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AN INNOVATIVE APPROACH TO SAFETY FORMAT OF NON-LINEAR ANALYSIS APPLIED TO STRUCTURAL ROBUSTNESS ASSESSMENT

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Abstract: Currently the CEN/TC250 is completing the development of a new (second) generation of Structural Eurocodes. The checking of robustness remains one of the important stages in the design of structural systems. Recommended design strategies for robustness checking are based on provisions given in actual EN 1991-1-7. A Non-linear pseudo-static analysis is widely used by following reasons: a non-linear structural analysis based on more realistic constitutive relations for basic variables makes possible a simulation of the real structural behaviour. Implementation of the non-linear pseudo-static analysis for assessment of the structural system in accidental design situation requires an alternate approach to safety format. The paper presents an innovative approach to safety format calibration for non-linear analysis of RC-structures subjected to accidental loads.

Słowa kluczowe: robustness, progressive collapse, reliability, safety format.
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1. Introduction

In accordance with the modern strategy adopted in the Mandate M/515EN each Structural Eurocode should contain special requirement related to assessment of Structural robustness.

As was shown in (Ellingwood and Dusenberry, 2004), prevention and mitigation of progressive collapse of the damaged structural system immediately after sudden column loss can be achieved using following methods: (1) TF-method (indirect tying-force provisions); (2) AP-method (direct Alternate Load Path method); (3) risk-based method; (4) key-element design method. The indirect (TF-method) consists of improving the structural integrity of building by providing redundancy of Load Path and ductile detailing. Currently, the EN 1991-1-7 allows the use of indirect method and some guidance is contained in the EN 1992-1-1. In this case criteria are devised to check the local resistance to withstand a specific postulated accidental load.

The AP-method consists in considering internal forces redistribution throughout the structural system following the sudden loss of a vertical support element based on non-linear analysis.

In general case, the proposed robustness assessment procedure consists of the following main steps: (1) determination of the non-linear static response of system considered: (2) dynamic assessment, using a simplified approach (Izzuddin et al., 2008; Vlassis et al., 2008) based on energy balance and obtaining pseudo-static response; (3) determination of the ultimate (pseudo-static) gravity load (response) for checking of the robustness of the structural system based on ultimate value of the static displacement u_{ult} (or ψ_{ult} for punching assessment of sudden column removal based on the (Olmati et al., 2017)); (4) ductility assessment of connections by means based on compatibility conditions between system and subsystem, and (5) safety format assessment for non-linear analysis of the damaged structural system. It should be pointed, that the first five steps and their modifications are considered widely in numerous international publications, but limited number of works are devoted for safety format assessment at accidental design situation.

This paper briefly presents the main steps of assessing the robustness of a structural system based on classical energy-conservation approach, while focusing on ensuring the target safety format when non-linear analysis using for obtaining pseudo-static response in accidental design situation.

2. Pseudo-static response of the damaged structural system

In accordance with approach, proposed by (Izzuddin et al., 2008; Vlassis et al., 2008), sudden column loss is considered similar in effect to sudden application of the gravity load on the damaged (modified) structure with removed column. This damaged system can be modelling as a single degree of freedom (SDOF) system consisting

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of the vertical deflections at the point of the removed column.

Assuming that the maximum dynamic deflection u_{dyn} at the point of the joint removed column is equal to ultimate static displacement u_{st} obtained from the non-linear static response, pseudo-static gravity load $F_{ps,u}$ can be calculated (see Fig. 1). Based on the proposed approaches (Izzuddin et al., 2008; Vlassis et al., 2008; Tur and Tur, 2018), the following assumption is formulated in present paper. In present work the following assumption was adopted.

A modified (damaged) structural system with SDOF has the required robustness level in accidental design situation, if the total gravity load applied immediately after sudden column loss does not exceed ultimate pseudo-static reaction (response) $F_{ps,u}$, obtained from the balance of the external work over dynamic displacement and internal energy absorbed by the system (substructure) over the maximum (ultimate) static deflection u_{ult}. Checking the Limit State of robustness is performing from the following inequality:

$$F_{st} \leq F_{ps,u} \tag{1}$$

where F_{st} is design value of the generalized gravity load, applied to structure immediately after sudden column loss.

$$F_{ps,u} = \frac{1}{u_u}\int_0^{u_u} P(u)\cdot du \tag{2}$$

where u_u is ultimate value of the static deflection (displacement) obtained based on non-linear static response.

In case of the flat slab robustness assessment, the following combined procedure can be recommended. In accordance with proposed (recommended) approach, the maximum dynamic displacement $u_{dyn,max}$, which is used for calculation of the pseudo-static ultimate gravity load $P_{ps,ult}$ in case of the bending failure mode is obtained from the corresponding pseudo-static rotation $\psi_{ps,u}$, calculated based on CSCT-model for punching shear (Micallef et. al., 2014) (see Fig. 1a).

It should be noted, that this approach proposed the system pseudo-static capacity as a single measure of structural robustness and so, the some criticism can be made for the energy-conservation approach. Nevertheless, the results of the detailed analysis given in (Olmati et al., 2017) shows that the implicit error due to all this simplifications is relatively small (no more than 5 to 8%) and slightly effects on the final result of robustness assessment.

3. Safety format for non-linear analysis / pseudo-static response

3.1. Reliability index

As was shown in (Ellingwood and Dusenberry, 2004), the first-generation probability-based Limit Design Criteria (*Limit State Design*) (such as, for example, EUROCODES) all are based, to varying degrees, on reliability of individual structural members and components.

However, to implement reliability-based design criteria against progressive collapse (for robustness limit state checking) in practice sense, the limit state probability (or reliability index) must be evaluate for structural system (!). In contrast to member reliability, this evaluation "is difficult (complicate) even at the present state of art and with computational resources available" (Ellingwood and Dusenberry, 2004).

As shown by (Ellingwood and Dusenberry, 2004), the probability of structural system failure is an order of magnitude less depending on the redundancy in the system and the degree continuity between members. The recommended value of the acceptable probability of failure β_{tag} depends on the design situation and it not specified usually in design codes.

These threshold values proposed by (Ellingwood and Dusenberry, 2004) assuming that the accepted unconditional probability of failure for extreme (accidental) loads is the same as the one accepted for the failure of structural elements subjected to appropriate load combinations. For example, if the mean rates of occurrence of the accidental event is equal $\lambda_i = 10^{-6}\ldots10^{-5}$ (according to (Ellingwood and Dusenberry, 2004)), than conditional failure probability for the structural system should be on order of $10^{-2}\ldots10^{-1}$, and the target value of reliability index β_{tag} should be the order of 1.5 (for state function $g(x)$ in case of the Normal distribution function for resistance).

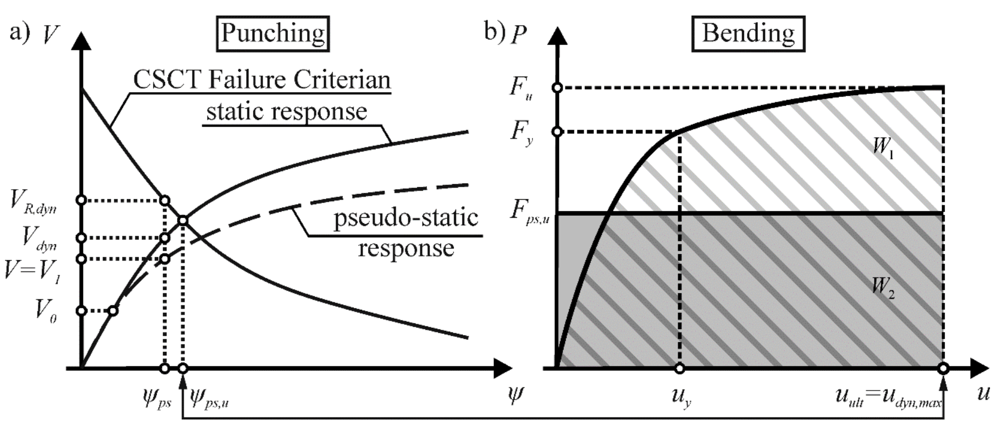

Fig. 1. The principle of assessing the robustness of a structural system with flat slabs based on a combined approach.

3.2. Assessment of resistance non-linear FEM-model uncertainties

Nonlinear analysis (static and dynamic) is most widely used as a main computational tool for checking of robustness of the structural systems in accidental design situations (Accidental Limit States Checking).

It should be noted that the different FEM-programs (software), which applied for nonlinear structural analysis, will have own different level of FEM-model uncertainties in addition to local cross-section resistance model, material and geometry uncertainties. Clearly, the approach is meaningful if structural model covers all relevant failure mechanisms. In our research (Tur and Tur, 2017) the coefficient of variations V_{Rd} of the computer model uncertainties was assessed based on theoretical background described in EN 1990. From these features, it is suggested to be derived from the comparison of the experimental tests data and numerical results, but though probabilistic consideration. The set of the test results obtained in experimental investigations of the different types of statically indeterminate structures demonstrates different failure mechanism was collected from some references and used for assessment of the coefficient variations V_{Rd} (see Fig. 2) and model uncertainly factor γ_{Rd}. The real properties of the material and specimens geometry characteristics obtained by testing used as an input data for nonlinear analysis. Based on results, obtained by numerical investigation was declared, that further research is need to recommended appropriate values of the model uncertainty for numerical simulation.

It should be noted, that for different FEM-programs (software) values of γ_{Rd} will be different. These values for FEM-program should be estimated based on full probabilistic approach, taking into account statistical parameters of the FEM-model uncertainties and consists of in Program Manual.

Using a full probabilistic method of finding a quasi-static (dynamic) response (resistance), N parallel simulations of strength characteristics of concrete and reinforcement steel are produced, for which the mean value and standard deviation are determined. The uncertainty of FEM-model obtained in (Tur and Tur, 2017) was considered as an additional basic variable (N - distribution with mean value 1.0 and V_{Rd} = 15.7% for beams and V_{Rd} = 6.6% for slabs). Then randomly, pairs of values of strengths "concrete-steel" are selected in each simulation and are used to describe the “strain-stress” relationship for materials. For the accepted characteristics of the cross sections, the "moment-curvature" relationships parameters was calculated and used for the creating of plastic hinges. Next, N nonlinear analyses are performed and, according to the methodology proposed in Section 2, values of the quasi-static resistance for each i-th analysis was calculated. As it was shown in (Tur and Tur, 2018) the full probabilistic analysis is general tool for safety assessment of RC-structures, and thus it can be applied in case of non-linear analysis.

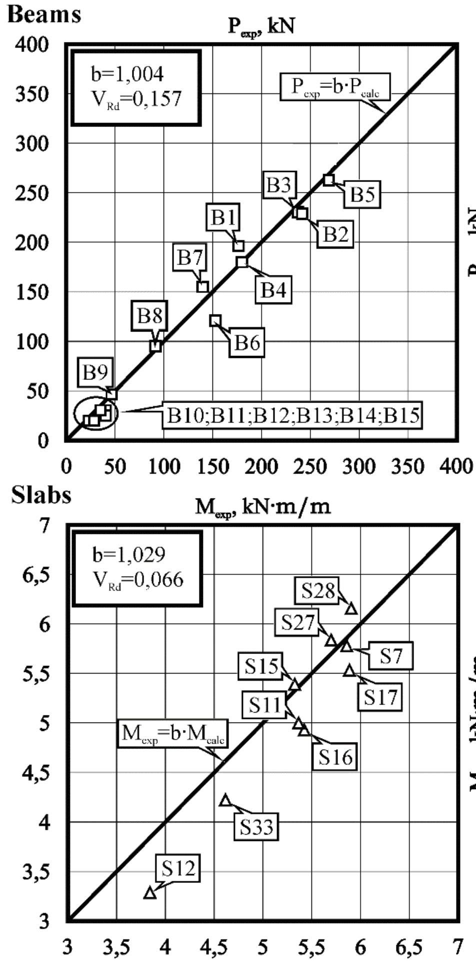

Fig. 2. For estimatiation of the coefficient V_{Rd} for FEM-model, according to (Tur and Tur, 2017).

3.3. Probabilistic analysis with usage non-parametric (order) statistics

In general case, probabilistic analysis based on numerical simulations including following steps: (1) numerical model formulation based on non-linear finite elements and this model describes the resistance function $r(\boldsymbol{r})$ and can be perform deterministic analysis of resistance for given set of input variables; (2) randomization of input variables (random properties are defined by random distribution type and its statistical parameters); (3) probabilistic analysis of resistance (this can be performed, for example, by numerical method of Monte-Carlo-type of sampling, such as LHS sampling). Results of this analysis provide set of random parameters of resistance (and actions); (4) Evaluation of safety using reliability index β or probability of failure. A disadvantage of this approach is in the fact that the target value of design resistance is located in the tail of probability distribution function (PDF), which is determined by the best fit from the sampling. The design value is obtained by extrapolation and strongly depends on the choice of PDF of resistance.

In accordance with proposed approach (Tur and Derechennik, 2018) the global resistance factor γ_{glob} should be determined by the following equations:

$$\gamma_{global} = \frac{R_{m(0.5)}}{R_{d(0.01)}} \quad (3)$$

where $R_{d(0.01)}$ is design resistance (0.01 – percentile of the probabilistic distribution function (*pdf*) of resistance); $R_{m(0.5)}$ is mean (median) value of resistance (as 0.5 – percentile). Based on the Order Statistic (nonparametric) Theory a original procedure for estimation of the desired p-th percentile of assuming arbitrary given confidence level (γ) was developed and presented in detail in (Tur and Derechennik, 2018). The main advantage of the order nonparametric statistics consists in its independence from the type of probability density function (PDF) as well as from the main statistical parameters of the continuous population.

According to proposed approach (Tur and Derechennik, 2018), the estimator of resistance $\hat{R}_{p,\gamma}$ (in case of accidental design situation in term of ultimate pseudo-static response $F_{ps,u}$) of *p-th* quantile (percentile) with required (desired) confidence level γ can be represented as a normalized linear combination of the first three order statistics:

$$\hat{R}_{p,\gamma} = R_{lowest} - \lambda_{(1),\gamma}\Delta_{2-1} - \lambda_{(2),\gamma}\Delta_{3-2} \quad (4)$$

where $R_{lowest} = R_{(1)}$ is the lowest value of resistance in the ordered sample (set of numerical results); $\Delta_{2-1} = R_{(2)} - R_{(1)}$ and $\Delta_{3-2} = R_{(3)} - R_{(2)}$ =are nonnegative differences; $R_{(1)}$, $R_{(2)}$, $R_{(3)}$ – first, second and third order statistics, respectively; $\lambda_1 = \lambda(\gamma, n)$; $\lambda_2 = \lambda(\gamma, n)$ – a dimensionless coefficient, which depends sample size n and specified confidence level γ.

Calibration of the coefficients λ_1, λ_2 for wide range of confidence level γ was performed using the set of n-size random sample obtained by numerical Monte-Carlo simulations, as shown in detail in (Tur and Derechennik, 2018).

Values of dimensionless coefficients λ_1, λ_2 (rounded to the hundreds place) for assessment of the 0.01 percentile with different confidence level γ are listed in Table 1.

Substituting Eq. (4) to Eq. (3) gives:

$$\gamma_{global(\gamma)} = \frac{1-\lambda_{1(0.5;\gamma)}\delta_1-\lambda_{2(0.5;\gamma)}\delta_2}{1-\lambda_{1(0.01;\gamma)}\delta_1-\lambda_{2(0.01;\gamma)}\delta_2} \quad (5)$$

with $\delta_1 = \frac{\Delta_{2-1}}{R_{lowest}}$; $\delta_2 = \frac{\Delta_{3-2}}{R_{lowest}}$.

Using the proposed approach to the assessment of robustness of damaged structural system, a nonlinear analysis of two-span frame (2×6 m) was performed in which beams has a cross-section 300×500 mm and reinforcement ratio $\rho_l = 0.33\%$ ($\rho_l' = 0.66\%$). The following input data was adopted: concrete compressive strength class C20/25, reinforcement steel B500, constitutive relationship "σ-ε" for materials was adopted in accordance with EN 1992-1-1.

3.4. Assessment of the global safety factor

As shown by preliminary analysis of obtained results, the calibrations procedure according to Approach 2 gives sufficiently larger values of the global safety factor γ_{glob} than according to Approach 1, especially with an increase in the confidence level of estimation. This is obviously due to the fact that the statistical parameters of a model uncertainty (μ_{Rd}, σ_{Rd}) in "varying degrees" affect the final value of the global safety factor when it is estimated based on Approach 1 or Approach 2.

When Approach 2 is applied, the model uncertainty can to become the dominant basic variable k_R, whereas according to Approach 1, calculation exp($\alpha_R \beta V_{Rd}$) when the coefficient of variation V_{Rd} changes from 6.6% to 16.7% leads to a change in the value of factor γ_{Rd} from 1.03 to 1.08 (only!). It was found that the global safety factor γ_{glob} values according to EN 1992-2, *fib* MC 2010 and ECOV-method are very close to value γ_{glob}, obtained by Approach 1 for difference confidence level (γ_{glob} from 1.22 to 1.29, see Table 2), but differ from γ_{glob} – values obtained by Approach 2.

Tab. 1. Values of the coefficient λ_1, λ_2 for different confidence level γ for *p-th* quantile (percentile) estimation (N = 35).

γ	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.75	0.8	0.9
	p = 0.01									
$\lambda_{1(\gamma)}$	-0.46	-0.28	-0.11	0.09	0.32	0.63	1.05	1.35	1.75	4.32
$\lambda_{2(\gamma)}$	-0.14	+0.03	0.19	0.37	0.58	0.86	1.26	1.53	1.9	4.29

Tab. 2. Influence of the confidence level γ on the global safety factor γ_{glob}.

Approach	Confidence level, γ						Notes
	0.5	0.6	0.7	0.75	0.9	unknown	
Approach 1	1.22	1.23	1.26	1.31	1.49	-	$\gamma_{glob} = \gamma_{Rd}\gamma_R$
Approach 2	1.41	1.46	1.49	1.73	3.00	-	from simulation with k_R as basic variable
ECOV (Cervenka 2013)	-	-	-	-	-	1.225	exp($\alpha_R \beta V$)
EN 1992-2 ***fib*** MC 2010	-	-	-	-	-	1.26	constant value with f_{cm}=0.85f_{ck}

The use of values of the global safety factors in accordance with Approach 1 to mean values of resistance $\hat{R}_m$ obtained from non-linear analysis can to increase risk of overestimation the design value of resistance $\hat{R}_d$. The application of the calibration procedure, based on proposed approach of interval estimation of quantile by the method of order (non-parametrical) statistics, creates the basis for more objective assessment of γ_{glob} – value, science it allows one to perform *p-th* quantile estimation with a desired confidence level γ without resorting to the selection of the resistance distribution function type.

4. Conclusions

The simplified pseudo-static column removal scenario with appropriate gravity load combination may be used for checking of the structural systems robustness and progressive collapse prevention in accidental design situation. When performing a nonlinear analysis (NLFEA) of a modified structural system, one of the main problems remains to ensure the required safety format. It should be noted, that for different FEM-software values of factor γ_{Rd} will be different and should be includes in Software Manual. An innovative calibration procedure of the global safety factor γ_{glob} is proposed based on Order (non-parametric) Statistics estimation method. The main advantage of the proposed approach is that the result of the percentile estimation does not depend on the choice of the probability distribution function (PDF). There are significant difference (up to 230% depending on the confidence level) in the γ_{glob} – value for the approach when the model uncertainty k_R is considered as a basic variable in the non-linear resistance model and for the approach when the value of the global safety coefficient is defined as the product γ_{Rd} γ_R.

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