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NPP SAFETY IN SLOVAKIA ACCORDING TO STRESS TESTS AFTER ACCIDENT IN FUKUSHIMI

BEZPEČNOSTI JE NA SLOVENSKU Z POHĽADU ZÁŤAŽOVÝCH TESTOV PO NEHODE VO FUKUSHIMI

Abstract

This paper presents the new requirements to test of the safety and reliability of the NPP structures due to the last nuclear accidents in the world. The accidents of the NPP in Chernobyl and Fukushima give us the new inspiration to verify the safety level of the NPP structures. The probabilistic assessment of NPP structures for PSA level 2 of VVER 440 in the case of LOCA accident is presented. The results of the probabilistic nonlinear analysis of the NPP structures are presented.

Keywords

Stress tests, nuclear accident, safety, probability, nonlinearity, RSM, VVER, NPP.

Abstrakt

Článok sa zaoberá novými požiadavkami na testovanie bezpečnosti a spoľahlivosti konštrukcií JE v dôsledku posledných jadrových nehôd vo svete. Havárie JE v Černobile a vo Fukushimi dávajú nové inšpirácie pre verifikovanie úrovne bezpečnosti konštrukcií JE. Rozoberá sa pravdepodobnostná metodológia konštrukcií JE pre PSA úrovne 2 reaktora VVER 440 v prípade havárie LOCA. Uvádzajú sa výsledky pravdepodobnostnej nelineárnej analýzy konštrukcií JE.

Kľúčové slová

Záťažové testy, jadrová nehoda, bezpečnosť, pravdepodobnosť, nelinearita, RSM, VVER, JE.

1 INTRODUCTION

The nuclear technology gets us the perspective and effective natural resources of the energy but from other side a same risk for the environment [1, 3, 4, 5, 6, 7, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 24, 25 and 27]. The first accident in nuclear research facilities was date on 21 may 1946. A nuclear criticality accident occurred at the Los Alamos Scientific Laboratory in New Mexico. Eight people were exposed to radiation, and one, Louis Slotin, died nine days later of acute radiation sickness. The first significant accident of nuclear power plant (NPP) was date on 28 March 1979 in Middletown, Pennsylvania. A series of human and mechanical failures nearly triggered a nuclear disaster. Contaminated coolant water escaped into a nearby building, releasing radioactive gasses, leading as many as 200,000 people to flee the region. The severe problems arrive after an accident in Chernobyl Nuclear Power Plant in Ukraine on 26 April 1986. An explosion and fire released large quantities of radioactive contamination into the atmosphere, which spread over much of Western USSR and Europe. The battle to contain the contamination and avert a greater catastrophe ultimately involved over 500,000 workers and cost an estimated 18 billion rubles. A 2006 report

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predicted 30,000 to 60,000 cancer deaths as a result of Chernobyl fallout. The last NPP accident was date on 11 March 2011 in Fukushima [7 and 23]. Following a major earthquake (a magnitude 7.1), a 15-metre tsunami disabled the power supply and cooling of three Fukushima Daiichi reactors, hence causing a nuclear accident. The population within a 20km radius had been evacuated three days earlier. Tepco had checked the radiation exposure of 7500 people who had worked on the site since 11 March.

Fig.1: The Fukushima Dai-ichi NPP accident after the great Japan earthquake and tsunami in 2011

In view of the analysis results of the Fukushima accident the owner and operator of nuclear power plants (NPPs) in Slovakia – joint stock company "Slovenské elektrárne a.s." (SE a.s.) has committed himself to perform so called stress tests on all units in operation or under construction [23]. Main plant equipment (except main circulation pumps) for all units was fabricated either in Czech Republic or in Slovakia.

During the process of plants'design, construction and operation significant safety improvements were implemented compared to initial design, including enhanced resistance to external hazards [5, 17 and 24]. Recently, in addition to previous safety improve-ments, a number of provisions have been included in the design of VVER 440/V213 units in Slovakia for mitigation of severe accidents as follows: Elimination of high-pressure core-melt scenarios, by fast reactor pressure vessel (RPV) depressurization even in station black-out (SBO) conditions; Adoption of measures to flood the reactor cavity in order to ensure the outside cooling of the RPV and hence its integrity in case of core melt scenarios.

Specifically the information about Fukushima Unit 1 with reactor thermal power (1380 MWt) similar to VVER 440 was taken as a basis for the comparison. Some of the observations are summarized below [23]:

Fukushima as well as Slovak sites have increased level of seismicity. However, seismicity level of Fukushima site is significantly higher, corresponding to 10^{o} MSK 64 (with maximum horizontal acceleration ~0,5 g), while for Bohunice it is 9^{o} MSK 64 (with 0.344 g) and for Mochovce it is 8^{o} MSK 64 (with 0,143 g). Nevertheless the seismicity is the relevant issue for Slovak sites which need to be addressed more in detail. Combination of an earthquake and flooding due to tsunami (main damaging factor in Fukushima) is irrelevant for sites in Slovakia and can be practically eliminated from further considerations. The only meaningful source of external flooding is extreme precipitation. In case of station black-out the heat removal from the primary circuit can be ensured in the Fukushima Unit 1 design by natural circulation of steam and water through 2 isolation condensers (normally separated from the reactor by isolation valves) containing altogether 212 t of water available for evaporation. In case of VVER 440/V213 residual heat is removed by natural circulation through permanently connected 6 steam generators containing about 330 t of water. Such features are fully covered by the design of EMO34 under construction, and their implementation is ongoing since in all operation units, with completion in V2 and EMO12 in 2013 and 2015, respectively.

4 SAFETY ASSESSMENT OF THE SEISMIC RESISTANCE OF NPP

On the base of the experience from the re-evaluation programs in the membership countries IAEA in Vienna the seismic safety standard No.28 was established at 2003 [4].

Seismic safety evaluation programs should contain three important parts

- ➲ The assessment of the seismic hazard as an external event, specific to the seismic-tectonic and soil conditions of the site, and of the associated input motion;
- ➲ The safety analysis of the NPP resulting in an identification of the SSSCs (Selected Structures, Systems and Components) 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.

4.2 Seismic Re-Evaluation Program in SR

A re-assessment of the seismic hazard specific to the seismic-tectonic conditions at the site was considered by the SAV (Slovak Academy of Sciences) based on IAEA NUSS SO-SG-S1 and S8 [2]; US NRC-RG 1.60 and NUREG/CR-0098 [17]. IAEA is providing technical assistance to the Slovak regulatory authorities for reviewing the work results. Therefore the RLE (Review Level Earthquake) should correspond to the SL-2 level (Second Seismic Level), directly related to ultimate safety requirements. This is a level of extreme ground motion that shall have a very low probability of being exceeded during the plant lifetime and represents the maximum level of ground motion to be used for design and re-evaluation purposes. For the probability of occurrence a typical value of 10^{-4}/yr is usually used and for the ground response spectra an elastic one is selected.

As formulated by the Slovak authorities, the main objective of the seismic re-evaluation programs of NPP is to enhance the seismic safety of the plant to the level generally accepted by the international community and in compliance with the valid standards and recognized practice. These programs should have three important components:

i. the re-assessment of the seismic hazard as an external event, specific to the site seismic-tectonic conditions;
ii. the evaluation of the plant specific seismic capacity to withstand the loads generated by such event;
iii. upgrading if necessary.

Regarding the first component (i), the geological stability and the ground motion parameters should be assessed according to specific site conditions and in compliance with criteria and methods valid for new facilities. In relation to the second component of the programs (ii) and considering that the plant has been originally designed for an earthquake level lower than the one would preliminarily be established for the site in compliance with IAFA NUSS 50-SG-S1 [17]. On the base of the results of the seismic analysis of the structure capacity the upgrade concept (iii) will be designed.

4.3 Safety Aspects

The decision should be made early on whether either the SPSA (Seismic Probabilistic Safety Assessment), SMA (Seismic Margin Assessment), or EPRI (Electric Power Research Institute) seismic safety evaluation methods are to be used [1, 4, 17 and 25]. These methods have an advantage in that the entire plant may be evaluated as an integrated unit, including system and spatial interactions, common cause failure, human actions, non-seismic failures and operating procedures. The seismic resistance of the existing building structures as well as the technological equipment can be executed by the SMA method, especially its variant known as CDFM (Conservative Deterministic

Failure Margin) depending on HCLPF (High Confidence Low Probability of Failure) determination of the seismic margin values. The CDFM method is based on an assumption that all the building structures and all the technological equipment components were designed properly for any non-seismic loads and conditions.

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 [25]. In simple terms it corresponds 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).

Estimating the HCLPF seismic capacity of a system, structure and component requires an estimation of the response, conditional on the occurrence of the RLE. Two candidate procedures to determine the HCLPF seismic capacities for NPP's structures and equipment components have been developed:

(1) the Fragility Analysis (FA), and

(2) the Conservative Deterministic Failure Margin (CDFM) method.

The HCLPF approach or an equivalent method may be used to verify the seismic capacity of Mochovce NPP. The general criteria for CDFM approach is contained in [25]. 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. Generally the value of HCLPF parameter must always be $HCLPF > PGA$

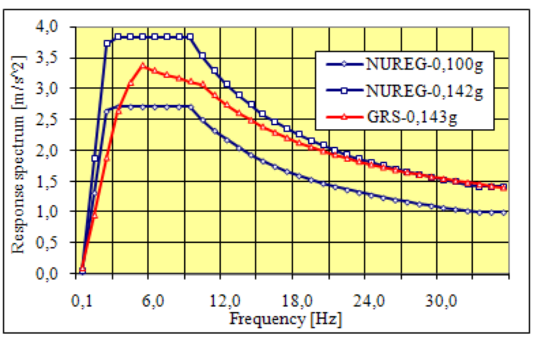
$$HCLPF\left(CDFM\right) = k_D.\left(FS\right)_{el}.PGA_{RLE=SL-2} \quad \text{and} \quad \left(FS\right)_{el} = \left(R - E_{NS}\right) \Big/ \left(E_{Si}^2 + E_{Sa}^2\right)^{1/2} \qquad (1)$$

where k_D is ductility coefficient ($k_D \geq 1.0$), E_{NS} is the nonseismic action, E_{Si}, or E_{Sa} is the seismic response to RLE (SL-2) inertial actions, or corresponding different seismic support movement, respectively, calculated according to linear elasticity.

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 (ed. 92) – Mandatory Appendix XIII). Generally, such calculation needs to be repeated several times before the seismic margin value is reached. No ductility coefficient is used in these non-linear calculations, of course (ductility coefficients are used only in linear elastic calculations).

4.6 Seismic Input Data

The seismic response can be calculated in the frequency (spectrum response analysis) or time domain (transient analysis) [1, 10 and 17].



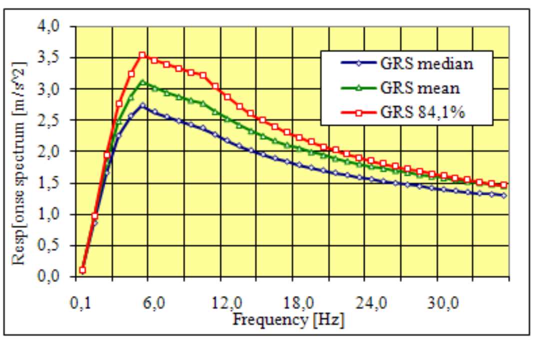


Fig. 2: Comparison of the horizontal and vertical acceleration response spectrum NUREG and GRS

Also, hence the earthquake input must be specified in terms of free-field ground motion accelerograms for time-history dynamic analyses [1, 3, 10 and 17]. The foundation of the reactor building NPP can be embedded into the subsoil. This embedment has generally two effects on the dynamic analysis of the building [17]:

- In comparison to a surface foundation the dynamic behavior of the foundation is different. In the case of rock these differences are minimal. The impedance analysis results in stiffness parameters and damping ratios for the foundation soil system, which are higher than those for a surface foundation.
- The second effect is that the acceleration time histories at foundation level are different from the control motions specified at the surface of the free field.

In the case where structure and soil are idealized in only one Finite Element System or a consistent substructuring analysis the control motion is specified at the top of the surface and the effect of the embedment on both impedance and free field motion are automatically taken into account [10 and 17].

4.7 Calculation model of NPP Structure

The NPP WWER 440 building consists of six objects - reactor building, bubbler tower, air-conditioning centre, turbine building, and lengthwise side electrical building and cross side electrical building [17]. The foundation plate (75,0/43,0m) under building on part V-D/10-22 is on two levels -8.5m. The foundation plate (39,5m/27,0m) under bubbler tower on part D-E/10-17 is on level -8.5 m too. The foundation strip and foot under columns are in the cross side electrical building and turbine building. The global geometry of the NPP structures in Jaslovské Bohunice and Mochovce is identical, but the bracing system and the section area of the steel elements are different.

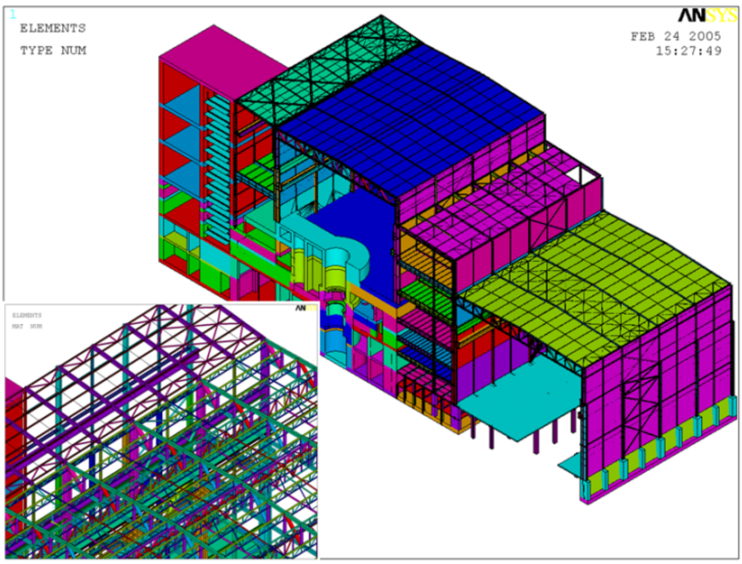


Fig. 3: Calculation model of NPP

The NPP building was discredited [17] by the 3D finite elements model to obtain realistic behavior of structure. The model (VUT Brno and STU Bratislava) consists of 161 856 elements with 440 531 degrees of freedom [15, 17 and 21]. The drawbars are modeled by bilinear elements and contact between bubbler tower and air-conditioning center by gap elements.

The seismic loading was considered by spectrum compatible 3D accelerograms at foundation level to response. The material damping occurring in the soil and the structure mainly involves a frictional loss of energy [17].

4.9 Seismic Resistance of Upgraded Structures of NPP Buildings

On the base of SMA methodology the seismic resistance of the NPP structures in Slovakia was calculated [17]. The seismic load for the NPP site was defined by peak ground acceleration (PGA) and local seismic spectrum in dependence on magnitude and distance from source zone of earthquake. The locality of J. Bohunice and Mochovce are in the different tectonic and seismic site. Also, hence the seismic risk level and the geological condition are different too. The original design seismic load was defined in accordance with the contemporary Soviet standards VSN - 15 - 78 and SNIP-2-7-81.

The following values were defined at that time for the seismic design of seismic category I buildings:

- Design Earthquake-DE, with an intensity 5 in MSK-64 scale;
- Maximal Calculation Earthquake - MCE, with an intensity 6 in MSK-64 scale, and PGA= 0,05g (e.g. PGA= 0,1g) for Mochovce (e.g. J.Bohunice).

During the last 30 years the seismic monitoring of these localities were realized under the supervision of the Slovak Academy of Sciences [2, 12 and 17]. On the base of the investigation results and the requirements of the IAEA NUSS SO-SG-S1 (Rev.1) and S8 the RLE correspond to the SL-2 level was established. The new value of the PGA was established using the probabilistic methodology and new results of the seismic monitoring of the locality [17].

In the actual time the new values of the peak ground accelerations were defined – PGA_{RLE} = 0,35g in J.Bohunice and PGA_{RLE} = 0,14g in Mochovce site for the re-evaluation of the seismic resistance of the NPP structures.

Tab. 1: Recapitulation of the seismic resistance of the principal structural elements of EMO12

HCLPF parameters for structural elements [g]					
Columns primary	Vertical bracing	Beams	Plane truss	Roof bracing	Anchors
SO 490 Tools Hall					
0,184	0,240	-	0,468	0,243	-
SO 800 Reactor Hall					
0,235	0,232	-	0,457	0,186	-
SO 800 Ventilation Hall					
0,157	0,173	1,095	-	0,244	-
SO 805 Longitude Gallery					
0,890	0,642	0,715	-	0,228	0,050*) 0,190
SO 806 Transversal Gallery					
0,368	0,235	0,264	-	1,008	0,190

The recapitulation of the HCLPF parameters of principal structure elements of the NPP buildings in Mochovce is demonstrated in [17]. The seismic safety of NPP building, after strengthening of the steel structures of gallery building floors to the concrete structure of the reactor building, is determined by the seismic resistance of the gallery anchors and secondary columns of the ventilating hall.

5 SAFETY ASSESSMENT OF THE NPP RESISTANCE TO LOCA ACCIDENT

The loss of coolant accident (LOCA) scenario was defined by VÚJE Trnava [17] in accordance with code MELCOR 1.8.5. The guillotine cutting of the ∅13 mm, ∅32 mm, ∅71 mm and the large break LOCA of the 2x∅500 mm 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 200 kPa (absolute value).

5.1 Failure pressure for Containment

The failure pressure p_u is 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

$$p_u = p_{LOCA}.k_r\left[\left(R - E_o\right)/E_p\right] \tag{2}$$

where p_u is failure pressure, p_{LOCA} is pressure in the case of LOCA effect (p_{LOCA} =150 kPa), k_r is reduction factor based on assumption of the stress redistribution due to nonlinear behaviour 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.

5.4 Probabilistic analysis of failure pressure

The general purpose of the probability analysis of the containment integrity was to define the critical places of the structure elements and to estimate the structural collapse. On the basis of previous investigations of VVER 440/213 reactor buildings, carried out in the USA, Slovakia and Hungary [13, 16, 17, 20 and 21] the following critical structures were identified:

- hermetic doors
- reactor dome
- covers of locks (rectangle and circle)
- tube penetrations
- boundaries of the hermetic compartment (reinforced concrete structures and the steel liner)

The simple steel structures of the hermetic zone (doors, dome, covers and tube) can be solved on the base of the linear theorie of elasticity, but the behaviour of the reinforced concrete structures depend on cracking and crushing process [20, 26 and 27]. The probability check of the structural integrity was realized for the critical places, which were defined from the previous deterministic analysis for LOCA loads. Probabilistic analysis was realized by numerical simulation on the base of LHS method using FReET software [22]. The uncertainties of input simples were taken in the form of histograms with the proposed statistical characteristics [17 and 18].

The probability density of pressure failure $\varphi\left(p_u\right)$ is defined in the following form

- steel beams under tanks in the bubbler tower

$$\varphi(p_u) = p_{LOCA} * \frac{f_{var} N_{\mathrm{R}}}{S_{var} \mathrm{N}_S}(1 - \frac{g_{var} S_{var} \mathrm{M}_S}{f_{var} M_{\mathrm{R}}}) * R_{var} \tag{3}$$

- reinforced concrete structures of containment

$$\varphi(p_u) = k_{var} k_{red} * p_{LOCA} * \frac{F_{var} f_{tm}}{S_{var} f_{sym}} * R_{var} \tag{4}$$

where variable parameters k_{var}, p_{var}, g_{var}, f_{var}, S_{var}, R_{var} are defined in the form of normalized histograms with mean values equal to one. These parameters present the probability density of input action effect p_{LOCA}, p_g, σ_{am}, f_{sym} and material resistance f_{aym}, f_{tm}, f_{aym} taken with their mean values. The model uncertainties are considered variable values of action effects S_{var} and resistance R_{var} .

The probability density of input values (table 2) is taken in accordance of requirements of literature [8, 9, 17 and 18] and international standards, Eurocodes, JCSS and OECD [17].

Tab.2: Variable coefficients of input parameter uncertainties

Variab.quantity x	Density	Mean μ_x	Variab.coef. σ_x	Note
Action effect				
k_{var}	Normal	1,0	0,100	Variability of force redistribution due to plasticity deformation
g_{var}	Normal	1,0	0,100	Dead load variability
q_{var}	Gumbel	0,6	0,210	Live load variability
t_{var}	Gumbel	0,6	0,210	Temperature effect variability
S_{var}	Normal	1,0	0,100	Model variability
Resistance of reinforced concrete structures				
$f_{c.var}$	Lognormal	1,0	0,111	Variability of concrete strentgh
$R_{o.var}$	Lognormal	1,2	0,150	Variability of bending resistance
$R_{v.var}$	Lognormal	1,0	0,100	Variability of shear resistance
$R_{n.var}$	Lognormal	1,2	0,150	Variability of compression strentgh
$R_{sp.var}$	Lognormal	1,0	0,150	Variability of connection resistance
Resistance of steel structures				
$f_{s.var}$	Lognormal	1,0	0,083	Variability of steel strength
$R_{o.var}$	Lognormal	1,0	0,050	Variability of bending resistance
$R_{v.var}$	Lognormal	1,0	0,100	Variability of shear resistance
$R_{n.var}$	Lognormal	1,2	0,100	Variability of compression strentgh
$R_{sp.var}$	Lognormal	1,15	0,200	Variability of connection resistance

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 [17, 18 and 22]. For a complex analysis of the concrete structure for different kind of loads, ANSYS software and the program CRACK (created by Králik) [13, 16, 17 and 20] 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 DOF (see Fig.4). The probability analysis of the concrete structure integrity was considered. The failure pressure is equal to $p_{u.0,95}$ = 486 kPa for 95% probability of penetration.

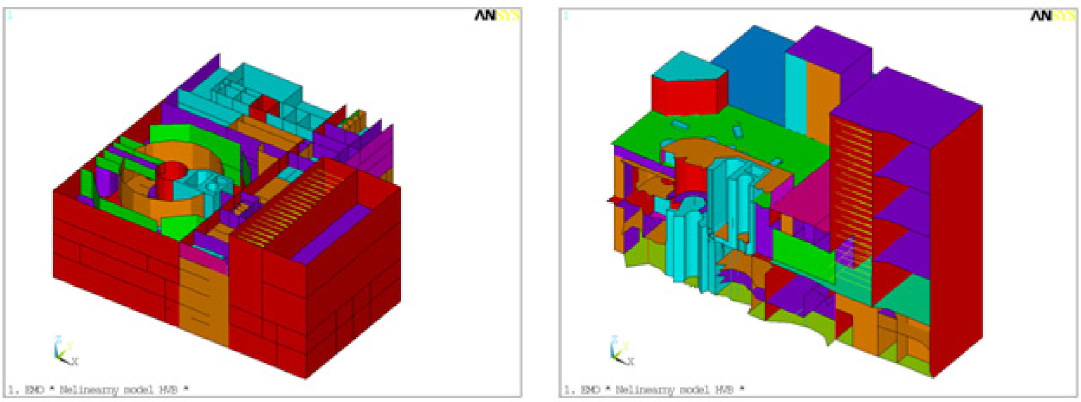


Fig.4: Calculation model of NPP building

6 CONCLUSION

This paper presents the results of the "Stress tests" in Slovakia on the base of Fukushima accidents [7, 23 and 24]. There were proposed the methodology of the risk analysis of the NPP hermetic structures penetration due to the accident events [5, 6, 20, 21 and 24]. There were summarized the works performed by the IAEA in the areas of safety review [17]. The methodology of the seismic reevaluation of NPP in Slovakia is based on the new results from the geological and seismic-tectonic monitoring of this site. The generation of the seismic loads on the base of probabilistic seismic risk analysis was described. The results from this analysis present the international level of the seismic resistance of the NPP structures in Slovakia. The methodology of the PSA 2 level analysis of the NPP hermetic structures penetration under accident events is discussed. 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. The critical steel segment was the reactor hermetic door with failure pressure $p_{u.0,95}$ = 839 kPa (95% failure probability). The critical concrete structures were the walls of the rooms A525 and A526 under steam generator box. Their failure pressure is equal to $p_{u.0,95}$ = 486 kPa (95% failure probability).

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