

NONLINEAR ANALYSIS OF THE EXTREME WIND FRAGILITY OF THE REACTOR HALL FRAME

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Abstract. This paper describe the methodology and the results of the safety analysis of the Nuclear Power Plant structures under impact of the extreme climatic loads. In the case of the Nuclear Power Plant structures, the design criteria are stronger. The requirements of the international agency IAEA and NRC standards are based on the probability of mean return period equal to one per 10^4 years. The fragility curve of the extreme wind was determined on the base of the nonlinear probability analysis of the steel hale frame considering material and geometric nonlinearity using ANSYS software. The failure mode of the NPP structures is expressed by High Confidence of Load Probability of Failure (HCLPF).

Keywords

NPP, Nonlinear, Safety, Fragility, Probability, Extreme Wind, ANSYS.

1. Introduction

This paper deals with the resistance of the steel hale frame of the nuclear power plant (NPP) with reactor VVER440. These analyses are based on the recommendations of IAEA and US NRC [1-14], experience from similar analyses of NPPs abroad [15, 16], new findings from probabilistic analyses of structures and own experience from previous analyses [16]. The methodology of probabilistic analysis of NPP structures are based on the analytical solution and simulation methods. [17-34]. These analyses are based on the current results of monitoring the material properties of the NPP structures, as well as the results of the analyses of the resistance of individual elements of the important structures of the NPP objects under the influence of various types of initiating events.

The international organization IAEA in Vienna set up the design requirements for the safety and reliability of the NPP structures. The extreme environmental events (e.g. wind, temperature, snow, explosion...) are the important

loads from the point of the NPP safety performance. The extreme wind loads are defined with the probability of mean return period equal to one per 10^4 years. This paper deals with the analysis of the steel hale frame loaded with extreme wind load. The IAEA [6-10] and NRC standards [11-13] require setting up the probability of the structure failure during the extreme loads.

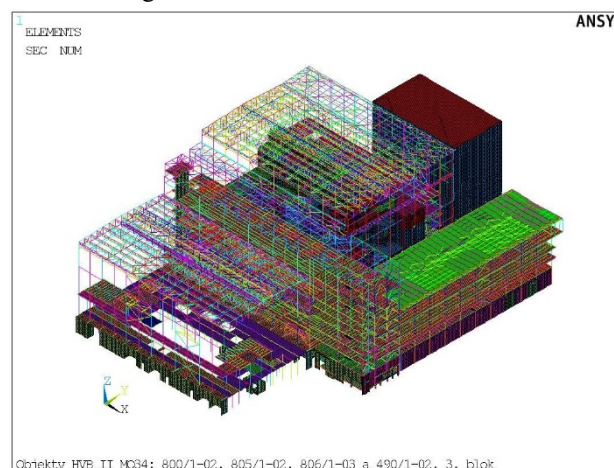


Fig. 1: FEM model of the NPP buildings.

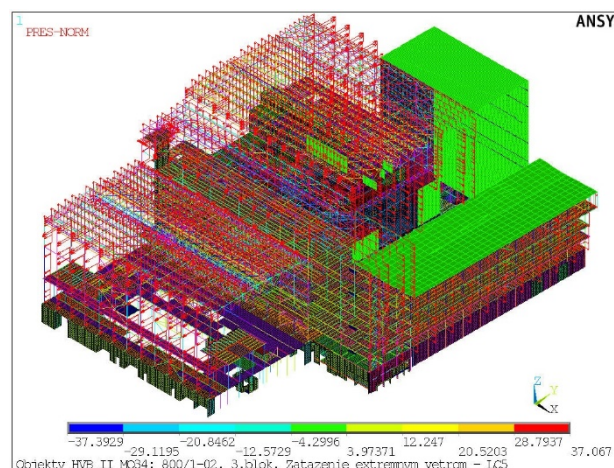


Fig. 2: FEM model of the extreme wind loads.

The NPP with the reactor VVER440 consist four buildings – reactor building, lengthwise side building, cross side building and turbine hall. The FEM model (Fig. 1, 2) of the NPP was created from following elements in software ANSYS – BEAM188, SHELL181, SURF154 and MASS21. The model has 996 917 elements with 444 426 nodes and 2 666 556 DOF.

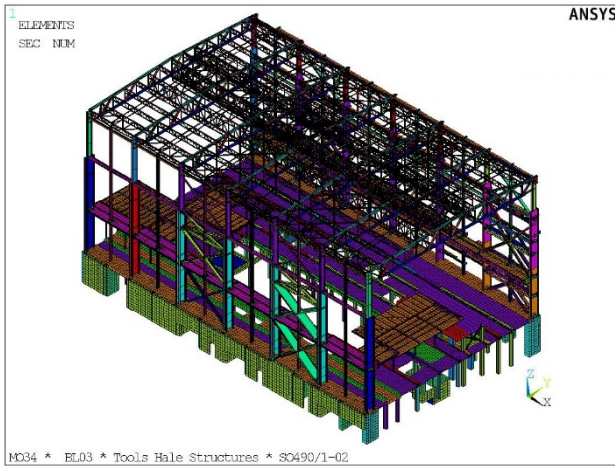


Fig. 3: FEM model of the NPP turbine hale.

The critical steel frame of turbine hall structure was investigated (Fig. 3). The FEM model of the steel hale frame consist the beam and mass elements of ANSYS program - BEAM188 and MASS21 (Fig. 4).

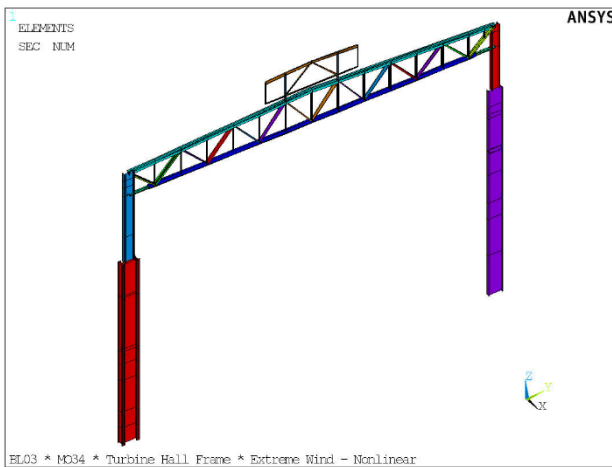


Fig. 4: FEM model of the critical steel hale frame.

The steel structures are made from steel S235 with following nominal material properties [15, 35]

$$f_y = 235 \text{ MPa} \text{ and } f_u = 360 \text{ MPa.} \quad (1)$$

The median material properties are defined with accordance to Eurocodes

$$f_{ym} = f_y (1 + 2\bar{\sigma}) \text{ and } f_{ym} = 1.2 f_y \text{ for } \bar{\sigma} = 0.1, \quad (2)$$

$$f_{um} = f_u (1 + 2\bar{\sigma}) \text{ and } f_{um} = 1.2 f_u \text{ for } \bar{\sigma} = 0.1.$$

The IAEA safety standards [2, 3] defines four acceptable levels of failure rate of structures, systems and

components of the NPP:

A. **High plastic failure** - large plastic deformation of structures, systems and components with probability of failure (collapse, crack) equal to or greater than 10^{-1} . The investigation of the state of the infringement must be carried out by non-linear analysis.

B. **Medium plastic failure** - large plastic deformation of structures, systems and components with probability of failure (collapse, crack) from 10^{-2} to 10^{-1} . The investigation of the state of the infringement must be carried out by non-linear analysis. Acceptable requirements for engineering element designs are defined in ASME and for AISC building structures.

C. **Small plastic failure** - limited plastic deformation of structures, systems and components with probability of failure (collapse, rupture) from 10^{-3} to 10^{-2} . The investigation of the structural safety is implemented with accordance of ASME's requirements.

D. **Without plastic failure** - elastic deformation of structures, systems and components with probability of failure (collapse, crack) equal in the range of 10^{-4} to 10^{-3} for acceptable steel strength criteria $0.8 f_y$ to $1.2 f_y$. For the probability of failure from 10^{-5} to 10^{-4} acceptable steel strength criteria are less than $0.8 f_y$.

The external event classification, if applied, does not imply different load levels for the external event scenarios and therefore the design of items classified for external events should refer only to the extreme values of design basic external events, or to a combination of these events where at least one of them is taken at, or close to, its extreme value.

2. Loads and load combinations

The load combination of the deterministic calculation was considered according to ENV 1990 [36] and IAEA requirements [6, 7] for the ultimate limit state of the structure as follows:

- Deterministic method – extreme design situation

$$E_d = G_d + Q_d + W_{Ed} \quad (3)$$

where G_d is the design value of the permanent dead loads, Q_d - the design value of the permanent live loads, W_{Ed} - the design value of the extreme wind load (with probability of getting higher value of 10^{-4}).

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

- Probabilistic method – extreme design situation

$$E = G + Q + W_E = g_{var} \cdot G_m + q_{var} \cdot Q_m + w_{var} \cdot W_{Em} \quad (4)$$

where g_{var} , q_{var} , w_{var} are the variable parameters defined in the form of the histogram calibrated to the load combination in compliance with Eurocode and G_m , Q_m and

W_{Em} are the median values of the permanent dead loads, live loads and extreme wind loads.

3. Probability nonlinear assessment

The safety of the building structures was determined by the safety function SF in the form [19, 20, 25 and 30]

$$SF = E/R \quad \text{and} \quad 0 \leq SF < 1 \quad (5)$$

where E is the action function and R is the resistance function.

The reliability function RF is defined in the form

$$RF = g(R, E) = 1 - SF = R - E > 0 \quad (6)$$

where $g(R, E)$ is the reliability function.

The probability of failure can be defined by the simple expression

$$P_f = P[R < E] = P[(R - E) < 0] \quad (7)$$

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

$$RF = g(X_1, X_2, \dots, X_n) \quad (8)$$

The failure function $g(\{X\})$ represents the condition (capacity margin) of the reliability, which can be either an explicit or implicit function of the stochastic parameters and can be single (defined on one cross-section) or complex (defined on several cross-sections, e.g., on a complex finite element model).

In the case of the nonlinear analysis, the correct solution of the elastic-plastic behavior of the structures is determined by the function plasticity. The HMM function of the plasticity was used for the nonlinear solution of the steel technology segments. This plasticity function is defined in the form

$$R = f_y \quad \text{and} \quad E = \sigma_{ef}, \quad (9)$$

where the effective stress (Von Mises stress).

The failure of the steel segments in the frame of the PSA analysis is defined by the limited values of the maximal strain deformation. This failure function is defined in the form

$$R = \varepsilon_{a,y,0} \quad \text{and} \quad E = \varepsilon_{ef}, \quad (10)$$

where the effective strain (Von Mises strain).

The failure probability is calculated from the evaluation of the statistical parameters and theoretical model of the probability distribution of the reliability function $Z = g(X)$ using the simulation methods. The failure probability is defined as the best estimation on the base of numerical simulations in the form

$$P_f = \frac{1}{N} \sum_{i=1}^N I[g(X_i) \leq 0] \quad (11)$$

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

4. Wind load

The load on a structure due to the wind is depended on both wind velocity and terrain roughness [37, 38]. The wind velocity and the velocity pressure are composed of a mean and a fluctuating component. The mean wind velocity v_m should be determined from the basic wind velocity v_b which depends on the wind climate and the height variation of the wind determined from the terrain roughness and orography. The fluctuating component of the wind is represented by the turbulence intensity.

According to STN EN 1991-1-4 [37] terrain corresponds to terrain category I, with the basic wind velocity $v_{b,0} = 24$ m/s, which correspond to basic wind pressure $q_b = 0,36$ kNm⁻².

The basic wind velocity should be calculated according to [37] as:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} = 1.1 \cdot 24 = 24 \text{ m/s} \quad (12)$$

where v_b is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category I, $v_{b,0}$ is the fundamental value of the basic wind velocity, c_{dir} is the directional factor, c_{season} is the season factor.

The roughness factor $c_r(z)$ accounts for the variability of the mean wind velocity at the site of the structure due to the height above ground level and the ground roughness of the terrain upwind of the structure in the wind direction considered and should be calculated according to [37] as:

$$c_r(z) = k_r \cdot \ln(z/z_0) = 0.19 \cdot \ln(30.85/0.05) = 1.007 \quad (13)$$

where z_0 is the roughness length, k_r terrain factor depending on the roughness length z_0 calculated using

$$k_r = 0.19(z_0/z_{0,II})^{0.07} = 0.19(0.05/0.05)^{0.07} = 0.19 \quad (14)$$

where $z_{0,II} = 0.05$ m (terrain category II), z_{min} is the minimum height, z_{max} is to be taken as 200 m, unless otherwise specified in the NA, z_0, z_{min} depend on the terrain category. Recommended values are given in [37] depending on five representative terrain categories. Where orography (e.g. hills, cliffs etc.) increases wind velocities by more than 5% the effects should be considered using the orography factor c_0 .

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity, v_b , and should be calculated according to STN ENV 1991 [37] as:

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b = 1.007 \cdot 1.24 = 24.16 \text{ m/s} \quad (15)$$

where $c_r(z)$ is the roughness factor, $c_0(z)$ is the orography factor, taken as 1.0 unless otherwise specified. The influence of neighboring structures on the wind velocity should be considered according to STN ENV 1991 [37].

External pressure coefficients for duo pitch roofs were taken according to STN ENV 1991 [37] for examined frame, with his location in middle of the hall, with angel of roof $\leq 5^\circ$ for different wind directions. The net pressure on a roof on the opposite surfaces was taken as a positive and directed towards the surface, pressure coefficient for the internal pressure $c_{pi} = +0.2$.

The extreme wind load (EWL) was determined from the maximum wind speed determined from the SHMU measurements [39] in the NPP locality for the return period 10^4 by year and the probability of non-exceedance 95%

$$v_{ewl,b} = 53.9 \text{ m/s and pressure } p_{ewl,b} = 1.816 \text{ kPa} \quad (16)$$

The pressure from the extreme wind load is 4.98x higher than the design basic value.

To extrapolate the maximum quantity of rainfall from meteorological measurement results on the quantity of rainfall (measured in the time period from 12 to 48 hours), for a mean time of recurrence 10^2 or 10^4 years, it is recommended to use the Gumbel probability distribution with the requirement that the probability of the excess quantity of rainfall with mean time repeat 10^2 , respectively 10^4 years, during the design of the power plant lifetime is less than 0.5, respectively 0.005.

The mean values of the extreme wind velocity and wind pressure are following

$$v_{ewl,m} = 47.6 \text{ m/s and pressure } p_{ewl,m} = 1.437 \text{ kPa} \quad (17)$$

5. Load Failure Effect

The failure effect of the extreme load on the yield strength p_y is determined for the selected critical elements of the structure, relying on the results of deterministic analyses for the design values of the load and structural resistance effects and the failure condition for the given stress case. In the case of probabilistic analysis, the mean values of the input variables, the median values of the static forces from the effects of the load E_m and the corresponding resistance value of the structure R_m . If we consider the load combination according to the relations (1) and the static quantity from the effect of the extreme load E_{ewl} to separate from the other effects of the other load E_o , then the failure effect of the load p_y gets in the following form

$$p_y = \eta_y \cdot p_{ewl} \quad (18)$$

where the parameter η_y is defined from the reliability function RF (Eq.4 and 8).

For the estimation of the failure (collapse of a part of the structure or the whole) it is also possible to use the philosophy of determination of the limit state by the parameter $HCLPF$ (High confidence of low probability of

failure) [11, 15], which is used mainly for evaluation of seismic resistance of the structure. Hence, $HCLPF$ is determined by the lowest resistance of an element of a given structure as a whole and expresses the relative resistance of the element to a specified extreme load expressed by the extreme load failure pressure. This methodology gives a good "estimate" of the structural resistance, but does not accurately reflect the margin of resistance of the elements to the combined stress cases because strength conditions are non-linear in nature.

The parameter $HCLPF$ is determined from the relationship

$$HCLPF(CDFM) = k_D \cdot P_y = k_D \cdot \eta_y \cdot P_{ewl}, \quad (19)$$

where k_D is the ductility factor expressing the capacity of a structural member or system.

6. Formulation of Fragility Curves

The probability rating is based on limit states. Limit states are possible types of faults in NPP object functions. The hermetic zone is interpreted as an emerging leak, which can be a small controlled leak or a large catastrophic crack or breakage. The median load-bearing capacity of a given type of disorder depends largely on several factors, including material properties, model assumptions, and failure criteria. These factors are characterized by uncertainty and variability, which must be taken into account in the probability evaluation of the pressure violation. The pressure carrying capacity for each type of disorder is expressed as a random variable with lognormal distribution when the uncertainty/variability is incorporated into the formulation (by definition, the random variable is lognormal divided if its natural logarithm is normally distributed).

In case of using analytical methods and FORM method for failure probability evaluation, pressure resistance can be expressed only from two parameters - model and material uncertainty. The failure resistance is described as:

$$p_u = p_m \cdot e_{var} \cdot r_{var} \quad (20)$$

where p_m is the median of the failure capacity, e_{var} is the logarithmically distributed random variable with the unit median and logarithmic standard deviation β_E (i.e., $E_m = 1.0$ and $\beta_E^2 = \text{Var}[\ln(E)]$), where $\ln(E)$ is the natural logarithm r_{var} is a lognormal distribution of a random variable with a unitary median value and a logarithmic standard deviation of β_R (i.e. $R_m = 1.0$ and $\beta_R^2 = \text{Var}[\ln(R)]$), representing the uncertainty of material properties.

Using the natural logarithm of equation (20) we get:

$$\ln(p_u) = \ln(p_m) + \ln(e_{var}) + \ln(r_{var}) \quad (21)$$

The first expression on the right is a constant (deterministic) number, while the second and third expressions are normally distributed random variables. According to the basic theory of probability, the sum of

normally distributed independent random variables is normally divided, with β_C^2 scattering equal to the sum of the deviations of random variable elements, i.e. in this case:

$$\beta_C^2 = (\beta_E^2 + \beta_R^2) \quad (22)$$

Based on the assumption that random modelling variables and random variables of material properties are independent, β_C can be considered as a "composite" logarithmic standard deviation.

Thus, the failure capacity of each type of disorder is defined in terms of three parameters: the median failure rate P_m , the logarithmic standard deviation β_E (corresponding to the random variable uncertainty of the load effect E) and the logarithmic standard deviation β_R (corresponding to the random variable uncertainty of the structural resistance R). Scattering is due to a lack of knowledge of the differences between the analytical model and the real design as well as material properties. Model indeterminacy is influenced by simplifying assumptions, model details, and their ability to capture glitch conditions on the actual structure (also design versus actual deviation). The uncertainty of the strength of the material lies in the lack of information on its properties as well as their variability. Examples of sources of such uncertainties include: variability in concrete strength, steel slip limit, stress-strain relationship, steel shell strength, and temperature influence on material strength. If experimental data on actual material strength properties are available, they can be used to estimate β_R .

In practice, however, the determination of the β_E and β_R parameters is largely a matter of good engineering estimation. In determining the realistic values of β , experience from previous probabilistic structural analyses, e.g. overpressure studies, seismic risk probability assessments, etc. and experience in determining the load-bearing capacity of various elements by test and analytical methods. From a practical point of view, the probability of a value of less than 90% is less than 5% when using a particular model / formula. In this case

$$\beta_E = -\ln(0.9) / 1.65 = 0.06 \quad (23)$$

The factor 1.65 is due to the fact that the value for a normally distributed random variable that is exceeded with a probability of 95% is the mean reduced by 1.65 times the standard deviation. If the given value is exceeded with 84% probability, $\exp(-0.006) = 0.94$, it is 94% of the best estimate value.

The probability of failure occurring at a pressure less than a specific value of internal pressure, p_u , is expressed in the case of lognormal distribution as

$$p_f = \Pr ob(p_{ewl} \leq p_u) = \Phi[\ln(p_u/p_m)/\beta_C] \quad (24)$$

where p_f is probability failure occurs at a pressure less than or equal to p_u , p_{ewl} is a random pressure capacity, $\Phi(\cdot)$ is a cumulative distribution function for a standard normal random variable, $\ln(\cdot)$ is a natural logarithm, p_m is a median pressure capacity, β_C is a logarithmic standard deviation of p .

The first step of a wind pressure evaluation is the identification of potential failure modes. Once the potential failure modes are identified, failure criteria are to be established from which the median capacities are estimated. For each failure mode, the median capacities are to be evaluated by conducting independent limit state analyses using the specific failure criteria with the applied loading consisting of wind pressure and dead load. Along with the pressure capacities for the leak type failure modes, leak areas are to be estimated in a probabilistic manner. The expected leak areas are failure mode dependent. After calculation of the fragility or conditional probability of failure of structure at different locations we must to consider a combination of wind pressure induced failure probabilities of different break or leak locations within building.

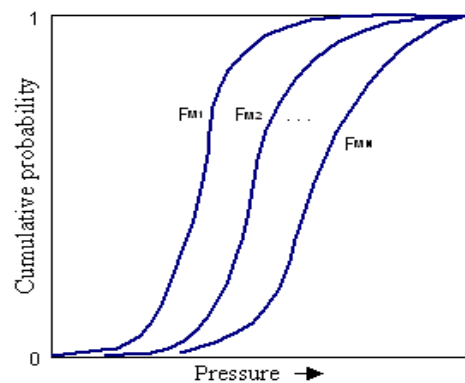


Fig. 5: Family of Fragility Curves Showing Modeling Uncertainty

Turbine hall may fail at different locations under different failure modes (Fig. 5). Consider two failure modes A and B , each with n - fragility curves and respective probabilities p_i ($i=1, \dots, n$) and q_j ($j=1, \dots, n$). Then the union $C = A \cup B$, the fragility $F_{Cij}(x)$ is given by

$$F_{Cij}(x) = F_{Ai}(x) + F_{Bj}(x) - F_{Ai}(x) \cap F_{Bj}(x) \quad (25)$$

where the subscripts i and j indicate one of n - fragility curves for the failure modes and x denote a specific value of the pressure within the containment. The probability p_{ij} associated with fragility curve $F_{Cij}(x)$ is given by p_i , q_j if the median capacities of the failure modes are independent. The result of the intersection term in (22) is $F_{Ai}(x) \cdot F_{Bj}(x)$ when the randomness in the failure mode capacities is independent and $\min[F_{Ai}(x), F_{Bj}(x)]$ when the failure modes are perfectly dependent.

The flow is and the consequence of an accident depends on the total leak area. Multiple leaks at different locations of the containment (e.g. bellows, hatch, and airlock) may contribute to the total leak area. Using the methodology described above, we can obtain the fragility curves for leak at each location.

For a given accident sequence, the induced accident pressure probability distribution, $h(x)$, is known. This is convolved with the fragility curve for each leak location to obtain the probability of leak from that location (P_{Li}). It is understood that there is no break or containment rupture at this pressure.

$$p_{Li} = \int_0^{\infty} h(x)[1 - F_b(x)]F_l(x)dx, \quad (26)$$

Here the $F_b(x)$ is the fragility of break at the location and $F_l(x)$ is the fragility of leak. The leak is for each location is only specified as a random variable with a probability distribution. This distribution of leak area is discretized into a number of ranges with associated probabilities (a_{ij} , P_{ij}). Therefore, the probability of a_{ij} occurring (i.e. location i and are range j) is $P_{Li} * P_{ij}$. This is accomplished using the DPD logic. Alternatively, a Monte Carlo simulation could be done by randomly sampling from each leak are distribution (from different locations) and summing them.

If we use simulation methods based on Monte Carlo or LHS methodology, several factors affecting the failure of the structural element can be taken into account in the calculation of the failure probability. Above all, it is a fact that other effects also condition the structural element capacity. The design of the NPP can be broken at different locations, with different types of extreme loads. If each type of failure leads to a collapse of the building structure or same principal structural system, it is not necessary to check multiple types of failure. If the consequences of the types of failures are the same, then the probability of failure could be calculated as the probability of a "group" of several types of failures.

7. Nonlinear analysis

The 2D model of the critical frame of the turbine hall structures was created from a spatial model by selecting using the substructure method [15]. The stiffness and weight of the elements as well as the load correspond to the values in the spatial model. The model contains 660 elements and 1074 nodes (Fig. 6). The model was tested by linear calculation from its own gravity and shows the same maximum deflection values. The limit state of the steel frame was considered to utilize the geometric and material nonlinearity in program ANSYS [15]. The geometric nonlinearity is based on the theory of the large strain, which is often used for elastic-plastic elements. The elastic-plastic model of steel material was taken in compliance with the Von Mises yield function [39-42]. The Newton-Raphson iteration method to solve nonlinear equations was considered.

The plasticity model is defined as shown on Fig. 7, as multilinear isotropic hardening material model.

The nonlinear analysis based on potential theory considering the isotropic material properties was made for the beam elements BEAM188 in the FEM model. The steel is typical isotropic material. The elastic-plastic behavior of the isotropic materials is described by the Hubert-Mises-Hencky (HMH) yield criterion.

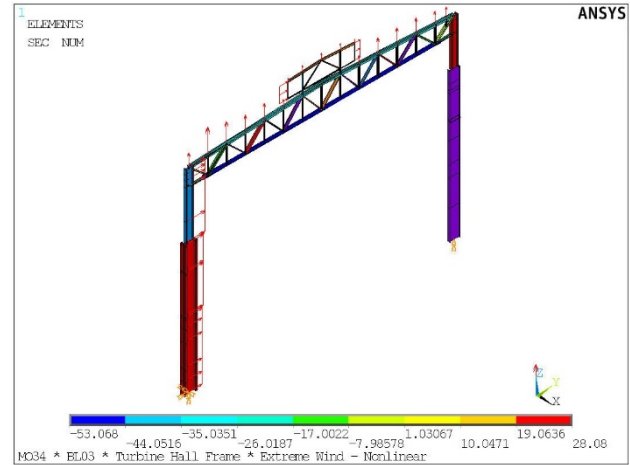


Fig. 6: Model of extreme wind loads.

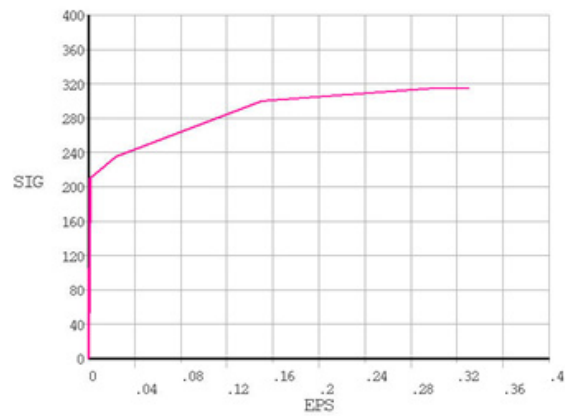


Fig. 7: Elasto-plastic Stress-Strain Curve.

Consequently the stress-strain relations are obtained from the following relations

$$\{d\sigma\} = [D_{el}]\left(\{d\varepsilon\} - \{d\varepsilon^{pl}\}\right) = [D_{el}]\left(\{d\varepsilon\} - d\lambda \left\{\frac{\partial Q}{\partial \sigma}\right\}\right)$$

$$\text{or } \{d\sigma\} = [D_{ep}]\{d\varepsilon\} \quad (27)$$

where $[D_{ep}]$ is elastic-plastic matrix in the form

$$[D_{ep}] = [D_e] - \frac{[D_e]\left\{\frac{\partial Q}{\partial \sigma}\right\}\left\{\frac{\partial F}{\partial \sigma}\right\}^T [D_e]}{A + \left\{\frac{\partial F}{\partial \sigma}\right\}^T [D_e]\left\{\frac{\partial Q}{\partial \sigma}\right\}} \quad (28)$$

The hardening parameter A depends on the yield function and model of hardening (isotropic or kinematic). HMH yield criterion is defined in the form

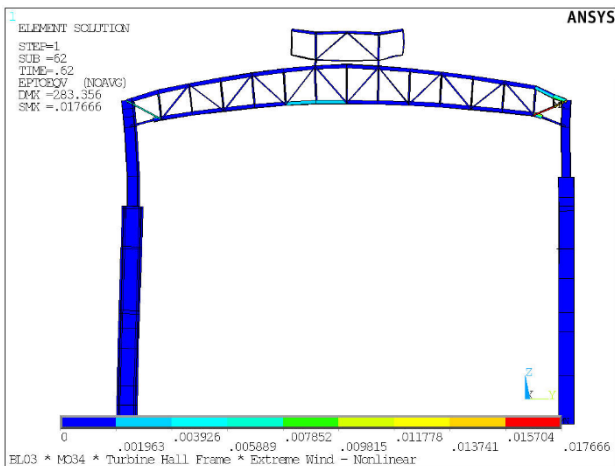
$$\sigma_{eq} = \sigma_T(\kappa), \quad (29)$$

where σ_{eq} is equivalent stress in the point and $\sigma_T(\kappa)$ is the yield stress depends on the hardening.

Tab.1: Calculation of *HCLPF* parameter from nonlinear analysis on 2D frame of the turbine hall

Limit state	Param. η	Loads increment	<i>HCLPF</i> [kPa]	
			50%	95%
f_y	4	0,35	2,542	1,828
f_u	4	0,62	4,576	3,290

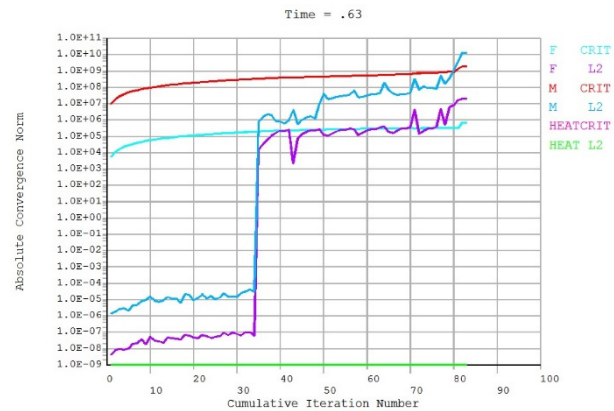
The nonlinear analyze of the critical frames of NPP turbine hall objects was performed by the Newton-Raphson method in 74 iteration steps of the 63 loads increments. The total load was considered to be a multiple of the design load from extreme wind of $\eta = 4$.

**Fig. 8:** Deformation of the frame for the *HCLPF*=4.576 kPa.**Fig. 9:** Equivalent stress of the frame for the *HCLPF*=4.576 kPa.

The failure capacity of frame on extreme wind load is obtained from plastic calculation of frame [15]. The fragility curve of the extreme wind was calculated on the base of the nonlinear deterministic analysis of the steel frame for median values of input data. The probability of the structure failure was calculated for the various levels of the wind loads. On the base of the nonlinear deterministic analysis the factor of failure was calculated on the steel hall frame. The Figs. 8-10 show results for 4x higher value of the extreme wind load. On base of the nonlinear deterministic calculation we have the median

value of the maximum wind load for elastic limit state (f_y) and plastic limit state (f_u) (Tab.1) as follows

$$p_{y,m} = 2.542 \text{ kPa and } p_{u,m} = 4.576 \text{ kPa} \quad (30)$$

**Fig. 10:** Maximal capacity for extreme wind.

The ductility factor can be determined as the ratio between the plastic limit load and elastic limit state

$$k_D = \eta_u / \eta_y = 2.48 / 1.40 = 1.77 \quad (31)$$

8. 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 [44] and Eurocode 1990 [36].

Tab.2: Probabilistic model of input parameters.

Name	Quantity	Charact. Value	Variable Paramet.	Histogram
Material	Young's Modulus	E_k	e_{var}	Normal
Load	Dead	G_k	g_{var}	Normal
	Live	Q_k	q_{var}	Gumbel
	Extreme Wind	W_k	w_{var}	Gumbel
Resistance	Steel Strength f_{sk}	F_k	f_{var}	Lognormal
Model	Action Uncertain	M_F	$m_{e,var}$	Normal
	Resistance Uncert.	M_R	$m_{r,var}$	Normal

Tab.3: Characteristic input data of the probabilistic model.

Name	Quantity	Mean	Stand. Deviation	Min. Value	Max. Value
Material	Young's Modulus	1	0.120	0.645	1.293
Load	Dead	1	0.010	0.755	1.282
	Live	0.60	0.200	0	1
	Extreme Wind	0.30	0.150	0.500	1.032
Resistance	Steel Strength f_{sk}	1	0.100	0.726	1.325
Model	Action Uncertain	1	0.100	0.875	1.135
	Resistance Uncert.	1	0.100	0.875	1.135

The stiffness of the structure is determined with the median value of Young's modulus E_m and variable factor e_{var} . Loads are represented by their characteristic values

G_m , Q_m , $W_{E,m}$ and variable factors g_{var} , q_{var} and w_{var} . The resistance of the steel is delimited by the characteristic values of the strength f_{ak} and the variable factor f_{var} . The uncertainties of the calculation model are considered by variable model factor and variable load factor for Gauss's normal distribution.

The reliability analyses of the structures are differentiated from the point of view of design quantities as the nonlinear deterministic analyses [24]. In stochastic analysis, the mean values and the standard deviation of the variable quantities are calculated analytically or numerically using Monte Carlo simulation, which gives us the more accurate results than deterministic values. Sensitivity analysis calculates the dominant impact to the output quantities and the probability of the failure is defined in comparison with the simulated quantities in probabilistic analysis.

9. Wind fragility curve of frame

The probability of the steel frame failure was determined by the probabilistic analysis based on nonlinear analysis using the analytical and simulation methods. The fragility curve was calculated for various levels of wind loads using the results from the nonlinear analysis of the steel hall frame.

The probability of frame failure was determined by two methods:

A. Analytical analysis based on the FORM method and considering the lognormal distribution of action effect E and resistance R ,

B. LHS simulation methods in software FREeT [31], considering the distribution of Gumbel's wind load, normal self-weight distribution, and lognormal for resistance (see Tab. 2-3).

A) FORM estimation of failure probability:

We consider the median value of the load limit effect $p_{wm} = 4.576$ MPa according to Tab. 1, the logarithmic standard deviation of load are considered by values $\beta_E = 0.1$ and resistance $\beta_R = 0.1$. We have following

$$\beta_C = \sqrt{\beta_E^2 + \beta_R^2} = 0.141 \quad \text{and} \quad (32)$$

$$HCLPF_{EWLN,95} = p_{wm} \exp(-1.65\beta_C) = 3.133 \text{ kPa}$$

In the case of extreme wind loads, it is an extreme load in the range of 10 min. impact on structure. In this case it is possible to consider the plastic reserve of the truss girder, or beam respectively. In the case of the simple console or beam simply supported the ductility factor is equivalent to $k_D = 1.5$. Then we get the value of the limit load capacity calculated from the linear analysis considering the ductility factor

$$HCLPF_{EWLD,95} = k_D p_{wm} \exp(-1.65\beta_C) = 1.836 \text{ kPa} \quad (33)$$

B) LHS simulation using the software FREeT [31]

The calculation based on LHS simulation method (Figs. 11-13) is based on the Normal load distribution of the self-weight, the distribution of the extreme wind load in the form of a Gumbel distribution, and the Lognormal distribution of the structural resistance. The HCLPF parameter determined by the probability analysis of the turbine hall frame under extreme wind load by LHS simulation method for 1000 simulations was obtained as follows

$$HCLPF_{EWLD,95} (LHS) = 1.8191 \text{ kPa} \quad (34)$$

The wind fragility curve of the steel hall frame is presented in Fig. 13. This curve was calculated by LHS method using the program FREET.

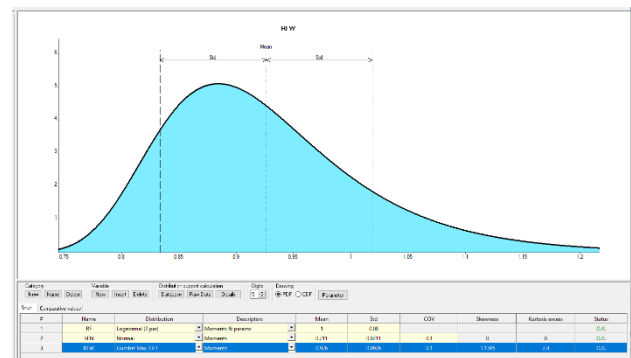


Fig. 11: Histogram of the extreme wind load.

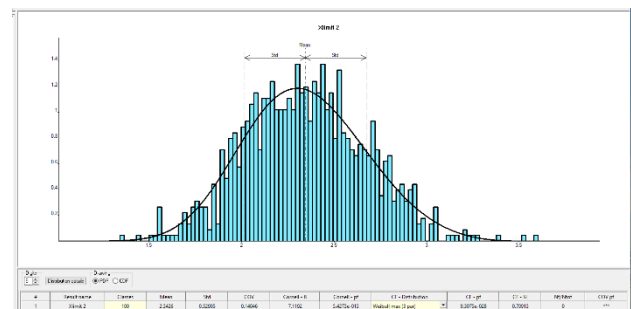


Fig. 12: Histogram of HCLPF parameter for extreme wind load.

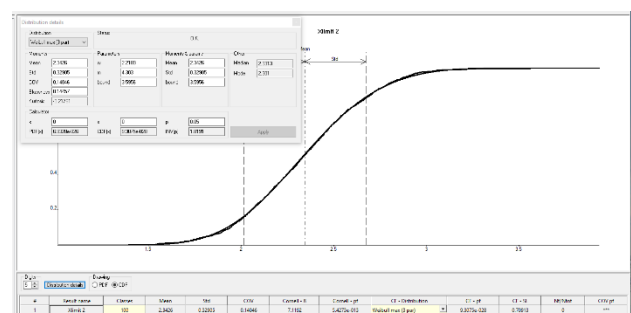


Fig. 13: Wind fragility curve of the steel hall frame.

10. Conclusion

This paper presents the reliability analysis of the steel hall frame resistance due to extreme wind loads [30]. The extreme loads were defined for mean return period equal

to one per 10^4 years in accordance of the IAEA requirements for NPP structures [5 to 10]. The geometric and material nonlinearity were taken into account. The deterministic and probabilistic analysis of the structure failure was investigated. The limit state (frame collapse) was obtained from deterministic analysis for the factor $\eta_u=2.48$. The probability of failure was calculated on program FREET using LHS method [31]. The probability of failure value is lower than 10^{-6} . In the case of the wind load multiplied by factor $\eta_u = 2.48$ the probability of failure is equal to 0.0087. The wind fragility curve of the steel hall frame was determined using LHS method for various level of the wind load.

The lowest failure load values (HCLPF) assuming a 95% probability of not exceeding the ultimate limit criterion on NPP structural elements based on linear analysis are as follows:

- Extreme wind: $HCLPF_{EWL,95} = 1.224$ kPa

Considering the minimum ductility value of $k_D = 1.5$ based on the assumption of the plastic reserve corresponding to the static uncertainty of the frame systems, with the short-term exposure to extreme loads, we have the following values:

- Extreme wind: $HCLPF_{EWD,95} = 1.819$ kPa

Lowest Failure Loads (HCLPF) assuming 95% probability of not exceeding the structural failure criterion for critical frames with duo-pitch Pratt Truss and beam based on nonlinear analysis are as follows:

- Extreme wind: $HCLPF_{EWLN,95} = 3.133$ kPa

The results of this analysis clearly confirm that turbine hall structures have significantly greater resistance to extreme snow and extreme wind effects at the origin of and the development of plastic deformations, considering the conservative linear calculation and assessment of one (weakest) cross-section of the critical element.

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