4.8 × 29.5 × 400	105	22.2	287	38.7
42	152.0 × 4.8 × 30.4 × 400	106	22.2	287	38.7
43	152.0 × 4.8 × 32.2 × 400	112	22.2	287	38.7

Table 2 Specimen labels, section capacities, ratio of the trial and calculated resistances and efficiency factor

No.	ξ	$N_{R,tr}$ [kN]	N_{Rm} [kN]	$N_{R,tr}$ / N_{Rm}	K_{ef}
1	1.310	1860	1857	1.0016	1.180
2	1.060	2000	1982	1.0091	1.194
3	0.975	2000	1990	1.0005	1.198
4	1.018	2022	2019	1.0015	1.196
5	1.212	1850	1845	1.0027	1.186
6	2.358	1460	1457	1.0020	1.148
7	4.637	1130	1125	1.0044	1.121
8	1.740	1510	1506	1.0027	1.162
9	1.773	1500	1495	1.0033	1.163
10	1.920	1450	1448	1.0014	1.158
11	2.413	1340	1330	1.0075	1.145
12	2.327	1350	1347	1.0022	1.149
13	1.375	1750	1753	0.9983	1.177
14	2.101	1485	1477	1.0055	1.166
15	1.248	1800	1793	1.0039	1.175
16	1.296	1750	1744	1.0034	1.101
17	1.838	1600	1598	1.0013	1.161
18	2.184	1380	1376	1.0029	1.151
19	1.290	1800	1783	1.0095	1.182
20	1.784	1020	1152	0.8854	1.027
21	1.703	1080	1176	0.9184	1.060
22	1.524	1291	1232	1.0479	1.226
23	1.524	1250	1233	1.0138	1.180
24	1.583	1299	1251	1.0384	1.250
25	1.600	1225	1211	1.0116	1.182
26	1.553	1241	1226	1.0122	1.183
27	1.568	1233	1213	1.0165	1.184
28	1.583	1315	1212	1.0850	1.259
29	1.568	1285	1239	1.0371	1.208
30	1.800	1160	1149	1.0096	1.170
31	1.510	992	1167	0.8500	0.991
32	1.430	1220	1275	0.9568	1.127
33	1.393	1210	1286	0.9411	1.078
34	1.470	1225	1254	0.9770	1.146
35	1.457	1282	1259	1.0183	1.152
36	1.470	1225	1254	0.9770	1.179
37	1.510	1270	1239	1.0254	1.199
38	1.583	1192	1213	0.9827	1.148
39	1.553	1220	1224	0.9971	1.163
40	1.583	1217	1209	1.0067	1.177
41	1.553	1425	1385	1.0286	1.343
42	1.553	1258	1223	1.0286	1.198
43	1.470	1357	1253	1.0823	1.259

Mean value: 1.00026

uncertainty. The rationality of tubular composite members under concentric compression demonstrates the efficiency factor

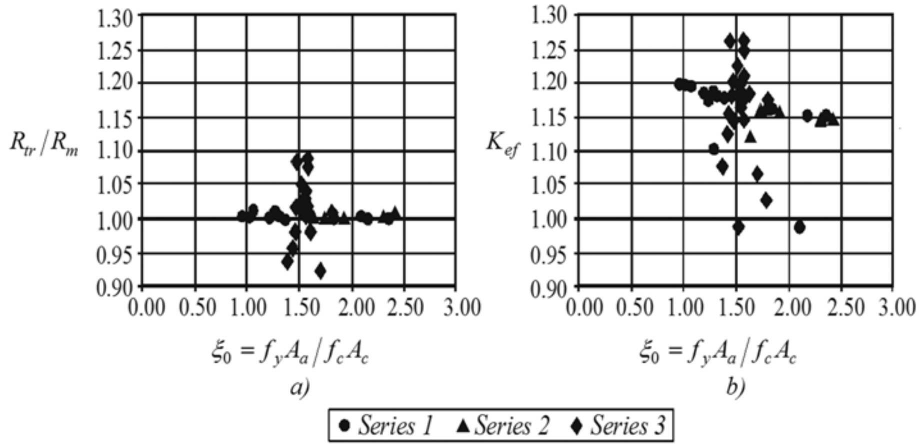

Fig. 5 Ratio R_{tr} / R_m (a) and efficiency factor K_{ef} (b) versus confinement factor $\xi_0 = f_y A_a / f_c' A_c$

$$K_{ef} = N_{R,tr} / (f_y A_a + f_c' A_c) \tag{5}$$

the mean value of which was equal to 1.17 (Fig. 5(b)).

The mean values of constraining factors $\eta_{cm} = 1.07$ and $\eta_{cm} = 1.32$ have slightly differed for compression and bending tubular members (Kvedaras 1999).

4. Modeling of the bearing capacity of beam-columns

4.1. Bending with a concentrical force

The modeling of the stress-strain state and bearing capacity of beam-columns as eccentrically loaded tubular composite members must assess the structural and mechanical features of annular steel tube and concrete cross-sections including their self-regulating strengths. A perfect bond between the annular spun concrete core and steel tube surfaces of bending tubular composite members of annular cross-sections (Kvedaras 1999) allows modeling the possible plastic stress distributions of steel and concrete strengths as it is shown in Fig. 6.

Vadlūga (1983) has demonstrated that the ultimate bending moment of eccentrically loaded annular concrete cross-sections reinforced by mild steel bars may be calculated by the expression:

$$M_R = 1.2 r_s (f_y A_s + N_R)(1 - a) \tag{6}$$

Here r_s is the radius of the reinforcement circle; N_R represents the ultimate concentric compressive force of these cross-sections;

$$\alpha = \psi / \pi = (f_y A_s + N_R)/(2 f_y A_s + f_c A_c) \tag{7}$$

is the conventional value of the compression zone area of cross-sections, where ψ is an angle of this zone, A_s and A_c are cross-sectional areas of steel and concrete, respectively.

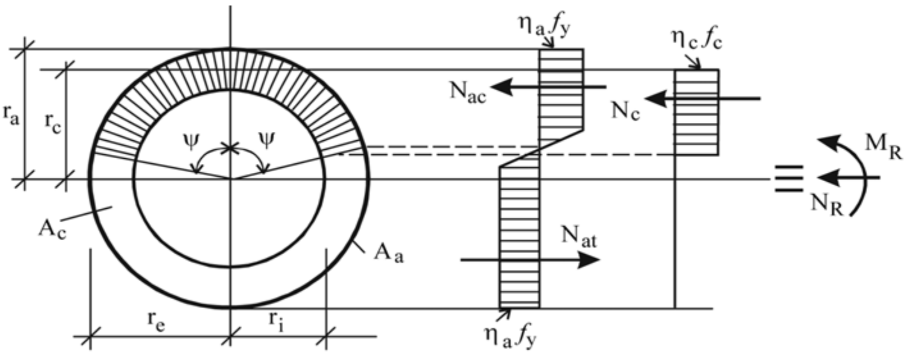

Fig. 6 Modeling of stresses in the steel tube and concrete cross-sections under bending with a concentric force

For the analysis of the bearing capacity of tubular composite steel and concrete columns, Eqs. (6) and (7) may be rewritten as follows:

$$M_R = 1.2r_a(\eta_a f_y A_a + N_R)(1-\alpha) \tag{8}$$

$$\alpha = (\eta_a f_y A_a + N_R)/\lfloor 2\eta_a f_y A_a + 0.5(1 + r_c/r_a)\eta_c f_c A_c \rfloor \tag{9}$$

where η_a and η_c are the constraining factors for steel and concrete interaction effect; r_a and r_c are the radii of the average circles of annular cross-sections of steel tube and concrete, respectively.

An analysis of Eq. (8) revealed its universality (see Fig. 10). When the compressive force $N = N_R = 0$, Eqs. (8) and (9) express the bearing capacity of tubular composite members of annular cross-sections subjected to pure bending. However, when the eccentricity of applied total compressive force of beam-columns $e < 2r_a$, in design practice it should be more expedient to treat tubular composite beam-columns as members exposed to compression with a bending moment.

4.2. Compression with a bending moment

The analysis of ultimate load effects of beam-columns may be based on a plane cross-section hypothesis and bi-linear steel and concrete stress-strain relations (Fig.4) when the tensile strength of concrete is ignored and the ultimate compressive concrete strain $\varepsilon_{ciu} = \varepsilon_{cu} = 3.5 \cdot 10^{-3}$. This value is close to the ultimate concrete strain of beam-columns recommended by Eurocode 2 (EN 1992-1-1) and by Hussaini, *et al*. (1993). Analogically to the assumptions of mechanical models recommended by Han and Yao (2004) and by Oyawa (2004) for the analysis of beam-columns and beams, respectively, the contribution of concrete tensile strength and the effect of shear forces on stress diagrams of materials may be neglected. According to the design model presented in Fig. 7, the conventional ultimate normal steel stress in the compression zone of annular cross sections may be expressed as:

$$\sigma_{ac} = \varepsilon_{au}E_a = \varepsilon_{cu}[1 + (r_a/r_c - 1)/(1 - \cos\psi)]E_a \tag{10}$$

where r_a and r_c are the radii of middle circles of steel tube and concrete cross-sections.

The intermediate angles of annular composite cross-sections characterizing the positions of design

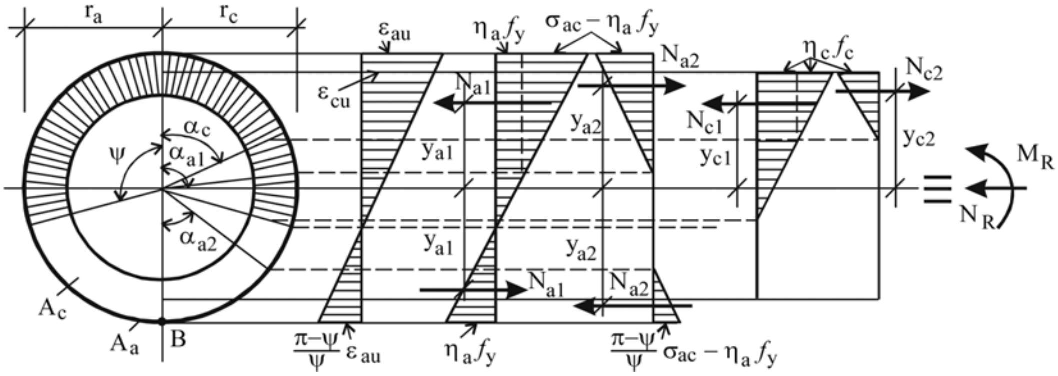

Fig. 7 Modeling of linear strain and stress distributions in the steel tube and concrete cross-sections

inelastic strains of steel (angles α_{a1} and α_{a2}) and concrete (angle α_c) may be calculated from the equations:

$$\cos\alpha_{a1} = 1-(1-\cos\psi)(1-\eta_a f_y/\sigma_{ac}) \tag{11}$$

$$\cos\alpha_{a2} = 1-(1+\cos\psi)\left[1-\eta_a f_y/\left(\frac{\pi-\psi}{\psi}\sigma_{ac}\right)\right] \tag{12}$$

$$\cos\alpha_c = 0.5(1+\cos\psi)$$

where the steel tube stress σ_{ac} is given by Eq. (10).

The internal forces and their distances from the cross-section centre point of beam-columns are expressed as:

$$N_{a1} = \sigma_{ac}A_a(\sin\psi-\psi\cos\psi)/[\pi(1-\cos\psi)] \tag{13}$$

$$y_{a1} = r_a(\psi-\sin\psi\cos\psi)/[2(\sin\psi-\psi\cos\psi)] \tag{14}$$

$$N_{a2} = (\sigma_{ac}-\eta_a f_y)A_a(\sin\alpha_{a1}-\alpha_{a1}\cos\alpha_{a1})/[\pi(1-\cos\alpha_{a1})] \tag{15}$$

$$y_{a2} = r_a(\alpha_{a1}-\sin\alpha_{a1}\cos\alpha_{a1})/[2(\sin\alpha_{a1}-\alpha_{a1}\cos\alpha_{a1})] \tag{16}$$

$$N'_{a1} = \frac{1+\cos\psi}{1-\cos\psi}\sigma_{ac}A_a[\sin\psi+(\pi-\psi)\cos\psi]/[\pi(1+\cos\psi)] \tag{17}$$

$$y'_{a1} = r_a(\pi-\psi+\sin\psi\cos\psi)/\{2[\sin\psi+(\pi-\psi)\cos\psi]\} \tag{18}$$

$$N'_{a2} = \left(\frac{1+\cos\psi}{1-\cos\psi}\sigma_{ac}-\eta_a f_y\right)A_a(\sin\alpha_{a2}+\alpha_{a2}\cos\alpha_{a2})/[\pi(1-\cos\alpha_{a2})] \tag{19}$$

$$y'_{a2} = r_a(\alpha_{a2} - \sin\alpha_{a2}\cos\alpha_{a2})/[2(\sin\alpha_{a2} - \alpha_{a2}\cos\alpha_{a2})] \tag{20}$$

$$N_{c1} = 2\eta_c f_c A_c(\sin\psi - \psi\cos\psi)/[\pi(1 - \cos\psi)] \tag{21}$$

$$y_{c1} = r_c(\psi - \sin\psi\cos\psi)/[2(\sin\psi - \psi\cos\psi)] \tag{22}$$

$$N_{c2} = \eta_c f_c A_c(\sin\alpha_c - \alpha_c\cos\alpha_c)/[\pi(1 - \cos\alpha_c)] \tag{23}$$

$$y_{c2} = r_c(\alpha_c - \sin\alpha_c\cos\alpha_c)/[2(\sin\alpha_c - \alpha_c\cos\alpha_c)] \tag{24}$$

where η_a and η_c are the constraining factors for tube steel and spun concrete strength.

The resultant internal force as the resisting compressive force of composite cross-sections and the resisting bending moment caused by this force with respect to the centre point of cross-section of beam-columns, respectively, are:

$$N_R = N_{a1} - N_{a2} - N'_{a1} + N'_{a2} + N_{c1} - N_{c2} \tag{25}$$

$$M_R = N_{a1}y_{a1} - N_{a2}y_{a2} + N'_{a1}\,y'_{a1} - N'_{a2}\,y'_{a2} + N_{c1}y_{c1} - N_{c2c2} \tag{26}$$

Thus, the eccentricity of the resisting compressive force of columns is:

$$e = M_R/N_R \tag{27}$$

The response factors, k_a and k_c, of annular steel and concrete cross-sections characterizing the level of the intelligent use of their compressive strengths in beam-columns may be calculated from the following equations:

$$k_a = \frac{N_{a1}(y_{a1} + r_a) - N_{a2}(y_{a2} + r_a) - N'_{a1}\,(r_a - y'_{a1}) + N'_{a2}\,(r_a - y'_{a2})}{\eta_a f_y A_a r_a} \tag{28}$$

$$k_c = \frac{N_{c1}(y_{c1} + r_a) - N_{c2}(y_{c2} + r_a)}{\eta_c f_c A_c r_a} \tag{29}$$

The bending moments of inner steel and concrete forces of eccentrically and concentrically loaded columns with respect to the bottom point B of annular cross-sections (Fig. 7) are presented in the numerators and denominators of Eq. (28) and Eq. (29).

In design practice, the values of steel and concrete response factors may be calculated by the formulae:

$$k_a = 1.10 - (0.53 - 0.35\xi)\left(\frac{e}{r_a}\right) \tag{30}$$

$$k_c = 1 - 0.21(1.7 - \xi)\left(\frac{e}{r_a}\right)^{0.667} \tag{31}$$

where

$$\xi = \eta_a f_y A_a / \eta_c f_c A_c \tag{32}$$

ξ is the transformed confinement factor of composite sections. The acceptability of Eqs. (30) and (31) in the resistance (bearing capacity) analysis of beam-columns of annular composite cross-sections is illustrated by curves presented in Fig. 8 and Fig. 9. The factor ξ by Eq. (32) and the point values of these figures are calculated using the mean values of design parameters as follows: $\eta_{am} = 1.07$, $f_{ym} = 262$ MPa, $\eta_{cm} = 1.32$, $f_{cm} = 58$ or 38 MPa, $A_{am} = 0.004046$ m^2, $A_{cm} = 0.034935$ m^2, $r_{am} = 0.161$ m, $r_{cm} = 0.139$ m. In any case, an increase of factor ξ leads to an increase of the response factors k_a and k_c given by Eq. (30) and Eq. (31).

When the eccentricity ratio $e/r_a \leq 2$, the resisting compressive force and bending moment of composite cross-sections of columns subjected to compression with a bending moment may be expressed as:

$$N_R = N_{Ra} + N_{Rc} = (k_a \eta_a f_y A_a + k_c \eta_c f_c A_c) r_a / (e + r_a) \tag{33}$$

$$M_R = N_R e \tag{34}$$

where N_{Ra} and N_{Rc} are the ultimate compressive forces in steel and concrete components of annular cross-sections; k_a is given by Eq. (30) and k_c - by Eq. (31). When cross-sections are concentrically loaded or the eccentricity of applied force by the first-order theory does not exceed $0.1r_a$, stub composite beam-columns may be treated as concentrically compressive members. Their buckling resistance may be expressed in the form of Eq. (4).

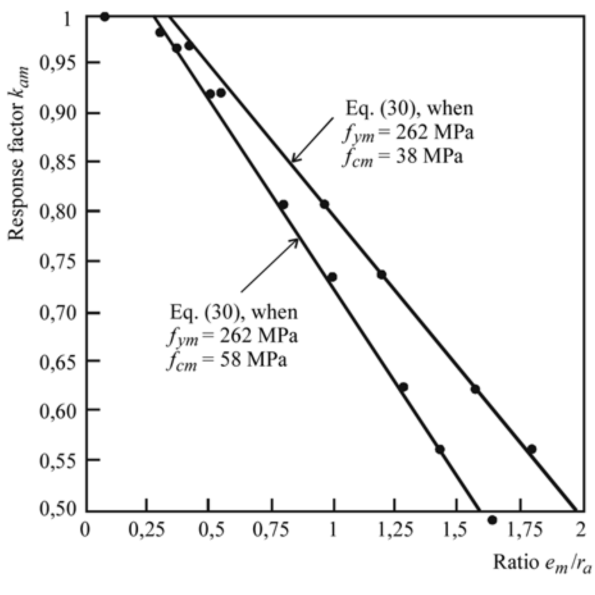

Fig. 8 Response factors of steel tubes k_{am} versus the eccentricity ratio e_m / r_a

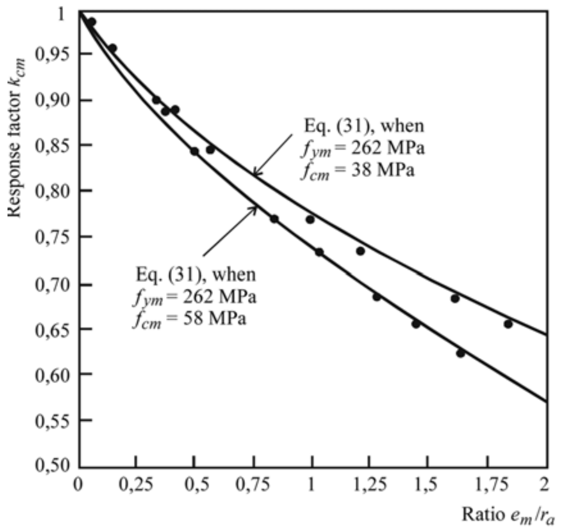

Fig. 9 Response factors of concrete k_{cm} versus the eccentricity ratio e_m/r_a

4.3. Comparison of modeling data

The comparison of the analytical interaction diagrams $N_R - M_R$ of beam-columns whose resisting bending moments are calculated by Eqs. (8) and (34) is given in Fig. 10. Curves 1 and 2 in this figure are drawn up using fairly unsophisticated model assumptions presented in Figs. 6 and 7 for eccentrically loaded members. Nevertheless, when the eccentricity ratio e/r_a is between 0.1 and 2.0, the interaction curves given there are very close.

The limit state design of composite steel and concrete beam-columns may be done in a simpler manner than it is recommended by international and many national codes. The response factors k_a by Eq. (30) and k_c by Eq. (31) for compressive annular steel tube and spun concrete sections based on the equilibrium of sectional forces and moments facilitate the prediction of resisting compressive forces of beam-column sections. Therefore, in many cases it is expedient to use in design practice unsophisticated Eq. (33) characterizing the resisting compressive force of tubular composite beam-columns of annular cross-sections and in this way to simplify their bearing capacity analysis.

5. Limit state design of beam-columns

5.1. Applied compressive forces and their eccentricities

The combined effects of eccentrically loaded tubular composite columns of buildings are caused by permanent G, sustained $Q_s(t)$ and extraordinary $Q_e(t)$ live loads, snow $S(t)$ and lateral wind $W(t)$ actions. The time-variant extreme live and climate loading of building structures belong to persistent design situations in spite of the short period of extreme events, it being much shorter than the design working life of buildings. The selected design situations and the relevant limit states in the design models of structures depend on the combination of variable actions that are considered to occur simultaneously.

The duration of annual extreme live loads, d_Q is fairly short and equal to 1~14 days for commercial and

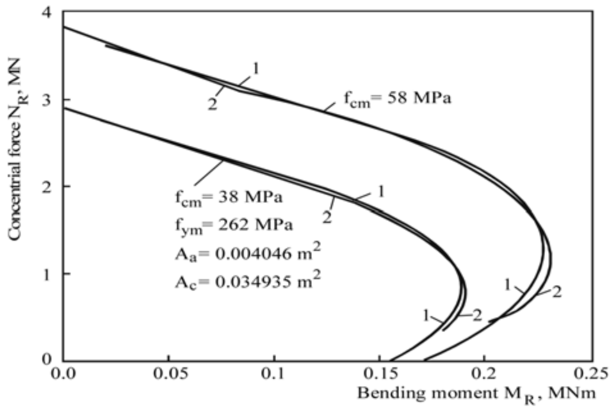

Fig. 10 Interaction diagrams of beam-columns: 1-M_R by Eq. (8); 2-M_R by Eq. (34)

1~3 days for other buildings (JCSS 2000). The renewal rate, $\lambda = 1/t_\lambda$ of annual extreme loads is equal to 1 / year. Thus, during $t = 50$ years service period, the average recurrence numbers of annual extreme live loads simultaneously on two and three storey of buildings, respectively, are equal to $n_2 = 2td_Q\lambda_Q^2 = 0.27 - 0.82$ and $n_3 = 3td_Q^2\lambda_Q^3 = 0.001 - 0.01$. Therefore, this unfavourable combination of loads for the columns of multistorey buildings may be neglected.

The duration of annual extreme snow and wind loads, respectively, are: d_s = 14~28 days and d_w = 8~12 hours (Ellingwood 1981, JCSS 2000). Therefore, the average recurrence numbers of joint annual extreme live or snow and wind loads during $t = 50$ years service period of office buildings are equal to $n_{QS} = t(d_Q + d_S)\lambda_Q\lambda_S = 2–4.2$, $n_{QW} = t(d_Q + d_W)\lambda_Q\lambda_W = 0.2–2$ and $n_{SW} = t(d_S + d_W)\lambda_S\lambda_W = 2–3.9$. The effect of these recurrences of joint variable loads on the limit state verification of structures is associated with the design factors for the combination values of variable actions (EN 1990, ASCE 7-05 2005).

Usually, the ends of pre-cast columns of frames are not fully rigidly restrained at in-situ concrete or composite foundations and floors. Both ends of pre-cast columns are only elastically restrained at pre-cast floors. Therefore, a slenderness criterion for pre-cast tubular composite columns of multi-storey buildings with in-situ or pre-cast concrete and composite beams may be checked, respectively, with their effective length $l = 0.75h_s$ and $l = h_s$, where h_s is a storey height. The first order eccentricities of applied total characteristic and design compressive forces N_{Ek} and N_{Ed} are:

$$e_{ok} = M_{oEk}/N_{Ek} \text{ and } e_{od} = M_{oEd}/N_{Ed} \tag{35}$$

where M_{oEd} and M_{oEk} are applied total characteristic and design bending moments.

The second order eccentricities of compressive forces may be calculated by the formulae:

$$e_k = e_{ok}\eta_k \geq 0.1r_a \text{ and } e_d = e_{od}\eta_d \geq 0.1r_a \tag{36}$$

Here

$$\eta_k = 1/(1 - N_{Ek}/N_{Bk}) \text{ and } \eta_d = 1/(1 - N_{Ed}/N_{Bd}) \tag{37}$$

are the factors of the second order moment effect, where the buckling loads of concentrically loaded columns are:

$$N_{Bk} = \pi^2(EI)_{ek}/l^2 \text{ and } N_{Bd} = \pi^2(EI)_{ed}/l^2 \quad (38)$$

According to Eurocode 4 (EN 1994-1-1) recommendations, the characteristic and design values of flexural stiffness of composite annular cross-sections may be calculated using the equations:

$$(EI)_{ek} = E_a I_a + 0.8(1 - 0.5N_{Gk}/N_{Ek})E_{ck}I_{cm} \text{ and } (EI)_{ed} = E_a I_a + 0.8(1 - 0.5N_{Gd}/N_{Ed})E_{cd}I_{cm} \quad (39)$$

where $E_{cm} = 20(0.1 \times f_{cm})^{0.3}$ and $E_{ck} = E_{cd} = E_{cm}/1.2$ are the mean, characteristic and design values of elastic modulus; N_{Ek}, N_{Ed} and N_{Gk}, N_{Gd} are applied compressive forces and their permanent components.

An analysis of Eq. (37) and Eq. (39) leads to the conclusion that the flexural stiffness and ductility of tubular composite columns decrease significantly with a decrease in the wall thickness of steel tubes. Therefore, an appropriate selection of steel tube sections for tubular composite steel and spun concrete beam-columns is indispensable as it is demonstrated in the investigations carried out by Lee (2007).

5.2. Reliability verification format

According to Eurocode (EN 1990) recommendations, the total design compressive force of column sections should be expressed as:

$$N_{Ed} = K_{F1}(N_{Gk}\gamma_G + N_{1k}\gamma_1 + N_{2k}\gamma_2\psi_0) \quad (40)$$

where $K_{F1} = 0.9$, 1.0 and 1.1 is the efficiency factor for unfavourable actions when the Reliability Class of structures is RC1, RC2 and RC3, respectively; N_{Gk}, N_{1k} and N_{2k} are the characteristic values of permanent, leading and accompanying variable compressive forces; $\gamma_G = 1.35$ and $\gamma_1 = \gamma_2 = 1.5$ are the partial safety factors for loads; $\psi_0 = 0.7$ is the factor for variable live actions and $\psi_0 = 0.6$ for wind loads in combinations of actions.

According to the current partial safety factors design (PSFD) used in European countries, the design values of a resisting bending moment by Eq. (8) and a compressive force by Eq. (33) may be expressed as follows:

$$M_{Rd} = 1.2r_a(\eta_a A_a f_{yk}/\gamma_a + N_{Ed})\left[1 - \frac{\eta_a A_a f_{yk}/\gamma_a + N_{Ed}}{2\eta_a A_a f_{yk}/\gamma_a + 0.5(1 + r_c/r_a)\eta_c A_c f_{ck}/\gamma_c}\right] \quad (41)$$

$$N_{Rd} = (k_{ad}\eta_a A_a f_{yk}/\gamma_a + k_{cd}\eta_c A_c f_{ck}/\gamma_c)r_a/(e_d + r_a) \quad (42)$$

where the constraining factors $\eta_a = \eta_{am} = 1.07$ and $\eta_c = \eta_{cm} = 1.32$; the response factors k_{ad} and k_{cd} for steel and concrete cross-sections are calculated by Eq. (30) and Eq. (31) using the design values of their strength $f_{yk}/1.1$ and $f_{ck}/1.5$; $\gamma_a = 1.1$ and $\gamma_c = 1.5$ are the partial factors for tube steel and core concrete properties recommended by Eurocode 3 (EN 1993-1-1) and Eurocode 2 (EN 1992-1-1), respectively.

According to the direction of the deterministic PSFD method, the structural design of columns is considered to be sufficient if their design limit state is not exceeded. This requirement is presented in the forms: $M_{Rd} \geq N_{Ed}e_d$ and $N_{Rd} \geq N_{Ed}$, where e_d is the second order eccentricity of destroying force N_{Ed} given by Eq. (40).

Using the partial factors for loads recommended by ASCE/SEI 7-05 (2005), the design value of the total compressive force of sway frame columns is presented as:

$$N_{Ed} = 1.2N_{Gk} + 1.6N_{Wk} + 0.5N_{Qk} \tag{43}$$

or

$$N_{Ed} = 1.2N_{Gk} + 1.6N_{Wk} + N_{Qk} \tag{44}$$

When uniformly distributed live loads are not more or more than 4.8 kN/m^2, the load factor γ_Q is defined as 0.5 and 1.0, respectively.

According to the current load and resistance factors design (LRFD) used in the USA and many other countries, the design values of products $\Phi M_{Rn} = \Phi M_{Rk}$ and $\Phi N_{Rn} = \Phi M_{Rk}$ may be expressed as follows:

$$M_{Rd} = \Phi M_{Rk} = \Phi 1.2 r_a(\eta_a A_a f_{yk} + N_{Ek})\left[1 - \frac{\eta_a A_a f_{yk} + N_{Ek}}{2\eta_a A_a f_{yk} + 0.5(1 + r_c/r_a)\eta_c A_c f_{ck}}\right] \tag{45}$$

$$N_{Rd} = \Phi N_{Rk} = \Phi(k_{ak}\eta_a A_a f_{yk} + k_{ck}\eta_c A_c f_{ck}) r_a/(e_k + r_a) \tag{46}$$

where M_{Rk} and N_{Rk} are the characteristic (nominal) values of a resisting bending moment by Eq. (8) and a compressive force by Eq. (33); the response factors k_{ak} and k_{ck} are calculated by Eq. (30) and Eq. (31) using the characteristic values of material strengths; Φ is the strength reduction factor as the global uncertainty factor for resistances (bearing capacities) of columns. According to ACI Committee 318 (2005) and Szerszen, *et al.* (2005) recommendations, for concrete columns subjected to compression with a small bending moment, the value of this factor is between 0.65 and 0.75 and may be selected as $\Phi = 0.7$. An analogy of design models for concrete and composite members of annular cross sections allows using the factor $\Phi = 0.7$ in the analysis of composite steel and concrete beam-columns of annular cross-sections.

The reliability of beam-columns is sufficient if the destroying moment $N_{Ed}e_k$ is not more than the product ΦM_{Rk} by Eq. (45) and the destroying compressive force N_{Ed} by Eq. (43) or Eq. (44) is not more than the product ΦN_{Rk} by Eq. (46).

5.3. Numerical illustration

The numerical analysis is considered as an illustration of the reliability verification of the tubular composite steel and concrete columns of annular cross-sections of unbraced multistorey frames of Reliability Class RC 2 by Eurocode 1 (EN 1990) or Category II by ASCE/SEI 7-05 (2005) using the load combination rules recommended by these European and American design codes. The analysed columns are under compression with a bending moment caused by permanent and variable gravity and lateral wind action effects. Their characteristic values are:

$N_{Gk} = 630$ kN, $M_{oGk} = 45.3$ kNm; $N_{Qk} = 72$ kN, $M_{oQk} = 30.2$ kNm; $N_{Wk} = 42$ kN, $M_{oWk} = 28$ kNm; $N_{Ek} = 630 + 72 + 42 = 744$ kN, $M_{oEk} = 45.3 + 30.2 + 28 = 103.5$ kNm; the multiplication factor $K_{F1} = 1.0$.

The design values of action effects of analysed columns are given in Table 3.

The geometrical parameters of columns are: $l = h_s = 3.6$ m; $r_a = 0.161$ m, $A_a = 0.004046$ m^2, $I_a = 5.245 \times 10^{-5}$ m^4; $r_c = 0.139$ m, $A_c = 0.034935$ m^2, $I_c = 3.445 \times 10^{-4}$ m^4.

The mechanical parameters of steel and fine aggregate spun concrete are: $f_{yk} = 235$ MPa, $\eta_a = 1.07$,

$E_a = 2 \times 10^5$ MPa; $f_{ck} = 50$ MPa, $\eta_c = 1.32$, $E_c = 2.824 \times 10^4$ MPa. According to Eq. (37), the characteristic and design values of a transformed confinement factor are:

$$\xi_k = 1.07 \times 235 \times 0.004046/(1.32 \times 50 \times 0.034935) = 0.4412;$$

$$\xi_d = 1.07 \times 235/1.1 \times 0.004046/(1.32 \times 50/1.5 \times 0.034935) = 0.6017.$$

The parameters of analysed columns are given in Table 4.

According to Eurocode (EN 1994-1-1) directions, the design values of resisting bending moment by Eq. (41) and compressive force by Eq. (42) are:

$$M_{Rd} = 1.2 \times 0.161(1.07 \times 0.004046 \times 235/1.1 + 0.9963) \times$$
$$\times \left[1 - \frac{1.07 \times 0.004046 \times 235/1.1 + 0.9963}{2 \times 1.07 \times 0.004046 \times 235/1.1 + 0.5(1 + 0.139/0.161)1.32 \times 0.034935 \times 50/1.5}\right] =$$

$= 0.1539$ MNm $> N_{Ed}e_d = 0.9963 \times 0.1449 = 0.1444$ MNm (the design value of a destroying bending moment), $N_{Rd} = (0.8125 \times 1.07 \times 0.004046 \times 235 / 1.1 + 0.7952 \times 1.32 \times 0.034935 \times 50 / 1.5)0.5263 =$ 1.039 MN $\approx N_{Ed} = 0.9963$ MN (the design value of a destroying compressive force by Eq. (40)).

According to AISC (2005) and ACI Committee 318 (2005) directions, the design values of a resisting bending moment by Eq. (45) and a compressive force by Eq. (46) as the products ΦM_{Rk} and ΦN_{Rk} are:

$$M_{Rd} = \Phi M_{Rk} = 0.7 \times 1.2 \times 0.16(1.07 \times 0.004046 \times 235 + 0.744) \times$$
$$\times \left[1 - \frac{1.07 \times 0.004046 \times 235 + 0.744}{2 \times 1.07 \times 0.004046 \times 235 + 0.5(1 + 0.139/0.161)1.32 \times 0.034935 \times 50}\right] =$$

$= 0.1379$ MNm $> N_{Ed}e_d = 0.8592 \times 0.1448 = 0.1244$ MNm when load factor $\gamma_Q = 0.5$ and $> N_{Ed}e_d =$ $0.8952 \times 0.1448 = 0.1296$ MNm when $\gamma_Q = 1.0$;

Table 3 The design values of first order action effects of analysed columns

Design code	Eqs	N_{Gd} (kN)	M_{Gd} (kNm)	N_{Qd} (kN)	M_{Qd} (kNm)	N_{Wd} (kN)	M_{Wd} (kNm)	N_{Ed} (kN)	M_{Ed} (kNm)
EN 1990	(40)	850.5	61.2	108.0	45.3	37.8	25.3	996.3	131.7
ASCE/SEI with $\gamma_a = 0.5$	(43)	756	54.4	36.0	15.1	67.2	44.8	859.2	114.8
ASCE/SEI with $\gamma_a = 1.0$	(44)	756	54.4	72.0	30.2	67.2	44.8	895.2	129.4

Table 4 The design and characteristic parameters of analyzed columns

Parameters	$(EI)_e$ by (39) (kNm^2)	N_B by (38) (kN)	η by (37)	$e_o = \frac{M_{oE}}{N_E}$ (m)	$e = \eta e_o$ (m)	$\frac{e}{r_a}$	k_a by (30)	k_c by (31)
Design value	14951	11386	1.096	0.1322	0.1449	0.9000	0.8125	0.7952
Characteristic value	14978	11406	1.070	0.1391	0.1448	0.9244	0.7528	0.7611

Table 5 The results of the numerical illustration of the structural design according to the methodology of appropriate codes

Design code	M_{Rd} (kNm)	M_{Ed} (kNm)	$\frac{M_{Rd}}{M_{Ed}}$	N_{Rd} (kN)	N_{Ed} (kN)	$\frac{N_{Rd}}{N_{Ed}}$
EN 1990	153.9	144.4	1.065	1038.8	996.3	1.042
ASCE/SEI with $\gamma_Q = 0.5$	137.9	124.4	1.108	929.0	859.2	1.081
ASCE/SEI with $\gamma_Q = 1.0$	137.9	129.6	1.064	929.0	895.2	1.038

$N_{Rd} = \Phi N_{Rk} = 0.7(0.7528 \times 1.07 \times 0.004046 \times 235 + 0.7611 \times 1.32 \times 0.034935 \times 50)0.5265 =$ $= 0.929$ MN > 0.8592 MN and > 0.8952 MN when load factor γ_Q is equal to 0.5 and 1.0, respectively.

The effect of three load combinations by Eq. (40), Eq. (43) and Eq. (44) on the analysis results of beam-columns is demonstrated in Table 5.

According to both limit state design methods (PSFD and LRFD), the analysed columns are suitable in service and their analysis results in design practice slightly depend on the methodological concepts close to the native European and American design limit state approaches. However, the predicted relations M_{Rd}/ M_{Ed} and N_{Rd} / N_{Ed} in Table 5 show that the drastic decrease of the load factor on live loads, γ_Q, from 1.0 to 0.5 is doubtful.

6. Conclusions

The experimental and analytical data on the concentrical and eccentrical compression of circular steel tubes with spun concrete hollow cores as composite members of annular cross-sections showed their structural and constructional efficiency. The fairly high effect of an interaction between steel tubes and spun concrete cores on their constraining factors $\eta_a = f_a / f_y$ and $\eta_c = f_c / f_c'$ and at the same time on the efficiency factor K_{ef} by Eq. (5) and the ultimate strength of compression and flexural composite members was established. It stimulated us to use them in design and construction practice as beam-columns of multistorey buildings subjected to permanent, variable and wind loads.

A homologous definition of the strain criteria for the limit state versions helped us assess the resistance of fairly complicated composite steel and concrete beam-columns in an unsophisticated engineering manner and recommend it to include into the Eurocode 4 and other international design codes or national standards.

The concepts of the bending with a concentrical compressive force and the compression with a bending moment may be successfully used in the bearing capacity analysis of beam-columns of annular cross-sections. Their ultimate bending moments, M_R, and compressive forces, N_R, may be calculated by unsophisticated fairly exact Eq. (8) and Eq. (33), respectively.

The limit state verification of analyzed beam-columns by the prevailing partial safety factors design (PSFD) and load and resistance factors design (LRFD) used in Europe and the USA, practically, lead to the same results despite the difference in their methodologies applied to assess the design values of resisting bending moments, M_{Rd} , by Eq. (41) and Eq. (45) or compressive forces, N_{Rd}, by Eq. (42) and Eq. (46). However, the drastic decrease of the load factor on live loads, γ_Q, from 1.0 to 0.5 recommended by ASCE/SEI 7-05 leads to the groundless overestimation of the reliability of structures.

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Notations

A_a and A_c	: steel tube and spun concrete cross-sectional areas
d_e and d_{ce}	: external diameters of steel tube and concrete annular cross-sections
E_a and E_c	: moduli of elasticity of steel and concrete
e_o and e	: first and second order eccentricities, respectively
f_c	: cylinder compressive strength of concrete
f_y	: yield strength of steel
h_s	: height of building storey
I_a and I_c	: second moments of areas of steel tube and concrete annular cross-sections
K_{ef}	: efficiency factor for variable actions
k_a and k_c	: response factors for compressive steel tube and spun concrete
l	: effective buckling length of columns
M_G, M_Q, M_W	: bending moments caused by permanent (G), live (Q) and wind (W) loads
N_G, N_Q, N_W	: compressive forces caused by permanent (G), live (Q) and wind (W) loads
N_B	: buckling load of columns
N_E and M_E	: applied total compressive force and bending moment, respectively
N_R and M_R	: resisting compressive force and bending moment, respectively
$r_a = 0.5(d_e - t)$	: radius of average circle of steel tube cross-section
$r_c = 0.5(r_e + r_i)$	: radius of average circle of spun concrete cross-section
r_e and r_i	: radii of external and internal circles of spun concrete cross section
t and t_c	: wall thickness of steel and concrete tubes, respectively
$\alpha = \psi / \pi$	: conventional value of compression zone area
α_a and α_c	: intermediate angles of cross-sectional compressive zone
γ	: partial factors for material properties and loads
ε_{xy} and ε_z	: longitudinal and tangential strains
ε_{au} and ε_{cu}	: ultimate steel tube and concrete core strains in compression for beam-columns
η	: factor of second order moment effect
η_a and η_c	: constraining factors for steel and concrete interaction effect
ξ_o and ξ	: initial and transformed confinement factors of composite sections
ξ_k and ξ_d	: characteristic and design values of a transformed confinement factor
σ_{ac}	: ultimate normal steel stress
Φ	: strength reduction factor
κ	: reduction factor for flexural buckling
ψ	: angle of cross-sectional compression zone
ψ_0	: factor for combination of variable actions