ISSN 1392 - 1207. MECHANIKA. 2006. Nr.5(61)

Research on structural resistance and safety of tubular composite members

A. Kudzys*, A.K. Kvedaras**
**Institute of Architecture and Construction of Kaunas University of Technology, Tunelio 60, 44405 Kaunas, Lithuania, E-mail: asi@asi.lt*
*** Vilnius Gediminas Technical University, Sauletekio al. 11, 10223 Vilnius, Lithuania, E-mail: akve@st.vtu.lt*

1. Introduction

The concrete-filled steel tubes obtain the wide application in construction practice. Eurocode 4 [1] presents design directions and recommendations only for the members the circle whether rectangular or square steel tubes of which are supplied with solid concrete cores. The research results carried out by Kvedaras et al. [2 - 4] illustrated a great structural efficiency of tubular composite members of annular cross-sections. The steel tubes with spun concrete cores, for which the self-regulating resistance property is characteristic, are economically and structurally rational composite structures for works, equipments and buildings. However, the structural performance and reliability of a new type of hollow concrete-filled circular steel tubular members are investigated not well enough.

The initial experimental data show greater than stub members' efficiency for slender differently loaded structures although such results are in some contradiction with limitation of Eurocode 4 allowance. Such and other code features come to light in design stage and safety assessment processes of new type composite structures. A lack of experimental and theoretical research results hampers the development of deterministic and probabilistic analysis methods of composite members. In some cases it can lead to groundless overestimation or underestimation of the reliability of designed and executed structures. Therefore, it is urgent matter analysing their robustness and structural safety both by deterministic ultimate limit state and probabilistic safety methods.

Time-variant extreme loadings of structures belong to persistent design situations in spite of the short period of extreme events being much shorter than the designed their working life. Using traditional deterministic approaches, it may be impossible or difficult to give objective and quantitative parameters for composite member reliabilities. It is expedient to base a reliability analysis on the probabilistic concept of life span assessment [5].

Probabilistic analysis of composite structures may be inevitable in cases when their effects of actions are caused by extreme recurrent service and climate loads. However, an estimation of effects of recurrent variable in time extreme actions on long-term safety of high-reliable tubular composite structures is connected with some methodological and mathematical features.

The object of this paper is turning an attention of engineers and researchers on the rationality of tubular composite steel and concrete members and expedience to appraise their long-term structural safety by the probabilistic methods.

2. Peculiarities of tubular composite members

2.1. Peculiarities of material properties

The self-regulating resistance property is characteristic for the tubular composite members. Because of the interaction between a steel tube's and a spun concrete contact surfaces occurring during coaxial compression is calling an increase in compressive strength of both components and of the robustness of the whole such composite members (Fig. 1).

The statistically-based assessment of the increased resistances of steel and concrete elements allows assessing and predicting the probabilistic safety of the tubular composite structures exposed to diverse action situations.

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 takes into account different values of Poisson's ratio. Homologous definition of the strain criteria allows an exact definition of the robustness of composite steel-concrete members evaluating their increase against the criteria determined by superposition of the resistances of member components.

The structural behaviour of hollow concrete-filled circular steel tubular members under axial compression is more complicated than that of composite members with solid concrete cores. The resistance analysis of annular 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 core – have to be expressed by [6] the formula

$$\sigma_x = (4/3) E_{im} \left(\varepsilon_{xy} + v\varepsilon_z \right) \quad (1)$$

where E_{im} is the secant modulus of elasticity; $\varepsilon_{xy} = 0.5\left(\varepsilon_z + \sqrt{3\left(\varepsilon_{iy}^2 - \varepsilon_z^2\right)} \right)$ and $\varepsilon_z = \left(\sigma_z - 0.5\sigma_x\right)/E_a$ are the ultimate values of longitudinal and tangential strains, respectively.

Because of the assumed equality of biaxial stress state in steel shell and in concrete core [6], the ultimate value of the generalized strain $\varepsilon_{iy} = 1.5\sigma_y / E_a$ will be the same for both materials. Thus, the ultimate normal stresses of steel σ_{ax} and concrete σ_{cx} defined from Eq. (1) represent the modified values of the steel and concrete resistances

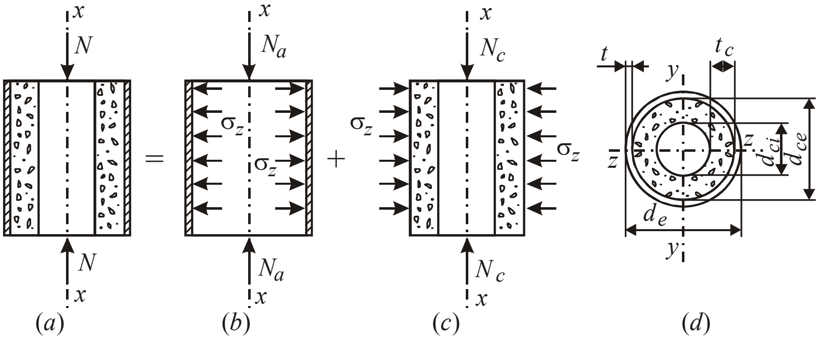

Fig. 1 Diagramatic sketch of axial actions on the hollow composite member (*a*), steel tube (*b*) and hollow concrete core (*c*): shape and dimensions of hollow cross-section (*d*)

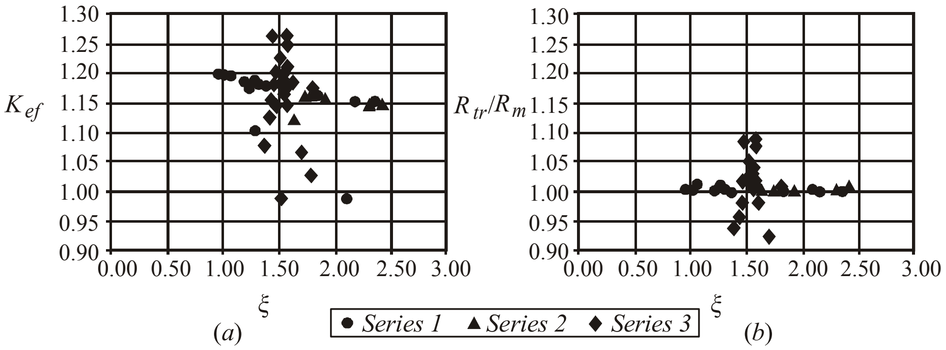

Fig. 2 Efficiency factor K_{ef} (a) and ratio R_{tr}/R_m (b) versus steel contribution factor $\xi = f_y A_a / f_c' A_c$

$$f_a = \sigma_{ax} = \eta_a f_y \text{ and } f_c = \sigma_{cx} = \eta_c f_c' \quad (2)$$

where $\eta_a = 1.07$ and $\eta_c = 1.32$ are the mean values of constraining factors as random variables values characterizing the interaction effect of member components on its load-carrying capacity; f_y and f_c' are the values of steel yield and concrete specified compressive strengths, respectively. When the thickness of a steel tube is less than its ultimate minimum, the critical strength f_{cr} has to be used instead of f_y. According to Lundberg and Galambos [7] and Raizer [8], the random variables of concrete-filled tube compressive resistance components f_y and f_c may be assumed as normally distributed.

2.2. Limit state criteria under axial and eccentric compression

The ultimate resistance of an annular cross-section of composite tubular members under axial compression may be introduced by the expression

$$R = \sigma_{ax} A_a + \sigma_{cx} A_c = f_a A_a + f_c A_c = \\ = \eta_a f_y A_a + \eta_c f_c' A_c \quad (3)$$

where A_a and A_c are the areas of steel and concrete cross-sections, respectively.

The mean and variance of this resistance distribution respectively are

$$R_m = f_{am} A_{am} + f_{cm} A_{cm} = \eta_{am} f_{ym} A_{am} + \eta_{cm} f_{cm}' A_{cm} \quad (4)$$

$$\sigma^2 R = A_{am}^2 \boldsymbol{\sigma}^2 f_a + f_{am}^2 \boldsymbol{\sigma}^2 A_a + A_{cm}^2 \boldsymbol{\sigma}^2 f_c + f_{cm}^2 \boldsymbol{\sigma}^2 A_c = \\ = (\eta_{am} A_{am})^2 \sigma^2 f_y + (\eta_{am} f_{ym})^2 \sigma^2 A_a + (f_{ym} A_{am})^2 \sigma^2 \eta_a + \\ + (\eta_{cm} A_{cm})^2 \sigma^2 f_c' + (\eta_{cm} f_{cm}') \sigma^2 A_c + (f_{cm}' A_{cm})^2 \sigma^2 \eta_c \quad (5)$$

Acording to the experimental investigations carried out in the Department of Steel and Timber Structures of Vilnius Gediminas Technical University, the mean and standard deviation of the ratio of trial and calculated by Eq. (4) resistances are: $\theta_{Rm} = R_{tr}/R_m = 1.005 \approx 1.0$ and $\boldsymbol{\sigma}\theta_R = 0.0485 \approx 0.05$. These values may be treated as the statistical parameters of additional random variables representing the resistance model uncertainty.

The rationality of tubular composite members under compression demonstrates the efficiency factor $K_{ef} = R_{tr}/(f_y A_a + f_c' A_c)$ the mean value of which was equal to 1.17 (Fig. 2).

The strength analysis of composite steel concrete beam-columns as eccentrically loaded members is carried out by the formula

$$\frac{N}{N_u} + \frac{N e}{M_u} \le 1 \quad (6)$$

where N_u and M_u are the ultimate values of axial internal force and bending moment, respectively; N is an acting axial force; e is its eccentricity.

An ultimate value of axial internal force N_u as a member ultimate resistance is

$$N_u = \eta_a f_y A_a + \eta_c f_c' A_c \quad (7)$$

The statistical moments of this force are calculated by Eqs. (4) and (5).

According to Kvedaras [9], the ultimate value of bending moment M_u is defined as follows

$$M_u = \eta_a f_a A_a e_u \tag{8}$$

where f_a is the strength of external shell's steel calculated by the formula:

$$f_a = f_y W_p / W \tag{9}$$

where W, W_p are the elastic and plastic sectional moments of steel tube, respectively; e_u is the ultimate value of an eccentricity of a longitudinal force N_u which is close to a radius of cross-section core.

The mean and variance of bending moments by Eq. (8) may be calculated by the formulae

$$M_{um} = B\eta_{am} f_{ym} A_{am} \tag{10}$$

$$\boldsymbol{\sigma}^2 M_u = (B\eta_{am} A_{am})^2 \boldsymbol{\sigma}^2 f_y + (B\eta_{am} f_{ym})^2 \boldsymbol{\sigma}^2 A_a + + (B f_{ym} A_{am})^2 \boldsymbol{\sigma}^2 \eta_a \tag{11}$$

where $B=e_u W_p/W$ is the geometrical parameter of a cross-section as its deterministic value.

According to Eq. (6), the cross-section resistance of hollow concrete filled steel tubular beam-columns may be defined as

$$R = 1 \Big/ \left(\frac{1}{N_u} + \frac{e}{M_u} \right) \tag{12}$$

where the components N_u by Eq. (7) and M_u by Eq. (8).

3. Time-dependent safety margin

The time-dependent safety margin of particular members (sections) may be defined as their performance process. According to Melchers [5] and JCSS [10], the safety margin may be expressed as a random process

$$Z(t) = g[\boldsymbol{X}(t), \boldsymbol{\theta}] = \theta_R R(t) - \theta_P S_P(t) - \theta_v S_v(t) \tag{13}$$

where $R(t)$, $S_p(t)$ and $S_v(t)$ are the stochastically independent component processes associated with resistance and loading specific features (Fig. 3).

The compressive resistance of members is

$$R(t) = R_o - \int_0^t v_R(\tau) d\tau \approx R_0 - v_R t \tag{14}$$

where $v_R(\tau)$ is the rate of resistance decrease induced by ageing and environmental aggressive actions; the action effects S_p and $S_v(t)$ are caused by permanent p, and variable v, actions including time-variant transient episodic and recurrent ones. The additional random variables θ represent the uncertainties of calculation models including uncertainties of their probability distributions. These variables may be modelled whether by the density functions or simply as their means $(\theta_{Rm}, \theta_{pm}, \theta_{vm})$ and standard deviations $(\boldsymbol{\sigma}\theta_R, \boldsymbol{\sigma}\theta_p, \boldsymbol{\sigma}\theta_v)$.

According to Ellingwood [11], Raizer [8] and Eurocodes [1, 12, 13], the permanent action effect $S_p(t)$

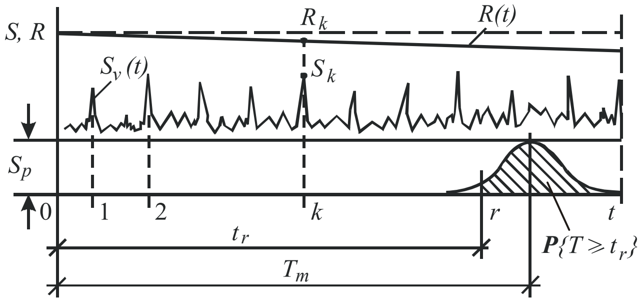

Fig. 3 Dynamic model for safety analysis of particular members

can be described by normal distribution law. Therefore, for the sake of design simplifications, it is expedient to present the Eq. (13) in the form

$$Z(t) = R_c(t) - S(t) \tag{15}$$

where the component process

$$R_c(t) = \theta_R R(t) - \theta_p S_p(t) \tag{16}$$

may be treated as the conventional member resistance which may be modelled by normal distribution irrespective of the fact that a distribution of the resistance $R(t)$ by Eq. (14) may only be close to this distribution [14]; $S(t) = \theta_v S_v(t)$ is the variable action effect process induced by extreme service live, ambient temperature or climate actions.

The means and variances of the probability distributions of random functions R_c and S (Fig.2) are

$$R_{cm} = \theta_{Rm} R_m - \theta_{pm} S_{pm} \tag{17}$$

$$\sigma^2 R_c = \theta_{Rm}^2 \sigma^2 R + R_m^2 \sigma^2 \theta_R + \theta_{pm}^2 \sigma^2 S_p + S_{pm}^2 \sigma^2 \theta_p \tag{18}$$

$$S_m = \theta_{vm} S_{vm} \tag{19}$$

$$\boldsymbol{\sigma}^2 S = \theta_{vm}^2 \boldsymbol{\sigma}^2 S_v + S_{vm}^2 \boldsymbol{\sigma}^2 \theta_v \tag{20}$$

where R_m by Eq. (4), $\sigma^2 R$ by Eq. (5), $S_{pm}, \sigma^2 S_p$ and $S_{vm}, \sigma^2 S_v$ are the means and variances of member resistance and action effects.

The member resistance $R(t)$ and permanent action effect $S_p(t)$, usually, represent fixed stochastic and stationary processes. Their numerical values are random functions only at the beginning of the process In their other cuts, the values of $R(t)$ and $S_p(t)$ changes according to deterministic laws.

The sustained and extraordinary components of occupancy live loads are modelled as time-variant stochastic processes. According to Rosowsky and Ellingwood [15], the annual extreme sum v of these components can be modelled as an intermittent rectangular wave renewal process and described by a Type 1 (Gumbel) distribution with the coefficient of variation $\boldsymbol{\delta}v = 0.58$ and mean equal to 0.47 v_k, where v_k is its characteristic value. According to JCSS [10] and Vrouwenvelder [16] recommendations, the annually recurrent short-term extreme action effect S_v may be assumed to be exponentially distributed.

It is proposed to model the annual extreme climate (wind and snow) actions by Gumbel distribution law with a mean equal to $v_k/\left(1+k_{0.98}\boldsymbol{\delta} v\right)$,where $k_{0.98}$ is the characteristic fractile factor [8, 10, 11, 16 - 18]. The coefficients or variation of wind forces and snow loads depend on the features of geographical area and are $\boldsymbol{\delta} v_w = 0.3\text{-}0.5$ and $\boldsymbol{\delta} v_s = 0.5\text{-}0.7$, respectively. It should be expedient to use this distribution law for extreme stresses caused by ambient temperature actions.

4. Structural safety prediction

4.1. Instantaneous survival probability

The time-dependent safety margin (15) may be treated as a random sequence and written as

$$Z_k = R_{ck} - S_k,\ k = 1,\ 2,\ 3, ..., r-l, r \tag{21}$$

where $R_{ck} = \theta_R R_k - \theta_p S_{pk}$ and $S_k = \theta_v S_{vk}$ are the conventional member resistance and variable extreme action effect at the sequence cut *k* the probability distributions of which are normal and exponential or of the Type 1, respectively; *r* is the number of extreme action effects during design working life t_r of the member (Fig. 3).

When R_{ck} and S_k are independent, the instantaneous survival probability of members at any cut *k* of their safety margin sequences, assuming that they were safe at time less than t_k, may be calculated using the formula

$$\boldsymbol{P}_k = \boldsymbol{P}\{Z_k > 0\} = \int_0^\infty f_{R_{ck}}(x) F_{S_k}(x)\ dx \tag{22}$$

where $f_{R_{c,k}}(x)$ is the density function of the conventional resistance $R_{c,k}$;

$$F_{S_k}(x) = exp\left[-exp\left(\frac{S_{km} - x}{0.7794\boldsymbol{\sigma} S_k} - 0.5772\right)\right] \tag{23}$$

is the Gumbel distribution function of the extreme action effect S_k the mean and standard deviation of which are S_{km} and $\boldsymbol{\sigma} S_k$.

When the extreme action effect S_v is distributed by exponential law, the probability (22) may be calculated by the analytical formula

$$\boldsymbol{P}_k = 1 - \boldsymbol{\Phi}\left(-\frac{R_{ck,m}}{\boldsymbol{\sigma} R_{ck}}\right) - exp\left(-\frac{R_{ck,m}}{S_{km}} + 0.5\frac{\boldsymbol{\sigma}^2 R_{ck}}{\boldsymbol{\sigma}^2 S_k}\right) \times \\ \times\left[1 - \boldsymbol{\Phi}\left(-\frac{R_{ck,m}}{\boldsymbol{\sigma} R_{ck}} + \frac{\boldsymbol{\sigma} R_{ck}}{\boldsymbol{\sigma} S_k}\right)\right] \tag{24}$$

where the means and variances of the random functions R_{ck} and S_k are calculated from Eqs. (17)-(20).

4.2. Long-term survival probability

The time-dependent survival probability of members as series systems may be calculated using the numerical integration and Monte Carlo simulation methods. The cuts of the random sequences of safety margins must be considered to be statistically dependent. However, it is more reasonable to use the unsophisticated analytical method of transformed conditional probabilities. When the conventional resistance of members is non-stationary process, their long-term survival probability can be written in the form

$$\boldsymbol{P}_r = \boldsymbol{P}\left\{\bigcap_{k=1}^{r}(Z_k > 0)\right\} \approx \prod_{k=1}^{r} \boldsymbol{P}_k \left[1 + \rho_{r...1}^3\left(\frac{1}{\boldsymbol{P}_{r-1}} - 1\right)\right] \times, ..., \\ \times\left[1 + \rho_{21}^3\left(\frac{1}{\boldsymbol{P}_1} - 1\right)\right] \tag{25}$$

where $\rho_{kl} = \rho(Z_k, Z_l) = Cov\left(Z_k, Z_l\right)/\left(\boldsymbol{\sigma} Z_k \times \boldsymbol{\sigma} Z_l\right)$ is the coefficient of auto-correlation of random safety margin sequence cuts the transformed value of which is $\rho_{r,...,l} = \left(\rho_{r,r-1} + ... + \rho_{rk} + ... + \rho_{rl}\right)/\left(r-1\right)$; $Cov(Z_k, Z_l)$ and $\boldsymbol{\sigma} Z_k, \boldsymbol{\sigma} Z_l$ are an auto-covariance and standard deviations of these cuts, respectively; $\boldsymbol{P}_k$ is the instantaneous survival probability by Eqs. (22) or (24).

When the member conventional resistance may be treated as stationary process, the expression (25) obtains the following form

$$\boldsymbol{P}_r = \boldsymbol{P}_k^r\left[1 + \rho_{kl}^3\left(\frac{1}{\boldsymbol{P}_k} - 1\right)\right]^{r-1} \tag{26}$$

The probabilistic analysis of structures subjected to two stochastically independent variable extreme actions is presented by Kvedaras and Kudzys [19, 20]. This analysis is based on the fact that a member failure may occur not only under joint action effects but also when the value of one out of two actions is extreme. For the practical sake, it is recommended to use the conventional bivariate distribution of two extreme action processes.

The survival probability of member $\boldsymbol{P}_r$, may be introduced by the generalized reliability index

$$\beta = \boldsymbol{\Phi}^{-1}\left(\boldsymbol{P}_r\right) \tag{27}$$

where $\boldsymbol{\Phi}^{-1}(\boldsymbol{P}_r)$ is the inverse of the standard normal distribution variable tabulated in statistics texts.

According to Eurocode 1 [12], for an ultimate limit state design of structural members the minimum values for the reliability index during 50 years reference period are: $\beta_{min} =$ 3.3, 3.8 and 4.3 when the reliability classes are RC1, RC2 and RC3, respectively.

4.3. Numerical illustration

The annular cross-section of not deteriorating composite members of reliability class RC2 is exposed to permanent and extreme snow axial compressive forces the characteristic values of which are: $N_{pk} = 900$ kN and

$N_{sk} = 174$ kN. The means and variances of these forces are:

$N_{pm} = N_{pk} = 900$ kN, $\sigma^2 N_p = (0.1 \times 900)^2 = 8100$ (kN)^2;

$N_{sm} = 174/(1 + 2.592 \times 0.5) = 75.78$ kN,

$\sigma^2 N_s = (0.5 \times 75.78)^2 = 1435.8$ (kN)^2.

The design axial force is:

$N_d = \gamma_P N_{pk} + \gamma_s N_{sk} = 1.35 \times 900 + 1.5 \times 174 = 1476$ kN.

The means and coefficients of variation of steel and concrete cross-section areas are: $A_{am} = 0.0042$ m^2; $\delta A_a = 0.03$; $A_{cm} = 0.024$ m^2; $\delta A_c = 0.15$. The parameters of steel and concrete strengths are: $f_{yk} = 235$ MPa; $f_{ym} = 262$ MPa; $\delta f_y = 0.07$; $\eta_{ak} = 1.03$; $\eta_{am} = 1.07$: $\delta\eta_a = 0.05$; $f'_{ck} = 30$ MPa; $f'_{cm} = 40$ MPa; $\delta f_c = 0.20$; $\eta_{ck} = 1.15$; $\eta_{cm} = 1.32$; $\delta\eta_c = 0.05$. The probabilistic parameters of material components are: $f_{am} = \eta_{am} f_{ym} =$ $=280.4$ MPa; $\sigma^2 f_a = 581.8$ (MPa)^2; $f_{cm} = \eta_{cm} f'_{cm} =$ $=52.8$ MPa; $\sigma^2 f_c = 118.5$ (MPa)^2.

According to Eqs. (4) and (5), the mean and variance of composite member resistance in compression are:

$R_m = 280.4 \times 0.042 + 52.8 \times 0.024 = 2.445$ MN,

$\sigma^2 R = 0.0042^2 \times 581.8 + 280.4^2 \times 1.5 \times 10^{-8} + 0.024^2 \times 118.5 +$ $+ 52.8^2 \times 12.96 \times 10^{-6} = 0.1158$ (MN)^2.

The design resistance of the cross-section is:

$R_d = \eta_{ak} f_{yk} A_{am} / \gamma_a + \eta_{ck} f'_{ck} A_{cm} / \gamma_c = 1476$ kN is equal to design axial force.

Thus, the requirement of partial factor design $R_d \geq N_d$ is satisfied.

The means and standard deviations of additional variable are: $\theta_{Rm} = \theta_{Pm} = \theta_{Sm} = 1.0$, $\sigma\theta_R = 0.05$, $\sigma\theta_P = \sigma\theta_S = 0.10$. According to Eqs. (17)-(20), the means and variances of random functions $R_c = \theta_R R - \theta_P N_P$ and $N = \theta_S N_S$ are: $R_{cm} = 2.445 -$ $- 0.90 = 1.545$ kN, $\sigma^2 R_c = 0.1158 + 2.445^2 \times 0.0025 + 0.081 +$ $+ 0.90^2 \times 0.01 = 0.147$ (kN)^2; $N_m = 0.0758$ kN, $\sigma^2 N = 14.36 \times$ $\times 10^{-6} + 0.0758^2 \times 0.01 = 14.93 \times 10^{-6}$ (kN)^2.

According to Eq. (22), the instantaneous survival probability of the members is: $\boldsymbol{P}_k = 0.999926$. The coefficient of auto-correlation of the safety margin sequence cuts is: $\rho_{kl} = 1/(1 + \sigma^2 N / \sigma^2 R_c) = 0.99$. According to Eq. (25), the long-term survival probability of members for 50 years reference period is

$$\boldsymbol{P}_r = \boldsymbol{P}_k^{50}\left[1 + \rho_{kl}^3\left(\frac{1}{\boldsymbol{P}_k} - 1\right)\right]^{49} = 0.99982$$

Therefore, the reliability index β is equal to 3.57. The slight difference from the target value $\beta_{min} = 3.80$ may be explained by increased uncertainties of spun concrete properties. Therefore, when a load-carrying capacity of the members is verified by the deterministic ultimate limit state method, the partial factor for spun concrete resistance should be increased to 1.6.

5. Conclusions

A homologous definition of the strain criteria for the limit state versions helps us to assess the resistance of tubular composite steel-concrete members subjected to axial and eccentric compressive forces in a simple engineering manner.

Inhomogeneity in a performance of tubular composite members is caused by features of their random robustness characteristics and load effects caused by extreme recurrent service, ambient temperature and climate actions as intermittent rectangular wave renewal processes. Therefore, the structural safety of these members may be objectively predicted only by probability-based approaches.

It is recommended to calculate the instantaneous and long-term survival probabilities of the members using respectively Eqs. (22), (24) and (25), (26) based on unsophisticated concepts of random sequences, conventional resistances and transformed conditional probabilities. For the sake of analysis simplifications, it is expedient to use the Gumbel distribution not only for annular extreme climate action effects but also for stresses caused by extreme live and ambient temperature actions.

The numerical example illustrates that it is expedient to increase the partial safety factor for spun concrete resistance to 1.6 when a load-carrying capacity of tubular composite members is verified by the deterministic ultimate limit state method.

The presented investigations on the compressive bearing capacity and structural safety of circular steel tubes with spun concrete will help engineers to use in practice new pre-cast composite steel and concrete structures not included in Eurocode 4.

References

1. Eurocode 4, EN 1994-1-1. Design of Composite Steel and Concrete Structures – Part 1-1: General Rules and Rules for Buildings.-Brussels: CEN, 2004.-180p.
2. **Kvedaras, A., Sapalas, K.** Carrying capacity determination of hollow composite members. Transactions of the Lithuanian Universities "Reinforced Concrete Structures". .-Vilnius, 11983, 2, p.101-112 (in Russian).
3. **Kvedaras, A., Mykolaitis, D., Sapalas, A.** In: Tubular Structures; VII: Farkas J and Jármai K (Eds). -Rotterdam/Brookfield: A.A. Balkema, 1996, p.341-348.
4. **Kvedaras, A.K., Kudzys, A., Vaitkevičius, V.** Efficient future strategies for constructing with steel in Lithuania. -J. Constr Steel Res, 1998, A 6(1-3), No.019, p.1-6.
5. **Melchers, R.E.** Structural Reliability Analysis and Prediction.-Chichester: John Wiley, 1999.- 437p.
6. **Kvedaras A.** Strength design of concrete filled steel tubes.-Transactions of the Lithuanian Universities "Construction Structures".-Vilnius, 1988, 14, p.121-135 (in Russian).
7. **Lundberg, J.E., Galambos, T.V.** Load and resistance factor design of composite columns.-Structural safety, 1996, 18, (2/3), p.169-177.
8. **Raizer, V.P.** Theory of Reliability in Structural Design. -Moscow: ACB Publishing House, 1998.-302p (in Russian).
9. **Kvedaras, A.K.** Light-Weight Hollow Concrete-Filled Steel Tubular Members in Bending. Light-Weight Steel and Aluminium Structures. ICSAS'99. Eds. P. Make-

lainen, P. Hassinen.-Elsevier, 1999. p.755-760.
10. JCSS (Joint Committee on Structural Safety). Probabilistic Model Code: Part 1 – Basis of design. 12th draft, 2000.-62p.
11. **Ellingwood, B.R.** Wind and snow load statistics for probabilistic design. -J. Struct Div, ASCE, 1981, 107(7), p.1345-1349.
12. Eurocode 1, EN 1990. Basis of Design and Actions on Structure.-CEN: Brussels, 2002.-87p.
13. Eurocode 3, EN 1993-1-1. Design of Steel Structures – Part 1-1: General Rules and Rules for Buildings.-CEN: Brussels, 2004.-91p.
14. **Kliukas, R., Kudzys, A.** Probabilistic durability prediction of existing building elements.-J. of Civil Engineering and Management.-Vilnius, 2004, X (2), p.107-112.
15. **Rosowsky, D., Ellingwood B.** Reliability of wood systems subjected to stochastic live loads.-Wood and Fiber Sci, 1992, 24(1), p.47-59.
16. **Vrouwenvelder, A.C.W.M.** Developments towards full probabilistic design codes. -Structural Safety. 2002, 24(2-4), p.417-432.
17. ISO 2394. General Principles on Reliability for Structures, 2nd ed.-Switzerland, 1998.-74p.
18. **Ellingwood, B.R., Tekie, P.B.** Wind load statistics for probability-based structural design.-J Struct Eng. -ASCE, 1999, 125(4), p.453-463.
19. **Kvedaras, A.K., Kudzys, A.** On performance and safety of composite steel-concrete framework. -Eurosteel, Proceedings, 2005. v.A, p.1.7-17-1.7-23.
20. **Kudzys, A.** On account of concurrent rectangular pulse actions in structural design. ESREL 2005, Advances in Safety and Reliability – Kolowrocki (ed.).-London: Taylor and Francis Group, 2005. p.1203-1206.

A. Kudzys, A.K. Kvedaras

VAMZDINIŲ KOMPOZITINIŲ ELEMENTŲ KONSTRUKCINIO ATSPARUMO IR SAUGOS TYRIMAS

R e z i u m ė

Aptariamos betono pripildytų plieninių vamzdžių teigiamybės ir tinkamumas konstrukcijose. Analizuojamas būtinumas ištirti vamzdinių kompozitinių konstrukcijų eksploatacinę kokybę ir patikimumą. Nagrinėjamas žiedinio skerspjūvio naujoviškų vamzdinių elementų tvirtinimas ir konstrukcinė sauga. Nagrinėjami ašiniai ir ekscentriškai gniuždomų vamzdinių kompozicinių elementų ribinio būvio kriterijai. Pateikta laikui bėgant kintanti elementų saugos riba ir jos komponentai kaip atsitiktinio atspario ir įrąžos procesai. Elementų išlikties tikimybės skaičiavimas remiasi atsitiktinės sekos, sutartinio atspario ir transformuotų sąlyginių tikimybių koncepcijomis ir iliustruojamas pavyzdžiu.

A. Kudzys, A.K. Kvedaras

RESEARCH ON STRUCTURAL RESISTANCE AND SAFETY OF TUBULAR COMPOSITE MEMBERS

S u m m a r y

An advantage and application of the concrete-filled steel tubes in structural practice is discussed. An urgency to investigate the structural performance and reliability of tubular composite structures is analysed. The robustness and structural safety of a new type of tubular members of annular cross-sections are considered. Limit state criteria of tubular composite members under axial and eccentric compression are investigated. The time-dependent safety margin of the members and its components as random resistance and action effect processes are presented. The survival probability analysis of the members is based on the concepts of random sequences, conventional resistances and transformed conditional probabilities and illustrated by a numerical example.

А. Кудзис, А.К. Квядарас

ИССЛЕДОВАНИЕ КОНСТРУКЦИОННОГО СОПРОТИВЛЕНИЯ И БЕЗОПАСНОСТИ ТРУБЧАТЫХ СОСТАВНЫХ ЭЛЕМЕНТОВ

Р е з ю м е

Обсуждаются достоинство и пригодность в конструкциях стальных труб заполненных бетоном. Анализируется неотложность исследования эксплуатационного качества и надежности трубчатых составных конструкций. Исследованию подвергаются прочность и конструкционная безопасность новых трубчатых элементов кольцевого сечения. Изучаются критерии предельного состояния трубчатых составных элементов подвергаемых осевому и внецентренному сжатию. Приводится изменяющийся во времени предел безопасности элементов и компонентов в виде случайных процессов сопротивления и нагрузки. Расчет элементов на безопасность опирается на концепциях случайной последовательности, общепринятого сопротивления и преобразованных условных вероятностей и иллюстрируется примером.

Received February 28, 2006