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PROBABILISTIC NONLINEAR ANALYSIS OF REINFORCED CONCRETE  
BUBBLER TOWER STRUCTURE FAILURE

**Abstract**

This paper describes the reliability analysis of concrete bubbler tower structure of nuclear power plant with the reactor WWER 440 under high internal overpressure. There is showed summary of calculation models and calculation methods for the probability analysis of the structural integrity considering degradation effects and high internal overpressure. The uncertainties of the resistance and the calculation model were taking in the account in the RSM method.

**Keywords**

Probability, Nonlinearity, Failure, Reinforced Concrete, NPP, ANSYS, RSM.

**1 INTRODUCTION**

The International Atomic Energy Agency set up a program [2, 7 and 22] to give guidance to its member states on the many aspects of the safety of nuclear power plant (NPP) reactors. The risk of the NPP performance from the point of the safety must be calculated by consideration of the impact of the all effects during plant operation. The probabilistic safety analysis (PSA) is one from the effective methods to analyze the safety and reliability of the NPP:

- (1) Accident frequency (systems) analysis,
- (2) Accident progression analysis,
- (3) Radioactive material transport (source term) analysis,
- (4) Offsite consequence analysis,
- (5) Risk integration.

The final stage of the PSA is the assembly of the outputs of the first four steps into an expression of risk as follows:

$$Risk_{In} = \sum_{h=1}^{n_{IE}} \sum_{i=1}^{n_{PDS}} \sum_{j=1}^{n_{APB}} \sum_{k=1}^{n_{STG}} f_n(IE_h) P_n(IE_h \Rightarrow PDS_i) P_n(PDS_i \Rightarrow APB_j) P_n(APB_j \Rightarrow STG_k) C_{ik} \quad (1)$$

where  $n$  is the sample number in the LHS scheme;  $n_{IE}$  - the number of initiating events;  $n_{PDS}$  - the number of plant damage states;  $n_{APB}$  - the number of accident progression bins;  $n_{STG}$  - the number of source term groups;  $Risk_{In}$  - the risk of consequence measure  $I$  for sample  $n$  (consequences/year);  $f_n(IE_h)$  - the frequency (per year) of initiating event  $h$  for sample  $n$ ;  $P_n(IE_h \Rightarrow PDS_i)$  - the conditional probability that initiating event  $h$  will lead to plant damage state  $i$  for sample  $n$ ;  $P_n(PDS_i \Rightarrow APB_j)$  - the conditional probability that plant damage state  $i$  will lead to accident progression bin 1 for sample  $n$ ;  $P_n(APB_j \Rightarrow STG_k)$  - the conditional probability that accident progression bin  $j$  will lead to source term group  $k$  for sample  $n$  and  $C_{ik}$  - the expected value of consequence measure  $i$  conditional on the occurrence of source term group  $k$ .

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The risk integration is shown in matrix formulation in Figure 1. The approximate numbers of PDSs, APBs, and STGs, and the number of consequences used in the different NUREG-1150 [20] PSAs are 20, 1000, 50 and 8, respectively.

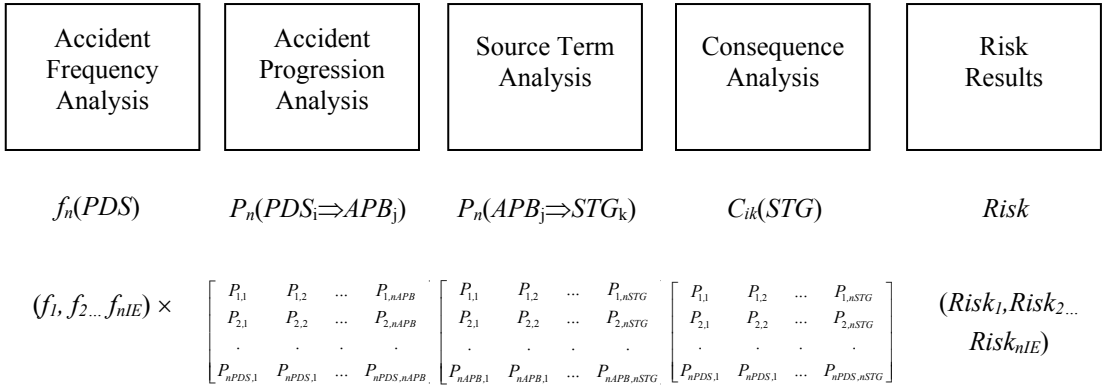


Fig. 1: Scheme of Latin Hypercube Sampling

The general purpose of the probability analysis of the containment integrity [15 and 16] was to define the critical places of the structure elements and to estimate the structural collapse.

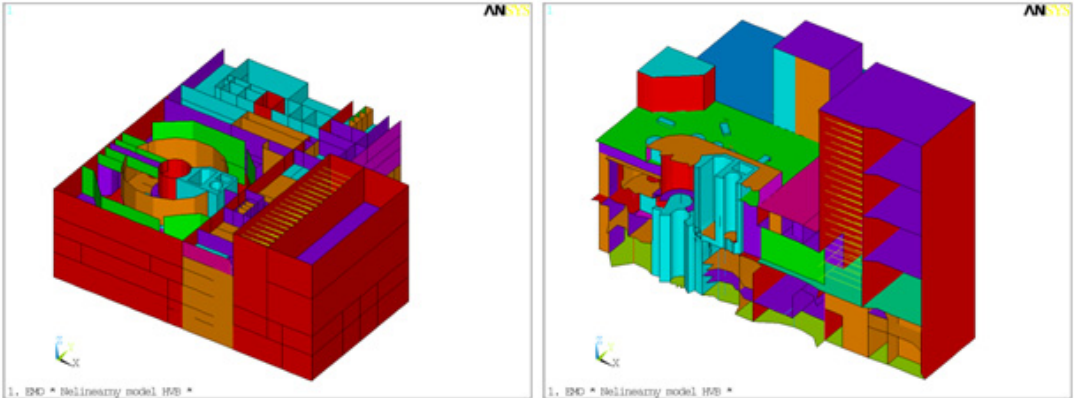


Fig. 2: Calculation model of NPP building

Following the results from Loss of Coolant Accident (LOCA) scenarios the probability check of the structural integrity may be realized for the random value of the loads and material properties by modified LHS method. For a complex analysis of the concrete structure for different kind of loads, ANSYS software and the program CRACK (created by Králik) [15 and 16] were provided to solve this task. The building of the power block was idealized with a discrete model consisting of 28 068 elements with 104 287 degrees of freedom (DOF) (see Fig.2).

The international standard NUREG-1150 [20] PSA defines the principal steps for the calculation of the risk of the NPP performance by LHS probabilistic method.

## 2 PROBABILISTIC SAFETY ASSESSMENT

Probabilistic safety assessment (PSA) level 2 [2, 7 and 16] is a systematic way to study, from the point of view of safety and with the restrictions of a specific methodology, the behaviour of a system (NPP under accident or quasi-accident conditions) when uncertainty is present and widespread. The starting point of level 2 is the result of a PSA level 1. The results of such study is a huge quantity of accident sequences that are grouped, according to different criteria regarding accident characteristics and potential containment responses, into a manageable number of plant damage states (PDS). After an appropriate screening of very low probability sequences, the

probabilistic progression of accidents is studied using event trees, commonly known as accident progression event trees (APET) or containment event trees (CET), under two possibilities: large event trees (virtually all questions regarding severe accident are included as top events) and small event trees (only main questions regarding severe accident phenomena are included as top events). The use of these event trees leads to getting a huge quantity of end states, that have to be grouped, as in the case of PDS's, to get a more manageable set of release categories, later used to estimate all the variety of different possible source terms. The appropriate combination of release categories and corresponding frequencies allows estimating the risk associated to the NPP. Uncertainty is really pervasive in a PSA level 2. The first matter of concern is the starting point. A lot of methods and tools do exist to study the influence of uncertainties on the results of severe accidents computer codes in use for PSA level 2. So we could say that uncertainty arises in three areas of the PSA level 2 - 1) Definition of plant damage states, 2) Simulation of the problem, including event tree construction and models (computer codes) used to simulate the physical-chemical processes involved, and 3) data used to feed models. This is what classically has been considered scenario, model and data uncertainty.

## 2.1 Plant damage state definition and quantification

The plant damage states (PDS) form the starting point for the level 2 analysis [16]. Each PDS consists of a collection of core damage sequences, which are expected to behave similarly following the onset of core damage. The purpose of grouping core damage sequences into PDS is to make the level 2 analysis more manageable and understandable. Accident progression was the first parameter considered in the grouping process. Four main source term groups were selected depending on the sequence type: a large LOCA, transients or small LOCA, interfacing LOCA, and open reactor (or fuel pool) sequences. All other parameters were considered within each of these main groups.

## 2.2 Probabilistic analysis of NPP structures

The containment overpressure study is part of the Level 2 PSA [2 and 7]. Consequently, the containment's pressure capacity must be expressed in probabilistic terms in such a form that it can be used as input in the overall probabilistic risk assessment.

The methodology of probabilistic analysis of integrity of reinforced concrete structures of containment results from requirements [7 and 16] and experiences from their applications [15].

The probability of loss integrity of reinforced concrete structure hence it will be calculated from the probability of no accomplishment condition of reliability  $RF$ ,

$$P_f = P(RF < 0), \quad (2)$$

where the reliability condition is defined by [4] in form

$$RF = R - E > 0, \quad \text{various in the form relative} \quad RF = R/E - 1 > 0 \quad (3)$$

where  $R$  is resistance of structure,  $E$  - effect of action defined by its density. In the case of calculus the resistance of reinforced concrete structure leads off the condition of section integrity.

The pressure value could be considered to be the containment ultimate capacity. This pressure capacity value can be determined through structural analysis methods. The conventional analysis is typically based on design configuration and specified design material property values, and as such is **deterministic** and the computed capacity is a **point estimate** of the capacity.

Approximation is always made in the analysis and the actual as-built building geometry and material properties deviate from the idealized design used as basis of the analysis. The uncertainty involved to the calculation has following two important implications:

1. The capacity description must include a quantified description of the uncertainty inherent in the point estimate. The fragility curves must be defined for Level 2 PSA overpressure studies.
2. The capacity estimates must be determined not only for the weakest link (with lowest point estimate), but also for other weak links along the pressure boundary. Once point estimates and the associated distributions, as well as the level of correlation between the different failure

modes, the aggregate overall description of the containment pressure capacity can be computed using probabilistic method.

Information from design calculation and engineering judgment may identify parts of the containment or doors, hatch covers, etc., as candidates limiting the overall containment pressure capacity. The components/mechanisms with low enough capacities must be analyzed to the level of detail considered reasonable for the purpose, eliminating conservatism as possible.

### 3 SAFETY ANALYSIS OF THE NPP STRUCTURES DUE TO LOCA ACCIDENT

The accident scenario was defined in accordance with code MELCOR 1.8.5 [12]. The guillotine cutting of the Ø13mm, Ø32mm, Ø71mm and the large break LOCA of the 2×Ø500mm (Fig.3) cold leg in the containment were considered.

The temperature in the containment increased during the LOCA accident. The peaks of the temperature are equal to 160°C in the Box SG (Steam generator) by the results of thermodynamic analysis. The effect of these temperature peaks is minimal during the accident and the acting of the overpressure loads. In the case of the harmonic amplitude of temperature the phase angle for concrete walls is superior to 24 hours. The strength of the concrete after LOCA accident increases about to 10% in consequence of the temperature loads during the accident. The peak of the pressure in the Box SG is equal to 200kPa (absolute value).

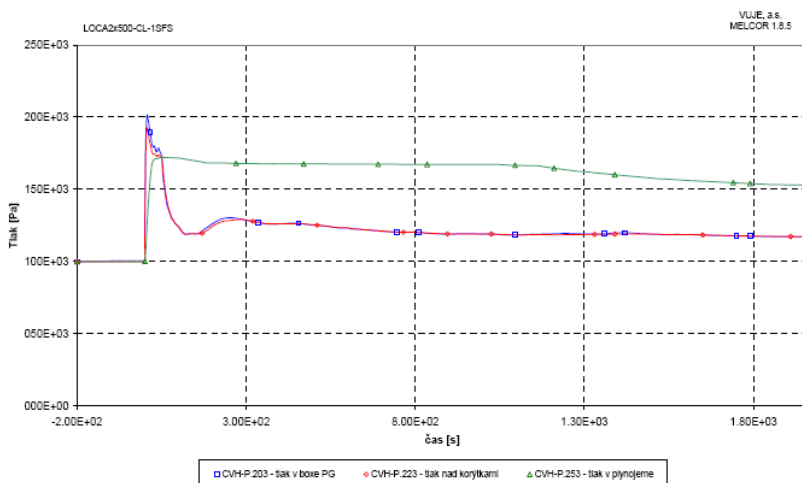


Fig. 3: Overpressure in the Box SG for guillotine cutting of pipe 2×Ø500mm [12]

#### 3.1 Failure pressure of containment

The failure pressure  $p_u$  can be determined from the assumption, that failure occurs when in the structure the mean resistance counted on the mean material strength  $R$  is reached assuming linear relation between the internal overpressure  $p$  and action effects  $E$  corrected by the action effect reducing coefficient

$$p_u = p_{LOCA} k_r \left[ (R - E_o) / E_p \right] \quad (4)$$

where  $p_u$  is failure pressure,  $p_{LOCA}$  is pressure in the case of LOCA effect ( $p_{LOCA} = 150\text{kPa}$ ),  $k_r$  is reduction factor based on assumption of the stress redistribution due to nonlinear behavior of material,  $R$  is structure resistance (capacity),  $E_o$  is effect of initial action (dead loads, temperature, performance loads),  $E_p$  is effect of pressure.

### 3.2 Failure pressure of containment

For the probability analysis of the steel and reinforced structures of NPP containment the statistical characteristic of material properties must be defined. In the case than the site-specific material strength test data are available median strength and variability can be obtained from the sample statistics. However, in the absence of site specific test data in the current study, the median material strengths and variability were estimated based on the nominal specification values adjusted based on generic data in the literature and experience from other containment investigations. The median values and variability were characterized assuming that all of the material strengths could be characterized by a lognormal distribution.

### 3.3 Modelling uncertainty

Uncertainties exist in the estimated pressure capacities due to differences between the analytical idealization of the structure and the real conditions. There are various possible sources of modelling uncertainties. The quality of calculation FEM model – meshing, approximation, boundary conditions – it has significant influence to value of internal force distributions. The uncertainty of internal force distribution, failure criteria, and used empirical formulae must be investigated. However, in many instances, the evaluation of these uncertainties would require very detailed analysis and/or extensive data which may not be available. As a result, it is necessary to use subjective evaluation and engineering judgment to estimate these uncertainties.

It is well known [14 and 16] that due to non-linear and especially plastic behaviour of reinforced concrete structure the different codes allow to take into account the redistribution of internal forces, primarily the bending moments in a different extent depending on the neutral axis depth, the quality of the concrete, the plastic behaviour of the reinforcement. The amount of bending moments redistributed in the codes is between 15-30%. The results are such a situation, where the capacity of all the cross sections of maximal moments is fully exhausted.

In the case of high internal overpressure the containment concrete walls and plates are loaded by tension forces and bending moments. The redistribution of internal normal forces in a box-like reinforced concrete structure is possible, even in case of tension if the capacities of the walls/slabs in one direction are not uniformly exhausted. Of course, the redistribution of the bending moments is possible too. The very high stresses of the range of the mean strengths cause high plastic deformations which also contribute to the redistribution.

This effect may be considered by conservative approach using reduction factor  $k_{red}$ . Summing up the foregoing arguments it was assumed that a  $k_{red} = 1.2$  which is consistent with a redistribution between 15-30 % [16].

## 4 EXPERIMENTAL TEST OF CONTAINMENT AIR TIGHTNESS

The bubbler tower (BT) is the most important structure in the case of the accident of the pipe coolant system in the Reactor hall (Fig.2). The extreme pressure and the steam radioactivity are eliminated in the space of BT. In this paper the nonlinear analysis of the concrete BT structure resistance for mean values of loads, material properties and higher overpressure than BDBA (Beyond Design Basic Accident) is presented. On the base of the IAEA requirements [14] the experimental test of the air tightness of hermetic zone must be realized each 10 years of NPP performance.

The stiffness of structure is tested during this experiment too. The experimental results were compared with the results of numerical analysis of the structures on the FEM calculation model. For a complex analysis of the concrete structure for different kind of loads, ANSYS software were provided to solve this task.

The building of the nuclear power block (NPP) was idealized with a discrete model consisting of 28.068 elements with 104.287 DOF (Fig.4). The air tightness of the hermetic zone and stiffness resistance of the structures was tested by compression of the interior space of NPP. The pressure increase with the speed of 25kPa by 2hours and each compression step (a'25kPa) were stabilized during 2hour.

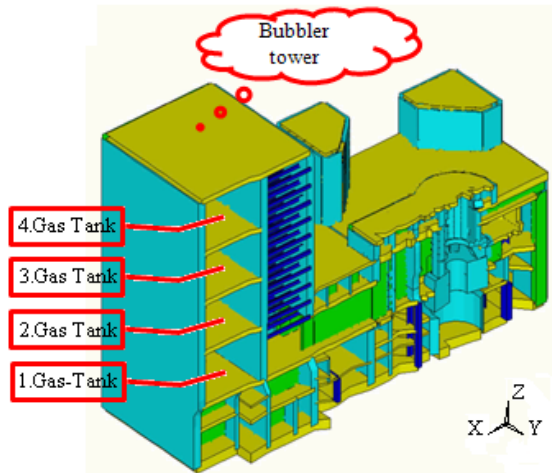


Fig. 4: Calculation FEM model of the NPP building

The pressure increase from the 0kPa to 100kPa and since the pressure decrease to 0kPa with the same tempo. The results of the measurements were recorded at pressure 0, 25, 50, 75 and 100kPa. The inspections of the critical places were realized by the experts (STU Bratislava, VUEZ Levice, VUJE Trnava, SE Bratislava) after each changing step.

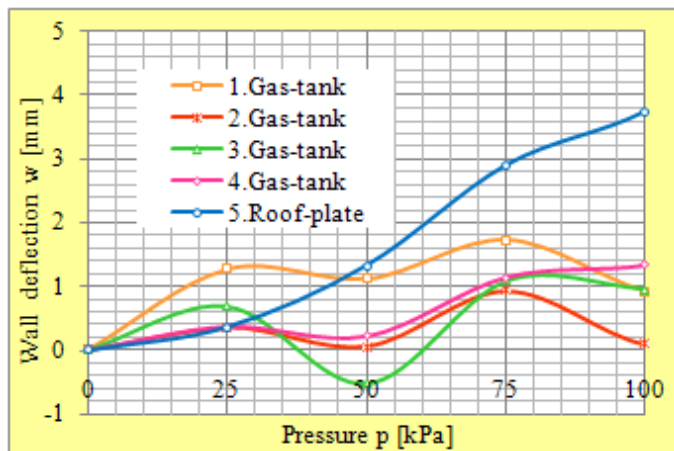


Fig. 5: Bubbler tower wall and roof measured deflection

The optical and mechanical methods were used to check the deformation of structures in the critical places during the pressure change inside the hermetic zone. The critical places of the structures were determined by the numerical analysis [14]. The mechanical indicators were installed in the wall centre of the gas-tank and the roof-plate of the bubbler tower.

## 5 NUMERICAL ANALYSIS

The safety and reliability of the NPP structures of the hermetic zone must be tested on the resistance to the LOCA accident. The DBA a BDBA loads were defined from the scenarios of the guillotine cutting of the  $\varnothing 13$  mm,  $\varnothing 32$  mm,  $\varnothing 71$  mm and  $2 \times \varnothing 500$  mm cold leg in the Box SG. The peaks pressure and temperature were considered on the base of the scenarios in program MELCOR by VUJE Trnava. The BDBA load case was defined for the pressure 150 kPa following

$$E = D + L + P_a + 0.7 T_o + R_a \quad (5)$$

where  $D$  - dead loads,  $L$  - live loads,  $T_o$  - performance temperature,  $R_a$  - reaction of the equipments,  $P_a$  - local effects of the LOCA.

The behavior of the intensity of bending moments  $m_x$  under pressure 150kPa is presented in Fig.6. The most exposed walls on the tension are the walls in the modulus “10” and “17” and wall bottom in the modulus “E”. The most exposed walls on the bending are the walls in the modulus “10” and “17” in the corner with wall in module “D” and wall bottom in the modulus “E” (Fig. 6).

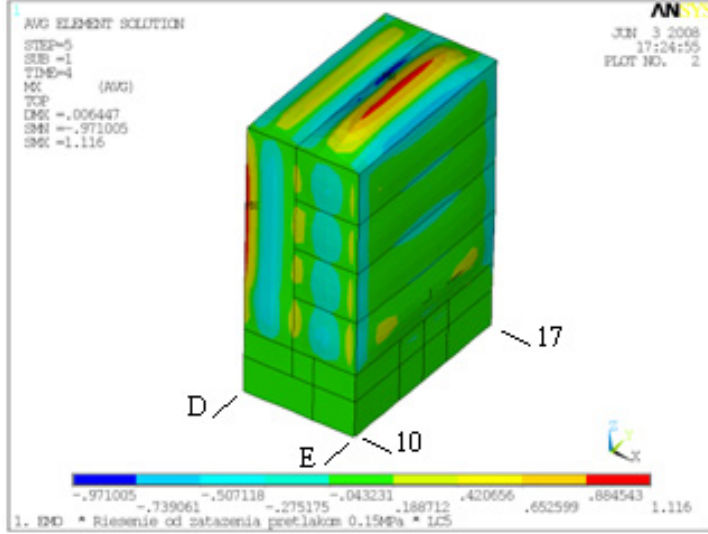


Fig. 6: Bending moments  $m_x$  under pressure 150kPa

## 6 NONLINEAR SOLUTION

The presented constitutive model is a further extension of the smeared crack model [1, 3, 21 and 23], model of the smeared reinforcements [14 and 21], which was developed in [15]. Following the experimental results [3, 11 and 18] a new concrete cracking layered finite shell element was developed and incorporated into the ANSYS system [16] using program CRACK. The layered approximation and the smeared crack model of the shell element are proposed. The matrix of the material stiffness is obtained from the proposition of smeared reinforcement and the rotated cracks in the direction of principal strain in each shell layer. The stiffness matrix of reinforced concrete for the  $l^{th}$ -layer can be written in the following form

$$[D^l] = [T_{c,\sigma}^l]^{-T} [D_{cr}^l] [T_{c,\sigma}^l] + \sum_{j=1}^n [T_s^l]^{-T} [D_s^l] [T_s^l] \quad (6)$$

where  $[T_{c,\sigma}]$ ,  $[T_{c,\varepsilon}]$ ,  $[T_s]$  are the transformation matrices for concrete and reinforcement separately.

The limit of damage at a point was controlled by the values of the so-called crushing or total damage function  $F_u$ . The modified Kupfer's condition [18] for  $l$ - layer of section is following

$$F_u^l = F_u^l(I_{\varepsilon 1}; J_{\varepsilon 2}; \varepsilon_u) = 0 \quad \text{and} \quad F_u^l = \sqrt{\beta(3J_{\varepsilon 2}) + \alpha I_{\varepsilon 1}} - \varepsilon_u = 0 \quad (7)$$

where  $I_{\varepsilon 1}$ ,  $J_{\varepsilon 2}$  are strain invariants; and  $\varepsilon_u$  is an ultimate total strain extrapolated from uniaxial test results;  $\alpha$ ,  $\beta$  are material parameters determined from Kupfer's experiment results ( $\beta = 1.355$ ,  $\alpha = 0.355\varepsilon_u$ ). In the rotated crack model, the direction of the principal stress coincides with the direction of the principal strain. If the principal strain axes rotate during the loading the direction of the cracks rotates, too.

The failure function [16] of the whole section will be obtained by the integration of the failure function through to whole section in the form

$$F_u = \frac{1}{h} \int_0^h F_u^l(I_{\varepsilon 1}; I_{\varepsilon 2}; \varepsilon_u) dz = \frac{1}{h} \sum_{l=1}^{N_{lay}} F_u^l(I_{\varepsilon 1}; I_{\varepsilon 2}; \varepsilon_u) h_l \quad (8)$$

where  $h_l$  is the thickness of the shell layer and  $h$  is the total shell thickness. This failure condition is determined by the maximum strain  $\varepsilon_s$  of the reinforcement steel in the tension area ( $\max(\varepsilon_s) \leq \varepsilon_{sm} = 0.01$ ) and by maximum concrete crack width  $w_c$  ( $\max(w_c) \leq w_{cm} = 0.3\text{mm}$ ).

In order to ensure the co-axiality of the principal strain axes with the material axes the tangent shear modulus  $G_t$  is calculated as

$$G_t = \frac{\sigma_{c1} - \sigma_{c2}}{2(\varepsilon_1 - \varepsilon_2)} \quad (9)$$

The nonlinear solution was realized using the layered shell element SHELL91 from the ANSYS library and program CRACK with concrete nonlinear model [15 and 16] and the experimental results [11]. The comparison of the influences of the plastic deformation and boundary effects is presented in the Fig. 7. The wall of the 4. gas-tank has dimension 39.0/13.61/1.5 m. The simple support and clamped was investigated. Also, the elastic and plastic behavior of concrete material was considered too.

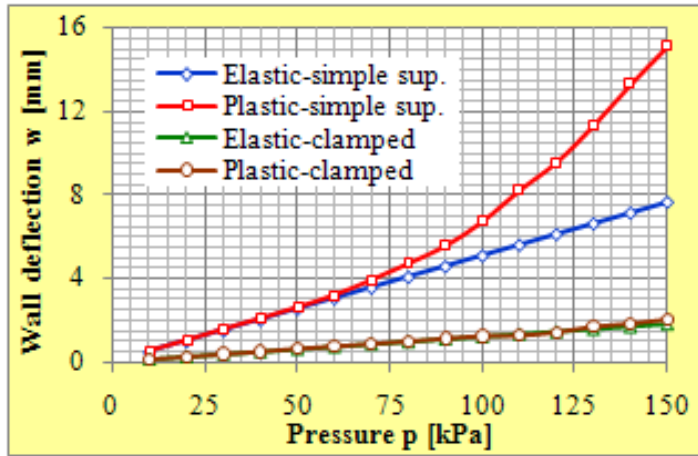


Fig. 7: Comparison the wall deflection of 4.gas-tank for elastic and plastic solution

## 7 PROBABILISTIC ANALYSIS OF THE STRUCTURE FAILURE

Recent advances and the general accessibility of information technologies and computing techniques give rise to assumptions concerning the wider use of the probabilistic assessment of the reliability of structures through the use of simulation methods [5, 6, 8, 9, 13, 16, 17, 19 and 24].

Reliability can be defined as the probabilistic measure of assurance of performance with respect to some prescribed conditions [4, 5, 6, 10 and 19]. A condition can refer to an ultimate limit state (such as collapse) or serviceability limit state (such as excessive deflection and/or vibration).

The probability of failure can be defined by the simple relation

$$P_f = P[R < E] = P[(R - E) < 0] = P[RF < 0] \quad (10)$$

where  $RF$  is a reliability function,  $E$  is a loading effects and  $R$  is a resistance of structure.

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  $E$ .

$$RF = g(X_1, X_2, \dots, X_n) \quad (11)$$

The failure function  $g(\{X\})$  represents the condition (reserve) of the reliability, which can either be 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).



For a system limit state defined by  $g(X_1, \dots, X_m) = 0$ , where  $X_i$  are the basic variables, the failure probability is computed as the integral over the failure domain ( $g(\mathbf{X}) < 0$ ) of the joint probability density function of  $\mathbf{X}$ . In general, the failure of any system can be expressed as a union and/or intersection of events. The failure of an ideal series (or weakest link) system [15] may be expressed following

$$F_{sys} = F_1 \cup F_2 \cup \dots \cup F_m \quad (12)$$

in which  $\cup$  denotes the Boolean OR operator.

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  $Z = g(\mathbf{X})$ . The failure probability is defined as the best estimation on the base of numerical simulations in the form

$$p_f = \frac{1}{N} \sum_{i=1}^N I[g(X_i) \leq 0] \quad (13)$$

where  $N$  is 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 to 0.

Variation of the failure function can be defined by Melchers [17] in the form

$$s_{p_f}^2 = \frac{1}{(N-1)} \left\{ \frac{1}{N} \left[ \sum_{i=1}^N I^2[g(X_i) \leq 0] \right] - \left[ \frac{1}{N} \sum_{i=1}^N I[g(X_i) \leq 0] \right]^2 \right\} \quad (14)$$

The various forms of analyses (statistical analysis, sensitivity analysis, probabilistic analysis) can be performed. Most of these methods are based on the integration of Monte Carlo (MC) simulations. Three categories of methods have been presently realized - **Direct methods** (Importance Sampling - IS, Adaptive Sampling - AS, Direct Sampling - DS), **Modified methods** (Conditional, Latin Hypercube Sampling - LHS) and **Approximation methods** (Response Surface Method - RSM). The advantages and drawbacks of these methods are described in detail in the work [17].

**Approximation methods - Response Surface Methods** are based on the assumption that it is possible to define the dependency between the variable input and the output data through the approximation functions in the following form:

$$Y = g(\{X\}) = c_o + \sum_{i=1}^n c_i X_i + \sum_{i=1}^n c_{ii} X_i^2 + \sum_{i=1}^{n-1} \sum_{j>i}^n c_{ij} X_i X_j + \varepsilon \quad (15)$$

where  $c_o$ ,  $c_i$ ,  $c_{ii}$ , and  $c_{ij}$  are regression coefficients which depend on  $\{\hat{X}\}$  and the derivatives of  $g(\{X\})$  in  $\{\hat{X}\}$ . Based on Equation (14), free types of polynomial are defined depending on the terms considered – full quadratic, reduced quadratic or linear.

On the base of experimental design, the unknown coefficients are determined due to the random variables selected within the experimental region. The true performance function  $g(\{X\})$  or  $\{Y\}$  in Equation (14) can be represented in the matrix form as

$$\{Y\} = [X]\{c\} + \{\varepsilon\} \quad (16)$$

where  $\{Y\}$  is the vector of actual responses, and  $[X]$  is the coefficient matrix.

The least squares estimates  $\{\hat{c}\}$ , defined as  $c_o$ ,  $c_i$ ,  $c_{ii}$  and  $c_{ij}$  in Equation (15), are obtained by solution of the least square (regression) analysis, i.e.,

$$\{\hat{c}\} = ([X]^T [X])^{-1} [X]^T \{Y\} \quad (17)$$

The design includes several statistical properties such as orthogonality that makes the calculation of  $[X]^T [X]$  term simple and rotability that insures the uniform precision of the predicted value.

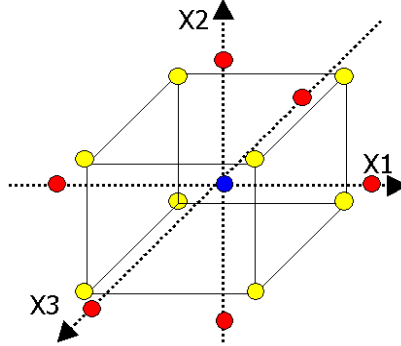


Fig. 8: Distribution schemes – CCD method

The central composite design (CCD) is based on the full quadratic polynomial. Hence it is composed of  $2^k$  factorial design,  $n_o$  centre points and  $2k$  axial portion of design.

The total number of design points is  $N= 2^k+2k+n_o$  which is much more than the number of the coefficients  $p=(k+1)(k+2)/2$ . The graphical representation for  $k=3$  and the matrix form of the coded values are represented in Figure 8.

## 8 PSA ANALYSIS OF STRUCTURE FAILURE

The probability of BT-structure failure is calculated from the probability of the reliability function  $RF$  [16] in the form,

$$P_f = P(RF < 0) \quad (18)$$

where the reliability condition is defined depending on concrete failure condition (4) as follows

$$RF = -F_u(I_{\varepsilon 1}; J_{\varepsilon 2}; \varepsilon_u) = 1 - F_u(I_{\varepsilon 1}; J_{\varepsilon 2}; \varepsilon_u) / \varepsilon_u, \quad (19)$$

where failure function  $F_u(\cdot)$  was considered in the form (7).

The previous design analysis, calculations and additions include various uncertainties, which determine the results of probability bearing analysis of containment structural integrity is presented in Table 1. On the base of mentioned inaccuracy of input data for probabilistic analysis of loss integrity of reinforced concrete containment structures were determined their mean values and standard deviations, various the variable parameters for normal and lognormal distribution.

On the base of the RSM simulation method the vector of the deformation parameters  $\{r_s\}$  is defined for  $s$ -simulation in the form

$$\{r_s\} = [K(E_s, \sigma_s, F_{\sigma_s})]^{-1} \{F(G_s, P_s, T_s)\} \quad (20)$$

and the strain vector

$$\{\varepsilon_s\} = [B_s] \{r(G_s, P_s, T_s, E_s, \sigma_s, F_{\sigma_s})\} \quad (21)$$

where  $F_{\sigma}$  is the Kupfer's yield function of the concrete defined in the stress components.

Table 1: Variable parameters of the input data [16]

	Loads			Model	
	Dead load	Pressure	Temperature	Action uncertainty	Resistance uncertainty
Characteristic values	$G_k$	$P_k$	$T_k$	$E_k$	$R_k$
Variables	$g_{var}$	$p_{var}$	$t_{var}$	$e_{var}$	$r_{var}$
Histogram type	N	N	N	N/LN	N/LN
Mean value $\mu$	1	1	1	1	1
Deviation $\sigma$ [%]	10	10	10	10	15
Minimum value	0.621	0.621	0.621	0.621/0.611	0.271/0.498
Maximum value	1.376	1.376	1.376	1.376/1.614	1.714/1.973

## 9 CONCLUSION

This paper proposed the methodology of the PSA 2 level analysis of the NPP hermetic structures penetration under accident events. The general purpose of the probabilistic analysis of the containment integrity was to define the critical places of the structure elements and to estimate the structural collapse. The uncertainties of the loads level (long-time temperature and dead loads), the material properties (concrete cracking and crushing, reinforcement, and liner) and other influences following the inaccuracy of the calculated model and numerical methods were considered. Resulting from variability of input quantity 25 simulation steps on the base of RSM method under system ANSYS-CRACK was realized [16]. The probability of loss BT-structure integrity was calculated from  $10^6$  Monte Carlo simulations for 25 steps of approximation method RSM on the full structural FEM model. The probability analysis was realized for structural FEM model considering the concrete cracking. The mean value of the failure pressure is equal to 609.7kPa and its 5% kvantil is equal to 369.3kPa.

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