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Juraj KRÁLIK[1], Juraj KRÁLIK, jr.[2]

DETERMINISTIC AND PROBABILISTIC ANALYSIS OF NPP COMMUNICATION BRIDGE RESISTANCE DUE TO EXTREME LOADS

Abstract

This paper presents the experiences from the deterministic and probability analysis of the reliability of communication bridge structure resistance due to extreme loads - wind and earthquake. On the example of the steel bridge between two NPP buildings is considered the efficiency of the bracing systems. The advantages and disadvantages of the deterministic and probabilistic analysis of the structure resistance are discussed. The advantages of the utilization the LHS method to analyze the safety and reliability of the structures is presented.

Keywords

Probability, extreme loads, steel bridge, bracing, FEM, LHS, ANSYS.

1 INTRODUCTION

This paper deals with the resistance of the steel bracing systems of the bridge between two buildings in the nuclear power plants (NPP) [10]. The international organization IAEA in Vienna [2, 3, 4 and 5] set up the design requirements for the safety and reliability of the NPP structures. The methodology of the seismic analysis of the structure behavior and the design of the structure under extreme loads are the object of the various authors [9, 11, 14, 18 and 22]. In the case of NPP structures the characteristic values of the seismic loads are determined on the base of the IAEA requirements [2] by mean return period of the extreme loads which is equal to one per 104 years [14]. The methodology of the probabilistic analysis of the structure reliability is described in various papers and practical applications [8, 11, 12, 13, 14, 15, 16, 17 and 20]. The reliability analysis is based on the partial factor methods in accordance of the Eurocode 1990 [2]. In the present the method of the partial factor is favourable in the practice. The Eurocode 1990 [2] and JCSS [6] recommends the use of three levels of the reliability analysis. Level III methods are seldom used in the calibration of design codes because of the frequent lack of statistical data. The measure of reliability in Eurocode 1990 [1] is defined by the reliability index β.. The reliability index depends on the criterion of the limited state. The standard JCSS [6] required the measure of reliability in dependency on the safety level. The probability of the failure Pf can be determined using simulation method on the base of MONTE CARLO, LHS and others.

2 RELIABILITY FUNCTION

Most problems concerning the reliability of building structures [1, 6, 8, 15, 16, 17, 19 and 20] are defined today as a comparison of two stochastic values, loading effects E and the resistance R,

[1] Prof. Ing. Juraj Králik, PhD., Department of Structural Mechanics, Faculty of Civil Engineering, Slovak University of Technology in Bratislava, Radlinského 11, 813 68 Bratislava, Slovakia, e-mail: juraj.kralik@stuba.sk.

[2] Ing. Juraj Králik, PhD, Institute of Architectural Constructions and Civil Engineering Structures, Faculty of Architecture, Slovak University of Technology in Bratislava, Námestie Slobody 19, 812 45 Bratislava, Slovakia, e-mail: kralik@fa.stuba.sk.

depending on the variable material and geometric characteristics of the structural element. In the case of a deterministic approach to a design, the deterministic (nominal) attributes of those parameters R_d and E_d are compared.

The deterministic definition of the reliability condition has the form

$$R_d \geq E_d \tag{1}$$

and in the case of the probabilistic approach, it has the form

$$RF = R - E > 0 \qquad \text{or} \qquad RF = 1 - E/R > 0 \tag{2}$$

The reliability function RF can be expressed generally as a function of the stochastic parameters X_1, X_2 to an used in the calculation of R and E.

$$RF = g(X_1, X_2, ..., X_n) \tag{3}$$

The probability of failure can be defined by the simple expression

$$P_f = P\left[R < E\right] = P\left[\left(R - E\right) < 0\right] \tag{4}$$

In the case of simulation methods the failure probability is calculated from the evaluation of the statistical parameters and theoretical model of the probability distribution of the reliability function $g_{(X)}$. The failure probability is defined as the best estimation on the base of numerical simulations in the form [15]

$$P_f = \frac{1}{N}\sum_{i=1}^{N} I\left[g\left(X_i\right) \leq 0\right] \tag{5}$$

where N in the number of simulations, $g_{(.)}$ is the failure function, I[.] is the function with value 1, if the condition in the square bracket is fulfilled, otherwise is equal 0.

Reliability of the bearing structures is designed in accordance of standard requirements STN ENV 1993-1-1 and ENV 1990 [1] for ultimate and serviceability limit state. Horizontal reinforced plane structures are designed on the bending and shear loads for ultimate limit state function in the next form

$$g(M) = 1 - M_E/M_R \geq 0 \qquad\qquad g(V) = 1 - V_E/V_R \geq 0 \tag{6}$$

where M_E, V_E are design bending moment and design shear force of the action and M_R, V_R are resistance bending moment and resistance shear force of the structure element.

In the case of the combination of the action of the normal forces and bending moments the yield function F(.) must be used as follows

$$g(N, M) = 1 - F\left(N_E, M_E\right) / F\left(N_R, M_R\right) \geq 0 \tag{7}$$

The failure function (7) for the linearized interaction diagram (Figure 1) may be defined in the form

$$\frac{N_E}{N_{Ru}} + \frac{M_E}{M_{Ru}} = 1 \tag{8}$$

where N_{Ru} and M_{Ru} are the values of limit normal force and moment on the axis of interaction diagram $N_{Ru} = N_R\left(M = 0\right)$ and $M_{Ru} = M_R\left(N = 0\right)$.

The total internal forces of the action effect are defined as follows

$$M_E = M_{NS} + M_S \qquad\qquad N_E = N_{NS} + N_S \tag{9}$$

where N_{NS}, M_{NS} are initial values of normal forces and moments due to no seismic load and N_S, M_S are normal forces and moments of the seismic load.

The moment of resistance M_R on the interaction diagram can be calculated from known normal force N in the form

$$M_R = M_{Ru} - \left(M_{Ru} / N_{Ru} \right) N \tag{10}$$

The moment of action effect M_E can be expressed for an initial values N_{NS}, M_{NS} and an increment of pressure N_S, M_S in the form

$$M_E = M_{NS} - \left(M_S / N_S \right) \left(\mathrm{N} - N_{NS} \right) \tag{11}$$

The failure condition will be fulfilled if we have

$$M_E = M_R \tag{12}$$

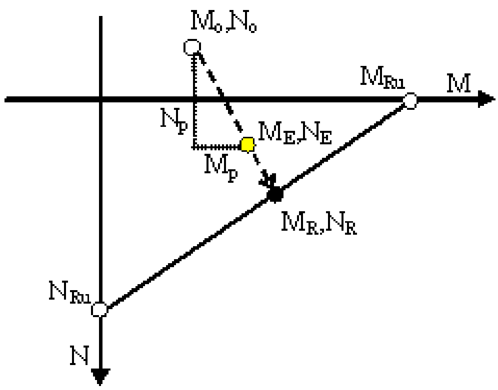

Fig. 1: Linearization of interaction diagram of RC section

If the relation (10) and (11) give (12) we have the value of normal force N on the interaction diagram $\left(N = N_R \right)$

$$N_R = \frac{M_{Ru} - M_{NS} + \frac{M_S}{N_S} N_{NS}}{\frac{M_{Ru}}{N_{Ru}} + \frac{M_S}{N_S}} \tag{13}$$

3 LOAD COMBINATION

The load combination of the deterministic calculation is considered according to ENV 1990 [1] and IAEA [4] for the ultimate limit state of the structure as follows:

Deterministic method – extreme design situation

$$E_d = G_k + Q_k + A_{Ed} \tag{14}$$

where G_k is the characteristic value of the permanent dead loads, Q_k - the characteristic value of the permanent live loads, A_{Ed} - the design value of the extreme loads, $A_{Ed.k}$ - the characteristic design value of the extreme loads.

In the case of probabilistic calculation and the ultimate limit of the structure the load combination [1] we take following:

Probabilistic method – extreme design situation

$$E = G + Q + A_E = g_{\text{var}} G_k + q_{\text{var}} Q_k + a_{\text{var}} A_{E.k} \tag{16}$$

where g_{var}, q_{var}, a_{var} are the variable parameters defined in the form of the histogram calibrated to the load combination in compliance with Eurocode and JCSS requirements.

The extreme wind was defined for the mean return period 10^4 years by the wind speed 54.47m/s and wind pressure 1.127 kN/m^2 [10]. The seismic load was considered for the same return period 10^4 years as SL-2 [2]. The peak ground acceleration was set up as 0.15g for the horizontal direction [14]. The spectrum acceleration response was calculated for the locality Mochovce in the three characteristic frequency values. The shape of the spectrum response acceleration is similar to the same in the NUREG 0098 [14]. The seismic response was solved by linear response spectrum method. The spectral analysis results from linear behavior of structures and the appropriate damping due to structure plasticity is considered by proportional damping for the whole structure or separately by materials. The seismic response for each direction of excitation was calculated particularly by spectrum response method using combination rule SRSS

$$E_i = \sum_{m=1}^{N_{\text{mod}}} E_{m.i} \,, \tag{16}$$

where "i" is excitation direction (i = X, Y, Z), "m" is the mode number from the modal analysis, "N_{mod}" is the total number of modes. Total seismic response was calculated by ASCE 4/98 in the form

$$E_{tot} = E_x + 0.4E_y + 0.4E_z \text{ or } E_{tot} = 0.4E_x + 0.4E_y + E_z \text{ or } E_{tot} = 0.4E_x + E_y + 0.4E_z \tag{17}$$

The maximum from all possibilities is taken to design structure.

4 UNCERTAINTIES OF INPUT DATA

The uncertainties of the input data – action effect and resistance are for the case of the probabilistic calculation of the structure reliability defined in JCSS and Eurocode 1990.

Tab. 1: Probabilistic model of input parameters

Name	Quantity	Charact. value	Variable paramet.	Histogram	Mean	Stand. deviation	Min. value	Max. value
Material	Young's modul.	E_{k}	e_{var}	Normal	1	0.120	0.645	1.293
Load	Dead	G_{k}	g_{var}	Normal	1	0.010	0.921	1.079
	Live	Q_{k}	q_{var}	Gumbel	0.60	0.200	0	1
	Earthquake	$A_{\text{E,.k}}$	a_{var}	Gama(T.II)	0.67	0.142	0.419	1.032
	Wind extrem	$A_{\text{W,k}}$	w_{var}	Gumbel	0.30	0.150	0.500	1.032
Resistance	Steel strength f_{sk}	F_{k}	f_{var}	Lognormal	1	0.100	0.726	1.325
Model	Action uncertaint	θ_{E}	Te_{var}	Normal	1	0.100	0.875	1.135
	Resistance uncert.	θ_{R}	Tr_{var}	Normal	1	0.100	0.875	1.135

The stiffness of the structure is determined with the characteristic value of Young's modulus E_k and variable factor e_{var} (Tab.1). A load is taken with characteristic values G_k, Q_k, $A_{E,k}$, $A_{W,k}$ and variable factors g_{var}, q_{var}, a_{var} and w_{var} (Tab.1). The uncertainties of the calculation model are considered by variable model factor θ_R and variable load factor θ_E for Gauss's normal distribution.

5 SEISMIC ANALYSIS OF THE NPP STRUCTURES

On base of the experience from the reevaluation programs in the membership countries IAEA in Vienna the seismic safety standard No.28 was established at 2003 [2].

- Seismic safety evaluation programs should contain three important parts
- The assessment of the seismic hazard as an external event, specific to the seismotectonic and soil conditions of the site, and of the associated input motion;
- The safety analysis of the NPP resulting in an identification of the selected structures, systems and components (SSSCs) appropriate for dealing with a seismic event with the objective of a safe shutdown;
- The evaluation of the plant specific seismic capacity to withstand the loads generated by such an event, possibly resulting in upgrading.

The earthquake resistance analysis of NPP buildings in Mochovce was based on the recommends of international organization IAEA in Vienna, EUROCODE 2, 7 and 8, CEB and Slovak National Standards. The seismic load for the Mochovce site was defined by peak ground acceleration (PGA) and local seismic spectrum in dependence on magnitude and distance from source zone of earthquake. Firstly the value of PGA was defined at 1994 ($PGA_{RLE}=0.1g$) follow in accordance of the results of seismological monitoring this locality at 2003 ($PGA_{UHS}=0.142g$ and $PGA_{HS}=0.143g$).

Methodology of structure resistance verification is elaborately described by Králik [14] . There are illustrated the procedures, requirements and criterion of calculation models and methods for design of structure reliability. There are two principal methodology available for seismic design of NPP structures - deterministic (SMA- seismic margin assessment) and probabilistic (SPRA – seismic probabilistic risk assessments. The objective of seismic margin assessment (SMA) is to determine for a nuclear power plant the high-confidence-of-a-low-probability-of-failure (HCLPF) capacity for a preselected seismic margin earthquake (SME), which is always chosen higher than the design basis input. In probabilistic terms, the HCLPF is expressed as approximately a 95% confidence of about a 5% or less probability of failure.

The concept of the HCLPF (High Confidence Low Probability Failure) capacity is used in the SMA (Seismic Margin Assessment) reviews to quantify the seismic margins of NPPs [6]. In simple terms it correspond to the earthquake level at which, with high confidence (≥ 95%) it is unlikely that failure of a system, structure or component required for safe shutdown of the plant will occur (< 5% probability).

The value of the HCLPF parameter depends on the equipment structure or component resistance (R) and the corresponding effect of action (E) using elastic or inelastic behavior. The following equation follows for the strength and response (R/E) in respect to linear elasticity

$$\left(R/E\right)_{el} = R\bigg/\left[\left(E_{si}^2 + E_{sa}^2\right)^{1/2} + E_{NS}\right] \tag{18}$$

where E_{Si} or E_{Sa} is seismic response to RLE (SL-2) inertial actions, or corresponding different seismic support movement, respectively, calculated according to linear elasticity. Then E_{NS} is a total response to all the co-incidental non-seismic bearings in the given combinations.

Analogically, considering the elastic-plastic effect

$$\left(R/E\right)_{ep} = R \Big/ \left\{ \left[\left(E_{Si}/k_D\right)^2 + \left(E_{Sa}/k_D\right)^2 \right]^{1/2} + E_{NS} \right\} \tag{19}$$

where k_D is ductility coefficient ($k_D \geq 1.0$). The partial seismic response E_{Sa} in equation (19) is really multiplied, not divided, by the ductility coefficient. If SME is greater than RLE (SL-2), then $\left(R/E\right)_{ep}$ is greater than 1.0 and vice-versa. However, the $\left(R/E\right)_{el}$ and $\left(R/E\right)_{ep}$ ratios do not define the multiplication factors for RLE (SL-2) to gain the HCLPF seismic margin value. These factors are calculated as follows:

$$\left(FS\right)_{el} = \left(R - E_{NS}\right) \Big/ \left(E_{Si}^2 + E_{Sa}^2\right)^{1/2} \tag{20}$$

$$\left(FS\right)_{ep} = \left(R - E_{NS}\right) \Big/ \left[\left(E_{Si}/k_D\right)^2 + \left(E_{Sa}/k_D\right)^2 \right]^{1/2} \tag{21}$$

The equation (10) is valid provided that $\left(FS\right)_{ep} > \left(FS\right)_{el}$ and it can be significantly simplified if the E_{Sa} response to different seismic support movement as a result of RLE (SL-2) is negligible or it does not need to be considered. Then

$$\left(FS\right)_{ep} = \left(FS\right)_{el} k_D \tag{22}$$

Generally it follows

$$HCLPF\left(CDFM\right) = \left(FS\right)_{ep} PGA_{RLE=SL-2} \quad \text{(in horizontal direction)} \tag{23}$$

and this value must always be HCLPF > ZPA.

The HCLPF seismic margin value can also be determined via a non-linear elastic-plastic calculation (e.g. limit analysis defined in the ASME BPVC Section III – Mandatory Appendix XIII).

6 COMPUTATIONAL MODEL OF THE BRIDGE STRUCTURE

The steel bridge connects the auxiliary building, reactor building and ventilating chimney of the JEMO NPP [10]. The length of bridge structures is equal 20.3 and 23 m. The bottom level of the bridge is at +6.0m and the top level at +10 m. The complex of the technology pipes is under bottom level. The total width of bridge is 5 672 mm and the height is 7 260 mm. The principal longitudinal beams are made from the steel profile I and 2U. The transversal beams are from the I profiles. The bridge is supported by columns from I profiles at modulus 4.7 m. The horizontal bracing system is made from 2L profiles at bottom and top level of bridge. The support structures of the technology pipes is from the 2T profiles. The roof panel of BDP are putting on steel profile panels type VSZ.

Tab. 2: Modal characteristics of the bridge

Model	Mode X		Mode Y		Mode Z	
	Frequency [Hz]	Mass fract. [%]	Frequency [Hz]	Mass fract. [%]	Frequency [Hz]	Mass fract. [%]
Original	3.89	54.10	1.81	39.69	7.37	6.71

The FEM model was set up by link, beam and shell elements in program ANSYS [11]. This model has 5858 elements and 4876 nodes. The comparisons of the modal characteristics are presented in the Table 2. The structure of the bridge is sensitive to the excitation in the direction Y (Figure 3).

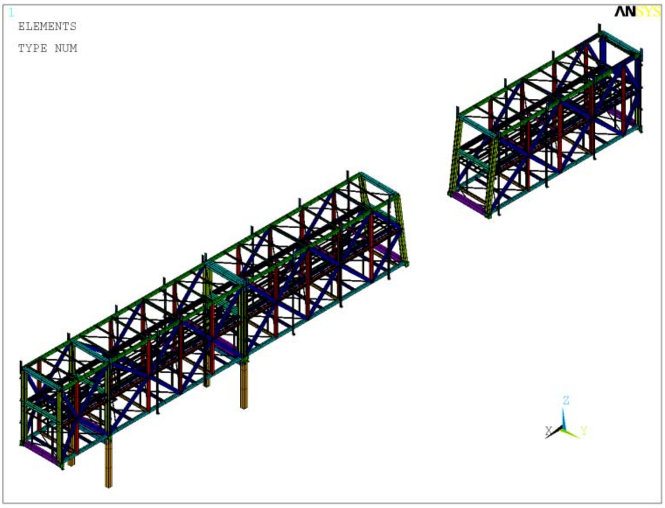

Fig. 2: The computational model of the steel bridge and the support system

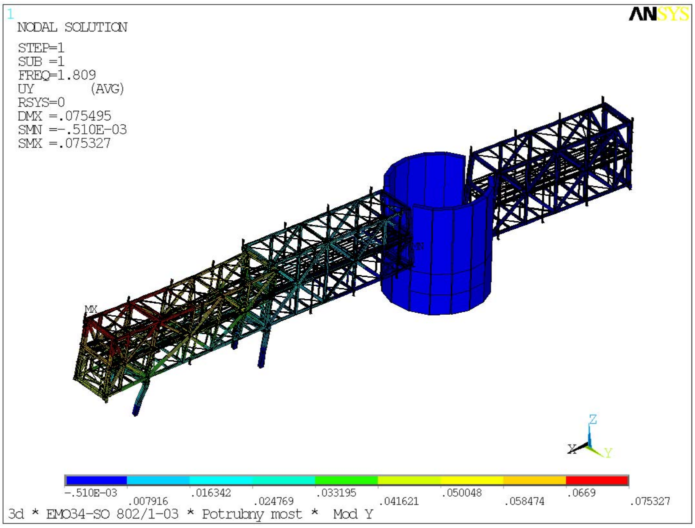

Fig. 3: The mode shape in direction Y for f1 = 1,809 Hz

7 RECAPITULATION OF THE NUMERICAL ANALYSIS

The elements of the bridge steel structure were designed in accordance of the Eurocode requirements described below. The results from the design check of the deterministic analysis are shown in Table 3. There are described the safety level of the critical elements of the bridge structures with the support in accordance of the Eurocode [1].

Tab. 3: Comparison of the design check of the original and upgraded bridge

Load case	Capacity ratio of Bridge Elements [%]			
	Column	Longitudinal Beam	Cross Beam	Bracing
Extreme wind	65.0	49.1	54.8	68.9
Earthquake	51.7	90.3	97.7	63.8

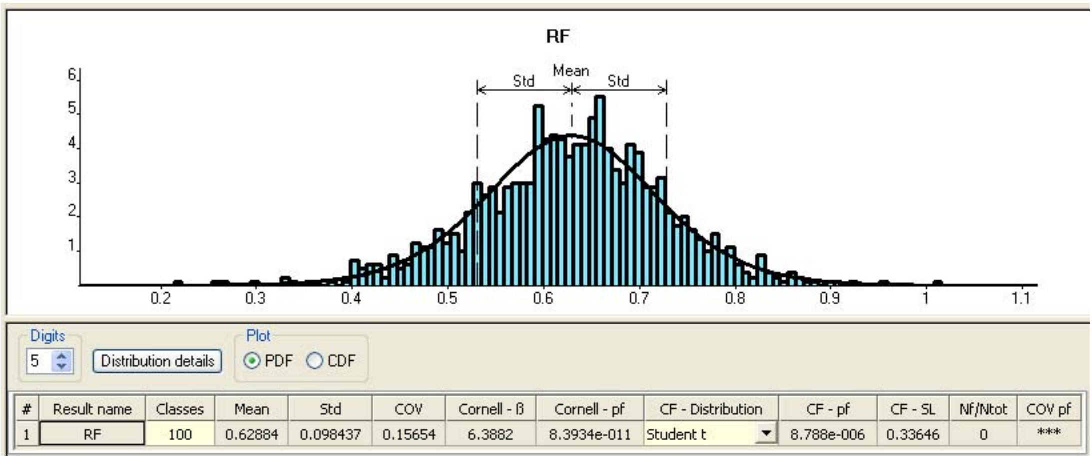

Fig. 4 The density of the reliability function *RF* – bracing system

The probabilistic analysis was realized using 1000 LHS simulations in program FReET [20]. The uncertainties of the input data was considered in the form of the histograms (see Table 3). The density of the probability of the failure (Figure 4) presents the reliability function in the form of the equation (5).

8 CONCLUSION

This paper presents the reliability analysis of the steel bridge support resistance due to extreme loads – wind and earthquake. The extreme loads were defined for mean return period equal to one per 104 years in accordance of the IAEA requirements for NPP structures. The reliability of the original and upgraded FEM model of bridge was calculated using the deterministic and probabilistic analysis. The uncertainties of the input data – action effect and resistance were considered by the partial factors in the case of deterministic analysis and in the form of the histograms on the base of the Eurocode and JCSS. The critical elements of the structure were identified on the base of the deterministic analysis. The effect of the extreme wind was worse than earthquake SL-2 with PGA=0.15*g*. The probability of the bridge bracing failure was equal to $P_f < 10^{-6}$ on the base of the LHS simulation.

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Reviewers:

Doc. Ing. Pustka David, Ph.D., Department of Building Structures, Faculty of Civil Engineering, VŠB-Technical University of Ostrava, Czech Republic.

Doc. Ing. Michal Tomko, PhD., Department of Structural Mechanics, Faculty of Civil Engineering, Technical University of Košice, Slovakia.