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BUILDINGS AND STRUCTURES OF CRITICAL INFRASTRUCTURE FACILITIES: NUMERICAL MODELLING AND ANALYSIS OF FLOOR RESPONSE SPECTRA

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Abstract. The paper presents a methodology and an example of determining the floor response spectrum of building structures of nuclear power plants under earthquake loads.

As the critical energy infrastructure facilities are considered, the issues of regulation and scientific support for seismic safety are quite complex. For this class of structures, a comprehensive approach to the study of the system (soil-foundation- structure) is required. The type of foundation and its connection with the base play an important role.

Existing regulations do not always take into account the specific features of the foundation, and the seismic resistance of the overground structure depends on the behavior of the foundation. For analyses of critical infrastructure objects, it is critical to take into account the soil properties and evaluate the seismicity of the territory. The paper presents peculiar features for dynamic analysis of energy infrastructure facilities and the main stages for analysis of floor response spectra. The paper demonstrates examples of design models for buildings and structures of critical infrastructure, namely, structural solutions for generating a model of a reactor. A mathematical model for dynamic analysis is presented. It takes into account both material damping and soil-foundation interaction [5]. The analyses apply the direct method of integrating the equations of motion, including the mathematical equations from which the min value of the damping coefficient is determined [6]. As a method for modeling the interaction between the base and the structure, the method of equivalent dynamic characteristics is adopted, according to which four independent stages are proposed for determining the design seismic impact; determining the coefficients of mass proportionality α and stiffness β; solving the problem of forced vibrations obtained from linear non-conservative oscillators, followed by obtaining floor response spectra.

Keywords: floor response spectrum, dynamic loads, earthquake, seismograms, damping, max design earthquake, buildings and structures of nuclear power plants.

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CALCULATION FEATURES OF ENERGY STRUCTURES FOR DYNAMIC INFLUENCES

The current regulatory standards, specifically DBN B.1.1.-12:2014 [1], permit the use of a variety of techniques for dynamic analysis of structures. They can be conditionally categorized into three groups: various methods based on spectral analysis; an approximate method for taking into account the physical nonlinearity of structures (Pushover analysis); and analysis by direct integration of the equations of motion (Time History Analysis). Additionally, current regulatory documents assume that the seismic acceleration of foundations (and the entire structure) and the base coincide. Experimental evidence, however, indicates that the acceleration of soils and foundations can vary by a number of times.

This can be explained by the fact that not all of the seismic energy is transferred from the soil to the foundation.

Due to the peculiarities of the connections between the foundation and the base, some part of the disturbance is transmitted. This can occur for several reasons:

- due to the damping effect (natural or engineered artificial nature) of the connections between the foundation and the base (including through seismic isolation);
- due to the "slippage" of a horizontal seismic wave under the foundation (overcoming friction forces and the specifics of one-sided connections between the foundation and the base);
- due to the variation in stiffness and mass values in the building models (high-rise and stylobate parts) [13].

In an earthquake analysis, various techniques for computer modelling are used to take into account the different stiffnesses of the high-rise and stylobate parts of the building. To determine the floor response spectra of building structures of nuclear power plants, we consider the analysis by direct integration of the equations of motion.

The analysis of floor acceleration response spectra (FRS) [8]consists of the following main steps:

- modelling the interaction between the foundation and the structure using the method of equivalent dynamic parameters [11];
- obtaining nodal accelerograms (NA) as a result of computing the forced vibrations of the building due to the action of the accelerograms;
- obtaining the response spectra of n-number of nodes as a result of computing the forced vibrations of linear non-conservative oscillators under the action of nodal accelerograms;
- obtaining the envelope of the floor response spectrum from the obtained nodal response spectra.

The influence of the foundation on seismic vibrations of a building has several aspects:

- the foundation transmits the earthquake load to the structure; the structure, due to its massiveness and stiffness, has a reverse effect on the movement of soil, so the law of seismic vibrations under the foundation slab differs from the "free field" vibrations;
- the soil base has its own mass and stiffness that reduce the free vibration frequencies of the dynamic system "structure - foundation";
- during an earthquake, seismic waves reflect off the foundation and dissipate on the base, thereby absorbing a certain amount of energy.

The influence of the foundation on the dynamic response of the building depends on the ratio of stiffnesses. This influence is negligible for flexible structures, and it is often assumed that the foundation is undeformed.

Even in relatively stiff soils, the influence of the foundation is significant on massive, rigid structures, such as the majority of NPP buildings.

According to the world practice of taking into account the interaction of the structure with the foundation, there are 3 primary aspects of influence that can be taken into account when determining the response spectra of a structure:

- considering the stiffness of the soil-foundation system, which is implemented

taking into account the influence of the base according to the recommendations of clause 6.4.13 of the DBN [1];
- filtering the soil vibrations, taking into account kinematic effects in accordance with recommendations [10], [11];
- removing the energy from the soil-structure system by radiation of elastic waves reflected from the structure and hysteresis losses in accordance with ASCE recommendations [10], [11].

MODELLING DESIGN MODELS OF CRITICAL BUILDINGS AND STRUCTURES

For high-risk structures (responsibility class CC3) and for residential and public buildings with a height from 73.5 m to 100 m (responsibility class CC3), it is important to know what technical condition the load-bearing structures and structural elements of the building will be in under a specified dynamic or earthquake load [9]. Since building structures can be quite complex and the number of installed equipment is quite significant, it might be challenging to simulate the design model of a building or structure.

In accordance with the technical documentation, 3D design models are generated to perform finite element analysis using software systems.

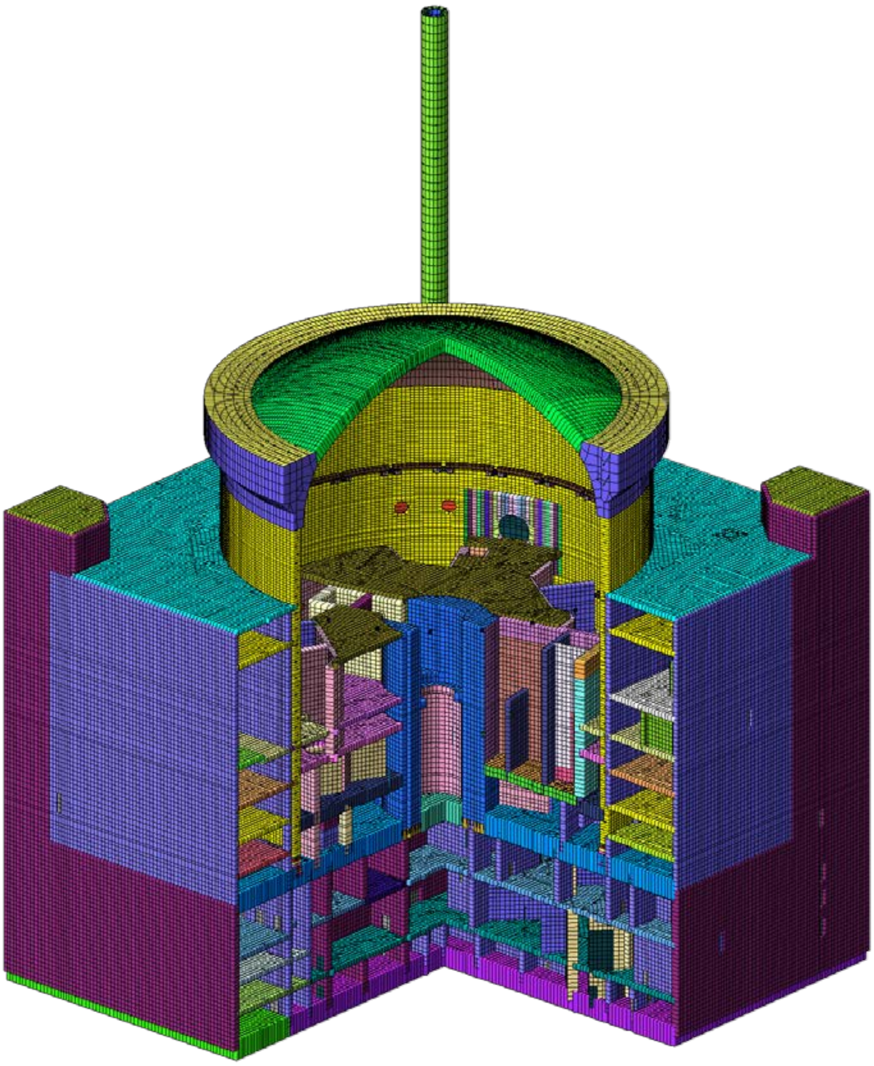

Fig.1. Design model of the reactor (section)
Рис.1. Комп’ютерна розрахункова модель реактора (розріз)

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DESIGN SOLUTIONS FOR THE REACTOR

The foundation part of the reactor is a rigid reinforced concrete box structure with dimensions of 66.00x66.00m in plan and a height of 17.4m from the -6,600 (bottom of the foundation (lower foundation slab)) to the +10,800 (top of the foundation (upper foundation slab)).

The bottom slab of the foundation is designed as a reinforced concrete monolithic slab with thickness 2400 mm and dimensions 68.18 x 68.18 m in plan.

The walls of the foundation are made of precast concrete in the form of wall cells with fixed formwork made of flat reinforced concrete slabs. The thickness of the external walls of the foundation is 900 mm. The internal walls have thickness 600 and 900 mm.

The floor slabs of the foundation are made of precast monolithic slabs with ribs installed with the edges upwards or flat slabs with subsequent monolithic reinforcement frames installed on top of the slabs. The slab thickness is 600 mm; in some sections – 740 mm.

The upper foundation slab from elevation 10.800 to elevation 13.200 is the support part for the containment and the reactor building. A 2.400 mm thick RC monolithic slab covers the foundation part.

The construction of the reactor compartment is placed around the protective shell on a common baseplate. The structural layout of the construction of the reactor compartment, which is cut off from the protective sealed shell by an anti-seismic joint, is a multi-storey box structure with horizontal seismic and shock wave loads transferred through the floor slab discs to the wall structures. Together with the floor slab discs, the internal walls are also subjected to horizontal forces.

BOUNDARY CONDITIONS AND METHODOLOGY FOR ANALYSIS OF THE FLOOR RESPONSE SPECTRA

The analysis was performed by direct integration of the equations of motion (direct dynamic method). The term "direct dynamic method" means that no equation is transformed before integration.

The dynamic analysis is based on solving a system of differential equations:

$$[M]\ddot{x}(t)+[C]\dot{x}(t)+[K]x(t)=\overline{q}(t) \qquad (1)$$

where

[M], [C] та [K] – the mass, damping and stiffness matrices of the system;

$\ddot{x}(t), \dot{x}(t), x(t)$ – the vectors of nodal accelerations, velocities and displacements at a certain time;

$\overline{q}(t)$ – the load c orresponding to time t.

The earthquake load (time history analysis) is applied as seismograms (time dependence of displacement).

The Newmark method [13] is used to solve the differential equations of motion for the corresponding nodes of the structure:

$$A=\frac{1}{\alpha\Delta t^2}+\frac{2\xi\omega}{\gamma\Delta t}+\omega^2\,; \qquad (5)$$

$$B_{i+1}=F(t_{i+1})+\left(\frac{1}{\alpha\Delta t^2}y_i+\frac{1}{\alpha\Delta t}\dot{y}_i+\left(\frac{1}{2\alpha}-1\right)\ddot{y}_i\right)+2\xi\omega\left(\frac{1}{\gamma\Delta}y_i+\left(\frac{1}{\gamma}-1\right)\dot{y}_i+\left(\frac{1}{2\gamma}-1\right)\Delta t\ddot{y}_i\right); \qquad (6)$$

$$y_{i+1}=B_{i+1}/A \qquad (7)$$

$$\dot{y}_{i+1}=\frac{1}{\gamma\Delta t}(y_{i+1}-y_i)+\left(1-\frac{1}{\gamma}\right)\dot{y}_i+\left(1-\frac{1}{2\gamma}\right)\Delta t\ddot{y}_i \qquad (8)$$

$$\ddot{y}_{i+1}=\frac{1}{\alpha\Delta t^2}(y_{i+1}-y_i)-\frac{1}{\alpha\Delta t}\dot{y}_i+\left(1-\frac{1}{2\alpha}\right)\ddot{y}_i.$$

In time history analysis, the Rayleigh dissipation matrix is used to consider the damping by setting the coefficients for mass proportionality $\boldsymbol{\alpha}$ and for stiffness $\boldsymbol{\beta}$ (Figure 2); these coefficients are calculated for the first two frequencies with a significant contribution to mass collection.

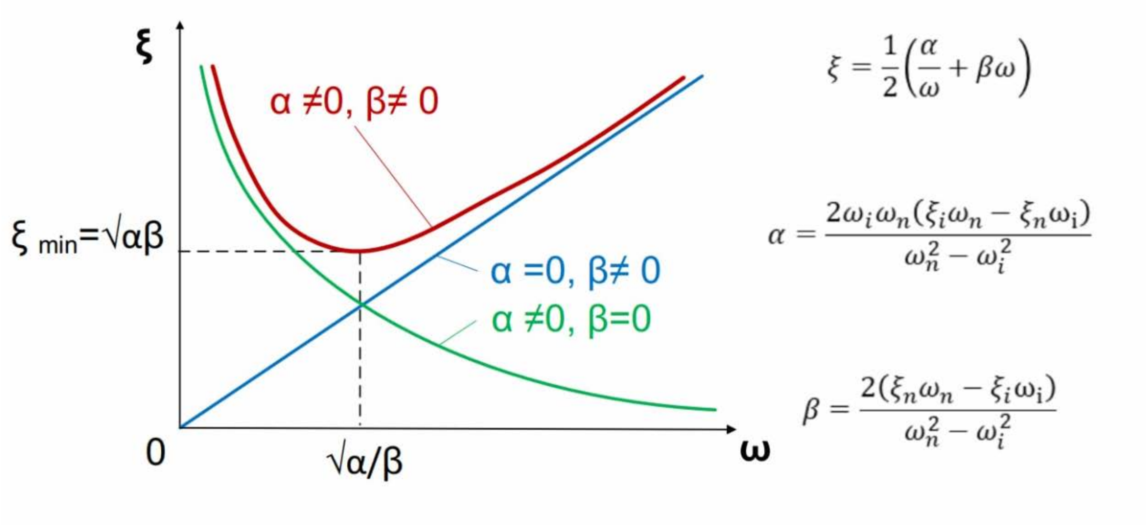

Fig.2. Dependence of damping on frequency when using the Rayleigh damping proportionality matrix
Рис.2. Залежність демпфування від частоти при використанні матриці пропорційності Релея

The damping coefficients due to the radiation of elastic waves reflected from the structure are calculated for horizontal, vertical displacements and rotation relative to the horizontal and vertical axes:

$$\xi_x = 0.5b_x(m\text{К}_x)^{-1/2}; \quad (9)$$

$$\xi_z = 0.5b_z(m\text{К}_z)^{-1/2}; \quad (10)$$

$$\xi_\varphi = 0.5b_\varphi(m\text{К}_\varphi)^{-1/2}; \quad (11)$$

$$\xi_\psi = 0.5b_\psi(m\text{К}_\psi)^{-1/2}; \quad (12)$$

The above-mentioned formulas determine the min damping coefficient through the direct method of integrating the equations of motion. The max values are standardized in global practice because the resulting damping might have considerable values (0.2÷0.4). For example, in Germany, $\xi_z \leq 0{,}35$; $\xi_x \leq 0{,}15$; $\xi_\varphi \leq 0{,}15$ are accepted.

The damping coefficients as a percentage of the critical one are taken in accordance with Table 3.3 [4]:

- for structures with a stress level greater than or equal to the yield strength, 10% for reinforced concrete structures and 7% for steel structures are assumed; the decrements of vibrations will be 0.63 for reinforced concrete structures and 0.44 for steel structures;
- for structures with a stress level less than the yield strength, 7% for reinforced concrete structures and 5% for steel structures are assumed; the decrements of vibrations will be 0.44 for reinforced concrete structures and 0.31 for steel structures.

The method of equivalent dynamic parameters is adopted as a method for modelling the interaction between the foundation and the structure. This method splits the problem of interaction between the foundation and the structure and response spectrum determination into the four independently solved problems listed below:

1) Determination of the design seismic impact on the foundation based on its shape, soil base structure, and the directions of seismic wave incidence.
2) To determine coefficients for mass proportionality α and for stiffness β; to

perform modal analysis of a structure.
3) Solving the problem of forced vibrations of a building under initial impact and foundation characteristics[18].
4) Research on forced vibrations of linear non-conservative oscillators with obtaining floor response spectra.

At the first stage, the earthquake loads (time history analysis) are determined, namely, a seismogram is applied at each node of the foundation slab in three directions. By adjusting the base line using a cubic polynomial method, the first accelerograms (Figure 3) are converted into seismograms in three directions for the level motions SL-1 and SL-2 [5]. These accelerograms either correspond to the seismic monitoring data (for buildings of nuclear power units) or to the accepted synthesized accelerograms according to the DBN [1,17] (when there is no monitoring data).

An example of such a seismogram in one direction is shown in Figure 4.

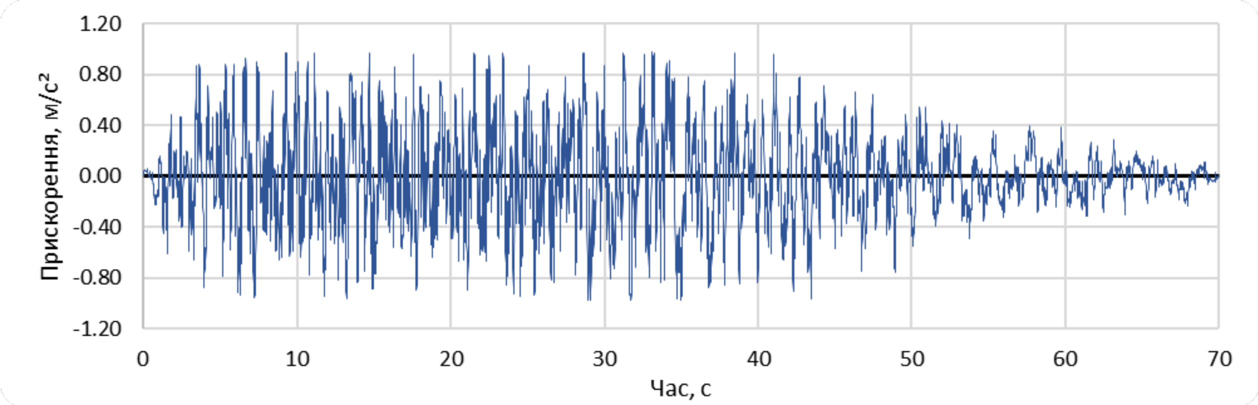

Fig.3. The initial accelerogram of the high-frequency spectrum of one direction (horizontal impact) with intensity SL-2 PGA = 0.10g

Рис.3. Початкова акселерограма високочастотного спектру одного напрямку (горизонтальний удар) з інтенсивністю SL-2 PGA = 0,10g.

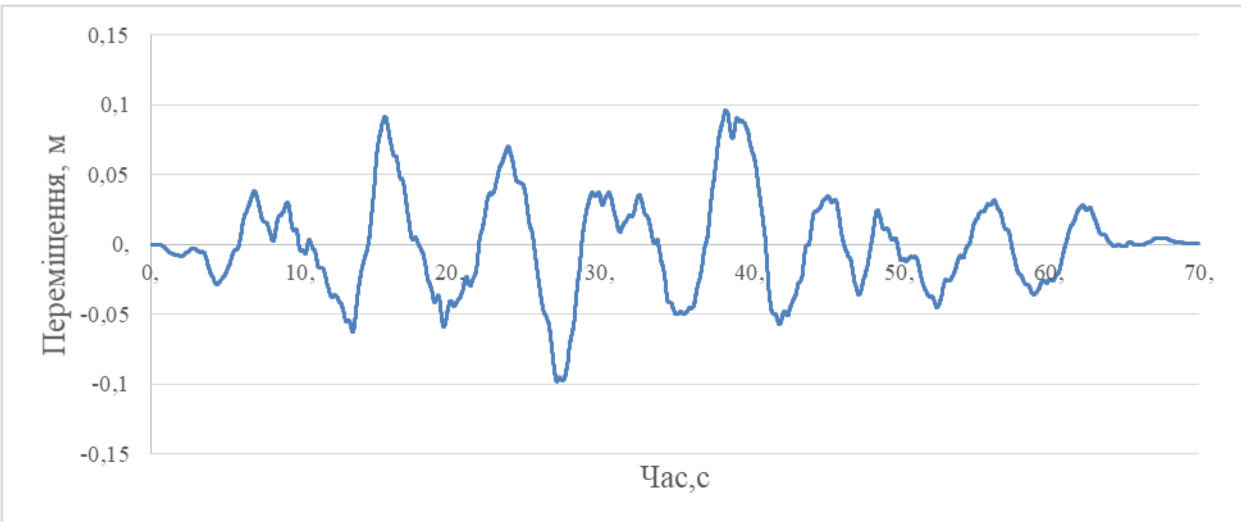

Fig. 4. Seismogram for one direction of the level motions SL-2 PGA= 0.10g

Рис. 4. Сейсмограмма для одного напрямку руху рівня SL-2 PGA= 0.10g

The seismogram can be obtained using the "Accelerogram Editor" program in SCAD or using the ReSpectrum module in the LIRA-FEM [7] software. The ReSpectrum module is mentioned to generate the response spectra of a single-mass oscillator from dynamic loads specified by accelerograms, seismograms, velocigrams and three-component accelerograms, as well as to mutually transform these effects.

At the second stage, a modal analysis of the structure [16] is performed to determine the first mode shapes for vibration of the building and further determine the coefficients of mass proportionality α and stiffness β.

The modal analysis is performed for the model without taking into account the soil behaviour (for the analysis of the building's natural frequencies), the number of active masses is at least 85% of all masses, which meets the requirements [1].

The calculations at the third stage are performed for all calculated nodes of the building (characteristic points of equipment installation) and all design seismograms.

Determination of equivalent dynamic characteristics of the base - a set of springs and dampers attached to the foundation slab, characterizing the stiffness and energy dissipation in the base. In general, there are twelve of them: six springs that define stiffness during translational and angular displacements of the foundation along three axes and six corresponding dampers. The parametric values of the stiffness of the springs and dampers are found by solving the problem of stamp oscillations on an elastic homogeneous or layered base, semi-infinite or underlain by rock.

The attached mass of the base is not taken into account, since for massive and rigid structures (NPP), the influence of this mass on the result is within the range of scatter due to the inaccuracy of the initial data.

The mass matrix is generated on the basis of the design combinations of loads with account of all loads according to [1,2,3,4]. After the mass matrix is generated, the earthquake loads are applied as the seismograms of stage 1.

As a result of the third stage, a set of nodal accelerograms should be obtained for each design point of the building.

The analysis of the floor response spectra is performed at the fourth stage. The forced vibrations of linear non-conservative oscillators are studied. The equation that describes the relative displacements of the mass in the coordinate system associated with the oscillator base is as follows:

$$\ddot{x} + 2\zeta\omega\dot{x} + \omega^2 x = -\ddot{x}_0(t) \qquad (13)$$

where

x – the relative displacement of the mass;

ζ – the damping coefficient;

ω – the natural circular frequency of the oscillator without damping;

$\ddot{x}_0(t)$ – acceleration of the oscillator base (nodal accelerogram).

The solution to equation (13) – the mass displacement function is expressed by the Duhamel integral that is calculated at successive time points t according to the formula (14).

$$x = -\frac{1}{\omega_D}\int_0^t \ddot{x}(\tau)e^{-\zeta\omega(t-\tau)}\sin\omega_D(t-\tau)d\tau\,, \qquad (14)$$

where

ω_D – the natural circular frequency of the oscillator with account of the damping, which is equal to:

$$\omega_D = \omega\sqrt{1-\zeta^2} \qquad (15)$$

The floor response spectra (FS) are calculated for the natural frequencies of the oscillator from the set given below (Table 1).

The set meets the requirements indicated in clause 6.4.12 of DBN [1]. The values of the frequencies specified to account for resonance are added to the building's computed natural frequencies.

Table 1. The set of natural frequencies of the oscillator

Табл.1. Сукупність власних частот осцилятора

Frequency *f*, Hz			Increase *Δ f*, Hz
1	**2**	**3**	**4**
0,2	-	3,0	0,10

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Table 1. (continued)
Продовження табл. 1.

1	2	3	4
3,0	-	3,6	0,15
3,6	-	5,0	0,20
5,0	-	8,0	0,25
8,0	-	15,0	0,50
15,0	-	18,0	1,00
18,0	-	22,0	2,00
22,0	-	34,0	3,00

At the fourth stage, the floor response spectra (FS) are calculated for two orthogonal variants of the horizontal seismogram direction and for n number of characteristic points at each elevation, resulting in a set of nodal accelerograms. For each FS, all oscillator frequencies specified above are considered (Table 1). At the same time, for each calculated frequency of the oscillator, the solution to equation (13) over the entire range of the accelerogram is the function of mass acceleration. Then, the max absolute value of the acceleration is selected. This value is the ordinate of the nodal response spectrum; the ordinate corresponds to the calculated frequency of the oscillator. As a result, the solution for all calculated frequencies of the oscillator is the nodal response spectrum of this nodal accelerogram at the given calculation point of the building.

The resulting floor response spectrum is determined by generating an envelope; i.e., at each calculated frequency, the max acceleration of all selected nodal response spectra is determined. The calculations at the second and third stages can be performed in the LIRA-FEM software [14], which implements the above method.

Uncertainties associated with the scatter of values of mechanical parameters of materials [12] and dynamic parameters of the building lead to errors in the analysis results. To compensate for the errors, the theoretical floor response spectra (FS) are processed. In the course of processing, the FSs are smoothed and their peaks are widened. The FS are processed according to the rules used in the design of nuclear power plants in the United States and indicated in [4], [8].

The peaks are expanded as follows: the peak is transformed into a 0.3fi wide area, with the centre of the area as the peak point. The peak accelerations are reduced by 15% [15].

The peak expansion is performed for damping values of 0.5%, 2%, 3%, 5%, 7%.

In combination with the expansion of the response spectrum peak, a 15% reduction in the narrow frequency peak amplitude is acceptable if the damping of the subsystem is less than 10% [11].

This 15% reduction applies only to narrow frequency peaks in the unexpanded response spectrum with the ratio of the frequency difference to the centre frequency, B, of less than 0.30:

$$B = \frac{\Delta f_{0.8}}{f_c} < 0{,}30 \qquad (16)$$

where:

$\Delta f_{0.8}$ - total frequency range with spectral amplitudes exceeding 80% of the peak spectral amplitude;

f_c - centre frequency for frequencies that exceed 80% of the peak amplitude.

The characteristics of the design floor response spectra (FS) are determined as a result of processing the theoretical floor response spectra (FS).

CONCLUSION

Due to the methodology applied for analysis and the functional capabilities of the LIRA-FEM program, nodal accelerograms were obtained for all nodes of the reactor model. Figures 5 and 6 show the values of max accelerations of the nodal accelerograms of the model fragments.

In the ReSpectrum module of the LIRA-FEM program, the obtained nodal accelerograms are processed according to the methodology described in the fourth stage of analysis. Figure 7 shows the floor response spectra along the X-axis at different damping values in the range from 0 to 34 Hz

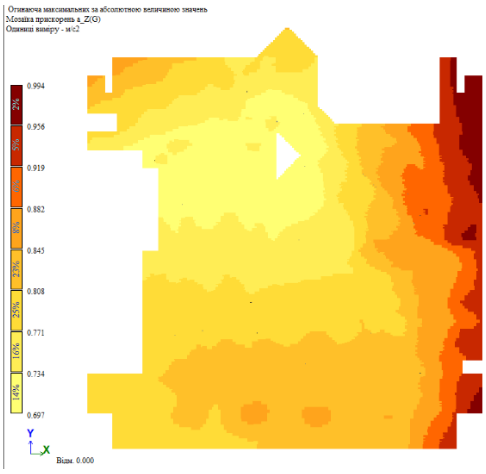

Fig. 5. Contour plots for max nodal accelerations of nodal accelerograms along the Z-axis

Рис.5. Контурні графіки (мозаїки) для максимальних вузлових прискорень вузлових акселерограм вздовж осі Z

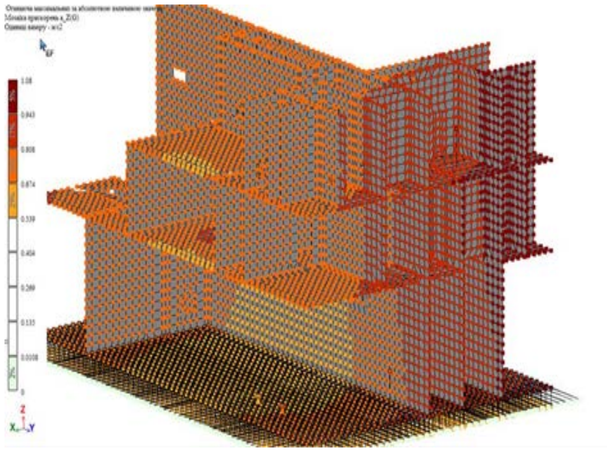

Fig.6. Contour plots for max nodal accelerations of accelerograms in the interior walls of the reactor along the Z-axis

Рис.6. Контурні графіки максимальних вузлових прискорень акселерограм у внутрішніх стінках реактора по осі Z

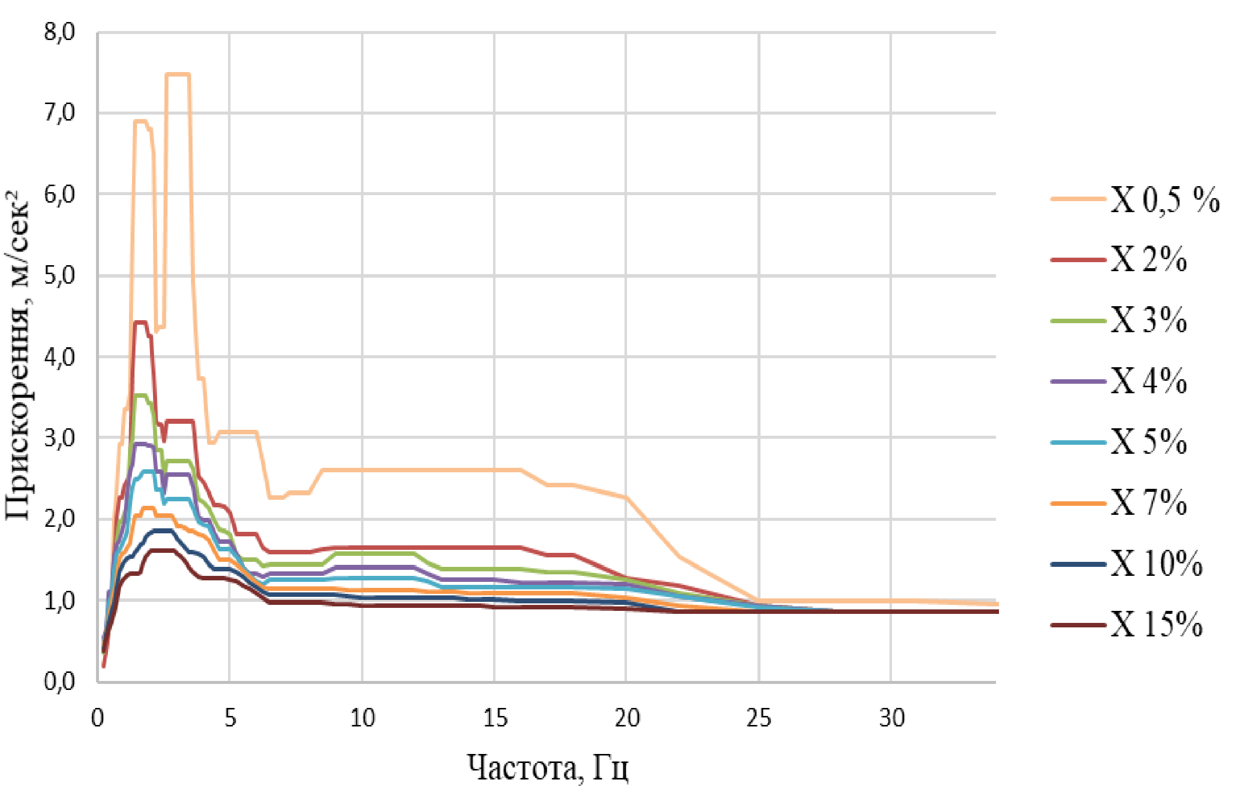

Fig. 7. Floor response spectra at different values of damping along the X-axis
Рис. 7. Поверхові спектри-відгуку при різних значеннях демпфування по осі X

In the future, when the seismic resistance of equipment is evaluated by the test method, the analysis method or the method of operational experience, it is possible to use the obtained floor response spectra depending on their location in the reactor.

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БУДІВЛІ ТА СПОРУДИ ОБ'ЄКТІВ КРИТИЧНОЇ ІНФРАСТРУКТУРИ: ЧИСЕЛЬНЕ МОДЕЛЮВАННЯ ТА АНАЛІЗ СПЕКТРІВ РЕАКЦІЇ ПЕРЕКРИТТЬ

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У статті наведено методику та приклад визначення спектра відгуку перекриття будівельних конструкцій атомних електростанцій при сейсмічних навантаженнях.

Оскільки розглядаються об'єкти критичної енергетичної інфраструктури, питання регулювання та наукового забезпечення сейсмобезпеки є досить складними. Для цього класу споруд необхідний комплексний підхід до дослідження системи "ґрунт-фундамент-

конструкція". Тип фундаменту та його зв'язок із основою відіграють важливу роль.

Існуючі нормативні документи не завжди враховують специфічні особливості фундаменту, при цьому сейсмоміцність надземної конструкції залежить від поведінки фундаменту. Для аналізу об'єктів критичної інфраструктури надзвичайно важливо враховувати властивості ґрунту та оцінювати сейсмічність території. У статті представлені особливості динамічного аналізу об'єктів енергетичної інфрастурк-тури та основні етапи аналізу спектрів відгуку перекриттів.

У статті наведені приклади розрахун-кових моделей будівель і споруд критичної інфраструктури, зокрема конструктивні рішення для створення моделі реактора. Представлено математичну модель для динамічного аналізу, яка враховує як матеріальне демпфування, так і взаємодію "ґрунт-фундамент" [5]. У аналізах викори-стовується прямий метод інтегрування рівнянь руху, включаючи математичні рівняння, з яких визначається мінімальне значення коефіцієнта демпфування [6]. Як метод моделювання взаємодії між основою та конструкцією прийнято метод еквівалентних динамічних характеристик, згідно з яким запропоновано чотири незалежні етапи для визначення розрахункового сейсмічного впливу; визначення коефіцієнтів пропорційності маси α та жорсткості β; розв'язання задачі вимушених коливань, отриманих від лінійних неконсервативних осциляторів, з подальшим отриманням спектрів відгуку перекриттів.

Ключові слова: спектр реакції перекриттів, динамічні навантаження, землетрус, сейсмограми, демпфування, максимальне розрахункове землетрусне навантаження, будівлі та споруди атомних електростанцій, метод скінченних елементів

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