DESIGN OF EARTHQUAKE RESISTANCE ENHANCEMENTS OF THE DUKOVANY NUCLEAR POWER PLANT BUILDING STRUCTURES

Z. Plocek\textsuperscript{a}, P. Štěpánek\textsuperscript{b}, V. Salajka\textsuperscript{b}, J. Kala\textsuperscript{b}, V. Kanický\textsuperscript{c}  
\textsuperscript{a}ČEZ Group, The Dukovany Nuclear Power Station, Czech Republic  
\textsuperscript{b}Brno University of Technology, Czech Republic  
\textsuperscript{c}KDV – Consulting Bureau, Czech Republic

Email address of main author: Stepanek.p@fce.vutbr.cz

Abstract. The paper deals with problems of enhancing earthquake resistance of safety related building structures of the NPP Dukovany. The plant operates for over thirty years and there is time to verify the actual state of structures. Two years ago a detailed seismic response analysis of structures considering the newly postulated earthquake loads has been performed. The computation model has been developed involving the complex of all interacting structures of the main production block with equipment. Seismic analysis has been performed using response spectrum method. The reliability assessment of structures has been carried out. Consequently feasible ways of enhancing the seismic resistance of structures have been designed. In order to verify the designs a second seismic analysis has been recently accomplished using acceleration time history method. The results of analyses are discussed in the paper.

1. Introduction

At the present time nuclear power plants of the WWER type operate for about three decades. In Czech Republic, the Dukovany NPP of the VVER-400213 type has been put into operation in 1985. Since that a steady high level of plant’s operational availability and reliability has been proved on international level. No noticeable problems with operational safety have been noticed. An effective way of managing updated safety requirements ensures a high safety level during the whole plant’s design life.

However, in accordance with general trends, there are increasing requirements of upgrading the plant and operating the plant in excess of its design life. The development of an effective system of management of ageing effects in structures and components is the key for maintaining the operational safety during the extended plant’s operating lifetime. Moreover, due to steadily increasing additional safety requirements, the operational safety has to be enhanced. Besides many other important problems, those related to earthquake have to be considered.

The earthquake resistance of building structures during the predicted residual life has to be investigated, considering both updated and newly postulated severed earthquake risks. The detailed seismic analysis of safety related structures of the plant constitutes the basic step in the assessment of the general reliability of an upgraded nuclear plant as a whole. Selected problems of enhancing earthquake resistance of building structures of the NPP Dukovany are dealt with in the presented paper.

2. Basic assumptions of the seismic analysis

Seismic response analysis of the selected nuclear plant building complex has been performed
using an extensive global model including all interacting structures. Study of past time analyses which were based on variant computations of separated individual structures involving approximately modeled interactions has revealed large dispersion of results. Consequently an overly conservative selection of results for application was inevitable. Such an approach has been fruitful in the plant building design phase, but it cannot be applied in the refined process of enhancing safety of an operating building complex.

Components of mutual constraints of analyzed structures as well as all exposed structural components have been modeled in detail, in order to get directly the loads for eventual redesign. The selection of components has been based on results of both past and revised seismic analyses using models of separated structures with simplified external interactions.

For the response analysis, seismic motion inputs have been defined in accordance with the International Atomic Energy Agency recommendations for seismic re-evaluation of operating nuclear power plants with VVER-type reactors [1].

Seismic analysis has been performed using the computer with the AMD Athlon 64 FX 51 processor with 4 GB RAM and 1000 GB of disk space. The 32 bit version of ANSYS 8.1 running under Windows XP 64 Bit Edition operating system has been used.

3. Computation model of structures

The computation model includes the reactor hall with the group of surrounding building structures constituting one of the four main production blocks of the nuclear plant. The building complex has a common reinforced concrete foundation plate. The reactor hall and the adjoining accident restraining tower represent massive reinforced concrete structures. The equipment housing buildings adjoining the reactor hall (along and across) are designed as steel structures. The turbogenerator hall situated in front along the reactor hall is designed as a classical steel structure, too.

A sophisticated spatial computation model has been developed particularly for the seismic structural analysis using finite element method of discretization [3]. Element library of the ANSYS program package has been used. The structure of the model is obvious from the graphical presentation (see Fig. 1 up to Fig. 6).

The main reinforced concrete structures have been modeled using spatial finite elements of the SOLID type. Roof structures, floors, partition walls and sheathings have been modeled by SHELL type elements. Steel structures have been modeled using mostly elements of the BEAM type. Important structural details such as anchoring elements, stiffeners, equipment supports, have been modeled using finite elements of the type LINK. Elements of the type MASS have been used to model the concentrated equipment masses.

For the seismic analysis, there is generally no need to use an overall extra fine discretization of the structure. The computation model has to represent the spatial distribution of both the stiffness and operating mass of the structure to an extent that ensures correct determination of significant features of structural seismic response [2]. Consequently, main load bearing structural components have been modeled in details, taking into account their stiffness as well as inertial properties. Non-structural components have been modeled with respect to their inertial properties only. However, in selected structure regions even secondary structural members have been modeled in detail in order to allow for a correct evaluation of local seismic components of displacements, deformations, forces and stresses.

The developed FEM-model involves 134983 elements localized by 133270 nodes with 427442 degrees of freedom.
The mechanical energy dissipation in structures has been modeled so as to correspond to the constant modal damping ratio of 7%. Consequently in analyses have been introduced either appropriately selected response spectra or Rayleighs model parameters. The relatively high damping corresponds to facts, that prevailing part of the structural complex is formed by reinforced concrete structures, and that the level of response deformations is expected to be high.

The interacting subsoil has not been included in the computation model. Thus, the subsoil-structure interaction has not been explicitly modeled. However, mechanical subsoil properties influencing the seismic response have been respected by using seismic motion inputs selected properly for the actual site subsoil category.

4. Natural frequencies of the modeled structural system

The modal analysis yields a deep insight of the dynamic behavior of the modeled structural system. Calculation of an appropriate number of natural frequencies and normal modes of vibration of the model represents the basic step in computing the seismic response by the method of response spectra. In the given case 3500 natural frequencies and normal modes of vibration have been computed in order to satisfy the condition that modal response components of up to 33 Hz have to be considered.

Normal mode of vibration with dominant displacements of the accident restraining tower \((f = 3.79 \text{ Hz})\) is shown in Fig. 7 for illustration. Normal mode with dominant displacements of the reactor hall roof structure \((f = 4.49 \text{ Hz})\) is shown in Fig. 8.

![Fig. 7. Mode of vibration – f = 3.743 Hz.](image1)

![Fig. 8. Mode of vibration – f = 4.942 Hz.](image2)

5. Seismic motion inputs

Seismic motion inputs have been defined in accordance with the IAEA recommendations for seismic re-evaluation of operating nuclear power plants with VVER-type reactors. As mentioned above, interacting subsoil body subjected to external seismic excitation has not been included in the computation model. Consequently, seismic motion input has to be defined directly for the foundation of the analyzed structural system.
For the seismic analysis of the computation model using time history method the seismic motion of the foundation plate has been defined by synthetized site-specific design acceleration time histories $a_x(t), a_y(t), a_z(t)$ (see [1]). The applied course $a_x(t)$ is shown in Fig. 9 for illustration.

For the seismic analysis of the model using response spectrum method the seismic excitation has been defined by site-specific acceleration design response spectra $S_{ax}(t), S_{ay}(t), S_{az}(t)$ for three orthogonal directions (see [1]. The response spectrum for the horizontal direction $x$ is shown in Fig. 10.

The spectra have been derived for the reinforced concrete foundation plate bearing a simplified model of the complex of analyzed structures (see [2]). The foundation plate has been subjected to seismic motion defined by the free field three-directional acceleration time history considering the actual site subsoil category.

6. Seismic response spectra analysis

In the first phase of the advanced seismic analysis of NPP Dukovany building complex, seismic response to newly specified earthquake loads has been computed using the response spectrum method (see Ref. [3]). Seismic responses have been computed separately for three orthogonal directions. The CQC rule has been applied for the combination of modal responses using all significant modes out of the 3500 modes of vibration that have been computed. Four alternative combinations of resultant directional responses have been used to assess the total seismic response extremes.

The extreme seismic responses (maxima and minima) have been combined with the response to operating steady loads using a simple superposition rule. The structure response to steady loads has been determined using the same model as that used in the seismic analysis. The extreme values of combined response quantities at individual components have been found. The feature of the static response is shown in Fig. 11. The dynamic response is illustrated by Fig. 12. The combined response is shown in Fig. 13. Principal stresses in reinforcement are illustrated by Fig. 14.
7. Assessment of the earthquake resistance of structures

In accordance with the relevant recommendations of IAEA the earthquake resistance of structures has been assessed using the High Confidence Low Probability of Failure (HCLPF) approach. The HCLPF value expresses the actual limit resistance of the structural component to earthquake load relative to the given earthquake load. The earthquake resistance of the complex of structures has been assessed according to the least value HCLPF of that computed for all main structural components or parts. For the analyzed case the HCLPF values should be greater than 0.1 g, otherwise appropriate structural modification have to be designed. Ten-thousands structural components and constructional details have been individually thoroughly investigated in order to get correct earthquake resistance assessments. Consequently feasible structural modifications, which shall enhance the earthquake resistance of the analyzed structures, have been designed [1].

For instance:

(a) The advanced seismic analysis has provided a realistic view on both the character and level of seismic loads of components realizing mutual constraints of structures in the building complex. Additional anchoring of the floor steel structures adjoining the reactor
hall in the reinforced concrete wall has been proposed.

(b) The analysis has indicated potential endangering of electrical installations due to relatively low ductility of adjoining partition walls. Additional cross-framing has been designed which increases substantially both bending stiffness and ductility properties of walls.

(c) The analysis has revealed high stresses in a number of both horizontal and vertical roof structure members. Although ductility properties of the structural steel present a satisfactory reserve with respect to the load limits, structural modifications involving additional members have been recommended.

8. **Seismic analysis using acceleration time histories**

The seismic response of the NPP Dukovany building complex has been computed using the response spectrum method. Extreme responses combined with respective static responses have been applied in the design of structural modifications assumed to enhance the seismic resistance of structures.

However, when deducing detailed conclusions from the analysis, some questions have arisen, related mainly to typical deficiencies of the response spectrum method (e.g. inaccuracies due to missing mass effect, displaying of the actual vibration shape, identification of stress components). It has been found as reasonable to carry out another seismic analysis in order to verify the conclusions of the seismic response spectra analysis prior to the final design of structural modifications.

Recently, seismic response analysis of the NPP Dukovany building complex has been accomplished using the time history method. Seismic excitation of the computation model has been defined by synthetized site-specific design acceleration time histories.

The computed responses have been stored in the form of time histories of the response quantities (displacements, stress resultants, stresses, forces). Consequently the envelopes of the seismic response time curves have been determined. Response extremes have been computed and combined with the responses to operating steady loads. The combined response extremes have been compared with the extremes obtained by response spectrum method.

The envelop of minimum dynamic response of displacements is illustrated by Fig. 15. Combined response is shown in Fig. 16. Stresses in steel roof members are presented in Fig. 17 and Fig. 18.
The verifying seismic analysis using acceleration time histories has proved its importance. With respect to the seismic response spectra analysis the new analysis has shown a more realistic view on the character of seismic responses. Although the changes of mean response levels for structural components are mostly not substantial, the newly computed local responses are in many cases considerably higher.

In general, all proposed structural modifications based on the seismic response spectra analysis have been proved to be principally correct. However, new data obtained by the seismic analysis using acceleration time histories have been respected in the final design of the proposed structural modifications.

9. Conclusion drawn from seismic analyses

Some important conclusions have been drawn from performed seismic analyses. The correctness of the seismic response analysis of NPP structures depends principally on the correctness of modeling physical process going on in the course of the earthquake event. Due to the nature of the process an objective direct assessment of the correctness of the computation model is practically impossible. There is meaningful only the comparative
assessment of results obtained through various approaches and computation procedures. With respect to the contemporary state of engineering computations the modeling of the process involves the modeling of the analyzed building structures, and modeling of the seismic motion of the base of structures.

It has been concluded, that the modeling of NPP structural complex as a whole represents the fundamental condition to get reliable results. The complexity of response displacement fields has shown practical impossibility to formulate boundary conditions for a correct solution of separated substructures. It has been proved that, due to very complicated vibration mode shapes, reliable results in stresses can be obtained only when even minute details of the structure are modeled. It has been shown, that simplified modeling of mutual constraints of substructures commonly used in standard analyses cannot be applied in the seismic analysis. Special constraining elements modeling unambiguously the load transfer have to be used.

With respect to the modeling of the seismic excitation of the analyzed structure, principally two basic approaches can be applied. In the first case, seismic input motion is specified by acceleration response spectra. In the second case, a set of acceleration time histories is used. For an actual NPP site these two seismic input motion specifications are principally identical. However, the design of structural modifications enhancing earthquake resistance of NPP building structures should be based on acceleration time history seismic excitation.

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