Wind and Structures, Vol. 12, No. 5 (2009) 425-439
DOI: http://dx.doi.org/10.12989/was.2009.12.5.425

On the member reliability of wind force-resisting steel frames designed by EN and ASCE rules of load combinations

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(Received March 11, 2008, Accepted May 13, 2009)

Abstract. The expediency of revising universal rules for the combination of gravity and lateral actions of wind force-resisting steel structures recommended by the Standards EN 1990 and ASCE/SEI 7-05 is discussed. Extreme wind forces, gravity actions and their combinations for the limit state design of structures are considered. The effect of statistical uncertainties of extreme wind pressure and steel yield strength on the structural safety of beam-column joints of wind force-resisting multistory steel frames designed by the partial factor design (PFD) and the load and resistance factor design (LRFD) methods is demonstrated. The limit state criterion and the performance process of steel frame joints are presented and considered. Their long-term survival probability analysis is based on the unsophisticated method of transformed conditional probabilities. A numerical example illustrates some discrepancies in international design standards and the necessity to revise the rule of universal combinations of loads in wind and structural engineering.

Keywords: wind engineering; wind-resisting frames; wind forces; combinations of actions; beam-column joints; structural safety.

1. Introduction

The Standards EN 1990 (2002) (in Europe) and ASCE/SEI 7-05 (2005) (in the USA) require that structures of buildings shall be designed with appropriate degrees of reliability. These standards are based on the limit state concept and, respectively, on the methods of the partial factor design (PFD) and the load and resistance factor design (LRFD). Researchers and designers have been doubting the adequacy of some backgrounds, requirements, design assumptions and approaches presented in these design codes and the International Standard ISO 2394 (1998). Sometimes it is difficult to perceive how it is possible to verify the reliability of structures subjected to complicated gravity and lateral actions by using their universal combination rules and generalized factors for loads and material properties. Practically, the reliability degree of structures designed by limit state and probabilistic concepts can be markedly different due to some conditionality of combination rules for variable actions of transient situations. This degree may be objectively defined only by full probability-based concepts and models.

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The roots of unexpected failures and residual deformations of structures can be traced back not only to gross human design or construction errors. They can be caused by some incorrect reliability issues, which can hardly be formulated and solved by limit state design methods. Ellingwood and Tekie (1999), Li and Li (2003) suggested that the statistical models of wind loads and factors achieving reliability degrees consistent with designs governed by wind and gravity load combinations recommended by ASCE 7 should be revised. Therefore, it should be expedient to corroborate some data on the reliability analysis of load-carrying structures designed by the recommendations of Standards EN 1990 and ASCE/SEI 7-05.

The main task of this paper is to prove that the rules of universal combinations of loads recommended by Standards EN 1990 and ASCE/SEI 7-05 may be used in the limit state analysis of wind force-resisting structures very cautiously. The overall structural reliability of multistory frame systems depends on their long-term response to extreme wind loads. This response is closely related to the load-carrying capacity, ductility and safety of beam-column joints. Panel zones as the main components of beam-column joints of multistory sway steel frames have a decisive effect on their performance during windstorms. The analysis is focused on the elastic behavior of beam-column joints of uniplanar frames of buildings situated in non-hurricane prone regions. Their elasticity helps us avoid the influence of stress redistributions on probabilistic analysis results. Analogous results may be obtained in the reliability analysis of other members of wind force-resisting structures. The numerical example allows us raise doubts about the combinations of wind and gravity loads and the generalized factors for wind loads and steel yield strength.

2. Actions and their combinations

2.1. Extreme wind forces

According to ISO 4354 (1997), the resultant lateral wind force, W, depends on the intensity of wind velocity, its aerodynamic pressure and its fluctuating pressure induced by the motion of structure due to the wind. The coefficient of variation of this force, δ_w, depends on the uncertainties of the said above quantities. The calculation results have confirmed the basic influence of these factors on the structural response of buildings and the value of coefficient of variation δ_w (Bosak and Flaga 1988).

It is proposed by JCSS (2000) to model the annual extreme storm forces, W, as an intermittent rectangular pulse renewal process using the Gumbel or extreme value Type 1 distribution law. This law is also recommended in the USA (Ellingwood 1981), in Russia (Raizer 1998) and by many researchers. At any case, the Type 1 is quite appropriate for the probabilistic analysis of structures exposed to extreme wind action effects in transient design situations of buildings. It is implanted explicitly in the EN 1990 (2002), ISO 2394 (1998), JCSS (2000) and many national design codes. According to the results obtained by using the bivariate estimation of extreme wind speeds as an additional mathematical tool, it is expedient to model the probability distribution of wind forces by this distribution law (Escalante-Sandoval 2008).

The mean, W_m, and characteristic (nominal), W_k, values and the load factor γ_w for the extreme wind forces are related subjectively to the target reliability index $\beta_{\min}$, its calibration rules, and they all objectively depend on wind load uncertainties. Significant variations in the estimates of the force W are corroborated by the analysis of international codes and standards (Zhou, *et al*. 2002). Any design of wind force-resisting structures needs to have more exactly defined aerodynamical, mechanical and statistical parameters of wind loads depending on the local terrain roughness. The coefficient of variation of annual extreme wind velocity, δ_v, is equal to 0.10-0.35 (JCSS 2000). The strong wind pressure conditions are characterized by smaller values of these coefficients. The coefficients of variation of other engineering quantities, δ_c, used in wind loading analysis are between 0.1 and 0.2 (ISO 4354 1997). Practically the coefficient of variation, of the annual extreme wind force $\delta_w = (\delta_v^2 + \sum \delta_v^2)^{1/2}$ is between 0.3 and 0.4.

2.2. Gravity actions

The permanent (dead) G, sustained Q_1, and extraordinary Q_2 live gravity loads may also have a significant effect on the reliability of wind force-resisting structures. The permanent load G can be described by a normal distribution (EN 1990 2002, ISO 2394 1998, JCSS 2000). The components Q_1 and Q_2 are modeled as time-variant stochastic processes. Lognormal, Weibull and Gamma distributions may be used for loads Q_1 and an extreme value distribution for loads Q_2 (ISO 2394 1998, JCSS 2000). The means and coefficients of variation of live floor loads per unit area of office buildings are: $Q_{1m} \approx 0.5\text{-}0.6$ kPa, δ_{Q_1}=50-90%, $Q_{2m} \approx 0.2\text{-}0.3$ kPa, δ_{Q_2}=50-110% (JCSS 2000, Chou and Thayaparan 1988).

Melchers (1999), Ellingwood and Tekie (1999) recommended modeling the sum maximum of gravity sustained and extraordinary floor action effects during a 50-year period according to the Type 1 distribution of largest values. The durations, d, and renewal rates, λ, of maximum extraordinary live loads and extreme wind forces are: $d_{Q_2} = 1\text{-}3$ days, $d_W = 8\text{-}12$ hours and $\lambda_{Q_2} = \lambda_W = 1.0$/year (JCSS 2000). The average recurrence number of coincident annual extreme gravity and wind loads is:

$$n_{Q_2W} = t(d_{Q_2} + d_W)\lambda_{Q_2}\lambda_W \tag{1}$$

For the t = 50-year reference period, the number n_{Q_2W} calculated by Eq. (1) is equal to 0.2-0.5 and the average recurrence numbers of Q_2 simultaneously on two and three stories of multistory buildings $n_2 = 2td_{Q_2} = 0.27\text{-}0.82$ and $n_3 = 3td_{Q_2}^2 = 0.001\text{-}0.01$ are rather small. Besides, the sum of gravity column forces $N_G + N_{Q_1}$ of multistory buildings markedly dominates over the component N_{Q_2}. In this case, the probability distribution of the resultant gravity force of columns

$$N_{gr} = N_G + N_{Q_1} + N_{Q_2} \tag{2}$$

is close to the normal or lognormal one with the coefficient of variation expressed as:

$$\delta_{N_{gr}} = [(\delta_G \times N_{Gm})^2 + (\delta_{Q_1} \times N_{Q_1m})^2 + (\delta_{Q_2} \times N_{Q_2m})^2]^{1/2} / N_{gr,m} \tag{3}$$

In this case, the conventional nominal value of compressive gravity force $N_{gr,n} = N_{gr,k} = N_{gr,m}(1 + \beta_{0.95}^{G}\delta_{N_{gr}})$ is close to the sum of the component forces $N_{Gn} + N_{Q_1n} + N_{Q_2n} = N_{Gm} + N_{Q_1m}(1 + \beta_{0.95}^{Q_1}\delta_{Q_1}) + N_{Q_2m}(1 + \beta_{0.95}^{Q_2}\delta_{Q_2})$, where $\beta_{0.95}$ are the 0.95 – fractile (quantile) of standardized distributions of gravity loads varying in time in different random manners.

All above-mentioned facts testify that it is expedient to analyze the reliability of columns and beam-column joints of multistory frames using the integrated gravity force N_{gr} (Eq. (2)). It helps us use simplified but fairly exact design procedures and diminish a discrepancy between the intricate theoretical concepts of full probabilistic model codes and the practical possibilities of designers. Besides, the normal probability distribution of the force N_{gr} with the coefficient $\delta_{N_{gr}} = 0.20$ was considered in the investigations on structural safety of multistory buildings carried out by Foschi, *et al*. (2002).

2.3. Action combinations

According to the recommendation of EN 1990 (2002) and EN 1993-1-1 (2003), the load combination for wind-resistant limit state design is:

$$U_1 = \gamma_G G_k + \gamma_W W_k + \psi_{oQ}\gamma_Q Q_k \tag{4}$$

where W_k , G_k and Q_k are the characteristic values of wind, permanent and variable gravity actions; γ_G=1.35, γ_W=1.5 and γ_Q=1.5 are the partial safety factors for actions; ψ_{oQ}=0.7 is the factor for imposed loads in buildings.

According to the ASCE/SEI 7-05 (2005) directions, the load combination for strength limit state design of structures subjected to the same loads is:

$$U_2 = \gamma_G G_n + \gamma_W W_n + \gamma_Q Q_n \tag{5}$$

where G_n, W_n and Q_n are the nominal (characteristic) values of loads; the load factors γ_G=1.2, γ_W=1.6, γ_Q=1.0 or 0.5 (when uniformly distributed live loads are more or not more than 4.8 kN/m^2, respectively).

The characteristic values of permanent loads are their means. The characteristic value of the wind force, W_k, is based upon the probability of 0.02 of its time-varying value being exceeded for a period of one year. Since the annual extreme wind pressure distribution is modeled on the Type 1 law, the mean value of wind load may be calculated by the formula:

$$W_m = \frac{W_k}{1 + \beta_{0.98}^{w}\delta_w} = \frac{W_d}{(1 + \beta_{0.98}^{w}\delta_w)\gamma_w} \tag{6}$$

where $\beta_{0.98}^{w} = 2.593$ is the 0.98-fractile (quantile); $W_d=\gamma_w W_k=\gamma_w W_n$ is the design value of wind load.

3. Beam-column joint behavior

3.1. Role of panel zone yielding

Wind force-resisting multistory steel frames are usually assembled from wide flange double-tee or box-section columns, double-tee or box-section beams, beams-to-column plate and bolt connections. The plastic hinges of multistory steel frames should occur in their beams rather than in the columns and joints. This requirement helps us ensure the ductile performance of the building skeleton and design more rational beam-to-column connections as it is shown by Arlekar and Murty (2004). The type of these connections depends on many factors characterizing the purpose of buildings, their dimensions in plane and elevation, column spacing, the intensity of gravity and lateral forces. In all cases, the bending resistance of beam-to-column connections must not be less than that of frame beams.

The beam-column joints of frames consist of joint cores (column panel zones and flanges) and connections between beam-ends and column flanges. There exists an interrelation in the behavior of panel zones and connectors. Inelastic deformations in panel zones of joints lead to local kinks in the column flanges and cause premature failure of welded-bolted connections on the beam-to-column interface (Krawinkler 1978). The low-cycle loading of four-type exterior joints carried out by Calado (1995), Calado and Lamas (1998) have confirmed an unfavorable effect of inelastic panel zone deformations on the behavior of beam-to-column connections. The analogous role of panel zone plastic deformations in the behavior and reliability of welded beam-to-column connections and frame system flexibility is presented by Song and Ellingwood (1999).

The detailed finite-element analysis carried out by El-Tawil, *et al*. (1999) has corroborated the detrimental effect of panel zone yielding on a greater potential for fracture of welded-bolted steel connections. It has been determined that the inelastic deformations of panel zones have a significant effect on the overall ductility and resistance of the beam-column joints. In general, only the elastic behavior of panel zones can guarantee the reliable behavior of beam-to-column connections and the overall ductility of frame joints under low-cycle or intensive lateral loading. Therefore, panel zone yielding is undesirable in multistory frames subjected to extreme wind loads.

3.2. Limit state criterion for panel zones

When the elastic structural analysis is based on the linear stress/state law and performed on the initial geometric quantities, the limit state criterion of elastic panel zones (Fig. 1) may be written in the following form:

$$\sigma_{pr} = [(\sigma_{gr} + \sigma_w)^2 + \nu_\tau \tau_w^2]^{1/2} = f_y \tag{7}$$

Here σ_{pr} is the maximum principal compressive stress; f_y is the steel yield strength; $\sigma_{gr} = N_{gr}/A_c$ is the normal stress caused by gravity permanent and variable actions;

$$\sigma_w = M_w z_v / I_c + N_w / A_c = V_w h_1 z_v / I_c + N_w / A_c \tag{8}$$

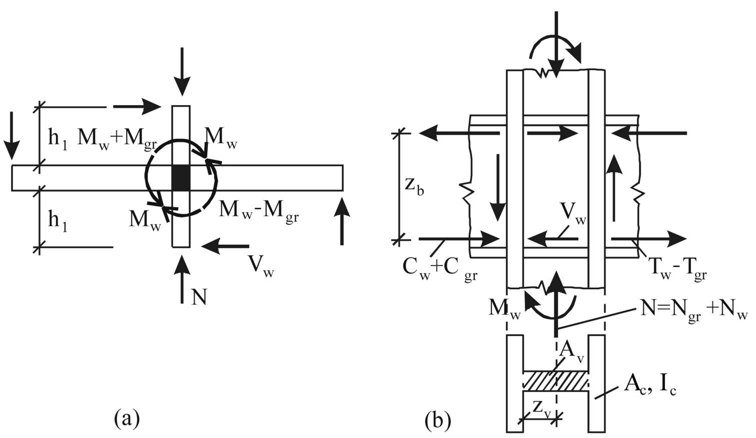

Fig. 1 The design diagram (a) and action effects (b) of interior beam-column joints of uniplanar steel frames subjected to gravity and lateral action effects

$$\tau_w = (2C_w - V_w)/A_v = V_w\left(\frac{2h_1}{z_b} - 1\right)/A_v \tag{9}$$

are the normal and shear stresses caused by wind forces; the coefficient $\nu_\tau = 3$, if the limit state analysis is based on the Huber's law, and $\nu_\tau = 4$, if the Tresca-St Venant criterion is preferred. It is logical to apply the coefficient $\nu_\tau = 3$ because the principal stress by Eq. (7) is caused by joint bending, shear and axial forces.

According to the wind tunnel experiment results presented by Zhao and Lam (2008), a strong interference effect on all structural members of a group of adjacent buildings as compared with isolated single building exists. Lim and Blenkiewicz (2007) found that the adverse aerodynamic interference effect caused by coupled twin tall buildings should be assessed and included in wind-resistant design.

Basically, any interference effect may lead to a decrease or an increase of the wind induced loads and their actions. Therefore, it is expedient to include in Eq. (7) the relative parameter $\alpha = \sigma_{gr}/\sigma_w$ characterizing an intensity of wind stresses. In this case, the role of interference effects on a probabilistic analysis model of wind force-resisting members becomes insignificant. Thus, the limit state criterion in Eq. (7) against composite compressive and shear failure of panel zones may be rewritten as follows:

$$[\sigma_w^2(\alpha + 1)^2 + 3\tau_w^2]^{1/2} = f_y \tag{10}$$

where the parameter $\alpha = \sigma_{gr}/\sigma_w$ is the random time-dependent ratio of compressive stresses caused by gravity and wind action effects depending on the features of site localities, architectural and structural concepts of buildings.

With the stresses σ_w from Eq. (8) and τ_w from Eq. (9), this above presented criterion may be expressed as:

$$\left[\left(\frac{h_1 z_v}{I_c} + \frac{N_w}{V_w A_c}\right)^2 (\alpha + 1)^2 + \frac{3}{A_v^2}\left(\frac{2h_1}{z_b} - 1\right)^2\right]^{1/2} V_w = f_y \tag{11}$$

where the geometrical quantities h_1, I_c, A_v, z_v, z_b are from Fig. 1. Thus, the shear force, which provokes steel yielding in the panel zone of beam-column joints, can be described by the following function:

$$V_w = f_y/B \tag{12}$$

where the component B is calculated by the equation:

$$B = \left[\left(\frac{h_1 z_v}{I_c} + \frac{N_w}{V_w A_c}\right)^2 (\alpha + 1)^2 + \frac{3}{A_v^2}\left(\frac{2h_1}{z_b} - 1\right)^2\right]^{1/2} \tag{13}$$

The effect of the lateral shear force on the limit state of joint panel zones depends on the parameter $\alpha = \sigma_{gr}/\sigma_w$. For two- and four-span multistory frames, the axial wind force of central interior columns $N_w \approx 0$. For interior columns of three-span frames the ratio of compressive and shear wind forces $N_w/V_w \approx 1$.

4. Structural reliability prediction

4.1. Performance criterion

It is expedient in the reliability analysis of the frame joints to change the limit state criterion in Eq. (7) into the formula:

$$\sigma_1 + \sigma_2 = f_y \tag{14}$$

Here the components of maximum principal stress may be expressed as:

$$\sigma_1 = \sigma_{gr} \tag{15}$$

$$\sigma_2 = [(\sigma_{gr} + \sigma_w)^2 + 3\tau_w^2]^{1/2} - \sigma_1 \tag{16}$$

the coefficients of variation of which are: $\delta_{\sigma_1} = \delta_{N_{gr}}$ (Eq. (3)) and $\delta_{\sigma_2} = \sqrt{D\sigma_2}/\sigma_{2m} \approx \delta_w$.

Thus, the performance process of the panel zone may be presented in the form of the safety margin function as follows:

$$M(t) = f_y - \theta_{gr}\sigma_1 - \theta_w\sigma_2(t) = f_{con} - \sigma(t) \tag{17}$$

Here

$$f_{con} = f_y - \theta_{gr}\sigma_1 \tag{18}$$

is the conventional compressive steel resistance and

$$\sigma(t) = \theta_w\sigma_2(t) \tag{19}$$

is the principal stress process caused by annual extreme wind forces, where the stresses σ_1 (Eq. (15)) and σ_2 (Eq. (16)); θ_{gr} and θ_w are the additional random variables introducing statistical uncertainties of design models, which give action effects. The means and coefficients of variation of these variables are: $\theta_{gr,m} \approx \theta_{wm} \approx 1.0$ and $\delta_{\theta_{gr}} = \delta_{\theta_w} \leq 0.10$ (Hong and Lind 1996, Gulvanessian, *et al.* 1998, Vrowenvelder 2002). In the elastic analysis of bar steel members and their joints the additional random variables θ_{gr} and θ_w may be ignored.

4.2. Survival probabilities

The conventional compressive resistance f_{con} (Eq. (18)) is a stationary function the probability distribution of which is close to the normal. The time-dependent principal stress $\sigma(t)$ (Eq. (19)) may be modeled as an intermittent rectangular pulse renewal process. Its probability distribution obeys the Gumbel distribution law. Thus, the performance process (Eq. (17)) may be treated as the finite statistically dependent random sequence and expressed as:

$$M_k = f_{con} - \sigma_k; \quad k = 1, 2, \ldots, n \tag{20}$$

where a number of sequence cuts n is the design working life of structures t_n in years.

Assuming that the joint was safe at the time less than t_k, its instantaneous survival probability at any cut k of the sequence in Eq. (20) may be calculated by the formula:

$$\boldsymbol{P}_k = \boldsymbol{P}(t_k) = \boldsymbol{P}\{f_{com} - \sigma_k > 0 \, \exists (t_k \in [t_1, t_n])\} = \int_0^{\infty} g_f(x) G_{\sigma}(x) dx \quad (21)$$

Here $g_f(x)$ is the normal density function of the variable f_{con} (Eq. (18)) with the mean $f_{con,m} = f_{ym} - \theta_{gr,m}\sigma_{gr,m}$ and the variance $Df_{con} = Df_y + \theta_{gr,m}^2(\delta_{\sigma_{gr}}\sigma_{gr,m})^2 + \sigma_{gr,m}^2(\delta_{\theta_{gr}}\theta_{gr,m})^2$;

$$G_{\sigma}(x) = \exp\left[-\exp\left(\frac{\sigma_m - x}{0.7794\sqrt{D\sigma}} - 0.5772\right)\right] \quad (22)$$

is the Gumbel distribution function of the variable $\sigma(t)$ (Eq. (19)) with the mean $\sigma_m = \theta_{wm}\sigma_{2m}$ and the variance $D\sigma = \theta_{wm}^2(\delta_w \sigma_{2m})^2 + \sigma_{2m}^2(\delta_{\theta_w}\theta_m)^2$.

A computation of instantaneous survival probabilities of members may be carried out by the numerical integration method and its computer programs. When the main uncertainty of a maximum principal compressive stress in each year comes from wind actions, according to EN 1990 (2002), the long-term survival probability of joints during the time period $[t_1, t_n]$ can be calculated by the very rough formula:

$$\boldsymbol{P}\{T \ge t_n\} = \boldsymbol{P}_k^n \approx \exp[-n(1 - \boldsymbol{P}_k)] \quad (23)$$

where the probability $\boldsymbol{P}_k$ is defined according to Eq. (21).

The random sequence of member performance should be treated as a highly correlated series system. Then, the long-term survival probability of members as its upper bound may be calculated using the numerical integration method presented by Ahammed and Melchers (2005). However, it is possible to correct Eq. (23) taking into account the statistical dependency of random sequence cuts. According to the method of transformed conditional probabilities (TCPM) (Kudzys 2007), the long-term survival probability of members as series systems with equicorrelated elements may be calculated by the following expression:

$$\boldsymbol{P}\{T \ge t_n\} = \boldsymbol{P}\left\{\bigcap_{k=1}^{n}(M_k > 0)\right\} = \boldsymbol{P}(M_1 > 0)\boldsymbol{P}(M_2 > 0 | M_1 > 0) \ldots \boldsymbol{P}\left(M_n > 0 \Big| \bigcap_{i=1}^{n-1} M_i > 0\right) \approx$$

$$\approx \boldsymbol{P}(M_1 > 0)\boldsymbol{P}(M_2 > 0)\left[1 + \rho_{kl}^a\left(\frac{1}{\boldsymbol{P}(M_1 > 0)} - 1\right)\right] \ldots \boldsymbol{P}(M_n > 0)\left[1 - \rho_{kl}^a\left(\frac{1}{\boldsymbol{P}(M_{n-1} > 0)} - 1\right)\right] =$$

$$= \boldsymbol{P}_k^n\left[1 + \rho_{kl}^a\left(\frac{1}{\boldsymbol{P}_k} - 1\right)\right]^{n-1} \quad (24)$$

Here the instantaneous survival probability of members $\boldsymbol{P}_k$ is calculated by Eq. (21);

$$\rho_{kl} = Cov(M_k, M_l)/(\sqrt{DM_k} \times \sqrt{DM_l}) = 1/(1 + D\sigma/Df) \quad (25)$$

is the coefficient of correlation of sequence cuts, where Df by Eq. (18) and $D\sigma$ by Eq. (19) are the variances of safety margin components. The correlation factor, ρ_{kl}^a, may be expressed from Eq. (24) as:

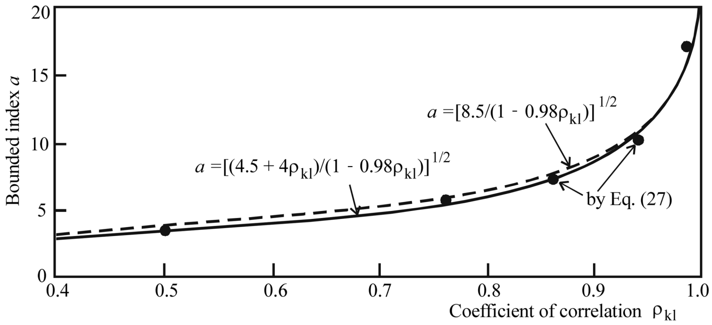

Fig. 2 The bounded index a versus the coefficient of correlation ρ_{kl} of equicorrelated cuts of random sequences of safety margins of particular members

$$\rho_{kl}^{a} = \left\{\left[\frac{\boldsymbol{P}(T \geq t_n)}{\boldsymbol{P}_k^n}\right]^{1/(n-1)} - 1\right\} \Big/ \left(\frac{1}{\boldsymbol{P}_k} - 1\right) \tag{26}$$

where its bounded index is:

$$a = \ln \rho_{kl}^{a} / \ln \rho_{kl} \tag{27}$$

According to our investigation data, the bounded index may be defined as:

$$a \approx \left(\frac{4.5 + 4\rho_{kl}}{1 - 0.98\rho_{kl}}\right)^{1/2} \approx \left(\frac{8.5}{1 - 0.98\rho_{kl}}\right)^{1/2}$$

According to Fig. 2, the position of Monte Carlo simulation points by Eq. (27) showed good agreement with the continuous and dotted curves of unsophisticated TCPM.

The structural safety of members may also be expressed by the following generalized reliability index:

$$\beta = \Phi^{-1}(\boldsymbol{P}\{T \geq t_n\}) = -\Phi^{-1}(1 - \boldsymbol{P}\{T \geq t_n\}) \tag{28}$$

where Φ is the cumulative distribution function of the standardized normal distribution. According to EN 1990 (2002), the load-carrying structures of residential and public buildings belong to the reliability class RC2 that is defined by the target value of the reliability index $\beta_{min} = 3.8$ for a 50-year reference period. In this case, the multiplication factor for unfavorable partial actions $K_{F1} = 1.0$. According to Ellingwood and Tekie (1999), for such buildings in the USA the target reliability index, β_{min}, should be equal to 3.2. These target index values correspond to the failure probabilities of members close to 0.00007 and 0.0007, respectively. This difference in target indices of international design codes is undesirable for structural engineers.

5. Numerical reliability analysis

5.1. Analysis parameters

The numerical analysis is considered as an illustration of reliability assessments of the joints of beams and central columns of wind-resisting multistory steel frames of Class RC2 by EN 1990 or Category II by ASCE/SEI 7-05 designed according to the load combination rules recommended by European and American standards. The geometrical properties of the wide flange double-tee column section (Fig. 1) are: $A_c = 394.4\times10^{-4}$ m^2, $A_v = 256.3\times10^{-4}$ m^2, $z_v = 0.175$ m, $I_c = 111.8\times10^{-8}$ m^4, $z_b = 0.7$ m, $h_1 = 1.40$ m.

The nominal and design values of the yield strength of the structural steel grade *Fe* 430 (EN 10025) are: $f_{yn} = f_{yk} = 275$ MPa and $f_{yd} = 275/1.1 = 250$ MPa. The probability distributions of conventional compressive resistance of steel f_{con} by Eq. (18) is close to a normal distribution. Two coefficients of variation $\delta_{f_y} = 6\%$ and 10% and, respectively, two means of the yield strength, f_{ym}, equal to 305.12 MPa and 329.15 MPa are taken into account. The mean and coefficient of variation of the resultant gravity load p are:

$$P_{gr,m} = G_m + Q_{1m} + Q_{2m} = 4.8 + 0.6 + 0.4 = 5.8 \text{ kPa},$$

$$\delta_{P_{gr}} = [(\delta_g \cdot G_m)^2 + (\delta_{q1} \cdot Q_{1m})^2 + (\delta_{q2} \cdot Q_{2m})^2]^{1/2} / P_{gr,m} =$$

$$= [(0.1 \times 4.8)^2 + (0.5 \times 0.6)^2 + (1 \times 0.4)^2]^{1/2} / 5.8 = 0.1195.$$

The limit state design values of shear and normal stresses of elastic panel zones of joints are presented in Table 1. The numerator and denominator values are calculated taking into account the load combinations for wind force-resisting structures given by Eqs. (4) and (5) recommended, respectively, by EN 1990 with $\gamma_Q = 0.7 \times 1.5 = 1.05$ and ASCE 7-05 with γ_Q equal to 1.0 and 0.5. Therefore, the factors for gravity load resultants are:

$$\gamma_{gr}^{EN} = (1.35 \times 4.8 + 1.05 \times 1.0)/(4.8 + 1.0) = 1.298,$$

$$\gamma_{gr}^{ASCE/1.0} = (1.2 \times 4.8 + 1.0 \times 1.0)/(4.8 + 1.0) = 1.165,$$

$$\gamma_{gr}^{ASCE/0.5} = (1.2 \times 4.8 + 0.5 \times 1.0)/(4.8 + 1.0) = 1.079.$$

The statistics of the conventional compressive steel resistance $f_{con} = f_y - \theta_{gr}\sigma_1$ and principal stresses $\sigma = \theta_w\sigma_2$ caused by extreme wind loads are: $f_{con,m} = f_{ym} - \sigma_{1m}$ and $Df_{con} = Df_y + D(\theta_{gr}\sigma_1)$; $\sigma_m = \sigma_{2m}$ and $D\sigma = D\sigma_2 + \sigma_{2m}^2 \times D\theta_w$, where the means and variances of the additional random variables of action effects are equal to $\theta_{gr,m} = \theta_{wm} = 1.0$ and $D\theta_{gr} = D\theta_w = 0.01$.

The long-term survival probabilities, $\boldsymbol{P}\{T \geq t_n\}$ of frame joints are calculated by Eq. (24).

5.2. Analysis results and reliability assessment

The main results as the generalized reliability index β (Eq. (28)) of analyzed frame joints exposed to compressive and shear wind stresses are presented in Figs. 3 and 4. The continuous curves belong to joints of considered steel frames designed according to EN 1990 (Fig. 3) and ASCE 7-05 (with the gravity variable load factor $\gamma_Q = 1.0$) (Fig. 4) load combinations. The dotted curves presented in Figures belong to the members designed according to ASCE 7-05 load combination rules with the factor $\gamma_Q = 0.5$. The target reliability indices for a 50-year reference period are represented by the broken horizontal lines with $\beta_{\min} = 3.8$ and $\beta_{\min} = 3.2$ for structures designed by PFD and LRFD methods, respectively. The intensity of the design wind stresses is expressed as the factor:

Table 1 The design and mean values of shear and compressive stresses of joint panel zones

Values	δ_w	γW	$\psi_{oQ}\gamma_Q$	Wind stress intensity variants 1	2	3	4	5	6
$\nu_d = \sigma_{wd}/(\sigma_{gr,d} + \sigma_{wd})$	-	-	-	0	0.0833	0.167	0.333	0.500	0.667
$\alpha_d = \sigma_{gr,d}/\sigma_{wd}$	-	-	-	0	11.000	5.000	2.000	1.000	0.500
$V_{wd} = f_{yd}/B$, MN by Eq. (12)	-	-	-	0	0.0948	0.188	0.363	0.518	0.647
$\tau_{wd} = V_{wd}(2h_1/z_b - 1)A_v$, MPa	-	-	-	0	11.10	22.00	42.53	60.63	75.78
$\sigma_{wd} = V_{wd}\, h_1 z_v/I_c$, MPa	-	-	-	0	20.77	41.19	79.63	113.45	141.85
$\sigma_{gr,d} = \sigma_{wd}\times\alpha_d$, MPa	-	-	-	250.0	228.49	205.95	159.26	113.45	70.92
$\tau_{wm} = \dfrac{\tau_{wd}}{(1+\beta^{w}_{0.98}\delta_w)\gamma_w}$, MPa	0.3	1.5	-	0	4.16	8.25	15.95	22.74	28.42
		1.6		0	3.90	7.73	14.95	21.32	26.64
	0.4	1.5	-	0	3.63	7.20	13.92	19.84	24.80
		1.6		0	3.41	6.75	13.05	18.60	23.25
$\sigma_{wm} = \dfrac{\sigma_{wd}}{(1+\beta^{w}_{0.98}\delta_w)\gamma_w}$, MPa	0.3	1.5	-	0	7.79	15.45	29.86	42.54	53.19
		1.6		0	7.30	14.48	28.00	39.89	49.87
	0.4	1.5	-	0	6.80	13.48	26.06	37.13	46.43
		1.6		0	6.37	12.64	24.43	34.81	43.52
$\sigma_{1m} = \sigma_{gr,m} = \sigma_{gr,d}/\gamma_{gr}$, MPa	-	-	1.05	192.56	176.01	158.64	122.68	87.39	54.63
			1.0	214.50	196.05	176.70	136.64	97.43	60.85
	-	-	0.5	231.63	211.70	190.82	147.56	105.11	65.71
$\sigma_{pr,m} \approx [(\sigma_{gr,m}+\sigma_{wm})^2 + 3\tau^2_{wm}]^{1/2}$, MPa	0.3	1.5	1.05	192.56	183.94	173.92	155.02	135.77	118.53
		1.6	1.00	214.50	203.76	191.65	166.66	142.19	119.96
		1.6	0.50	231.63	219.10	205.74	177.46	149.63	124.45
	0.4	1.5	1.05	192.56	182.91	172.57	150.68	129.18	109.81
		1.6	1.00	214.50	202.51	189.70	162.65	136.11	111.88
		1.6	0.50	231.63	218.15	203.79	173.47	143.58	116.42
$\sigma_{2m} = \sigma_{pr,m} - \sigma_{1m}$, MPa	0.3	1.5	1.05	0	7.93	15.28	32.34	48.38	63.90
		1.6	1.00	0	7.71	14.95	30.02	44.76	59.11
		1.6	0.50	0	7.40	14.92	29.90	44.52	58.74
	0.4	1.5	1.05	0	6.90	13.93	28.00	41.79	55.18
		1.6	1.00	0	6.46	13.00	26.01	38.68	51.03
		1.6	0.50	0	6.45	12.97	25.91	38.47	50.71

$$\nu_d = \sigma_{wd}/(\sigma_{gr,d} + \sigma_{wd}) \tag{29}$$

where σ_{wd} and $\sigma_{gr,d}$ are the design values of normal stresses caused by wind and gravity loads. Figs. 3 and 4 show that the reliability indices of considered joints greatly depend on the factor of design wind stress intensity ν_d (Eq. (29)).

When the normal stresses of structural members caused by gravity loads dominate ($\nu_d \le 0.4$), the continuous and dotted curves of reliability indices, β, representing EN 1990 and ASCE 7-05 (with $\gamma_Q = 0.5$), respectively, load combination effects on the safety of wind force-resisting structures of multistory office and residential buildings are fairly alike in their shape but rather different in their

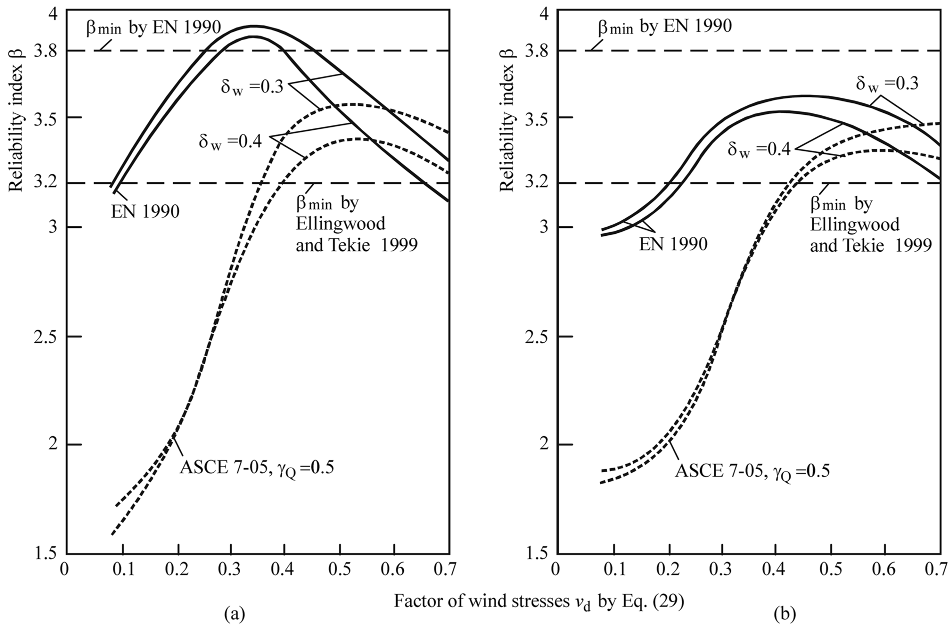

Fig. 3 Reliability indices β by EN and ASCE load combinations versus wind factor ν_d when δ_{f_y} is equal to 6% (a) and 10% (b)

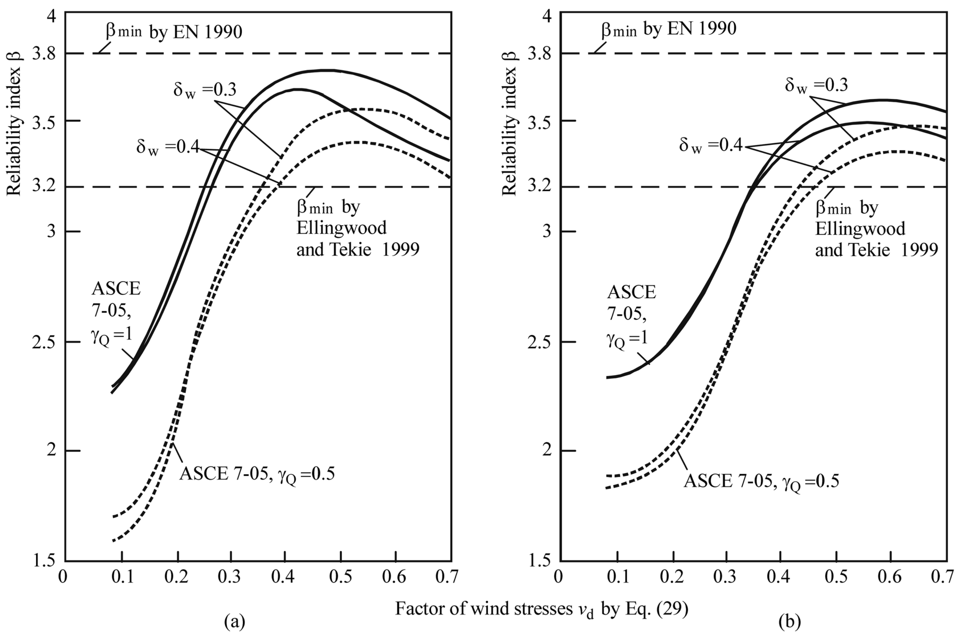

Fig. 4 Reliability indices β by ASCE load combinations versus wind factor ν_d when δ_{f_y} is equal to 6% (a) and 10% (b)

position (Fig. 3). The analysis of wind-force resisting steel structures by EN and ASCE load combinations may lead to fairly equal values of reliability indices only when lateral wind forces distinctly dominate, i.e. when the factor ν_d (Eq. (29)) is more than 0.4. Therefore, the predicted failure probabilities of designed structural members may exceed their target values by a factor of many times. It is not difficult to make sure that the failure probability of wind force-resisting structures designed according to EN 1990 and ASCE 7-05 directions may be higher, respectively, by the factors of 20 and 50 times than, respectively, it was planned by authors of PFD and LRFD methods. The curves in Fig. 3 urge us to approve of the substantiated recommendations of Ellingwood and Tekie (1999) claiming that the use of a single wind load factor, γ_w, for all wind force-resisting structures, regardless of a site, is not suitable to guarantee the sufficient desirable safety of buildings and construction works.

Despite the features of load combinations recommended according to EN 1990 and ASCE 7-05 design codes, Figs. 3 and 4 show that the negative effect of increased variation of wind loads, $\delta_w = 0.4$, on the structural reliability index of members is significant when the wind intensity factor ν_d (Eq. (29)) is more than 0.4. Besides, the uncertainty of steel yield strength characterized by its coefficient of variation, δ_{f_y}, may significantly decrease the structural safety of members of wind force-resisting structures. Fig. 4 shows that the factor of gravity variable load $\gamma_Q = 0.5$ is inadmissible to use in ASCE 7-05 load combinations.

6. Conclusions

The combinations of wind and gravity loads recommended by the existing PFD (in Europe) and LRFD (in the USA) methods presented in Standards EN 1990 and ASCE 7-05 for the limit state design of load-carrying structures cannot be treated as universal regulations of wind and structural engineering. The absolute values of reliability indices, β by Eq. (28), of structural members designed by these methods may be different in principle. In certain cases, these load combinations cannot guarantee the expected sufficient reliability of designed structural members of multistory buildings and construction works subjected to gravity loads and extreme lateral wind forces as rectangular pulse renewal processes. The failure probability of members of wind force-resisting structures designed according to the directions of these International Standards may exceed its target value by the factor of 20-50 and more.

The probability-based analysis data have shown that the reliability degree of members of wind force-resisting steel frames designed by limit state approaches significantly depend on the wind intensity factor, ν_d, expressed by Eq. (29), statistical uncertainties of extreme wind forces and steel yield strength. When this factor is equal to 0.4 and less, i.e. design the gravity stresses distinctly dominate, the reliability indices of members designed according to PFD and LRFD methods are inadmissibly too small. When the factor ν_d is more than 0.4, i.e. when the design extreme wind stresses distinctly dominate, the different load combination rules leads approximately to the same values of the reliability index. However, in all cases it is inexpedient to use ASCE 7-05 load combinations with the factor of gravity variable loads, γ_Q, equal to 0.5.

Despite the differences in load combinations recommended according to Standards EN 1990 and ASCE 7-05, an increase in the uncertainties of wind loads may, usually, slightly decrease the reliability index of members. However, their reliability index and at the same time their structural safety degree may decrease significantly, when an uncertainty of member resistance or the coefficient of variation of steel yield strength, δ_{f_y}, increases.

The target reliability index, β_T, depends on the consequences of failure or malfunction of the structure and is associated with the loss of human life. Therefore, the great difference between its target values 3.8 and 3.2 for structural members designed by PFD and LRFD methods, respectively, seems to be unacceptable from the standpoint of humaneness.

It is not complicated to predict the long-term reliability index for steel frame joints and other members of wind force-resisting structures by simplified but fairly exact probability-based approaches including the method of transformed conditional probabilities demonstrated by Eq. (24). The unsophisticated approaches will help designers understand the merit of probabilistic reliability predictions and avoid black box engineering the undesirability of which is splendidly phrased by Sexsmith and Hirata (2002).

References

Ahammed, M. and Melchers, R.E. (2005), "New bounds for highly correlated structural series systems", *Proc. of ICOSSAR 2005*, G. Augusti, G.J. Schuëller, M. Campoli (eds), Millpress, Roterdam, 3656-3662.

Arlekar, J.N. and Murty, C.V.R. (2004), "Improved truss model for design of welded steel moment-resisting frame connections", *J. Struct. Eng.* ASCE, **130**(3), 198-510.

ASCE/SEI 7-05 (2005), *Minimum design loads for buildings and other structures*, American Society of Civil Engineers, Structural Engineering Institute, Reston, Virginia.

Bosak, G. and Flaga, A. (1998), "Response of high-rise buildings to wind action in conditions of aerodynamical and mechanical feedbacks", *Proc. of 2nd EECWE'98*, **2**, 399-406.

Calado, L. (1995), "Experimental research and analytical modeling of the cyclic behavior of bolted semi-rigid connections", *Steel Structures-Eurosteel 95*, A.A.Balkema, Roterdam, Brookfield, 541-554.

Calado, L. and Lamas, A. (1998), "Seismic modeling and behavior of steel-to-beam connections", *J. Constr. Steel Res.*, **46**(1-3).

Calgaro, J.A. and Gulvanessian, H. (2001), "Management of reliability and risk to the Eurocode system", *Safety, Risk and Reliability – Trends in Engineering*, Malta, 155-160.

Chou, K.C. and Thayaparan, P.A. (1998), "Nonlinear structural response to combined live load processes", *J. Struct. Eng.* ASCE, **114**(5), 1135-1151.

Ellingwood, B.R. (1981), "Wind and snow load statistics for probabilistic design", *J. Struct. Div.* ASCE, **107**(7), 1345-1349.

Ellingwood, B.R. and Tekie, P.B. (1999), "Wind load statistics for probability-based structural design", *J. Struct. Eng.* ASCE, **125**(4), 453-463.

El-Tawil, S., Vidarsson, E., Mikesell, T. and Kunnath, S.K. (1999), "Inelastic behavior and design of steel panel zones", *J. Struct. Eng.* ASCE, **125**(2), 183-193.

EN 1990 (2002), *Eurocode – Basis of structural design*, CEN, Brussels.

EN 1993-1-1 (2003), *Eurocode – Design of steel structures*, CEN, Brussels.

Escalante-Sandoval, C. (2008), "Bivariate estimation of extreme wind speeds", *Struct. Saf.*, **30**(6), 482-492.

Foschi, R.O., Li, H. and Zhang, J. (2002), "Reliability and performance-based design: a computational approach and applications", *Struct. Saf.*, **24**(2-4), 205-218.

Gulvanessian, H., Holicky, M., Cajot, L.G. and Schleich, J.B. (1998), "Reliability analysis of a steel beam under fire design situation", *Eurosteel, 2nd European Conf. on Steel Structures*, Praha.

Holmes, J.D., Kasperski, M., Miller, C.A., Žuranski, J.A. and Choi, E.C.C. (2005), "Extreme wind prediction and zoning", *Wind Struct.*, **8**(4), 269-281.

Hong, H.P. and Lind, N.C. (1996), "Approximate reliability analysis using normal polynomial and simulation results", *Struct. Saf.*, **18**(4), 329-339.

ISO 2394 (1998), *General Principles on Reliability for Structures*, Switzerland.

ISO 4354 (1997), *Wind actions on structures*, Switzerland.

JCSS (2000), *Probabilistic Model Code: Part 1 – Basis on Design*, Joint Committee on Structural Safety, 1-62.

Kasperski, M. and Geurts, C. (2005), "Reliability and code level", *Wind Struct.*, **8**(4), 295-307.

Krawinkler, A. (1978), “Shear in beam-column joints in seismic design of steel frames”, *Eng. J.* AISC, **3**, 82-91.
Kudzys, A. (2007), “Transformed conditional probabilities in analysis of stochastic sequences”, *Summer Safety and Reliability Seminars*, SSARS 2007, Gransk-Sopot, Poland, 243-250.
Li, J.J. and Li, G.Q. (2003), “Reliability-based integrated design of steel portal frames with tapered members”, *Struct. Saf.*, **26**(2), 221-239.
Lim, J. and Blenkiewicz, B. (2007), “Wind-induced response of structurally coupled twin tall buildings”, *Wind Struct.*, **10**(4), 383-398.
Melchers, R.E. (1999), *Structural reliability analysis and prediction*, Chichester, John Wiley.
Raizer, V.D. (1998), *Theory of Reliability in Structural Design*, ACB Publishing House, Moscow.
Sexsmith, R.G. and Hirata, K. (2005), “Future issues for codes and standards”, *ICOSSAR 2005*, G. Augusti, G. J. Schuëler, M. Campoli, Millpress, Rotterdam, 3225-3230.
Song, J. and Ellingwood, B.R. (1999), “Seismic reliability of special moment steel frames with welded connections”, *J. Struct. Eng.* ASCE, **125**(4), 357-371.
Vrouwenvelder, A.C.W.M. (2002), “Developments towards full probabilistic design codes”, *Struct. Saf.*, **24**(2-4), 417-432.
Zhao, J.G. and lam, K.M. (2008), “Interference effects in a group of tall buildings closely arranged in an L- or T-shaped pattern”, *Wind Struct.*, **11**(1), 1-18.
Zhou, Y., Kijewski, T. and Kareem, A. (2002), “Along-wind load effects on tall buildings: comparative study of major international codes and standards”, *J. Struct. Eng.* ASCE, **128**(6), 788-796.

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