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ПРИЛОЖЕНИЕ НА МЕТОДА НА МОНТЕ КАРЛО В ЧИСЛЕН МОДЕЛ ЗА ИЗСЛЕДВАНЕ НА ПРОГРЕСИВНО РАЗРУШАВАНЕ НА СЪЩЕСТВУВАЩИ СТ.БЕТ. КОНСТРУКЦИИ УСИЛЕНИ С ОБТЕГАЧИ, С ОТЧИТАНЕ НА ВХОДНИ ПАРАМЕТРИ НАТОВАРЕНИ С ВЕРОЯТНОСТНИ НЕОПРЕДЕЛЕНОСТИ

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A MONTE CARLO BASED NUMERICAL APPROACH FOR PROGRESSIVE COLLAPSE OF EXISTING RC STRUCTURES STRENGTHENED BY TIES UNDER UNCERTAIN-BUT-BOUNDED INPUT PARAMETERS

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Abstract:
The case of uncertain-but-bounded input parameters for the problem of avoiding a progressive collapse of reinforced concrete (RC) buildings is treated numerically. Existing old RC buildings are sometimes subjected to obligatory removal of some structural element-members, e.g. columns, and so to the risk of a progressive collapse. In order to avoid such a progressive collapse., a modification of the structural response and a redistribution of internal actions can result to a requirement for strengthening the remaining structure after the removal of the degraded elements. The present study deals with such a case, which concerns the computational analysis of framed RC structures under the removal of some columns and the so-induced requirement of a strengthening by ties (tension-only elements). The unilateral behaviour of these cable-ties, which can undertake only tension, is strictly considered, and the response of the remaining structure strengthened by ties is computed. The Monte Carlo method is used for treating the uncertainty concerning input parameters. Finally, in a practical case of a framed RC structure, the effectiveness of the proposed methodology is shown

Keywords:
Uncertain input parameters, progressive collapse of old RC structures, removal of columns, strengthening by ties.

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1. INTRODUCTION

In existing Reinforced Concrete (RC) buildings and in other Civil Engineering structures, e.g. highway bridges, the removal and/or the local failure of a primary structural component can lead to the progressive collapse of the entire structure [1]. Local failure can be caused by an accidental action, involving exceptional conditions for the structure. These conditions can be the exposure of the structure to overloading, explosion, impact, vehicle collisions, environmental degradation, seismic excitation etc. So, a progressive collapse can be triggered by a variety of causes, including design and construction errors as well as loading events that are beyond the standard design envelope and are seldom considered by the structural engineer. Such extreme events are typically associated with abnormal loads, e.g. gas explosions, bomb detonations, aircraft impacts and vehicular collisions, structural actions induced by severe fires, excessive values of environmental loads that stress the building well above the anticipated design [1-5].

As concerns existing old RC buildings, they are sometimes subjected to obligatory removal of some structural element-members, e.g. columns, and so to the risk of a progressive collapse. In order to prevent and avoid such a progressive collapse., a modification of the structural response and a redistribution of internal actions can result in to a requirement for strengthening the remaining structure after the removal of the degradated elements.

Among the available strengthening methods [6-8], cable-like members (tension-only tie-elements) can be used as a first strengthening and repairing procedure [9,10]. Cables can undertake tension but buckle and become slack and structurally ineffective when subjected to a sufficiently large compressive force. Thus the governing conditions take an equality as well as an inequality form and the problem becomes a high non-linear one. So, the problem of structures containing as above cable-like members belongs to the so-called Inequality Problems of Mechanics, as their governing conditions are of both, equality and inequality type [11-12]. A realistic numerical treatment of such problems can be obtained by mathematical programming methods (optimization algorithms) [9-13].

For the numerical analysis of such old RC structures, many uncertainties for input parameters must be taking into account [14-18]. These mainly concern the holding properties of the old materials that had been used for the building of such structures, e.g. the remaining strength of the concrete and steel, as well as the cracking effects etc. Therefore, an appropriate estimation of the input parameters and use of probabilistic methods must be performed.

In the present study, the problem of RC structures strengthened by tention-ties in order to prevent and avoid progressive collapse is analyzed in a numerical stochastic way. Emphasis is given to the uncertainty concerning the input parameters. For this purpose, the input-parameters are considered as interval parameters with known upper and lower bounds, characterized in Civil Structural Engineering as uncertain-but-bounded parameters [19,20]. The herein numerical stochastic approach is based on Monte Carlo simulation methods, see e.g. [21-25]. Finally, an application is presented for a simple typical example of an industrial RC frame strengthened by bracing ties after the removal of some ground floor columns.

2. THE PROBABILISTIC COMPUTATIONAL APPROACH

The probabilistic approach for the analysis of existing RC frame-buildings may be obtained through Monte Carlo simulations. As well-known [21-25], Monte Carlo simulation is simply a repeated process of generating deterministic solutions to a given problem. Each solution corresponds to a set of deterministic input values of the underlying random variables. A statistical analysis of the so obtained simulated solutions is then performed. Thus the computational methodology consists of solving first the deterministic problem for each set of the random input variables and finally realizing a statistical analysis.

2.1. Numerical Treatment of the Deterministic Problem

A general dynamic analysis of reinforced concrete (RC) framed structures containing cable-like members is in details described in [9,10]. Generally, a double discretization is applied: in space by finite elements and in time by a direct time-integration method. The RC structure is discretized to frame elements with generally non-linear behavior. For the cables, pin-jointed bar elements with unilateral behavior are used. The rigorous mathematical investigation of the problem can be obtained by using the variational or hemivariational inequality concept, see Panagiotopoulos [11,12]. So, the behavior of the cables and the generally non-linear behavior of RC elements, including loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading - reloading effects, can be expressed mathematically by the subdifferential relation:

$$s_i(d_i) \in \partial \mathrm{S}_i(d_i)\,. \tag{1}$$

Here, for the example case of a typical i-th cable element, s_i and d_i are the (tensile) force and the deformation (elongation), respectively, ∂ is the generalized gradient and S_i is the superpotential function [11-13].

Next, the incremental dynamic equilibrium for the assembled structural system with cables is expressed in matrix form by the equation:

$$\underline{\mathrm{M}}.\Delta\underline{\ddot{\mathrm{u}}}(\mathrm{t}) + \underline{\mathrm{C}}.\Delta\underline{\dot{\mathrm{u}}}(\mathrm{t}) + \underline{\mathrm{K}}_{\mathrm{T}}.\Delta\underline{\mathrm{u}}(\mathrm{t}) = \Delta\underline{\mathrm{p}}(\mathrm{t}) + \underline{\mathrm{T}}.\Delta\underline{\mathrm{s}}(\mathrm{t})\,. \tag{2}$$

Here $\underline{u}(t)$ and $\underline{p}(t)$ are the time dependent displacement and the given load vectors, respectively. $\underline{C}$ and $\underline{K}_T(\underline{u})$, are the damping and the time dependent tangent stiffness matrix, respectively. Dots over symbols denote derivatives with respect to time. $\underline{T}$ is a transformation matrix. By $\underline{s}(t)$ is denoted the time dependent cable stress vector with elements satisfying the relations (1)-(2).

The above matrix equation combined with the initial conditions consist the problem formulation, where, for given $\underline{p}(t)$, the vectors $\underline{u}(t)$ and $\underline{s}(t)$ are to be computed. For the numerical treatment of the above problem, the structural analysis software Ruaumoko [24] is herein used, as described in [9,10]. When the static case of the problem is only to be investigated, a Dynamic Relaxation approach [27] is appropriately used.

2.2. Numerical Treatment of the Probabilistic Problem

In order to calculate the random characteristics of the response of the considered RC buildings, the Monte Carlo simulation is used [21-25]. As mentioned, the·main element of a Monte Carlo simulation procedure is the generation of random numbers from a specified distribution. Systematic and efficient methods for generating such random numbers from several common probability distributions are available. The random variable simulation is implemented herein by using the technique of Latin Hypercube Sampling (LHS) [14,15].

In more details, a set of values of the basic design input variables can be generated according to their corresponding probability distributions by using statistical sampling techniques. As concerns the uncertain-but-bounded input parameters for the stochastic analysis, these are estimated here by using available upper and lower bounds, denoted as U_B and L_B respectively. So, the mean values are estimated as $(U_B + L_B)/2$.

Such design variables for the herein considered RC buildings are the uncertain quantities describing the backbone diagrammes of non-linear constitutive laws, e.g. plastic-hinges behavior, and the spatial variation of input old materials parameters. Concerning the plastic hinges in the end sections of the frame structural elements, a typical normalized moment-normalized rotation backbone is shown in Figure 1, see [17]. This backbone hardens after a yield moment M_y, having a non-negative slope of a_h up to a corner normalized rotation (or rotational ductility) μ_c where the negative stiffness segment starts. The drop, at a slope of a_c, is arrested by the residual plateau appearing at normalized height r that abruptly ends at the ultimate rotational ductility μ_u. The normalized rotation is the rotational ductility $\mu=\theta/\theta^{yield}$.

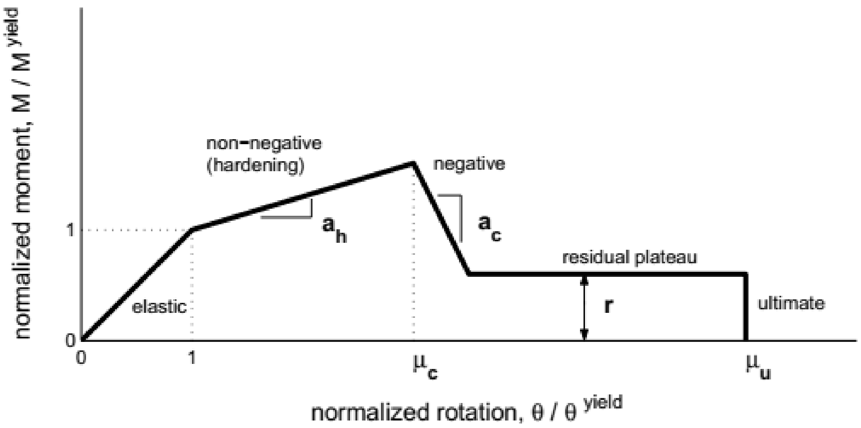

Figure 1. Representative moment-rotation backbone diagramme for plastic hinges [17].

The above six backbone parameters in Fig. 1, namely a_h, a_c, μ_c, r, μ_u and a_{My},= M/M_y are assumed to vary independently from each other according to a truncated Normal distribution,. Typical distribution properties for these uncertain parameters concerning plastic hinges according to [17] are given in Table 1. The table values concern the mean value, the coefficent of variation (COV) and the upper and lower bounds of the truncated Normal distribution.

Table 1**.** The uncertain-but-bounded parameters for a typical plastic hinge [32]

	Mean	COV	L_B (min)	U_B (max)	Distr. type
a_{My},	1.0	20%	0.70	1.30	Normal-tr.
a_h	0.1	40%	0.04	0.16	Normal-tr.
μ_c	3.0	40%	1.20	4.80	Normal-tr.
a_c	-0.5	40%	-0.80	-0.20	Normal-tr.
r	0.5	40%	0.20	0.80	Normal-tr.
μ_u	6.0	40%	2.40	9.60	Normal-tr.

3. NUMERICAL EXAMPLE

In Fig. 1 is shown an old industrial RC plane frame structure, which had been initially constructed with two more internal columns in the ground floor. These columns are shown as dashed lines and have been removed due to degradation caused by environmental actions. Following [1, 2], the axial loads, which were initially undertaken by these two columns, are now shown as the two applied vertical concentrated loads of 180 kN and 220 kN.

Due to removal of the above two columns, and after structural assessment and in order to prevent a progressive collapse, the initial RC frame F0 of Fig. 1 is strengthened by ten (10) steel cables (tension-only tie-elements) as shown in Fig. 3. In the so formulated system, it is wanted to be computed which of the cables are activated and which are not, under the considered critical static loading of Fig. 2. This critical loading includes the wind-horizontal loading of 8 kN/m and the static-equivalent seismic loads of shown 24 kN and 18 kN.

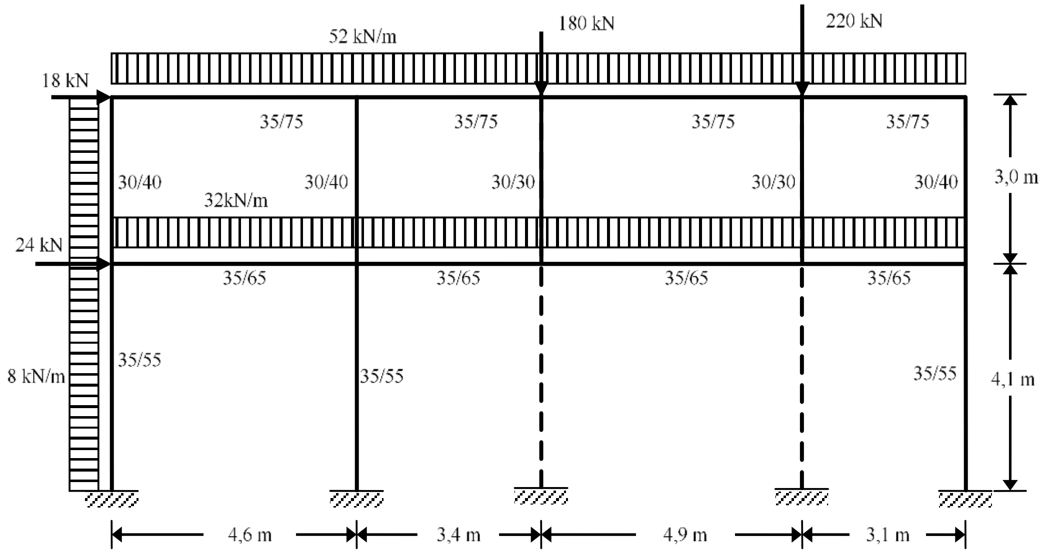

Figure 2. The initial RC frame F0.

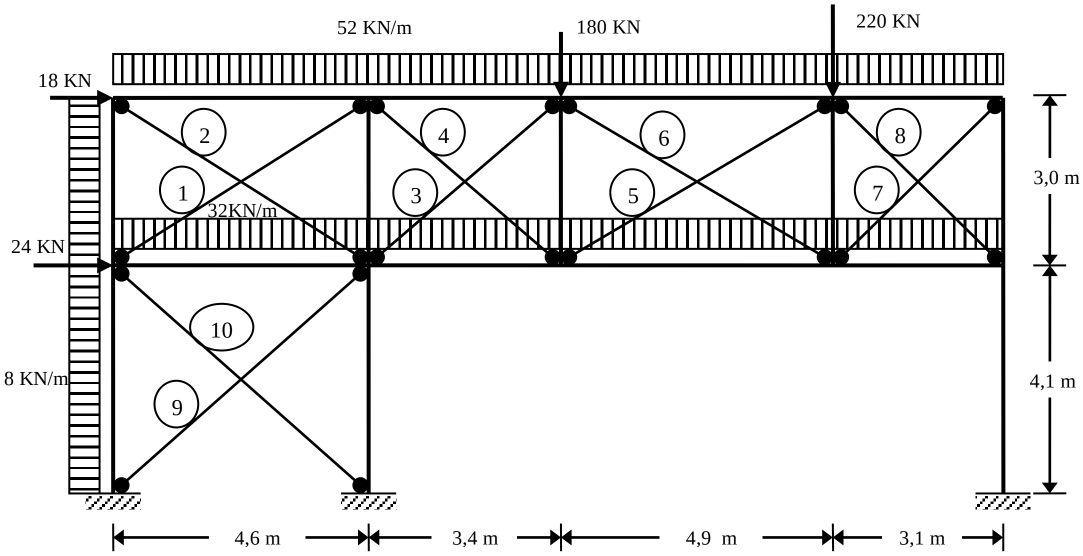

Figure 3. The RC frame strengthened by 10 cables.

As concerns the uncertain-but-bounded input parameters for the stochastic analysis, these are estimated here by using available upper and lower bounds, denoted as U_B and L_B respectively The concrete class of initial frame is estimated to be C12/15 and the steel class is S220. According to JCSS (Joint Committee Structural Safety), see [18], concrete strength and elasticity modulus follow a Normal distribution and the steel strength follows the Lognormal distribution. So the statistical characteristics of the input random variables concerning the building materials are estimated to be as shown in Table 1. By COV is denoted the coefficient of variation.

Using Ruaumoko software [24], the columns and the beams of the frame are modeled by prismatic frame RC elements. The effects of cracking on columns and beams are estimated by applying the guidelines of [6,7,28]. So, the stiffness reduction due to cracking results to effective stiffness I_{eff} with mean values of 0.60 I_g for the external columns, 0.80 I_g for the internal columns and 0.40 I_g for the beams, where I_g is the gross inertia moment of their cross-section. Nonlinearity at the two ends of the RC frame structural elements is idealized by using one-component plastic hinge models, following the Takeda hysteresis rule [26].

Table 1. Statistical data for the building materials treated as random variables

	Disribution	mean	COV
Compressive strength of concrete	Normal	12.0 MPa	15%
Yield strength of steel	Lognormal	191.3 MPa	10%
Elasticity modulus, concrete	Normal	26.0 GPA	8%
Elasticity modulus, steel	Normal	200 GPA	4%

The strengthening cable members have a cross-sectional area F_r = 8 cm^2 and are of steel class S1400/1600 with elasticity modulus E_s = 210 GPa. The cable constitutive law concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. hysteretic behavior, has the diagram depicted in Fig. 4.

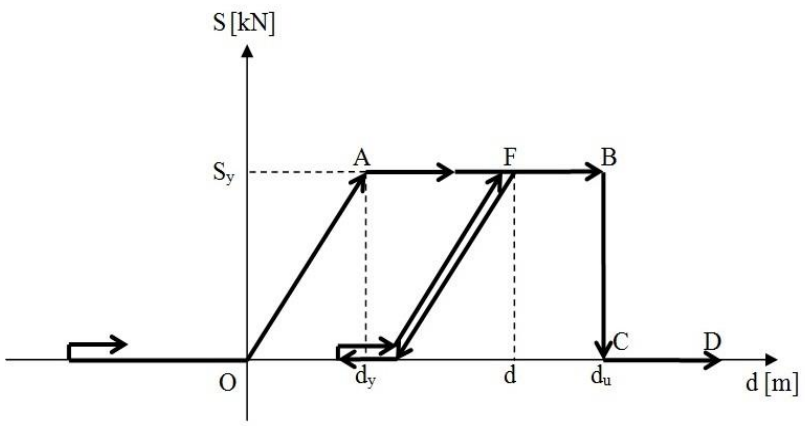

Figure 4. Constitutive law of the cable-elements

The application of the proposed numerical procedure by using 250 Monte Carlo samples and applying a pseudo Dynamic Relaxation approach [27] gives the following representative results for the cable-elements :

a. The elements of the slackness vector **v**, where: **v** = $[v_1,\ldots, v_{10}]^T$, are computed to have the following mean values of the slackness of the no activated cable-elements:
$v_1 = 0.837*10^{-3}$ m, $v_3 = 10.298*10^{-3}$ m, $v_5 = 1.027*10^{-3}$ m,

$v_8 = 9.468*10^{-3}$ m, $v_{10} = 1.498*10^{-3}$ m.
The relevant coefficient of variation is COV=17.84%.

b. The elements of the stress vector **s**, where: **s** = $[S_1, S_2, \ldots, S_{10}]^T$, are computed to have the following stress-values (in kN) for the non-active cables:
$S_1 = S_3 = S_5 = S_8 = S_{10} = 0.0$,
whereas for the active cables, the mean tension values are:
$S_2 =$ 10.32 kN, $S_4 =$ 347.12 kN, $S_6 =$ 19.08 kN,
$S_7 =$ 343.84 kN, $S_9 =$ 26.04 kN.
The relevant mean coefficient of variation is COV=22.37%.

Thus, cables 2,4,6,7 and 9 are the only ones which have been activated, appearing non-zero tension. The other cables 1,3,5,8 and 10 cannot contribute to the system resistance against progressive collapse under the given loads of Fig. 2.

Obviously, by parametric investigation of the characteristics of the cable-element (sectional area, elasticity modulus etc.), a parametric upgrading investigation of the strengthened structure can be obtained, see [10], in order to avoid a progressive collapse.

4. CONCLUDING REMARKS

The presented computational approach can be used effectively for the probabilistic numerical investigation concerning the inelastic behaviour of existing RC framed-structures subjected to removal of some degraded structural elements and strengthened by cable elements. The case of uncertain-but-bounded input-parameters, considered as interval parameters with known upper and lower bounds, is treated effectively. Finally, by parametric investigation of the characteristics of the cable-elements (sectional area, elasticity modulus etc.), the required upgrading of the remaining structure can be obtained in order to avoid a progressive collapse.

REFERENCES

[1] Starossek, U. (2017). *Progressive collapse of Structures*. 2nd edition. Thomas Telford Ltd, London.
[2] Zoli, T. & Woodward, R. (2005). *Design of long span bridges for cable loss*. In Proceedings of IABSE Symposium, Structures and Extreme Events, Lisbon, 2005.
[3] Izzuddin BA, Vlassis AG, Elghazouli AY, Nethercot DA. (2008). *Progressive collapse of multi-storey buildings due to sudden column loss — Part I: Simplified assessment framework*. Engineering Structures 2008;30(5): 1308–18.
[4] Vlassis AG. (2007). *Progressive collapse assessment of tall buildings*. Ph.D. thesis. Imperial College London (UK); 2007.
[5] Miao, F., Ghosn, M. (2016). *Reliability-based progressive collapse analysis of highway bridges*. Structural safety, 63, 33-46.
[6] Penelis, G., Penelis, Gr. (2019). *Concrete Buildings in Seismic Regions*. 2nd edition. CRC Press, Taylor and Francis Group Ltd, London.
[7] Fardis, M.N. (2009). *Seismic design, assessment and retrofitting of concrete buildings: based on EN-Eurocode 8*. Springer, Berlin.
[8] Dritsos, S.E. (2017). *Repair and strengthening of reinforced concrete structures*, (in greek). University of Patras, Greece.
[9] Liolios, A., Liolios, Ast., and Hatzigeorgiou, G., (2013). *A Numerical Approach for Estimating the Effects of Multiple Earthquakes to Seismic Response of Structures*

Strengthened by Cable-Elements. Journal of Theoretical and Applied Mechanics, Vol.43, No. 3, pp. 21-32, 2013.

[10] Liolios A., Partov D., Liolios K., Folic B. and Skodras G., (2019). *Progressive Collapse of Existing RC Structures due to Environmental Effects: A Numerical Approach for Strengthening by Ties*. In: Georgiev V. et al (Eds.), *Proceedings of the VSU 2019 XIX Intern. Scient. Conference*, Article 4.01, 17-19 October 2019, Sofia, Bulgaria.

[11] Panagiotopoulos, P.D. (1985). *Inequality Problems in Mechanics and Applications. Convex and Nonconvex Energy Functions*, Birkhäuser Verlag, Basel, Boston.

[12] Panagiotopoulos, P. (1993). *Hemivariational Inequalities. Applications in Mechanics and Engineering*, Springer Verlag, Berlin.

[13] Mistakidis, E.S. and Stavroulakis, G.E. (1998). *Nonconvex optimization in mechanics. Smooth and nonsmooth algorithmes, heuristic and engineering applications*. Kluwer, London.

[14] Papadrakakis, M., & Stefanou, G. (Eds.), (2014). *Multiscale modeling and uncertainty quantification of materials and structures*. Springer, Berlin, 2014.

[15] Thomos, G. and Trezos C. (2006). *Examination of the probabilistic response of reinforced concrete structures under static non-linear analysis*. Engineering Structures 28 (2006), pp. 120–133.

[16] Strauss, A., Hoffmann, S., Wendner, R., & Bergmeister, K. (2009). *Structural assessment and reliability analysis for existing engineering structures, applications for real structures*. Structure and Infrastructure Engineering, 2009, vol. 5(4), pp. 277-286.

[17] Vamvatsikos, D., & Fragiadakis, M. (2010). *Incremental dynamic analysis for estimating seismic performance sensitivity and uncertainty*. Earthquake Engineering & Structural Dynamics, vol. 39(2), pp. 141-163.

[18] JCSS, (2001). Joint Committee on Structural Safety. *Probabilistic Model Code-Part 1: Basis of Design (12th draft)*., March 2001. Available from: http://www.jcss.ethz.ch/.

[19] Muscolino G. and A. Sofi, (2011). *Stochastic analysis of structures with uncertain-but-bounded parameters*. In: G. Deodatis, P.D. Spanos (Eds.), *Computational Stochastic Mechanics*, Research Publishing, Singapore, 2011, pp. 415–427.

[20] Impollonia, N. and Muscolino, G. (2011). *Interval analysis of structures with uncertain-but-bounded axial stiffness*. Comput. Methods Appl. Mech. Engrg., vol. 200, pp. 1945–1962.

[21] Králik, J. (2013). Deterministic and Probabilistic Analysis of Steel Frame Bracing System Efficiency, Applied Mechanics and Materials, Vol. 390, pp 172-177.

[22] Krejsa, M., & Kralik, J. (2015). Probabilistic computational methods in structural failure analysis. Journal of Multiscale Modelling, 6(03).

[23] Dimov, I. T. (2008). *Monte Carlo methods for applied scientists*. World Scientific.

[24] Ang, A. H., & Tang, W. H. (1984). *Probability concepts in engineering planning and design, vol. 2: Decision, risk, and reliability*. New York: Wiley.

[25] Kottegoda, N., & Rosso, R. (2000). *Statistics, probability and reliability for civil and environmental engineers*. McGraw-Hill, London.

[26] Carr, A.J. (2008). *RUAUMOKO - Inelastic Dynamic Analysis Program*. Dep. of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

[27] Papadrakakis, M. (1981). *A method for the automatic evaluation of the dynamic relaxation parameters*. Computer Methods in Applied Mechanics and Engineering, 25(1), 35-48.

[28] Pauley T. and Priestley, M.J.N., (1992). *Seismic design of reinforced concrete and masonry buildings*. Wiley, New York.