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Reliability Approaches to Modeling of the Nonlinear Pseudo-static Response of RC-structural Systems in Accidental Design Situations

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The nonlinear structural analysis is considered as a basic design procedure, which is used for checking of the structural robustness in accidental design situation. It is explained by following reason: a nonlinear structural analysis based on realistic constitutive relations for basic variables (average values) makes possible a simulation of a real structural behavior. It should be pointed that, implementation of the nonlinear structural analysis in design of concrete structures requires an alternate approach to safety verification. The paper presents a new approach to safety format for nonlinear analysis of RC structures subjected to accidental loads.

Keywords: nonlinear analysis, progressive collapse, safety format, robustness, reliability.

Introduction

In recent years structural engineers try to use nonlinear analysis while designing a new complex structural system as well as for checking of the existing structures.

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).

As it was stated in Červenka (2013a), “evaluations of the nonlinear analysis are supported by rapid increase of computational power as well as new capabilities of the available tools for numerical simulations of structural performance”.

The first published works dealing with nonlinear finite element analysis of concrete systems emerged in the late 1960. These studies focused on various aspects of element formations, including crack propagation and the bonding of reinforcement. In general case two basic FEM-methods are used for non-linear modeling: 1) so-called stiffness Method (Modified Stiffness Model); 2) Layered Model.

The stiffness adaptation analysis is purposed to be an alternative for a full nonlinear analysis (Layered Model) for calculating load distributions, deformations, crack patterns and crack-width in reinforced concrete structures.

In stiffness adaptation analysis both standards linear elastic material as well as non-linear material behavior can be defined (Hu and Schnobrich 1991). Nonlinear materials can be defined through

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a uni-axial stress-strain curve, both in the tensile and in the compressive state. Nonlinear stress-strain curves may also be defined for bar and grid reinforcement.

In case of the Layered model approach each concrete layer is assumed to be in a state of plane stress and the actual stress distribution of the concrete section is modeled by a piecewise constant approximation.

In general case, for the reinforcement concrete section, the final form of the stress resultant constitutive matrix at an integration point can be written as:

$$\begin{Bmatrix} \{N_{Rd}\}^T \\ \{V_{Rd}\}^T \\ \{M_{Rd}\}^T \end{Bmatrix} = [D] \begin{Bmatrix} \{\varepsilon_0\}^T \\ \{\gamma_t\}^T \\ \{\varphi\}^T \end{Bmatrix}, \qquad (1)$$

where: $[D]$ – is the stiffness matrix that can be established by assembling the contributions of all the concrete layers, all steel layers and transverse shear stiffness.

As it was shown above, nonlinear analysis take into account the nonlinear deformation properties of RC-sections, based on physical constitutive relations (" ó – å " for material properties) and makes possible a simulation of a real structural behavior. If reflects an integral response, where all local sections interact and therefore it requires an adequate approach for safety verification (note, that in partial safety factor (PSF) method (EN 1990: 2006) we assume a failure probabilities of separate materials, but do not evaluate the failure probability on the structural level). It should be underlined, that nonlinear analysis offers a verification of *global resistance* and requires a safety format for global resistance (Červenka 2013b). In accordance with Červenka (2013b), the term *global resistance* (global safety) is used for "assessment of structural response on higher structural level than a cross-section". The term *global resistance* is introduced in Červenka (2013b) in order to distinguish the newly introduced check of safety on global level, as compared to local safety check in the partial safety factor method (PSF-method) in accordance with EN 1990 (2006).

Safety format for nonlinear analysis in accordance with actual codes provision

he historical review (from CEM MC78 to *fib* MC2010) of the non-linear safety format development was described in detail in Sangiorgio (2015).

With the implementation of the new *fib* MC2010 (2010), a different perspective was placed on nonlinear analysis and safety assessment. The design condition to be used in safety format for nonlinear analysis is written in the external actions and resisting internal forces domain:

$$E_d \le R_d, \qquad (2)$$

where: E_d – is the design value of the action; R_d – is the design value of resistance.

Three different approaches are proposed to evaluate the design resistance R_d (depending on various levels of implementations of probabilistic theory): (1) full probabilistic method, recommended by *JCSS* as a basic method; (2) the global resistance method; and (3) the partial factor method (PSF-method).

In the global resistance format, the resistance is considered on a global structural level. Two alternative methods are mentioned in *fib* MC2010 (2010) for the derivation of the design resistance R_d : (1) global resistance factor method (which was adopted from EN 1992-2 (2005), slightly modified); and (2) ECOV-method, proposed by Červenka (2013b) and Sykora and Holicky (2011) (estimations of coefficient variation for resistance).

In this case, the safety margin can be expressed by the global safety factor as:

$$R_d = \frac{R_m}{\gamma_R}, \qquad (3)$$

where: R_m – is the mean resistance.

The global safety factor γ_R cover all uncertainties and can be related to the coefficient of variations of resistance V_R (according a LN- distribution (!) according EN 1992-2 (2005)) as $\gamma_R = \exp(\alpha_R \beta V_R)$.

A simplified formulation was proposed in *fib* MC2010 (2010), where in denominator of the right hand side in eq. (3) is product of two factors $\gamma_R = \gamma_m \cdot \gamma_{Rd}$ (Sykora and Holicky 2011). The first factor γ_m is related to material uncertainty and can be established by probabilistic analysis. The second factor γ_{Rd} is related to model and geometrical uncertainties and recommended value are in range 1.05...1.1 only (!) (as suggested by EN 1992-2(2005)).

As it was stated in Sangiorgio (2015), after the new *fib* MC 2010 (2010), although the topic is still controversial, only few contributions were found in literature (Schlune et al. 2011, Allaix et al. 2013).

The first contribution was presented by Schlune et al. (2011). Design resistance R_d is then derived by division of the obtained mean resistance R_m by global resistance factor γ_R:

$$R_d = \frac{R(f_{ym}, f_{cm}, a_{nom})}{\gamma_R}, \tag{4}$$

where: $\gamma_R = \dfrac{\exp(\alpha_R \beta V_R)}{\theta_m}$ – again based on the assumption of a lognormal distributed resistance.

Model uncertainties are explicitly taken into accounts thought the use of the bias factor $è_m$, which is defined as the mean ratio of experimental to predicted resistance (in accordance with Schlune et al. (2011)) its value varies between 0.7 and 1.2 for failure in compression, bending and shear). The coefficient variation of structural resistance V_R is written as follows:

$$V_R = \sqrt{V_g^2 + V_m^2 + V_f^2}, \tag{5}$$

where: V_g, V_m, V_f – are the coefficients of variations of the geometrical, model and material uncertainties respectively, estimated in accordance with Schlune et al. (2011).

According with the second contributions, proposed by (Allaix and Mancini 2007, Allaix et al. 2013) the design resistance R_d is derived by divisions of obtained resistance factor γ_R and the model uncertainty factor γ_{Rd}:

$$R_d = \frac{R(f_{ym}, f_{cm}, a_{nom})}{\gamma_R \gamma_{Rd}}, \tag{6}$$

In this case, the global resistance factor γ_R is derived from coefficient of variations of the structural resistance V_R (estimated by probabilistic method or based on Červenka method ECOV):

$$\gamma_R = \exp(\alpha_R \beta V_R), \tag{7}$$

where: β is the reliability index in accordance with EN 1990; α_R – is the sensitivity factor for resistance.

The model uncertainty factor γ_{Rd} takes into account the difference between the real behavior of the structure and the results obtained based on a numerical model. The model uncertainty factor γ_{Rd} can be derived using the following expression from Schlune et al. (2011):

$$\gamma_{Rd} = \exp(\tilde{\alpha}_R \beta V_{\vartheta R}), \tag{8}$$

where: $\tilde{\alpha}_R = 0,4\alpha_R$ – is the sensitivity factor for resistance model uncertainty ($\tilde{\alpha}_R$ <1 in order to account for separate safety assessment of resistance); $V_{\vartheta R}$ – is the coefficient of variations of the resistance model uncertainly.

The value of this coefficient of variations can be obtained based on experimental results according to EN 1990 (2006).

For checking of the RC-structural system in accidental design situation, two main issues must be solved: (1) to calculate the pseudo-static response of the modified structural system under accidental loads; (2) to determined target value of the reliability index for accidental design (required level of reliability).

Pseudo-static response of the structural system with a removed vertical load bearing elements

As was stated in Ellingwood (2002) prevent and mitigation of progressive collapse can be achieved using two different methods: (1) TF-method (indirect Tie-Force method); (2) AP – method (direct Alternate Load Path 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 direct method, referred to as "Alternate Load Path" (AP - method), is most widely used in the practical design and based on criteria for evaluating the capability of a damaged structure to bridge over or around the damaged volume of area without progressive collapse developing from the local damage. The AP-method consists in considering internal force (effect of the actions) redistributions throughout the structure following the loss of a vertical support element (UFC 4-023-03: 2010).

As was shown in UFC 4-023-03 (2010), an AP-method analysis may be performed using of the following basic nonlinear procedures: Nonlinear Dynamic (*NLD*) and Nonlinear Static (*NLS*) procedures. In case of the Nonlinear Static procedure after materially-and-geometrically nonlinear model is built, the accidental load combination are magnified by a dynamic increase factor (*DIF*) that accounts for inertia effects and the resulting load is applied to model with removed vertical load bearing elements. If a dynamic increase factor (*DIF*) is known, for deformation-controlled actions, the resulting deformations are compared to the expected deformation capacities; for the force controlled action, the member strength is not modified and shell not be less than the maximum internal member forces (demands). Otherwise, calculation procedure based on the energetic approach should be used. The basic provisions of this procedure are described in detail in Tur (2012). The purpose here is to analyze the structural response of RC-structural systems subjected to a sudden column loss.

The procedure, which is used for obtaining of the pseudo-static nonlinear response of the structural system, consists of the following main steps: (1) Calculate the static non-linear response "F–δ" for the modified structural system with a removed vertical load bearing element according to certain rules (Tur 2012, Vlassis 2009) (see Fig. 1, line 1); (2) Calculate the pseudo-static re-

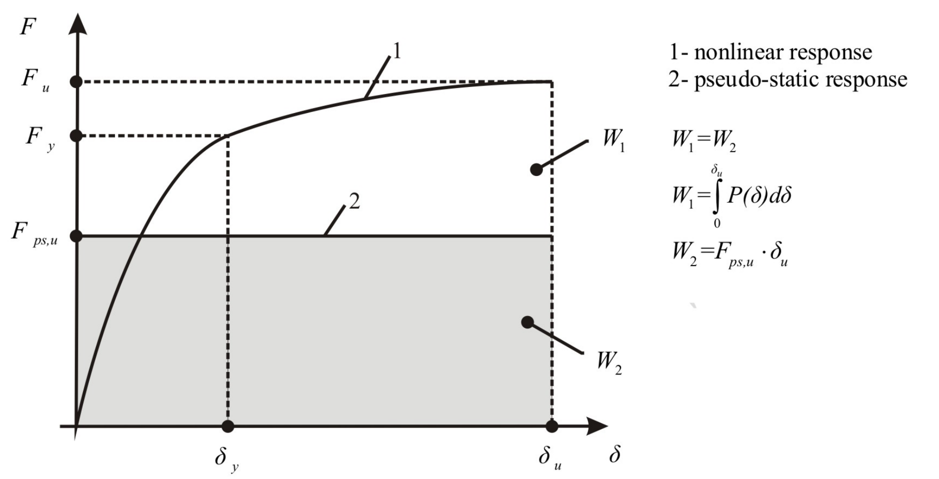


Fig. 1

To assessment of the pseudo-static response of the structural system in accordance with (Tur 2012, Vlassis 2009) sponse, that taking into account inertia effects, caused by suddenly applied gravity load.

In general case, based on energetic consideration (see **Fig. 1**):

$$\int_0^{\delta_u} P(\delta)d\delta = F_{ps,u} \cdot \delta_u \,, \tag{9}$$

Pseudo-static response is equal:

$$F_{ps,u} = \frac{1}{\delta_u}\int_0^{\delta_u} P(\delta)d\delta \,, \tag{10}$$

Required level of reliability for accidental design situation

In general case, the probability of structure collapse due to postulated abnormal event can be written as:

$$P(F) = P(F|DH_i)P(D|H_i)P(H_i) \,, \tag{11}$$

As was shown in Ellingwood (2002), in a "specific local resistance" design strategy, the focus is on minimizing probability $P(F|DH_i)$, that is, to minimize the likelihood of initiation of damage that may lead to progressive collapse.

This strategy may be difficult or uneconomical, and may leave some significant hazards unaddressed.

Accordingly, it is likely that $P(D|H_i)$ will very close to 1,0 in many practical cases, meaning that the collapse probability becomes, approximately:

$$P(F) = P(F|DH_i)P(H_i) \,, \tag{12}$$

It is in minimizing the conditional probability $P(F|H_i)$, that the science and art of the structural engineer becomes paramount (Ellingwood 2002).

It may be assumed that the occurrence of the abnormal event H_i can be modeled as a Poisson process with yearly mean rate of occurrence λ_i. The probability of occurrence of this abnormal event during some reference period *T*, is thus approximately $P(H_i) = \lambda_i T$ (for very small λ_i) (Ellingwood 2002). In the case of fire, gas explosion and some other accidental loads, parameter λ_i may be related to building floor area ($\lambda_i = p \cdot A_f$, in which A_f - floor area and $p = p_1 \cdot p_2$, where term p_1 – probability of occurrence of hazard per unit area and $p_2 < 1.0$ represents effect of warning and control systems).

Mean rates of occurrence for gas explosions, bomb explosions and vehicular collisions in accordance with (Ellingwood and Corotis 1991) are approximately:

- Gas explosions (per dwelling): $2x10^{-5}$/yr;
- Bomb explosions (per dwelling): $2x10^{-6}$/yr;
- Vehicular collisions (per dwelling): $6x10^{-4}$/yr;
- Full developed fire (per building): $5x10^{-8}$/yr.

As it was shown in (Ellingwood 2002), to evaluate $P(F|DH_i)$, one must postulate a mathematical model, *G*(**X**) (state model), of the structural system based on principles of mechanics and supplemented, where possible, with experimental data (!). The load and resistance variables are expressed by vector **X**. We must then determine the probability distribution of each variable and integrate the joint density function of **X** over that region of probability space where *G*(**X**) <0 to compute in accordance with EN 1990 conventional limit state probability. But, we must to remember that it is very difficult and complex way (especially for structural systems).

Alternatively, FORM – analysis may be used to compute a conditional reliability index β defined as:

$$\beta = \frac{\mu_G}{\sigma_G}, \tag{13}$$

where: μ_G and σ_G – is mean and standard deviation of $G(\mathbf{X})$.

According to Ellingwood (2002), the reliability index is related to $P(F|DH_i)$ through:

$$\beta = \Phi^{-1}[P(F|DH_i)], \tag{14}$$

in which $\Phi^{-1}[P(F|DH_i)]$ is the percent-point function of the standard Normal probability distribution.

With $P(H_i) = \lambda_i T$, eq. (14) can be rewritten as:

$$\beta = \Phi^{-1}[P(F/\lambda_i T)], \tag{15}$$

As was shown in Ellingwood (2002), the first-generation probability-based Limit State Design Criteria (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 in practice sense, the limit state probability (or reliability index) must be evaluated for a *structural system* (!). In contrast to member reliability, this evaluation is difficult (!) even at the present state of art and with computational resources available (Ellingwood 2002, Tur 2012).

Assuming that an analysis of a damaged structure can be performed, an acceptable value of β upon which to base design for conditional limit states is suggested by eq. (15).

As shown by Ellingwood (2002), 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.

For example, if $\lambda_i = 10^{-6}$ to 10^{-5}, than the conditional failure probability for the structural system should be on the order of $10^{-2}...10^{-1}$, and the target value of reliability index β_{tag} should be the order of 1,5. Load and resistance criteria can be developed to be consistent with the reliability.

Assessment of the global resistance and global safety factors for pseudo-static response

At the *first stage* of analysis the value of the global resistance factor γ_R was defined in accordance with (Sykora and Holicky 2011) from eq. (7). As it was shown above, the ECOV-method is based on idea that the random distribution of resistance, which is again described by the coefficient variation V_R, can be estimated from mean R_m and characteristic R_k values of resistance (pseudo-static response of the structural system). In this case, coefficient variations of resistance V_R can be obtained from following equation:

$$V_R = \frac{1}{1,64}\ln\left(\frac{R_m}{R_k}\right), \tag{16}$$

where: R_m, R_k – are the mean and characteristic values of resistance (pseudo-static response, as was shown in section 3), obtained by two separate non-linear analysis using mean and characteristic values of input material parameters respectively.

The results of the nonlinear analysis of the statically undetermined an encastre RC-beam and values of the coefficient variations V_R and global resistance coefficient γ_R obtained by calculations are presented in Table 1.

The result, presented in Table 1 was obtained with FEM-computer program most widely used in practical design and declared about possibilities for nonlinear analysis of reinforced concrete

Table 1

The results of estimation the coefficient γ_R based on ECOV

Element	$\frac{r_1}{r_1'}$ [%]	Resistance, kN/m		$V_R = \frac{1}{1,64}\ln(\frac{R_m}{R_k})$	$\gamma_R = \exp(1,2V_R)$
		R_m	R_k		
RC-beam (an encastre)	$\frac{0,48}{1,05}$	119,5	109,4	0,054	1,07

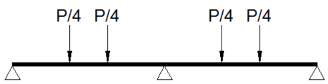
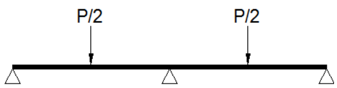
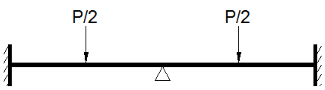
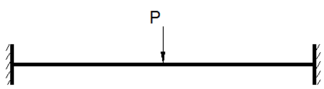
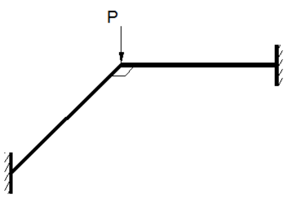
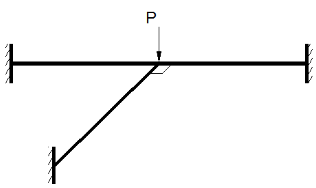
Notes: Materials properties: concrete class C25/30, f_{cm}=33 MPa, steel B500, f_{ym}=1,1f_{yk}=550 MPa; section 300x350 mm; $\beta_{tag} = 1,5$ for accidental design situation.

structures. As it was declared in software manual, FE-program is capable of a "realistic simulation of RC-structure" behavior in the entire loading range with ductile as well as brittle failure modes (Sykora and Holicky 2011, Schlune et al. 2011).

As was shown in Allaix and Mancini (2007) the result of investigation depends on assumption and criteria underlying the model used in the non-linear analysis. 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. So, effects of model uncertainties should be treated separately (!).

Table 2

Loading arrangement for experimental specimens

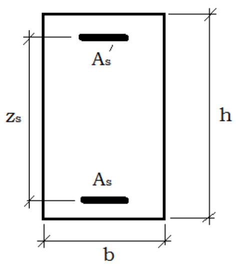
Loading scheme	Beam, slabs Series	Reference
	B1, B2, B3	Monnier (1970)
	B4, B6, B7, B8, B9	Saleh and Barem (2013), Ashour and Habeeb (2008), Maghsoudi and Bengar (2009), Mahmoud and Afefy (2012), Dalfré and Barros (2011)
	B5	Saleh and Barem (2013)
	B10, B11, B13	Farhangvesali et al. (2013), Parmar et al. (2015)
	B12	Qian and Li (2012)
	B14, B15	Rashidian et al. (2016)
Slab	S7, S11, S12, S15, S16, S17, S27, S28, S33	Cardenas and Sozen (1968)
Cylindrical Shell	SH11, SH31	Duddeck et. al. (1978)

At the *second stage* of analysis the coefficient of variations $V_{\vartheta R}$ of the computer model uncertainties was assessed based on theoretical background described in Annex D (EN 1990: 2006). From these features, it is suggested to be derived from the comparison of the experimental tests data and numerical calculations 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 (see Tables 2, 3) was

Table 3

Basic parameters of the experimental specimens (input data for non-linear analysis)

Beam/Slab Series	Cross-section	Dimensions Size, mm					Material properties			
							Concrete		Steel	
		b	h	z_s	r_l	r'_l	f_{cm}, f_{ctm}, MPa	E_{cm}, GPa	f_{ym}, MPa	E_s, GPa
1	2	3	4	5	6	7	8	9	10	11
B1		150	260	236	0,64	0,91	34,4 / 4,51	32	440	200
B2					0,91	0,64	32,4 / 3,95	31,9	435	
B3					0,91	0,64	33,9 / 4,41	33,5	433	
B4			250	210	0,66	0,46	28 / 2,5	28	520 (Ø10) 580 (Ø12)	
B5							29 / 2,64			
B6		200	300	265	0,85	0,85	26,6	28,3	511	
B7		150	250	170	1,28	1,28	74,2	48,3	412	
B8			250	210	0,46	1,17	25	28	445	
B9		375	120	68	0,64	1,85	30,1	31,5	447	
B10		180	180	118	0,59	0,59	30,5	31,6	592	
B11							59	54,3	550	
B12		100	180	150	0,87	0,87	40	41,2	575	
B13		900	150	130	1,06	0,83	33	28,9	450	
B14, B15		200	140	90	0,66	0,44	26 / 1,5	28	530	
S7	Slabs	2290	105	1143	0,008	0,0086	35,5	33,7	345	
S11			100		0,008	0,004	32,3			
S12			100		0,008	0,004	35,6			
S15			104		0,0079	0,0087	32,3			
S16			105		0,008	0,0088	35,6			
S17			100		0,0081	0,0088	35,2			
S27			105		0,008	0,0087	35,5			
S28			107		0,0081	0,0087	35,3			
S33			101		0,008	0,0021	35,3			
SH11	Cylindrical Shell	1677	5,02	447	0,611	0,611	43,0 / 2,0	16,4	670,0	201
SH31			5,0		0,895	0,326				

collected from some references and used for assessment of the coefficient variations $V_{\vartheta R}$ and model uncertainly factor γ_{Rd}. The model uncertainty factor γ_{Rd} takes into account difference between the real behavior of structure and the results of a numerical modeling suitable for specific structure.

The real properties of the material and specimens geometry characteristics obtained by testing were used as an input data for nonlinear analysis. The main characteristics of the analyzed test specimens are presented in Tables 2, 3.

As it can be seen from the Table 4, the estimated values of coefficient of variations V_{Rd} for model uncertainties are much higher than recommended in codes (for example, in *fib* MC2010, values in range 1,05...1,1).

The same results and conclusions were obtained by Schlune et al. (2011). Schlune concluded that model uncertainties of nonlinear analysis are much higher than in standard design based on engineering formulas and are strongly dependent on modes of failure and adopted failure criteria. Reported in Schlune et al. (2011) coefficient of variation due to model uncertainty for bending failure in range 5...30%, for shear 15...64%. Schlune concluded that due to the lack of data, the choice of model uncertainty often depends on engineering judgment and can be subjective.

Note, that coefficient of variations V_m due to material uncertainty (variability) has not a fixed value. In the case of concrete, the mean value of the concrete compressive strength for different classes according to EN 1992-2 (2005) is calculated as: f_{cm}=f_{ck}+8 MPa (where $8MPa = 1,64\sigma_c$, which standard deviation $\sigma_c = 4,88MPa$). For fixed value of standard deviation (as a basic characteristic of the production quality control) $\sigma_c = 4,5MPa$, coefficient of variation $V_{m,c}$ of concrete compressive strength will be in range from 8,6 % (C50/60) to 21 % (C16/20) and coefficient of variation for materials V_m will be in range from V_m=10,48 % to 21,84 % (with fixed value of coefficient of variations V_s=6 % for steel).

Table 4

Estimated values of the global coefficient γ_R

Structures	Shlune model						Allaix model				
	Coefficient of variation, %				θ_m	γ_R	coeff. var., %		factors		
	V_m^*	V_g	V_{Rd}	V_R			V_{R0}	V_{Rd}	γ_{R_0}	γ_{Rd}	γ_R
Beams, Frames	var	det	15,7	17,8... 30,6	1,004	1,55... 1,97	5,8	15,7	1,19	1,21	1,44
Slabs	var	det	17,3	17,8... 30,6	1,03	1,52... 1,87	5,8	6,56	1,19	1,08	1,28

Note: Value of V_m due to material variability in range from 8,6 % (C50/60) to V_m=21 % (C16/20).

Further research is need to recommended appropriate values of the model uncertainty for numerical simulation. It should be noted, that for different FEM-programs 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.

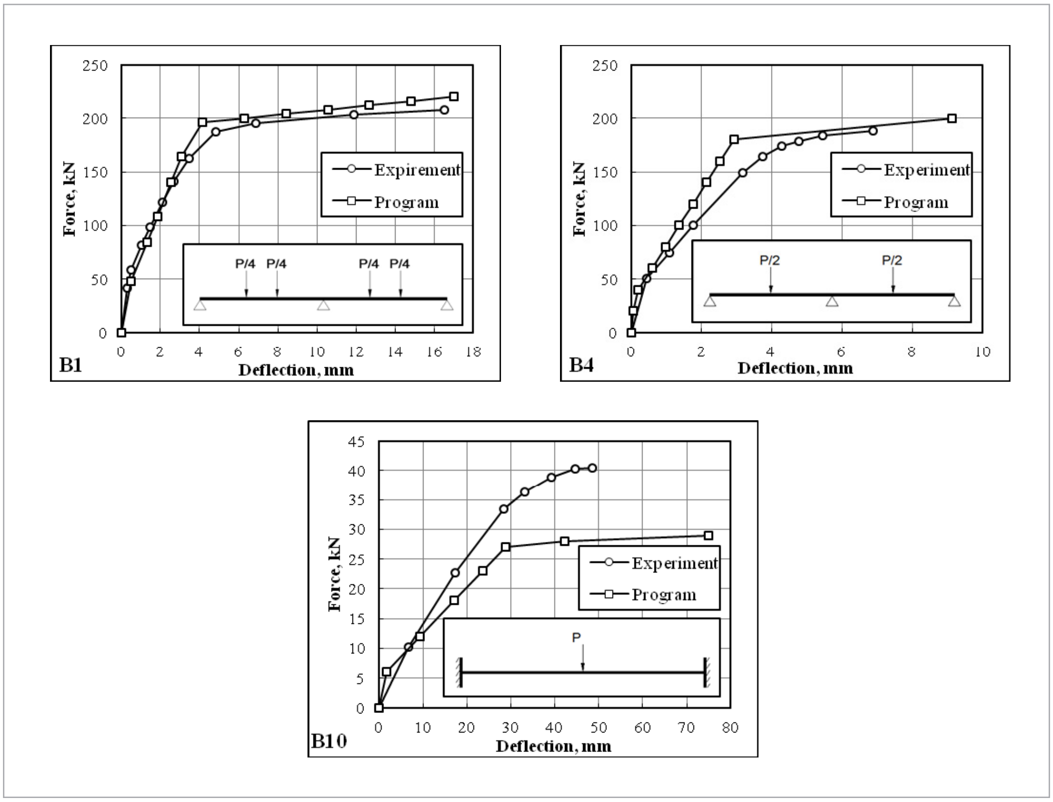


Fig. 2

Some typical examples of the experimental and predicted force-deflection response of the analyzed specimens (see **Tables 2, 3** for designation of the specimens)

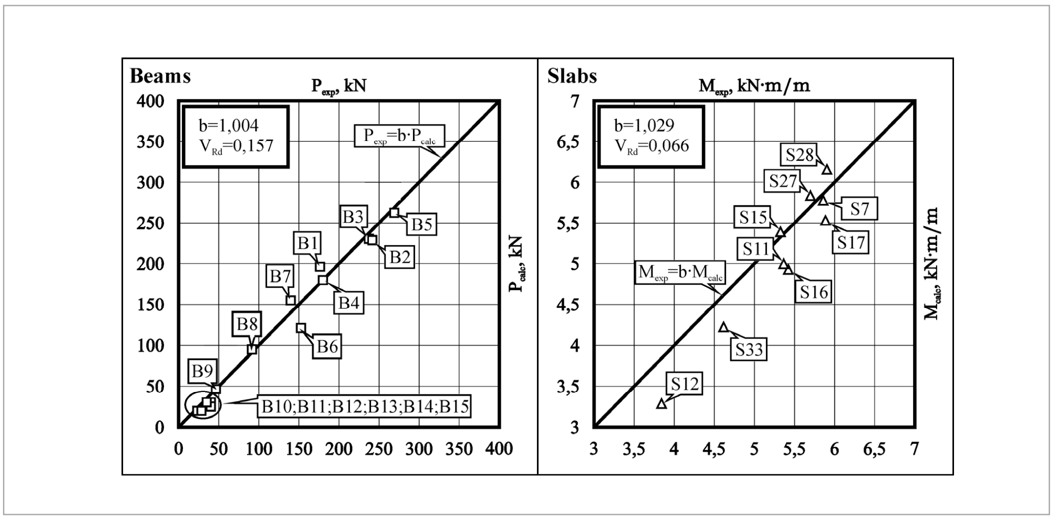


Fig. 3

For estimatiation of the coefficient V_R for FEM-model (see with **tables 2, 3**)

Conclusions

Safety format suitable for nonlinear analysis (pseudo-static response) that based on global resistance in accordance with *fib* MC2010 concept are presented.

The following conclusions can be adopt: (1) the differences between proposed methods are not significant; (2) fixed value of global safety factor $\gamma_R = 1,27$ in accordance with *fib* MC2010 (2010) and EN 1992-2 (2005) is not good approach for safety assessment and sometimes can be unconservative results; (3) the values of the global resistance factor γ_R should be estimated separately for different computer programs, which are used for non-linear analysis (pseudo-static response of the structural system), based on experimental results. These values for separate computer programs should be estimated based on full probabilistic approach, taking into account statistical parameters of the *FEM*-model uncertainties. For accidental design situation load and resistance criteria can be developed based on the target value of reliability index $\beta_{tag} = 1,5$.

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