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THE SAFETY DESIGN METHODOLOGY OF BEAMS OF HYPERSTATIC BAR STRUCTURES

An assessment of structural and technological features in the design methodology of hyperstatic precast reinforced concrete and composite steel-concrete structures is discussed. Permanent and variable service, snow and wind loads of buildings and their extreme values are analysed. Two loading cases of precast reinforced concrete and composite steel-concrete continuous and sway frame beams as propped and unpropped members are considered. A limit state verification of hyperstatic beams by the partial factor and probability-based methods is presented. It is recommended to calculate a long-term survival probability of beams by the analytical method of transformed conditional probabilities.

Introduction. Composite steel-concrete structures utilise distinct advantages of steel and concrete components using the properties of materials as defined in Eurocode 2 [1] and Eurocode 3 [2]. A propping of horizontal members during their construction period is characteristic not only for composite structures but also for precast reinforced concrete continuous and frame beams. Hyperstatic composite and concrete systems of buildings exposed to extreme gravity and lateral actions belong to high-reliable structures. Usually, a failure probability of these systems may be assessed as subjective predicted degree of dangerous event occurrence which cannot be observed frequently.

Composite steel-concrete and cracking reinforced concrete hyperstatic structures, usually, cannot collapse without warning. The potential damage of propped and unpropped structures should be limited reducing the hazards which their members are to sustain during construction and service periods. The hazards and structural failures can be caused not only by irresponsibility and gross human errors of designers and buildings engineers but also by some imperfect of recommendations and directions presented in design codes and standards.

According to Eurocodes [1-3], the reliability required for load-carrying structures can be achieved by appropriate execution (construction-erection) and quality management measures. Unfortunately, real proposals, recommendations and specific features considering the effect of construction technology on structural safety of buildings are passed over in silence. This shortage is visually revealed in the analysis of the load-carrying capacity and safety of structures with propped and unpropped members.

It is difficult to assess quantitatively the reliability of hyperstatic systems and their members by deterministic design code recommendations. Therefore, in some cases it can lead to groundless overestimation or underestimation of the reliability of designed and existing structures. The probability-based concepts and approaches allow us to calculate quantitative reliability indices. However, it is difficult to implant the probabilistic methods in design practice due to some methodological and mathematical troubles.

The purpose of this paper is to turn an attention of structural engineers to design features of hyperstatic structures consisted of propped and unpropped members and to encourage designers having a minimum appropriate skill and experience to use the probability-based methods in their design practice.

The actions of hyperstatic bar structures. Continuous beams with three or more supports and continuous slabs belong to the simplest hyperstatic concrete structures. Continuous beams, usually, are constant in cross-section, have effective reinforcement at internal supports and may generally be analysed on the assumption that the supports provide no rotational restraints and do not transfer bending moments to the beams.

Single-storey and multi-storey sway frames as the complex hyperstatic systems are capable to response to bending and torsion moments, axial and shear forces caused not only by gravity but also by lateral variable actions. Multi-storey moment-resisting sway systems are used as load-carrying frameworks of offices, residential and industry buildings. To these systems also belongs a combination of reinforced concrete floor slabs and walls with rigid floor-wall joints. Beam-column and floor-wall joints may be treated as rigid because their deformations have no significant influence on the distribution of internal moments and forces.

Composite steel-concrete columns, beams and slabs of hyperstatic systems consist of concrete and structural or cold-formed steel sections. The steel sections of composite beams are either continuous over internal supports or are joined by full-strength and rigid connections. The steel sections of composite beams may be propped until the concrete components are able to resist action effects (loading case *A*). The weight of concrete components may also be applied to steel beams (loading case *B*). Analogically, the precast reinforced concrete beams may be presented as propped or unpropped members. Beams of precast concrete frames may be treated as unpropped members in which the weight of floor structures is applied before beam and frame joints are able to resist action effects.

Usually, action effects of load-carrying structures of buildings are caused by the mass of erected members g_1, additional permanent mass of superstructures $g_2 = g - g_1$, time-dependent sustained $q_1(t)$ and extraordinary $q_2(t)$ or snow $s(t)$ variable loads and wind actions $w(t)$. All service loads which do not belong to sustained actions may be treated as extraordinary live load components. According to Rosowsky and Ellingwood [4], the annual extreme sum of sustained and extraordinary loads $q(t) = q_1(t) + q_2(t)$ can be modeled as an intermittent rectangular pulse process and described by a Type 1 (Gumbel) distribution with the mean $q_m = 0.47 q_k$ and coefficient of variation $\delta q = 0.58$, where q_k is the characteristic extreme load.

The probability distribution of permanent loads g is close to a Gaussian distribution with the coefficient of variation $\delta g = 0.05\text{-}0.15$. It is proposed to model the annual extreme snow and wind loads by Gumbel distribution with the coefficients of variation, respectively, $\delta s = 0.30\text{-}0.70$ and $\delta w = 0.30\text{-}0.50$ [3, 5, 6, 7, 8, 9, 10, 11, 12, 13].

The joints of propped continuous or braced frame beams are able to resist all action effects caused by permanent and variable actions. Quite the reverse, the action effects at joints of redundant systems erected with unpropped precast beams are caused only by additional permanent and all variable gravity and wind actions. The mentioned features of construction technology of hyperstatic systems have some influence on their action effects and must be assessed in their bearing capacity and structural safety analysis.

The bending moments of continuous beams. The bending moments at support (1-1) and span (2-2) sections of propped and unpropped continuous or non-sway frame beams are presented in Fig. 1. It is not difficult to satisfy oneself that the span moment of unpropped beams in construction stage may be much greater than that predicted using classical structural mechanics methods and ignoring the role of permanent load features.

The bending moment distribution given by an elastic analysis of continuous concrete and composite beams may be redistributed. The total modified bending moments of propped (loading case *A*) and unpropped (loading case *B*) middle beams are:

$$M_{1A} = M_{1g_1} + M_{1g_2} + M_{1q_1} + M_{1q_2} = \delta_A p_A l^2 / 12 \,, \tag{1}$$

$$M_{2A} = M_{2g_1} + M_{2g_2} + M_{2q_1} + M_{2q_2} = p_A l^2 \left(1/8 - \delta_A / 12 \right), \tag{2}$$

$$M_{1B} = M_{1g_2} + M_{1q_1} + M_{1q_2} = \delta_B p_B l^2/12\,, \tag{3}$$

$$M_{2B} = M^*_{2g_1} + M_{2g_2} + M_{2q_1} + M_{2q_2} = \left(p_A/8 - \delta_B p_B/12\right)\, l^2\,. \tag{4}$$

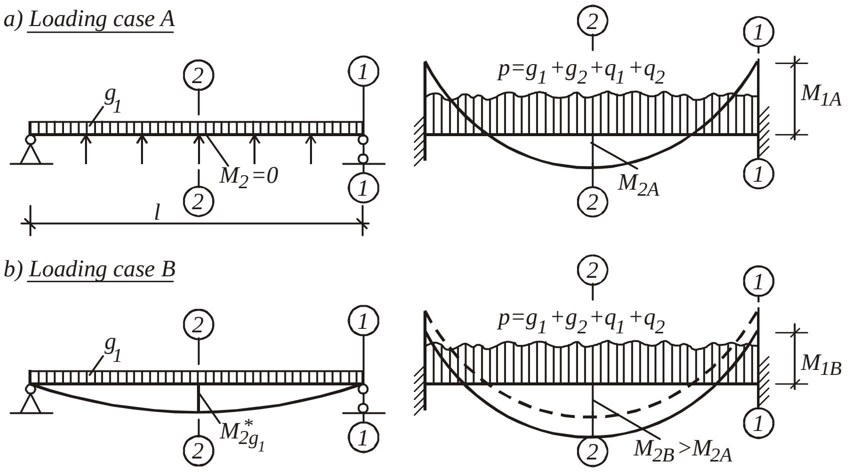

*Fig. 1***.** *Gravity loads and bending moments of middle propped (a) and unpropped (b) continuous or braced frame beams*

Here $M^*_{2g_1}$ is the bending moment of a single beam caused by permanent load g_1; $\delta_A \approx 0.8$ [14] and $\delta_B = 0.9 - 1.0$ [15] are the moment reduction factors for propped and unpropped beams, respectively;

$$p_A = g_1 + g_2 + q\,, \tag{5}$$

$$p_B = g_2 + q\,, \tag{6}$$

where $q = q_1 + q_2$ and $q = s$ are the variable loads when floor and roof beams of buildings are under consideration.

Bending moments of sway frame beams. The bending moments of sway frame beams (Fig. 2) and their redistributions are closely related to lateral wind loads.

The total modified bending moments of propped (loading case A) and unpropped (loading case B) middle frame beams are:

$$M_{1A} = \delta_A\left(p_A l^2/12 + M_w\right), \tag{7}$$

$$M_{2A} = p_A l^2\left(1/8 - \delta_A/12\right) + \left[2\delta_A^2 M_w^2/\left(p_A l^2\right)\right], \tag{8}$$

$$M_{1B} = \delta_B\left(p_B l^2/12 + M_w\right), \tag{9}$$

$$M_{2B} = \left(p_A/8 - \delta_B p_B/12\right) l^2 + \left[2\delta_B^2 M_w^2/\left(p_A l^2\right)\right], \tag{10}$$

where p_A and p_B are distributed gravity loads by (5) and (6). The quantities in square brackets may be ignored when the wind moment $M_w \le 0.02\,pl^2$.

The safety margins of continuous and frame beams. Structural reinforced concrete, steel and composite steel-concrete beams must be analysed at a sufficient number of cross and oblique sections to ensure that the requirements of design codes are satisfied at all sections along the beams and columns. The critical support and span sections may be treated as the particular members of load-carrying structures.

The performance as the safety margin process of the particular member may be written in the form:

$$Z(t) = \theta_R R - \theta_g\left(S_{g_1} + S_{g_2}\right) - \theta_q S_q(t) - \theta_w S_w(t)\,. \tag{11}$$

Here R is the member resistance; S_{g_1} and S_{g_2} are the action effects caused by the mass of the load-carrying structures and additional permanent loads, respectively; $S_q(t)$ and $S_w(t)$ are the action effects caused by the gravity and lateral extreme variable actions; θ_R, θ_g, θ_q and θ_w are the additional variables representing the uncertainties of analysis models which give the values of resistance and action effects.

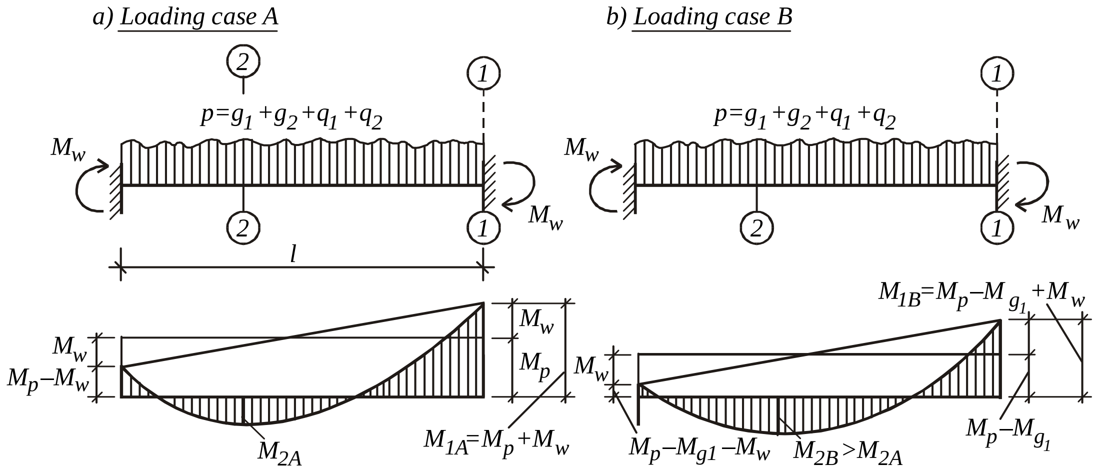

Fig. 2. Loads and bending moments of in-situ or propped (a) und unpropped (b) precast and composite middle beams of sway frames

The time-dependent performance of particular members should be assessed taking into account all construction features of precast concrete and composite hyperstatic systems. When longitudinal forces may be ignored and the loading case A is considered (Fig. 2 a), the time-dependent safety margin of the normal section 1-1 of beams can be expressed as:

$$Z_{1A}(t) = R_{1CA} - M_1(t)\,. \tag{12}$$

Here the conventional resistance R_{1CA} and the bending moment $M_1(t)$ are:

$$R_{1CA} = \theta_R R_{1A} - \theta_g \left(M_{g_1} + M_{g_2}\right)\,, \tag{13}$$

$$M_1(t) = \theta_q M_q(t) + \theta_w M_w(t)\,. \tag{14}$$

The probability distribution law of the conventional resistance R_{1CA} is close to the normal one.

When the loading case B exists (Fig. 2 b), the equations (12) and (13) can be re-expressed as:

$$Z_{1B}(t) = R_{1CB} - M_1(t)\,, \tag{15}$$

$$R_{1CB} = \theta_R R_{1B} - \theta_g M_{g_2}\,, \tag{16}$$

where the moment $M_1(t)$ by (14). For non-sway frames and continuous beams the extreme bending moment is:

$$M_1(t) = \theta_q M_q(t)\,, \tag{17}$$

or

$$M_1(t) = \theta_s M_s(t)\,, \tag{18}$$

when extreme action effects are caused by live service or snow loads.

The survival probabilities of particular and structural members. The recurrent rates of the extreme values of live service, snow and wind loads are $\lambda_q = \lambda_s = \lambda_w = 1$/year [6, 16]. Therefore, it is expedient to consider the random safety margin process of particular members as the random sequence written in the form:

$$Z_k = R_C - S_k\,, \quad k = 1,\ 2, ..., r\,, \tag{19}$$

where R_C is the conventional resistance of normal or oblique sections, S_k is their bending moment or shear force and r is the design working life of structures in years. A stochastical dependency of the sequence cuts is represented by the coefficient of correlation as:

$$\rho_{kl} = \mathrm{Cov}\left(Z_k, Z_l\right)/\left(\sigma Z_k \times \sigma Z_l\right) = 1/\left(1 + \boldsymbol{\sigma}^2 S_k / \boldsymbol{\sigma}^2 R_C\right), \tag{20}$$

where $\mathrm{Cov}(Z_k, Z_l)$ and σZ_k, σZ_l are the covariance and standard deviations of the random sequence cuts.

Resistances and action effects of beam sections may be treated as statistically independent. Therefore, their instantaneous survival probability is:

$$P_k = P\{Z_k > 0\} = P\{Z_k > 0 \quad \mathrm{E}\, t_k [t_1, t_r]\} = \int_0^\infty f_{R_c}(x) F_{S_k}(x) dx, \tag{21}$$

where $f_{R_c}(x)$ and $F_{S_k}(x)$ are the density and distribution functions of conventional resistances and action effects, respectively. Its value may be calculated by Monte Carlo simulation, the numerical integration and limit transient action effect [17] methods.

According to the method of transformed conditional probabilities (TCPM) [18, 19], the long-term survival probability of particular members may be calculated by the formula:

$$\boldsymbol{P}_i = \boldsymbol{P}_i\{Z(t) > 0\} = \left\{\bigcap_{k=1}^{r} (R_c - S_k > 0) \exists\, Z_k \in [Z_1, Z_r]\right\} = \boldsymbol{P}_k^r \left[1 + \rho_{kl}\left(\frac{1}{\boldsymbol{P}_k} - 1\right)\right]^{r-1} . \tag{22}$$

where $\boldsymbol{P}_k$ is the probability by Eq (21); r – the number of annual extreme events; ρ_{kl} – the coefficient by (20) and is its bond index

$$x = P_k \left[4.5/\left(1 - 0.98\rho_{kl}^{1/3\rho_{kl}}\right)\right]^{1/2} \tag{23}$$

The continuous and frame beams should be idealized as the auto-systems representing multicriteria failure mode due to various responses of particular members. The auto-systems of beams are characterised by stochastically dependent conventional elements in mixed connections (Fig. 3). The survival probabilities of their elements as particular members of beams may be expressed as: $\boldsymbol{P}_1 = \boldsymbol{P}\{Z_1(t) > 0\}$, $\boldsymbol{P}_2 = \boldsymbol{P}\{Z_2(t) > 0\}$, $\boldsymbol{P}_3 = \boldsymbol{P}\{Z_3(t) > 0\}$.

Due to system redundancy, the reaching of the limit state in any one normal section 1 or 2 of beams does not mean their failure. But the failure of beams in any oblique section 3 implies the failure of the auto-system.

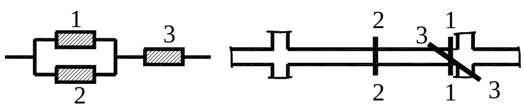

Fig. 3. Mixed auto-system representation

The stochastical dependency of auto-system elements depend on the structural concept and construction technology features of hyperstatic structures and an intensity of extreme actions. For in-situ reinforced concrete and precast or composite beams, the coefficient of correlation ρ_{12} of safety margins of particular members, usually, is equal to 0.6-1 and 0.3-0.8, respectively. The coefficients of correlation ρ_{13} and ρ_{23} are equal from 0.3 to 0.8.

According to the TCPM, the survival probability of beam normal sections 1-1 and 2-2 as the series autosystems is:

$$\boldsymbol{P}_{12} = \boldsymbol{P}_{12}\{T \geq t_r\} = \boldsymbol{P}_1 + \boldsymbol{P}_2 - \boldsymbol{P}_1\boldsymbol{P}_2\left[1 + \rho_{12}^{x}\left(1/\boldsymbol{P}_{1/2} - 1\right)\right], \tag{24}$$

where $\boldsymbol{P}_{1/2}$ is the greater value from the probabilities $\boldsymbol{P}_1$ and $\boldsymbol{P}_2$,

$$\rho_{12} = \mathrm{Cov}(Z_1, Z_2)/(\boldsymbol{\sigma} Z_1 \times \boldsymbol{\sigma} Z_2), \tag{25}$$

is the coefficient of cross-correlation of beam safety margins Z_1 and Z_2.

The total survival probability of continuous or frame beams as the mixed auto systems may be calculated by the formula:

$$\boldsymbol{P}_{123} = \boldsymbol{P}_{123}\{T \geq t_r\} = \boldsymbol{P}\left[(Z_1(t) > 0 \bigcup Z_2(t) > 0) \bigcap Z_3(t) > 0\right] = \boldsymbol{P}_{12}\boldsymbol{P}_3\left[1 + \rho_{3|12}^{x}\left(1/P_{3/12} - 1\right)\right], \tag{26}$$

where $\boldsymbol{P}_{12}$ by (24), $\rho_{3|12} = (\rho_{31} + \rho_{32})/2$.

The generalised reliability index is

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

where Φ^{-1} is the inverse standardised normal distribution. For beams of hyperstatic systems of reliability class RC2, the index β must be not less as β_{tar}=3.8. For their normal sections as particular members, this index may be decreased to 3.5.

Numerical illustration. Resistance and load parameters. Consider as an example the verification of availability during 50 years period of normal sections of precast members as frame middle beams of reliability class RC2 (Fig. 4) the span of which is l =5.7 m. The cross-sectional area of reinforcing bars (3 ∅ 25), the coefficient of variation, mean and variance of yield strength and its characteristic value are:

$A_{s1} = A_{s2} = A_s = 14.72$ cm^2, $\boldsymbol{\delta} f_y = 8$ %, $f_{ym} = 460$ MPa, $\boldsymbol{\sigma}^2 f_y = 1354$ (MPa)^2, $f_{yk} = 400$ MPa. The design yield strength is: $f_{yd} = f_{yk}/\gamma_M = 400/1.15 = 347.8$ MPa.

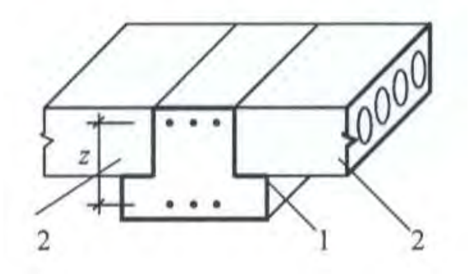

Fig. 4. Precast floor beam (1) and slabs (2)

The mean and variance of couple arms of bending moments are: $z_m = 32$ cm^2, $\boldsymbol{\sigma}^2 z = 2.56$ cm^2. Thus, the design resistance of bending sections 1-1 and 2-2 is: $M_{Rd} = f_{yd} A_s z = 163.83$ kNm.

When the parameters of the additional variable θ_R are equal to $\theta_{Rm} = 1.0$ and $\boldsymbol{\sigma}^2\theta_R = 0.01$, the mean and variances of section resistances are: $(\theta_R R)_m = R_m = f_{ym} A_s z_m = 216.68$ kNm,

$\boldsymbol{\sigma}^2 R = (A_s z_m)^2 \boldsymbol{\sigma}^2 f_y + (f_{ym} A_s)^2 \boldsymbol{\sigma}^2 z = 417.84$ (kNm)^2, $\boldsymbol{\sigma}^2(\theta_R R) = \boldsymbol{\sigma}^2 R + R_m^2 \boldsymbol{\sigma}^2 \theta_R = 887.34$ (kNm)^2.

The parameters of permanent and variable loads are: $g_{1k} = g_{1m} = 23.2$ kN/m, $g_{2k} = g_{2m} = 8.0$ kN/m, $\boldsymbol{\delta} g_1 = \boldsymbol{\delta} g_2 = 10$ %; $q_k = 18.0$ kN/m, $q_m = 0.47 q_k =$ 8.46 kN/m, $\boldsymbol{\delta} q = 58$ %. The parameters of wind moments are: $M_{Wk} = 16.8$ kNm, $M_{Wm} = M_{Wk}/(1 + k_{0.98}\boldsymbol{\delta} W) = 9.45$ kNm. $\boldsymbol{\delta} W = 30$ %. The parameters of the additional variable θ_M are: $\theta_{Mm} = 1.0$ and $\boldsymbol{\sigma}^2\theta_M = 0.01$.

Verification by the partial factor method. Using the partial factor method, no ultimate limit state may be exceeded when design values for beam resistances M_{R_d} and bending moments M_{E_d} are considered. Design bending moments of beam normal sections 1-1 and 2-2 are calculated by Equations (1)-(4) and (7)-(10) using design values of gravity loads p_{Ad} by (5), p_{Bd} by (6) and wind moments M_{wd} with the multification factor K_{F1}=1.0.

The live load q is a leading variable action. According to (5) and (6), the design gravity loads are:

$$p_{Ad} = g_{1k}\gamma_G + g_{2k}\gamma_G + q_k\gamma_Q = 23.2 \times 1.35 + 8.0 \times 1.35 + 18.0 \times 1.5 = 69.12 \text{ kN/m},$$

$$p_{Bd} = g_{2k}\gamma_G + q_k\gamma_Q = 8.0 \times 1.35 + 18.0 \times 1.5 = 37.8 \text{ kN/m}.$$

The design wind moment is:

$$M_{wd} = M_{wk}\psi_o\gamma_w = 16.8 \times 0.7 \times 1.5 = 17.64 \text{ kN·m} < 0.02 pl^2 .$$

According to (7)-(10), the design values of bending moments are:

$M_{1AE_d} = 0.8\ \left(69.12 \times 5.7^2/12 + 17.64\right) = 163.82$ kN·m $\approx M_{1R_d}$ = 163.83 kNm (a rational solution),

$M_{2AE_d} = 69.12 \times 5.7^2 (1/8 - 0.8/12) = 131$ kNm $< M_{2R_d}$ = 163.83 kNm (a logical solution),

$M_{1BE_d} = 1.0 \ (37.8 \times 5.7^2/12 + 17.64) = 119.98$ kNm $<< M_{1R_d}$ = 163.83 kNm (an irrational solution),

$M_{2BE_d} = 69.12 \times 5.7^2/8 - 1.0 \times 37.8 \times 5.7^2/12 = 178.37$ kN·m $> M_{2R_d}$ = 163.83 kNm (an inadmissible solution).

According to the partial factor method, the unpropped precast members are irreliable for considered frame beams.

Verification by the probability-based model. The results on safety design of the normal sections of frame middle beams are presented in Tables 1 and 2.

Table I

The parameters of bending moments

Beams	Sections	Load g		Load q		Wind w		$q+w$		R_c	
		$(\theta M_g)_m$ kNm	$\boldsymbol{\sigma}^2(\theta M_g)$ (kNm)^2	$(\theta M_q)_m$ kNm	$\boldsymbol{\sigma}^2(\theta M_q)$ (kNm)^2	$(\theta M_w)_m$ kNm	$\boldsymbol{\sigma}^2(\theta M_w)$ (kNm)^2	M_m kNm	$\boldsymbol{\sigma}^2 M$ (kNm)^2	R_{cm} kNm	$\boldsymbol{\sigma}^2 R_c$ (kNm)^2
Propped	1-1	67.58	91.34	18.32	116.32	7.56	5.72	25.88	122.04	149.1	978.68
	2-2	59.13	69.93	19.48	131.45	–	–	19.48	131.45	157.55	957.27
Unpropped	1-1	21.66	9.38	22.91	181.81	9.45	8.93	32.36	190.74	195.02	896.78
	2-2	105.05	200.30	11.45	45.45	–	–	11.45	45.45	116.63	1087.64

Table II

The survival probabilities and reliability indices

Beams	Sections	ρ_{kl} by (20)	$\boldsymbol{P}_k$ by (21)	$\boldsymbol{P}_i$ by (22)	Indices β	Indices β_{tar}	Reinforcing
Propped	1-1	0.8891	0.9^3823	0.9^2436	2.53	3.50	Irreliable
	2-2	0.8792	0.9^4614	0.9^2873	3.02	3.50	Irreliable
Unpropped	1-1	0.8246	0.9^5266	0.9^3635	3.38	3.50	Irreliable
	2-2	0.9599	0.9^2847	0.9707	1.89	3.50	Inadmissible

Contrary to the results of the partial safety factor design (section VII.2), the reliability indices presented in Table 2 show that not only unpropped but also propped precast members must be treated as irrationally reinforced irreliable beams of considered frames.

Conclusions. The analysis of hyperstatic reinforced concrete and composite steel-concrete structures subjected to action effects caused by service and climate actions depends on the features of structural systems and construction technologies. Therefore, different design approaches and models must be used in load-carrying capacity and reliability predictions of hyperstatic systems consisted of propped and unpropped bending members.

The values of annual extreme service, snow and wind loads may be treated as basic action variables. In addition, they are closely related with characteristic values of actions used in the partial factor method. Therefore, when a limit state verification of hyperstatic structures is carried out by probability-based approaches, it is recommended to use extreme variable effects of actions

For the sake of design simplifications, it is expedient to base the structural safety analysis of members on the concepts of conventional resistances and safety margin sequences. The long-term survival probabilities of normal or oblique sections as particular members having one single failure mode and beams as structural mixed auto systems representing multicriteria failure mode may be calculated by the method of unsophisticated transformed conditional probabilities.

In some cases, it may be expedient to design and erect precast concrete and composite steel-concrete beams as propped members of hyperstatic structures. These beams may be temporarily supported until their joints are able to resist stresses.

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