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ASSESSING SEISMIC ADEQUACY OF EXISTING NUCLEAR POWER PLANT STRUCTURES

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ABSTRACT

Nowadays Russia's specialists perform a huge amount of works to revaluate the NPP safety. These works are certain to include refinement of NPP safety assessment under the effects of specific dynamic loads, earthquake effects included. It should be noted, that a number of Russian NPPs now in operation had been designed either with no account of these loads, or under the requirements which are underestimated as compared with the modern requirements on the external load composition and rate. Revaluation of NPP seismic safety is based on the results of the works taken under orderly sequence on assessment of (1) seismic input and ground effects; (2) structure response and state; (3) equipment and pipelines response and state.

The paper considers the methods of NPP structures response and state assessment. Therewith we assume that ground motion predicted behavior at the construction basement has been preset for the SSE and OBE conditions and the effects of soil-structure interaction, including the situation of possible soft soil liquefaction. Necessity to determine both the reaction of a construction and its state as a whole as well as its elements reaction, to evaluate their bearing capacity and destruction zones formation makes it necessary to make up a detailed structural model, which is usually a finite-element one. Since seismic revaluation is to be performed for the existing structures, characteristics of which can substantially differ from the design ones, revealing the actual state of this structures becomes critical. If the real values of physical and mechanical properties of the structure materials, connections of elements etc. are used as initial data in a structural model this permits to increase the design assessment credibility and reliability substantially.

The paper analyzes the results of determining these initial assessments while inspecting several Russian NPPs on the basis of a "combined" method. This method is realized at two consecutive stages. The first stage is the procedure of integrated assessment of state and conditions of fixing the separate elements, structures or a construction as a whole. It is based on the results of analysis of natural oscillation, which are induced by the pulse load. The second stage is presented by the technique of determining the elastic characteristic and material strength as a result of analysis of shallow wave parameters, which are excited by the pulse shock load too. Comparison of the results of seismic structural analysis with the design and real characteristics of material and the state of connections in several cases illustrates a pronounced effect of wear on their reaction to a seismic input. It is precisely these data that are to be taken as the initial data for seismic revaluation of equipment and pipelines reaction and state.

KEY WORDS: safety assessment, seismic adequacy, building structures, tool examinations, reevaluation.

INTRODUCTION

The increasing seismicity level in several areas of Russia makes it necessary to solve the following problems for existing NPP:

- to perform special works on seismic strengthening of NPP buildings and equipment in order to bring their seismic stability to the level required;
- to carry out design evaluation of the possibility of NPP safety provision in case of considerable damages of buildings and structures.

Solving the problem of seismic strengthening of buildings and structures has been the objective of much research, all of this was reflected in standard material and procedure specifications, in construction and renewal procedures. By this is meant that most of existing types and structural configurations of buildings have practically testified designs for seismic strengthening. However, real financial limitations give no way to do our utmost in the shortest possible time. Therefore an in-site safety assurance evaluation in case of considerable damages of buildings, first, will allow to receive a realistic prognosis of earthquake consequences, second, it will serve as the basis for the feasibility study for carrying out the works on the new requirements on seismic stability of buildings and structures. Thus, design evaluation can be performed independently or in parallel with operations on seismic strengthening.

Analysis of the results of seismic revaluation of existing NPP buildings and structures permits to state that standard designs, which are used in engineering practice, seem to be insufficient to evaluate expected damage degree of building supporting elements under the design basis earthquakes. More often than not it is required to design on the basis of real (or synthesized) records of ground motions and complicated 3D models of buildings to judge reasonably the possibilities of a required safety level provision. Furthermore the major factor of credibility of the design evaluation received is the quality and completeness of the data on examination of the existing buildings actual state, which are to be taken into account when designing. Thus, building foundation differential settlement causes considerable stresses in the walls, these stresses (even if the monolithic nature of masonry is preserved under the normal operation conditions) at earthquakes will result in considerable supporting element damages. Furthermore, the background seismicity and other impacts (transport vibrations, industrial explosions etc.) create a damage accumulation effect in supporting elements and change the real ability of buildings to resist a seismic input.

APPLICATION OF TOOL EXAMINATIONS TO EVALUATE THE REAL STATE OF NPP BUILDING STRUCTURES

Nowadays the most commonly used nondestructive methods of determination of physical and mechanical characteristics of building materials and structures (mechanical ones: rebound, plastic deformation, chip, explosion, etc, ultrasonic characteristics) have substantial drawbacks: they require an access to the structure surface under testing and permit to determine the required material characteristics at shallow depths (usually up to several mm). The exception is provided by a through ultrasonic testing, but it could be performed only given the access to two building structure surfaces, which is not always possible. When inspecting the building structures which are under operation these methods are frequently hindered or inefficient for the following reasons:

- the structure surface may have coverings (for example, plaster or metal isolation) and their removal and further recovery involve huge expenditures;
- the shallow layer characteristics of the material structure may differ substantially from the characteristics of the body of masonry as a result of carbonization, weathering, frost action, effect of high temperature or corrosive media etc;
- the building could be a multilayer structure.

To examine such structures we need special methods, enabling both to determine the characteristics of structural materials at great depths, furthermore, it is desirable to be on a large (several meters) base and without coverings removing, as well as to assess the conditions of connecting the structural components among themselves.

We had developed a combined method of evaluating the mechanical characteristics of materials, which are used for various structural elements, and the nature of their fixing (types of connections), the method is based on vibrations and wave dynamics theory. The evaluation procedure consists of two parts:

- the procedure of integrated evaluation of state and pattern of fixing the separate elements, structures or buildings as a whole on the basis of analysis of natural oscillation, induced by pulse shock load;
- the procedure of determination of acoustic characteristics of structural material at the area under testing and on the basis of these results the procedure of determination of elastic characteristics (shear modulus and elastic modulus), as well as the strength of material using the correlation dependencies.

The procedure in question is based on the analysis of shallow wave parameters, which are excited by pulse shock load.

Procedure for integrated evaluation of state and fixing pattern of the structural elements is based on exciting the structure natural vibrations by a shock pulse load, measuring these vibrations in various points of the structure, detecting vibrations at various frequencies by spectral analysis methods and analyzing these vibrations (defining the frequencies and vibration decrements, constructing their diagrams, i.e. dependencies of the amplitude on the measurement point coordinates).

Giving no consideration to the peculiarities of natural vibrations excitation and record, the processing and interpretation results, I'd like to note that the schemes of excitation and measurement of vibrations, modes of identification and detection of various vibration modes had been developed in details, these schemes enable us to determine frequencies with sufficient accuracy and to construct the diagrams for two or three (sometimes even more) first modes of structural vibrations.

Using this procedure permits to find a sound structural model, this model in its turn is the basis to defining elastic material characteristics of a structure (if its dimensions are known), which are averaged through the entire structure or its elements, as well as large defects which affect the form of diagrams of the vibration modes revealed.

Procedure for determination of acoustic characteristics of structural material is based on excitation of shallow waves by shock pulse load, measurement of these waves on the structure surface and analysis of their parameters.

It is known, that the velocity of shallow waves, Rayleigh waves as well, is related to its dependence on the length (variance curve), which is determined by wave propagation velocity distribution in structural material (by dependence of a shear wave velocity and Poisson's ratio on the distance to the surface). Shear wave velocity in its turn is closely related to the shear modulus, and taking into account the Poisson's ratio it is related to the elastic modulus. Thus it is believed that given the variance curve, a shear modulus of various structure layers is determinable.

Such inverse problems in seismic acoustics have no unambiguous solution. However when inspecting the building structures the problem is frequently simplified by the fact that we usually dispose information on geometrical characteristics and material of various structural layers, which permits to receive unambiguous solution.

The purpose of the investigations conducted was to analyze for a possibility to construct shallow wave variance curves from the realizations, derived due to pulse effect (impact). With this aim in view a mathematical modeling had been conducted, in which a multilayer plate response to pulse effect in a point was simulated. The problem has been solved as a plane one. These versions had been simulated: (1) uniform half space; (2) uniform plate; (3) one- and two-layer plate on the half space at various relations of layer thickness and longitudinal wave velocities, and Poisson's ratios in layers and half space; (4) two- and three-layer plate at various relations of layer thickness and longitudinal wave velocities and Poisson's ratios in layers.

The first two versions were simulated to compare the results of variance curve construction with the theoretical data. The obtained realizations of vibrations in various points of surface were filtered through several band filters with 0,5 octave band pass over a wide range of frequencies, after that the filtered-out realizations were used to determine the coordinates of variance curve points. For all the versions variance curves had been constructed, furthermore, for the half space and for the uniform plate the results were in complete agreement with the theoretical data.

In more complicated cases the variance curve form does not entirely coincides with the forms in [1]. The difference is in that variance curves, constructed due to pulse processes, as a rule are free of branches, they are the result of averaging the parameters of waves, propagating in different layers. The one exception is provided by the versions of "soft layer on the rigid base" and "rigid interlayer in the soft soil" at velocities of waves in the layers, which differ several times, where two branches of the variance curve are observed at short waves.

Thus, mathematical modeling had shown that construction of variance curves for shallow waves phase velocities from the pulse realizations creates no problems, and their construction accuracy permits to assess acoustic characteristics of building structure materials (multilayer structure included) with reasonable resolving power. The algorithms developed were used at processing the real objects inspection data, and the modeled variance curves were used to analyze the object variance curves.

Using this procedure permits to assess acoustic and elastic characteristics of materials of various elements of building structures or their separate sections.

The procedures described above could be used independently or in combination. In case of their complex use the elastic characteristics and the material strength are assessed from the shallow waves, and the structural model is selected so that its design dynamic responses (frequencies and forms of resonant vibrations diagrams) most closely agree with the data which had been received experimentally for natural vibration. As an example of such a complex use of this procedures let's consider some results of examination of brick building walls of machine hall of Leningrad NPP.

METHODICAL BASES OF SEISMIC REEVALUATION OF EXISTING NPP BUILDINGS AND STRUCTURES

Evaluation of the building state after design earthquake is carried out by method of direct dynamic load design with the use of real earthquakes records, analog or synthesized accelerogramms. In this condition the actual geology of foundation soils (according to the microseismic zoning data) and their capability to transform a seismic input are taken into account. We had developed a procedure in which soil mechanical behavior during the process of interaction with seismic waves is described on the basis of viscoelastic models (for the magnitude 6 as per the MSK-64 scale and below) and on the basis of elasto-plastic hysteretic models or visco-elasto-plastic models for the magnitudes more than 6. The validity of the procedure developed had been confirmed when solving the problem of verification of parameters of seismic inputs once they had passed through the foundation soils when designing buildings and structures of nuclear power engineering objects.

When evaluating the same building design models with increasing degree of complexity and details are used in succession. The need for such approach stems from the numerous facts that there is no direct dependence of building damage degree (including the same-type buildings) at earthquakes on such parameters as number of stories, structural scheme, etc., therefore successive improvement of the design model will allow to reveal the influence of various factors on building behavior under seismic effects with due regard to damages. The principal condition of design evaluation is accounting for the changes in model states during the earthquake effect under limiting states of separate elements or the building as a whole. In this condition it is necessary to consider the following possible changes of the model state:

a). Building separation into large parts, this separation is marked by through cracks which pierce the entire building in vertical or horizontal directions (adjacent walls separation, building separation and displacement along its foundation, displacement of floor disks and building separation into parts within the stories, etc).

It is evident, that building separation into large parts has a pronounced effect on its natural vibration frequencies. Hence, according to the in-situ measurement data the lateral-direction natural period values in brick buildings (being 0,2 s before Gazly earthquakes) had increased up to 0.4 s after these earthquakes.

At present solving the problem of description of building separation into large parts during seismic input involves the problems related, mainly, to absence of limiting state criteria and proper design models. According to the above procedures in the first stage a building dynamic load design with the use of 3D finite-element model without regard for failures will be performed. Then the building zones in which internal forces (stresses or deformations) exceed the limiting values for the code on aseismic building are to be determined. The design model is supplemented with the elements, which work only up to a certain level load, following which they are eliminated and dynamic load design is performed once again. If new zones of possible failures arise the further model improvement is to be performed. For example, if in the first design stage we find the possibility of vertical cracking and end building wall separation from the spine walls because tensile stresses exceed their limiting values, then at the sites of their junction the elements are introduced, which are to be eliminated provided that:

$$\sigma_P \geq R^H{}_{P,}$$

where $R^H{}_P$ - standard tension resistance for the masonry.

b). Changing the pattern of load perception by building supporting structures due to failure (if some components show a notable loss of bearing capacity) or changing the conditions of element junctions (horizontal cracking in upper and/or lower part of partition walls, diagonal cracking in the blind walls within a story etc).

If such a mechanism of building failure is realized in its design model then either changing the element connection conditions (unilateral constraint forming, sliding friction, etc) is performed, or stress-strain modulus alters locally, or the elements which correspond the destroyed part of structure are eliminated from the design model. In all the conditions specified a change of building stiffness and natural vibration frequency takes place. Once a number of elements had been eliminated from the design model a calculation of the building changed state under static loads is to be performed and the criteria of limiting state are to be specified.

c). Changing the masonry monolithic nature, which, as a rule, decreases the natural vibration frequencies of buildings, that is, their stiffness. These changes are related to numerous internal processes in the masonry under dynamic load input, which are associated with formation of tiny cracks and its continuity disturbance. It is to be noted, that masonry is a non-uniform elastic plastic body which consists of solution-filled stones and joints. It is obvious that in this case the choice of integrated parameter which accounts the entire set of factors will be a proper one. Numerous investigations use as such a parameter a stress-strain modulus of masonry, multiplied by degradation factor, dependent on the stressed-deformed state in building structures.

Under compressing stresses in masonry ($\sigma_P \geq 0.3R_{cm}$) the current stress-strain modulus is recommended to be determined as follows

$$E_1 = \alpha\ (R_{cm} - 0.91\sigma).$$

where (α - masonry elastic characteristic according to Russian norms and standards, R_{cm} - standard compression resistance of masonry.

Similar analyses for determination of masonry degradation factors due to various conditions are under investigation in many countries. The paper [2] shows the analysis for the most commonly used degradation models and comparison of design evaluations with research data on wide and narrow partition walls.

PRACTICE OF SEISMIC RECERTIFICATION OF NPP BUILDING STRUCTURES

Seismic recertification of a water fire extinguishing pump building

As example of the procedures above is the design results of a relatively simple NPP water fire extinguishing pump (WFEP) building for SSE of magnitude 7 as per the MSK-64 scale. During dynamic load design a 3D finite-element building model was used. As a result we received the displacements of all the model units and the forces in building structural elements. The designs carried out gave the evaluation of additional loads influence on meeting the strength conditions, the loads arising due to seismic input to building structures. The standard criteria of strength used at building model evaluation allowed to determine the possibility of its separation into large parts and to reveal the zones of localized failures.

WFEP building overall view is presented in Fig. 1. This building plan view is a rectangle which measures 20.51x10.2 m. Building height is 8.6 m at the 0.0 m level. In design the WFEP building includes three parts which differ in their stiffness and inertial properties: modular and monolithic reinforced concrete foundation from the building base up to 0.0 m level; masonry walls above the 0.0 m level, and the building roof, which is made from reinforced concrete panels and is at the + 8.3 m level.

Development of a finite-element WFEP building model was carried out from the operating drawings. In doing this all building design features (windows and doorways, brick masonry of various thickness, floor plates scheme etc) were reproduced in the model. In the places of possible building separation (in the places of outside walls junctions among themselves and with internal partitions, as well as in the zone of building walls connection with foundation) the design model was added with the elements, which were eliminated as soon the limiting value of forces had been achieved. Besides, the changes in the masonry stress-strain modulus when achieving the limit value of compressing stresses was taken into account.

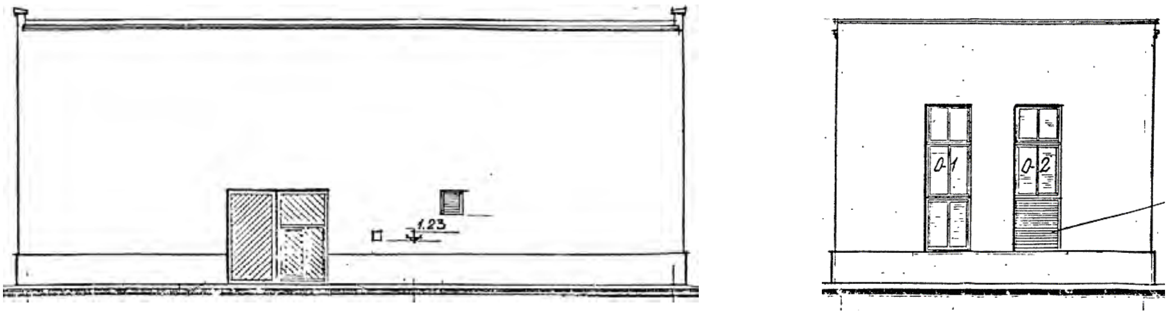

Figure 1. Pump building overall view

Analysis of building structural configuration had shown that absence of symmetry planes in plan view does not result in substantial displacement of building centre of gravity relatively its geometrical centre, this displacement was equal ~0.03 m in lateral direction and ~0.24 m in longitudinal direction. Building stiffness in longitudinal and lateral directions with due regard to ground foundation stiffness was $2.287 \cdot 10^9$ n/m and $0.912 \cdot 10^9$ n/m respectively.

Analysis of the static design results had shown that the stresses acting in brick masonry sections (compression, shear, etc) and deformation are substantially lower than allowable standard values, this agrees with the building examination data.

As the design seismic input we took the accelerogramms of 9.03.1949 Holister earthquake. The parameters of seismic input at the building foundation had been determined due its passing through the foundation soils, the structure and physical characteristics of which were determined from the microseismic zoning data. Both accelerogramms and velocity and soil-surface displacement curves, as well as response spectra corresponding to the design basis earthquake are shown in Fig. 2.

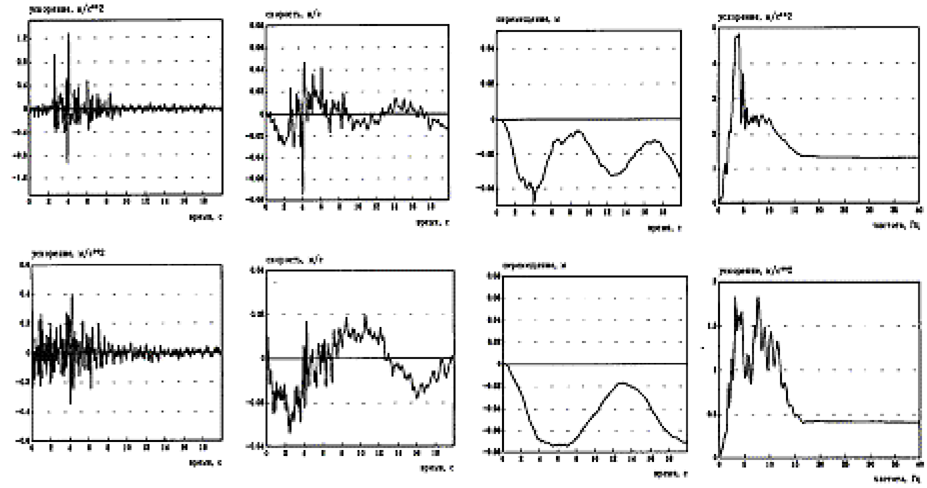

Figure 2. Design seismic impact

The displacement diagrams show that at earthquake the soil surface is rocking with amplitude ~2 cm relatively the common non-compensated soil displacement, equal to ~2 cm in horizontal and ~4 cm in vertical direction. As for their value the hard soil surface displacements correspond those observable at earthquakes of magnitude ~ 7 (MSK-64 scale). Analysis of the effect spectra permits to state that:

- the heaviest rate of seismic input is displayed within the range of frequencies from 3 to 15 Hz both in vertical and horizontal directions, where the dynamic coefficient of the effect is ~4.
- for the horizontal effect component the presence of one peak of acceleration in the spectrum at frequency ~ 5 Hz (4.8 m/s^2) is typical;
- for the vertical effect components in the spectrum two peaks of approximately equal rate (~ 1.7 m/s^2) at frequencies 4 and 9 Hz are observed.

From the results of WFEP building dynamic load design for the earthquake of magnitude ~7 (MSK-64 scale) due to Holister accelerogramms we had determined:

- failure of the building brick part connections with reinforced concrete foundation takes place practically immediately, this serves the decrease of wall loads;
- maximum displacements of brick part of WFEP building relatively the foundation are: 1.5 cm along the short building side and 1.1 cm along the long building side;
- strength of brick WFEP building walls at the preset seismic input is obtained with the exception for the partition, wall crosswise the building, in its upper part the conditions of lateral shear strength are disturbed and a crack is formed, however the partition doesn't separate from the outside building walls at its whole height;
- reducing the building stiffness due to change in masonry deformation module, related to its continuity disturbance and tiny cracks formation, doesn't exceed 5%.

Seismic recertification of tank for a foam fire extinguisher

Tank of foam fire extinguisher is a banked thin-walled reinforced concrete structure which is cylindrical in plan 12 m in diameter, used as a tank for water reserve. Tank is embedded into the ground for -4.2 2 m depth. Relation of the sides of natural soil prism banking is 1:1.5, that allows for its stability under standard operation conditions. The goal of the study was tank strength analysis under magnitude 6 SSE effect as per the scale MSK-64 with due regard to surrounding soil action.

The overall view of the design finite-element model for the tank-surrounding soil system and its section are presented in Fig. 3. Structural thin-walled tank elements (bottom, cover and walls) were simulated by shell elements. Columns were simulated by beam elements. Surrounding soil was simulated by spatial elements. In the places where soil contacts with external tank surfaces displacement continuity condition was accepted, i.e. soil displacement relatively the tank in contact places was not taken into account. Precomputation of the tank had indicated, that soil stiffness under bottom was high enough and soil stresses due to structures pressure for a depth damped quickly, therefore when simulating it would suffice to be limited with the soil layer for 1 m depth below the pit bottom.

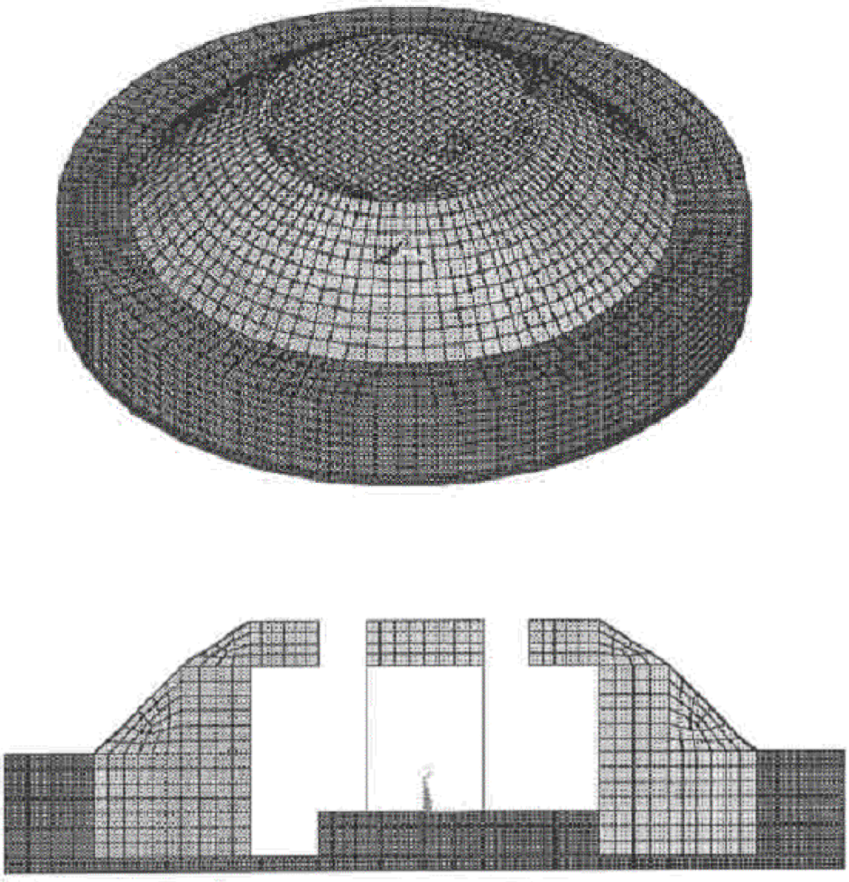

Figure 3. finite-element model for the tank-surrounding soil system and its section.

Seismic stability of tanks is limited by strength of their structural members. Fencing structures should be estimated by limiting state for the second group, and the columns - by limiting state of the first group. For fencing structures the strength disturbance condition is the condition of through shake occurrence, for columns - excess of actual stresses over the design strength of extension $R_{bt,n}$ for tension area of concrete with simultaneous excess of design compressive strength R_{bn} for compression area of concrete. Values of design resistance $R_{bt,n}$ and R_{bn} were used in calculation with regard to coefficient of operation conditions $\gamma_b = 1.1$

In accordance with Russian standards it is allowed to conduct determination of stresses and deformation on the assumption of static effect of designed earthquake loading on equipment.

Thus, tank design for seismic input includes the following procedures:

- calculation of seismically-triggered inertial loads and hydrodynamic pressures on tank walls and bottom with due regard to target reaction spectra in the base of supporting structure and the values of natural frequency of system vibrations;
- calculation of stressed-deformed state of the tank at preset loads;
- check of tank strength under the action of static and earthquake loads with due regard to accepted criteria.

When designing the tank in normal operation conditions the following loads were taken into account:

- from own mass of tank structures;
- water hydrostatic pressure (filling of tank to 4.8 m level);
- pressure on tank covering from the back-fill;
- pressure on outer surface of tank walls from the surrounding soil.

As a result of calculations the stresses in tank structural member sections have been received: for shell, bottom, covering and columns.

Comparative analysis of calculation results has indicated, that simplified formulae give more conservative evaluation of stresses in the shell. When using in calculation the model developed the stresses in the shell will be smaller, than when using of simplified formulae, as compensating soil pressure to the shell external surface and deformation restriction because of bottom and covering are taken into account. As a whole in both cases tension stress turned out to be lower than allowable values for concrete B15 (1.265 MPa).

Flexural stresses in columns practically are equal to zero, and compression stresses are ~ 7.4 MPa, which is also lower than allowable values for compression area of concrete (12.1 MPa).

Strength of bottom and covering are ensured as well. In normal operation conditions occurrence of cracks is possible in the following places (stresses in tension zone exceed the limiting value, however compression area of concrete remains in section, that ensures absence of thorough shakes):

- in the places of columns contact with bottom and covering;
- along the perimeter of pit and bottom connection;
- in the area of covering and vertical walls connection.

When designing for combination of NOC + SSE loads apart from the loads, acting under standard conditions, the loads, arising under earthquake action were applied to the system additionally. The calculation was carried out separately for vertical and horizontal directions of seismic input.

When designing the embedded tank for the effects of combination of NOC + SSE loads the following values of stresses, arising in tank structural members had been received:

- in the tank shell from the heaviest pressure side (hydrostatic and hydrodynamic pressure sum) the tensile tangential stresses on internal shell surface were ~ 2 MPa, and on the outside ~ 1.6 MPa;
- in the areas of concentration in the wall at tank bottom and covering, where the influence of local flexural stresses is considerable the tensile tangential stresses were:
    - in the wall near covering: - 2.038 MPa on the shell outer surface, - 7.965 MPa on the internal surface;
    - in the wall near the bottom: - 2.602 MPa on the internal surface and 1.416 MPa - on the outside surface;
- in the tank bottom the heaviest stresses act in the area of pit and bottom connection and in the area between the internal pit edges and the column nearest to it due to considerable bending deformations of the bottom, arising at seismic input and pressure, transferred from the columns to the bottom in their support areas; in this condition maximum stresses in compression area of concrete were ~ 24 MPa, and the stresses in tension area ~ 13.675 MPa;
- two columns which support the tank covering and are located in plane of pressure, react a load in horizontal direction, this load is transferred at motion of covering with soil (two other columns practically do not work), in this condition the maximum stresses (due to compressing stresses from the longitudinal force) were ~ 23 MPa, and tensile stresses ~ 3 MPa;
- in tank cover in the places of its connection with vertical walls and in the areas, contiguous with support parts of the columns, the heaviest stresses arise in tensile area of concrete:
    - near tank walls - 4.8 MPa (outside of the tank);
    - in column support places - 10.2 MPa (outside of the tank).

Analysis of results of tank design for seismic input had allowed to reveal adverse factors, which has caused occurrence of stresses in the tank structural members, the stresses which exceed the allowable values. These factors are as follows:

- large mass of back-fill soil on the tank covering (soil thickness is 1.5 m) and involved considerable inertial loads to the upper section of the walls at seismic input which are not compensated by lateral soil pressure (because of its practical absence in upper section of tank);
- decrease of lateral soil pressure at seismic input (due to its own motion) on the external tank shell surface with simultaneous increase of pressure on the internal surface (hydrostatic pressure is added with hydrodynamic), especially in wall lower part;
- lack of symmetry of stiffness distribution along the tank bottom (because of the pit, which is designed for water intake) and centre-point loads in the places of column support result in localization and rise of flexural stresses in these areas;
- when designing the tank the possibility of occurrence of considerable bending moments in the columns which support the covering was not taken into account, therefore safety margin under standard operation conditions is exhausted entirely under off-design loads.

CONCLUSION

At present the works on seismic recertifications of Russian NPP structures and buildings on the basis of the above procedure are in progress. The results received are used at NPP safety analysis based on the results of seismic revaluation of equipment and pipelines response and state.

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