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CONSIDERATION OF THE SEISMIC IMPACT'S WAVE NATURE IN THE DESIGN JUSTIFICATION OF HIGH-RISE BUILDINGS' SEISMIC RESISTANCE

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Abstract: This paper presents the seismic resistance calculation results for a high-rise building that considers the wave nature of the impact, its heterogeneity and irregularity in the structural plan, performed in SCAD Office. The analysis of normative requirements to consider the wave action and existing calculation methodologies is carried out. Recommendations are given on possible consideration of the seismic impact's irregularity using the direct dynamic method to solve the MDE-level seismic impact on the structures.

Keywords: seismic impact's wave nature, rotational components of seismic motion, linear-spectral method, direct dynamic method, SCAD Office, seismic resistance

УЧЕТ ВОЛНОВОГО ХАРАКТЕРА СЕЙСМИЧЕСКОГО ВОЗДЕЙСТВИЯ ПРИ РАСЧЕТНОМ ОБОСНОВАНИИ СЕЙСМОСТОЙКОСТИ ВЫСОТНЫХ ЗДАНИЙ

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Аннотация: В настоящей статье представлены результаты расчета на сейсмостойкость высотного здания с учетом волнового характера воздействия и его неоднородности и неравномерности в плане сооружения, в программном комплексе SCAD Office. Проведен анализ нормативных требований в части учета волнового воздействия и существующих методик расчета. Даны рекомендации по возможному учету неравномерности сейсмического воздействия при расчете сооружений прямым динамическим методом на сейсмическое воздействие уровня МРЗ (КЗ).

Ключевые слова: Волновой характер сейсмического воздействия, ротационные составляющие сейсмического движения, линейно-спектральный метод, прямой динамический метод, ПК SCAD Office, сейсмостойкость

1. INTRODUCTION

Paragraph 6.1.4 of SP 14.13330.2018 [1] demands for the buildings with reinforced concrete framing that, if the design seismicity of the construction site is or above 7-8 points, the distances between aseismic joints should not exceed 80 m. If this condition cannot be satisfied, *analysis of structures should consider the seismic impact's wave nature, its inhomogeneity and irregularity in the structural plan according to the established methodologies*. This requirement first appeared in Amendment 1 to SP 14.13330.2018 and simply stated the need consider the seismic impact's wave nature without describing any approaches to carry out such calculations or referring to any methodologies.

To date, there are still no consistent approaches or methodologies accepted by the engineering community for earthquake-resistant buildings and structures analysis that consider wave-like spatial and temporal seismic effects that can be applied in engineering design practice, as well as adopted methodological recommendations to SP 14.13330.2018.

A number of works by Yu. P. Nazarov on the application of the linear-spectral method to solve these issues [2, 3, 4, 5, 6] formed the basis of the manual [7]. It presents:

– an engineering methodology for determining the prevailing wavelengths and selecting a computational model of seismic ground motion;

– a methodology for calculation of rotational components for the integral dilatational-rotational model of seismic ground motion;

– a modified linear-spectral method for calculation of earthquake-resistant buildings and structures for the integral dilatation-rotation model of seismic ground motion.

In the EU seismic standards [8] there is also a requirement to take into account rotational components of seismic ground motion for high-rise structures in regions with high seismicity, but it is only applicable for such structures (towers, masts, chimneys, etc.) which utilise the obsolete cantilever dynamic model.

It should be noted that all the described methodologies are applicable to the analysis of buildings using the linear-spectral theory. The most widespread software packages in the Russian construction industry (SCAD Office, LIRA-SAPR, LIRA 10) currently lack convenient tools for their application in the calculation practice. Which, in fact, makes it practically impossible to deviate from the normative requirements in terms of distances between aseismic joints.

SP 14.13330.2018 demands that analysis of buildings for seismic effects of control earthquake-level (CE), or, per custom, maximum design earthquake-level (MDE), should be performed using the direct dynamic method. In practice, rather complicatedly implemented approaches are used for the design justification of structures' seismic resistance in terms of wave action [9]. Few works consider the seismic impact's wave nature in the direct dynamic method excluding the foundation. It's primarily due to the complexity of the impact's character at the "foundation-structure" boundary, which should account for the velocity of the seismic wave propagation, depending on the type of soil [10].

This paper describes a simplified method to consider the seismic impact's wave nature in the analysis of a building using the direct dynamic method in SCAD Office. This approach can be used in addition to the aforementioned methodologies when calculating the seismic resistance of high-rise buildings.

2. PROBLEM STATEMENT

The building considered in this article is a 99-meter-tall multifunctional hotel complex with *XY*-dimensions of 104×87 m and an underground 3-storey car parking (*Figure 1*).

The frame-wall structural system with a load-bearing core is adopted in this project. The foundation is a continuous monolithic 1.8-meter-thick slab on a natural bed. The frame is formed by a system of vertical elements (columns, pylons, walls and a load-bearing core, which consists of stairwells and lift shafts) and horizontal floor slabs. All joints of the frame elements (walls, diaphragms, columns, beams, floor slabs) are connected using fixed supports, which ensures the continuity of the frame elements' systems. Spatial rigidity is ensured by the joint work of load-bearing structures of slabs, reinforced concrete columns, pylons and the vertical load-bearing core, which are all connected to the foundation slab using fixed supports.

All vertical structures are made of B40 concrete, all horizontal load-bearing structures are made of B30 concrete.

Seismicity of the construction site, according to the geophysical surveys, is 8 points. The soils belong to the II (second) category of seismic properties. A three-component accelerogram was presented for the initial impact (as per the geophysical surveys), which was multiplied by the coefficients K_0 =1.3, K_1 =1.0 and K_ψ =1.0 before being used in the calculation. Due to the fact that in SCAD Office's direct dynamic method the seismic impact is specified in the form of displacements [11], the corresponding seismograms were obtained from the initial accelerograms.

For dynamic calculations, a finite-element (FE) model was developed in SCAD Office. The structures of the building (reinforced concrete foundation slab, floor slabs, walls, cover disc) were modelled by shell 44 FE. The beam elements (columns) were modelled by a spatial beam FE, which implements the Timoshenko beam theory. The elements are connected using fixed supports. Computational dimensionality of the model is 822 295 nodes (4 588 118 degrees of freedom) and 894 287 FE.

As part of the considered building's seismic resistance design justification, calculations for the seismic impact of a design earthquake-level (DE) or, per custom, a project earthquake-level (PE), were carried out using the linear-spectral method with the response spectra obtained from synthesised three-component accelerograms (with 5% damping). From this calculation's results, the reinforcement of elements was chosen, and the seismic impact of the MDE-level was calculated using the direct dynamic method. The implementation of a simplified method to consider the impact's wave nature in SCAD is presented below. A comparison of the results of the platform scheme (when the same impact is applied to all nodes of the foundation simultaneously) and the designed wave scheme is also presented.

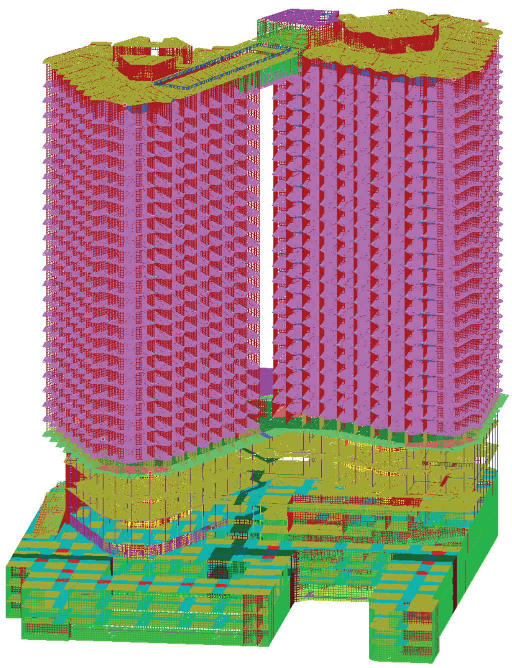


Figure 1. General view of the calculation model of a high-rise building

3. METHODOLOGY FOR THE SEISMIC IMPACT WAVE NATURE'S CONSIDERATION

1) The calculation of dynamic stress-strain state of load-bearing structures under special combinations of loads and influences (MDE-level) is considered. The dynamic calculation of an impact represented by accelerograms of ground vibrations at the base of the structure is done by numerical integration of the motion equations.

2) The dynamic characteristics of the foundation obtained from geophysical survey data are used.

3) The wave-induced seismic loading is represented by a discrete set of loads, which are represented as "bands" in the structural plan. The width of these bands is determined by the geophysical survey data regarding the transverse wave velocity in the foundation bed. In this case:

– transverse wave velocity $V = 760$ m/s;

– integration step: $h_{st} = 0.01$ s;

– strip width: $\Delta l = V \times h_{st} = 760 \times 0.01 = 7.6$ m (*Figure 4*).

4) Loads for the horizontal and vertical components of the seismic load are specified separately as single displacements along the global axes of the model. The displacements are applied to the nodes of one-way tie elements (type 352, *Figure 2*). Horizontal displacements are applied to the top tie node, and vertical displacements to the bottom tie node. It is possible to correct the direction of horizontal loads using cosine guides.

5) The impact's wave nature is considered by applying the load to different bands with a time offset, which corresponds to the chosen integration step (so they are included in the oscillation process one by one).

The integration step (at known wave propagation velocity) determines the width of the band to which the seismic load is applied.

6) Each band should be assigned both a horizontal and a vertical seismic load component.

7) In order to account for the most dangerous direction of seismic action, the bands should be oriented perpendicular to the direction of the first mode of the system in the structural plan.

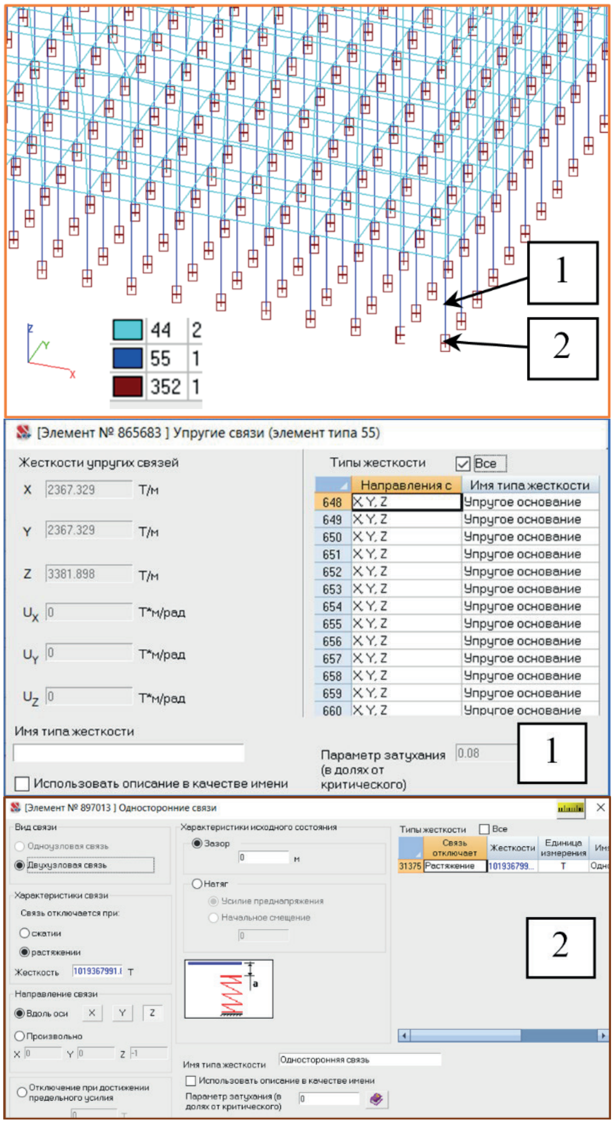


Figure 2. Structure and parameters of a dynamic elastic foundation

The three-component seismogram used is shown in *Figure 3* (seismogram along *Z* axis is given in green, *X*-axis in red and *Y*-axis in blue).

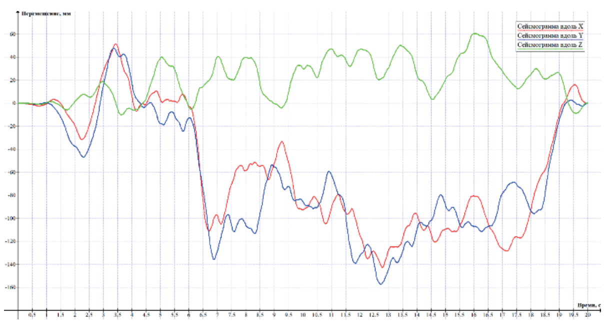


Figure 3. Three-component seismogram. Time step 0.01 s. Range 0-20 s

The most dangerous direction of horizontal seismic impact, determined from the analysis of the eigenfrequencies and modes, is shown in the *Figure 4*. The time offset of the bands is 0.01 s (corresponding to the integration step). Investigation of influence of the integration step (and, consequently, the width of the "bands") on the final dynamic response of the building has not been carried out in this study, but is planned for the future. The direction of the first mode of the system is shown in purple.

Thus, the *Y* component is selected for the horizontal component and the *Z* component for the vertical component.

4. RESULTS

Following the results of the performed dynamic calculations for two schemes (wave and platform), considering the physical nonlinearity, the displacements at the nodes with maximum amplitudes in the nonlinear analysis (486700, 466484, 223778, 486540, 487389), as well as the transient changes of internal forces for a number of finite elements (FE 830115, FE 819373) are compared (*Figure 5*).

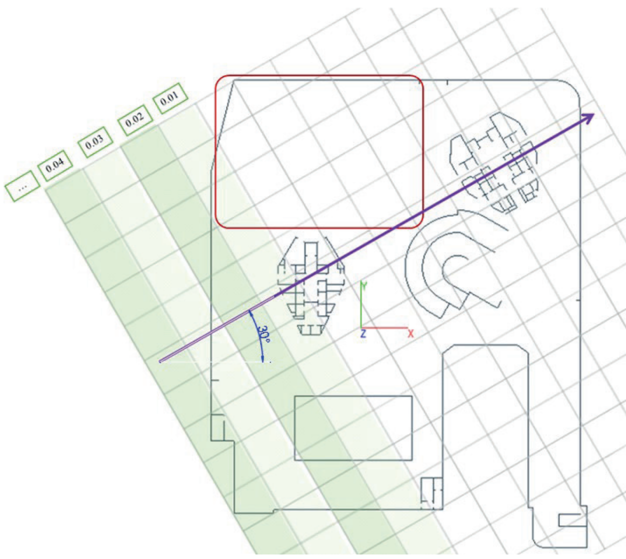


Figure 4. Configuration of the "bands" (t = 7.6 s) and their orientation in plan

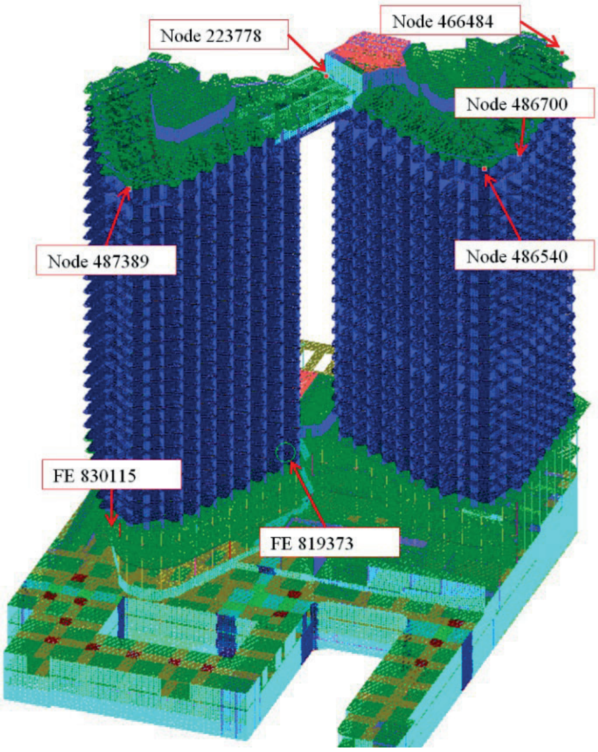


Figure 5. *Locations of nodes/elements for the post-processing*

Figure 6 shows a plot of the linear horizontal displacements of node 466484 for the two schemes, and *Table 1* summarises the results for all nodes and shows the relative differences in values.

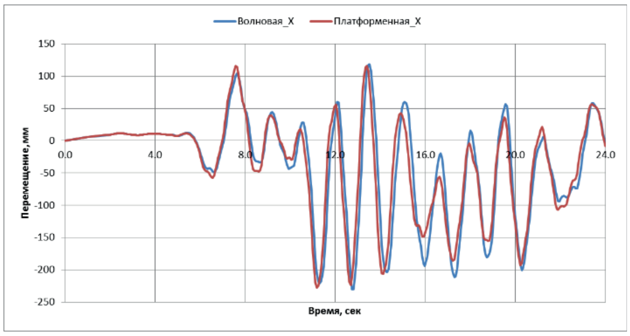


Figure 6. *Nonlinear analysis. Comparison of absolute displacements UX for the platform and wave scheme for the node 466484*

Table 1. Comparison of peak displacements in the selected nodes

Node No.	*Min/max*	*Platform scheme. Displacements, mm*			*"Wave" scheme. Displacements, mm*			*Relative difference, %*		
		UX	*UY*	*UZ*	*UX*	*UY*	*UZ*	*UX*	*UY*	*UZ*
486700	*min*	*-367.3*	*-345.6*	*-49.9*	*-338.6*	*-337.2*	*-51.4*	*7.81*	*2.43*	*-3.01*
	max	*211.8*	*173.1*	*46.3*	*223.9*	*185.3*	*47.9*	*-5.71*	*-7.05*	*-3.46*
466484	*min*	*-227.5*	*-300.2*	*-46.5*	*-230.1*	*-290.6*	*-46.1*	*-1.14*	*3.20*	*0.86*
	max	*115.7*	*152.5*	*58.2*	*117.8*	*164*	*57.7*	*-1.82*	*-7.54*	*0.86*
223778	*min*	*-207.9*	*-206.8*	*-61.3*	*-215.6*	*-223.1*	*-57.9*	*-3.70*	*-7.88*	*5.55*
	max	*145.7*	*86.8*	*63.8*	*157.5*	*100.3*	*68.6*	*-8.10*	*-15.55*	*-7.52*
486540	*min*	*-425.6*	*-311.6*	*-58.5*	*-398.6*	*-302.3*	*-59.6*	*6.34*	*2.98*	*-1.88*
	max	*241.6*	*156.9*	*31.4*	*256.38*	*168.7*	*32.6*	*-6.12*	*-7.52*	*-3.82*
487389	*min*	*-385*	*-181.4*	*-67*	*-386.6*	*-206.4*	*-68*	*-0.42*	*-13.78*	*-1.49*
	max	*215.7*	*51.1*	*35.1*	*209.2*	*75.6*	*35.4*	*3.01*	*-47.95*	*-0.85*

+	*The platform scheme values are bigger than the wave scheme values.*
-	*The wave scheme values are bigger than the platform scheme values.*

The forces in element 819373 (beam 60*80 at +11.700) for the platform scheme are shown in *Figure 7* and *Figure 8*. The normal stresses in the beam section at the moment of time (12.86 s) are given in *Figure 9*.

The forces in same element 819373 for the "wave" scheme are shown in *Figure 10* and *Figure 11*. The normal stresses in the beam section at the moment of time (12.98 s) are given in *Figure 12*.

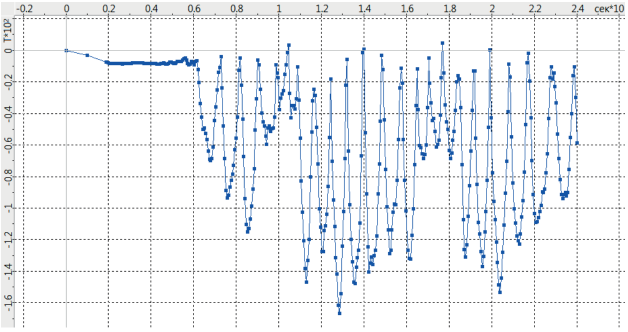


Figure 7. FE 819373. Longitudinal force.
N_{min}= -166.8 m (t = 12.86 s),
N_{max}= 4.68 m (t = 17.7 s)

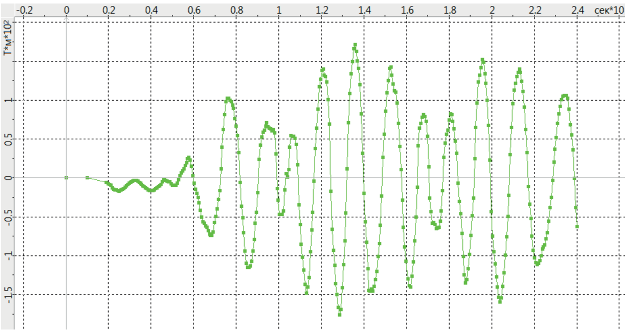
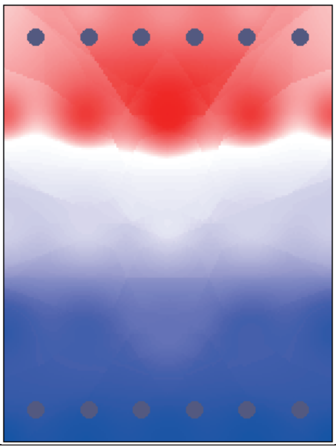


Figure 8. FE 819373. Bending moment.
My_{min}= -176.40 m (t = 12.86 s),
My_{max}= 171.65 m (t = 13.58 s)



	Concrete compression. Min - 1307.66 t/m^2
	Concrete tension. Max 54.43 t/m^2

Figure 9. Normal stresses in the beam section, (t=12.85 s)

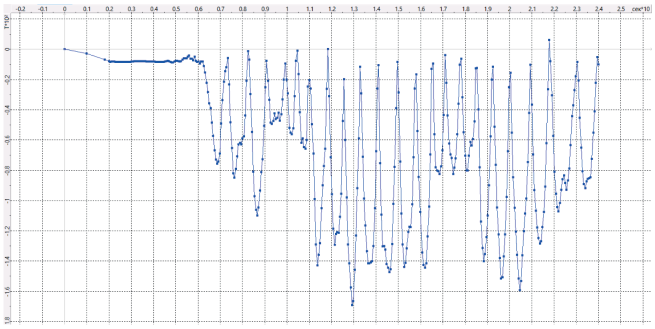


Figure 10. FE 819373. Longitudinal force.
N_{min}= -169.11 m (t = 12.92 s),
N_{max}= 6.22 m (t = 21.78 s)

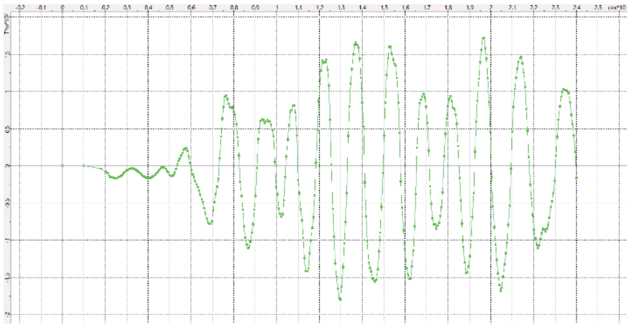
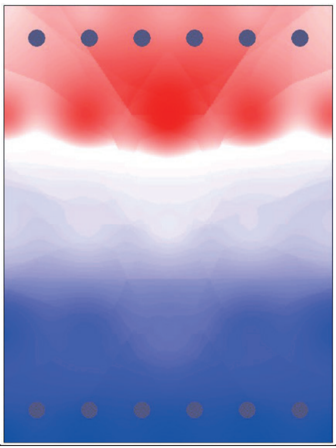


Figure 11. FE 819373. Bending moment.
My_{min}= -179.77 m (t = 12.98 s),
My_{max}= 171.69 m (t = 19.68 s)



	Concrete compression. Min - 1322.63 t/m^2
	Concrete tension. Max 52.46 t/m^2

Figure 12. Normal stresses in the beam section, (t=12.98 s)

5. CONCLUSIONS

The following conclusions can be drawn from the presented results of the comparative analysis:
1) Qualitatively, structural displacements are generally similar for both the platform and the wave scheme;
2) With the described methodology to apply the impact's wave nature, a slight delay of the displacement peaks can be seen in comparison with the platform scheme. It reaches 0.15 s in displacements for different parts of the model;
3) For individual local peak displacements, the difference reaches up to 18.1%, but for absolute maximum displacements on the roof the difference does not exceed 10%.

Taking into account the available ground conditions (wave propagation velocity) and the foundation dimensions (104×87 m), the influence of the wave action on the stress-strain state of the building is insignificant. In the development of this work, it is planned to carry out design studies using the described approach on a structure with more extensive foundation dimensions.

Also, in the considered example, the seismic wave propagation velocity according to geophysical survey data is 760 m/s, and it is recommended [1] to consider the wave nature at smaller phase velocities' values of $V < 400$ m/s.

The methodology for consideration of the seismic impact's wave nature in the analysis of high-rise buildings using the direct dynamic method can be applied with the methodologies that utilise the linear-spectral theory.

In light of the current architectural evolution of modern buildings and structures, and in the presence of the paragraph's 6.1.4 of SP 14.13330.2018 [11] requirements, it is extremely necessary to develop engineering methods to consider the seismic impact's wave nature, as well as to introduce them into the software used in the industry.

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