Methods for subsoil modelling under dynamic impacts and multicomponent damping in SCAD FEA software with geophysical monitoring

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Abstract. In this paper, an integrated technology for performing finite element calculations in SCAD Office FEA-software and identification of dynamic characteristics of Soil-Structure Interaction models with the use of geophysical hardware REGISTR and SINUS is proposed.

Introduction
In the modern construction legal field of the Russian Federation, a dual situation has formed. On the one hand, as part of the harmonization process with Eurocodes, new federal laws and updated design codes were introduced, requiring the analysis of the interaction of the superstructure, foundation and subsoil without specifying the criteria when this requirement should be applied. Indeed, such an interaction can be significant for statically indefinable and flexible structures on a flexible foundation or under dynamic conditions for a certain ratio of the stiffness of the structure and the soil. On the other hand, since the suspension of the harmonization process of building norms in 2014, new methods for modelling dynamic stiffness and damping of natural and pile foundations have not been presented.

In this study, the authors propose resolving a situation of inconsistency in regulatory requirements. For the Near Field Soil, which is affected by the Soil-Structure Interaction (SSI), the most optimal is the combined direct physical model of solid high-precision isoparametric finite elements on the underlying infinite half-space layer. The presented combined method corresponds to the modern approach to the SSI models in various versions of SASSI class programs (figure 3), and also allows calculation of flexible foundations on a natural and pile foundation in SCAD FEA software. To identify the conditions for using the new combined method, the authors give a criterion for assessing the presence of the inverse effect of the structure on the ground vibrations.

The new approach to subsoil modelling proposed by the team of authors includes two methods integrated with the model for geophysical identification of dynamic parameters of soil media and structures for the stages of design and construction.

1. The criterion for assessing the presence of the inverse effect of the structure on soil vibrations
In the first study of the Soil Structure Interaction accounting criteria, the authors considered two independent foreign approaches [1]. The graph-analytical method proposed by Japanese researchers
Osawa, Kitagawa, and Iri [2] used the dynamic characteristics of soils determined by the average speeds of seismic shear waves. The four seismicity categories according to SP 14.13330.2014 “Construction in seismic areas” according to the formulas from monograph of A. N. Birbraer [3] have been considered.

The second considered analytical method is reflected in the American standards for designing nuclear facilities ASCE 4-16 “Seismic Analysis of Safety-Related Nuclear Structures and Commentary” [4]. It uses the same dynamic characteristics of soils, but is easier to use. Its essence lies in determining the natural frequency of the inertia-free base as the simplest oscillator with the mass of an absolutely rigid structure and comparing it with the dominant frequency of a flexible structure on an absolutely rigid foundation. If these frequencies differ by less than a factor of two, then accounting for SSI effects is necessary. For example, if the rigidity of the tent structure is several times lower than the rigidity of the underlying rock base, then it is obvious that the opposite effect is impossible. A more conservative condition on the difference in frequencies up to three times made it possible to achieve a match between ASCE 4-16 and the Japanese method in 100% of cases from the considered 20 models.

An additional study [5] made it possible to compare the dynamic equivalent stiffness and damping parameters for the simplest model of a homogeneous subsoil under massive foundations according to SP 26.13330 “Foundations of machines with dynamic loads” modified by the authors, the OA Savinov’s empirical method and analytical dependencies ASCE 4-16, used for preliminary calculations of the subsoil dynamic stiffness. For soils of III and IV seismicity categories, the dynamic parameters of all the methods turned out to be close, which allows applying the provisions of SP 26.13330 for the validation of complex models. This study also proved the validity of the criterion [1] for subsoil models of solid FEs and of simplified models with equivalent characteristics according to SP 26.13330.

2. Principles of creating direct physical models of solid finite elements

Geotechnical models of foundations under static conditions usually have a rectangular geometry of the external boundary contour (figure 1) with an array height to the depth of the compressible thickness \( H_c \) or to a low compressible base with a deformation modulus \( E > 100 \) MPa. The width of the array is taken with the vertical boundaries indented at a distance of 3-4 maximum foundation dimensions \( D \) but not more than 1.5\( H_c \). From the assumption that the edge effects beyond this “influence area” do not affect the stresses under the structure [6]. As a rule, a model has tetrahedral triangulation due to the simplicity of splitting an inhomogeneous multilayer soil model using Delaunay methods. However, distortion of the geometry of tetrahedral elements and reduced accuracy, by analogy with triangular FEs, increase the errors of calculations, therefore, prismatic elements are often used in scientific research.

The simplest way to stretch a mesh of prismatic finite elements is to lengthen the elements as they approach the outer boundaries. At the same time, the calculation errors in the stretched elements increase, however, their effect on the deformation of the base immediately below the foundation of the structure tends to zero (figure 1.a).

![Figure 1. Finite element mesh optimization methods in direct physical models](image)
The second grid stretching method for this example is 27% more efficient and uses the transition from a rectangular prismatic element through two triangular and one trapezoidal planar finite element to the next row of prismatic elements with a triple increase of the elements’ size in each transition (figure 1.b). The third method is available in SCAD Office using up to 20-node isoparametric finite elements with additional intermediate nodes on the faces. In this method, the stretching rate is much higher due to a double increase at each transition, as a result of which the total number of elements is 36% less than in the first method. At the same time, the quality of calculations is improved due to the increased accuracy of 20-node solid FEs (figure 1.c).

In the practice of subsoil analysis under dynamic impacts when creating direct physical models, the cylindrical geometry of the outer boundary is used to absorb normal-reflected waves by viscous dampers. However, such “absorbing boundaries” are not able to exclude surface seismic waves. Therefore, the indentation of the boundaries of the simulated subsoil must be at least five maximum dimensions of the foundation $D$ in the horizontal direction and three $D$ in depth [3, 4, 6]. Similar modelling rules are formalized in the American standard ASCE 4-16 for the design of nuclear power facilities under seismic conditions. At the same time, two design complexes were verified for use in the design of nuclear power plants in the USA. These are LS-DYNA with direct physical modelling and SASSI with a simplified modelling method [7]. It should be noted that even in the simplest example considered, with the transition from the foundation edge to the outer borders through four rows of finite elements, the dimension of the problem increased by 12.5% (figure 1.d). When assigning boundaries to $5D$, the number of degrees of freedom will increase by several orders of magnitude.

In the Russian standards there are no recommendations on the permissible degree of mesh stretching at the external boundaries of the model. In contrast, in ASCE 4-16, the size of the finite elements of a volume dynamic model is strictly regulated. The maximum dimension of an element at the outer boundary shall not exceed one eighth of the length of the slowest shear seismic waves in weak layers of the soil base. Previously [1, 5], the authors considered four types of foundations by seismicity categories. If the average shear wave velocity $V_s$ of 100 m / s is typical for soil category IV, then the size of the final element in the base model should be no more than $0.125L_{V_s}$ at a wavelength of $L_{V_s}=V_sT_{\min}$.

Unfortunately, this ASCE 4-16 rule does not specify the minimum period for which a seismic wave in one oscillation cycle must pass eight finite elements. It is impossible to accept the predominant period of seismic impact, since when analyzing the spectrum of the acceleration response, it is almost always possible to identify more than one clearly pronounced peak and the corresponding period. According to A G Tyapin minimum significant period should be chosen according to the energy principle. In figure 2, the authors give an example of identifying the minimum significant period of seismic impact using the power spectrum in the SCAD Office service application “Accelerogram Editor” for the synthesized accelerogram of seismic impact in the Zalari town of the Irkutsk region. The prevailing period $T_0$ for a peak frequency of 2.04 Hz is 0.49 s. In the case of choosing the lower limit for considering the peaks of seismic impact power not lower than 33% of the maximum value, the minimum significant period $T_{\min}$ will be 0.34 s.

![Figure 2](image-url) Determining the smallest significant period from the accelerogram power spectrum
If the lower limit is reduced to 7%, then $T_{\text{min}}$ will be 0.26 s. At the external boundary of the model, the maximum size of the finite element $L_{\text{FE max}}$ for this accelerogram on soft soils of category IV with a lower power limit of 7% will be $0.125V_s T_{\text{min}}$, or 3.25 m. The minimum element size should not exceed one third of $L_{\text{FE max}}$, equal to 1.08 m.

The number of corner segments in a cylindrical model with a finite element width at the outer boundary less than $L_{\text{FE max}}$ should not exceed an integer from $11\pi D (L_{\text{FE max}})^{-1}$ and be a multiple of four. With a smooth stretching of the grid from $0.33L_{\text{FE max}}$ near the edge of the foundation to $L_{\text{FE max}}$ long the width of the transition region $L_{NF} \geq 5D$, the size of intermediate FEs $L_{FEj}$ for an integer number of steps $N \geq 1.33L_{NF} L_{\text{FE max}}^{-1}$ can be found by the formula (1):

$$L_{FEj} = L_{\text{FE max}}[1 - j(0.667N^{-1})]$$

To reduce the size of SSI models in civil engineering, the authors recommend considering a simplified modeling approach implemented in the ACS SASSI 2010 or MTR / SASSI SSI Software program. As noted earlier, in LS-DYNA, the simulated fragment of the Near Field Soil affected by SSI should have a height $H_{NF}$ of more than three dimensions of the foundation $D$ and a total diameter of more than 11$D$ (figure 3.a). This inevitably leads to increased hardware requirements when solving direct dynamics problems with a stiffness matrix of several million degrees of freedom. In contrast, the SASSI foundation model is modeled by traditional solid finite elements only in the area of replaced, reinforced subsoil or of pile foundation. The rest of the subsoil is represented by infinite horizontal layers on the underlying half-space layer (figure 3.b). In SASSI FEA software, structural systems are linear in physical properties and analyzed without taking into account geometric or structural nonlinearities, without local detachments of the foundation or sliding of the contact surface [8].

During the development of SASSI, the authors performed a comparison with many existing analytical and numerical approaches to calculating SSI problems. In 1987, the US government atomic energy departments conducted “blind testing” of various SSI methods based on the results of extensive field instrumental studies in Lothung (Taiwan) on an in-depth 1: 4 scale model of a nuclear power plant construction. Comparison showed that the best prediction of experimental results is observed in models of the SASSI program [9].

In the SASSI model, it is necessary to transfer the accelerogram from the “day” surface of the Free-Field to the specified depth of the “evening” surface. This task is performed in programs of SHAKE type [4, 10], the analytical dependencies of which are implemented in SCAD Office.

Figure 3. Simplified combined dynamic model of a multilayer subsoil
3. The underlying layer of infinite half-space

The geological structure of the ground base with increasing depth becomes more regular compared with the surface layers of sedimentary and fractured rocky soils. As the depth increases from the bottom of the slab foundation or from the lower level of the pile field, the stresses at the subsoil also tend to a uniform distribution. According to the Saint-Venant principle, this depth can be comparable with the foundation size.

In direct dynamic calculations, the depth of the model should ensure the attenuation of the reflected waves during its passage. In SASSI, the dimension of the problem is reduced by replacing the lower homogeneous region with a half-space layer. The Winkler subsoil model is not applicable for this purpose, since it does not reflect shear stiffness affecting angular oscillations and rolls in the area of the sedimentary funnel beyond the foundation contour (figure 4.a). Therefore, in the traditional finite element method, the half-space is modelled by a surface of shells and auxiliary one- and two-node infinite contour elements, under which the properties of the elastic half-space are assigned (figure 4.b).

Three or four nodal shell finite elements of the first type have vertical displacements \( u \), length \( a \), and width \( b \) (figure 4.c). The potential energy \( E_0 \) of such shell elements of conditional surface can be excluded from the calculation by setting a low stiffness or zero thickness. Thus only potential energy of the underlying elastic half-space \( E_1 \) will remain along surface of the foundation bottom. Its rigidity can be described by a two-parameter model, which reflects the dependence of the settlement \( u_z \) on pressure \( p \) using bed ratios for compression \( C_1 (\text{MN/m}^3) \) and for shear \( C_2 (\text{MN/m}) \). This dependence is the mathematical “folding” of the elastic Boussinesq half-space to the elastic surface using the Laplace operator \( \Delta \) [6]:

\[
\begin{align*}
    u_z &= \frac{p}{c_1-c_1\Delta} = \frac{p}{c_1-c_1\left(\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}\right)} \\
    \text{where } u &= u_i(x, y),
\end{align*}
\]

(2)

The second type is double-node contour finite elements with potential energy \( E_2 \), the formula of which is identical to the energy of half-space at the contact with the shells. Two-node elements have one degree of freedom, the length \( a \) between nodes and an infinite width \( b \) along the normal to the contour of the slab (figure 4.d). The potential energy of single-node contour elements of the third type \( E_3 \) for angular sectors is conveniently expressed in the polar coordinate system (figure 4.e). The total potential energy \( E \) of all 6 shell, 12 double-node and 4 single-node elements in figure 4.b would be as follows:

\[
\begin{align*}
    E &= 18 \times \frac{1}{2} \int_0^a \int_0^b \left\{ c_1 u^2 + c_2 \left[ \left( \frac{du}{dx} \right)^2 + \left( \frac{du}{dy} \right)^2 \right] \right\} dx \, dy + 4 \times \frac{1}{2} \int_0^{\pi/2} \int_0^\infty \left\{ c_1 u^2 + c_2 \left[ \frac{1}{\rho} \left( \frac{du}{d\rho} \right)^2 + \left( \frac{du}{d\phi} \right)^2 \right] \right\} \rho \, d\rho \, d\phi
\end{align*}
\]

(3)

where \( u = u_i(x, y) \) is the function of elastic settlement over the subsoil area of the finite element \( \Omega^e \).

![Figure 4. Finite elements and basics of two-parameter model of the underlying elastic half-space](image-url)
The practical use of an accurate mathematical description of a two-parameter model of a homogeneous elastic half-space is complicated by the heterogeneity of real soils and by the conventional assumption that the elasticity theory is applicable for irreversible linear strains. Therefore, none of the many methods for calibrating the coefficients $C_1$ and $C_2$ for a uniformly layered subsoil with limited compressible thickness was not included in the Russian Design Codes. Table 1 compares the three more reliable methods considered within this study.

Table 1. Comparison of two-parameter models of the underlying elastic half-space

| Pasternak, Vlasov, Leontiev (1960-63) | Mednikov, Shashkin (1967, 1999) | Piskunov, Fedorenko (1994) |
|--------------------------------------|---------------------------------|-----------------------------|
| $C_1 = \left( \int_0^H \frac{dz}{E(z)} \right)^{-1} \sum_{i=1}^{n} E_i \frac{h_i}{h}$ | $C_1 = \left( \int_0^H \frac{dz}{E(z)} \right)^{-1} \sum_{i=1}^{n} E_i \frac{1}{h_i}$ | $C_1 = \sum_{i=1}^{n} E_i h_i (1 - e^{-2\gamma h_i})$ |
| $C_2 = \left( \int_0^H \frac{dz}{E(z)} \right)^{-2} \left[ \int_0^H \frac{G(z)}{E(z)} \frac{dz}{z} \right] \left( \frac{dz}{E(z)} \right)^{-1} \sum_{i=1}^{n} G_i h_i (\delta_i + \delta_i + \delta_i + \delta_i)$ | $C_2 = \sum_{i=1}^{n} G_i B_i (1 - e^{-2\gamma h_i})$ |

Here $H = \sum_{i=1}^{n} h_i$ is the total depth of the deformable thickness of the ground base, consisting of a finite number of layers $n$, linearly deformable and constant in thickness $h_i$; $E_i = E_0 (1 - \nu_i) (1 + \nu_i) (1 - 2\nu_i)^{-1}$ is the reduced isometric modulus of deformation of layer $i$, taking into account the assumption of a slight effect of transverse deformation; $E_0$ is the general deformation module of layer $i$ according to geological survey data or reference literature; $\nu_i$ is the modulus of the proportional transverse deformation of the layer (Poisson’s ratio); $G_i = K_i E_0 [2(1 + \nu_i)]^{-1}$ is the shear strain modulus of layer $i$ and $G(z)$ is the reduced deformation modulus and shear modulus in the corresponding soil layer at depth $z$; $K_i < 1$ – correction coefficient according to laboratory tests, always less than unity and taking into account the deviation of soils from perfectly elastic bodies, which, if ignored and a single value, leads to overestimated values of $C_2$ (the equivalent $K_i$ can be calculated by comparing with stamp tests or by absence with the method of layer-by-layer summation); $\gamma_i = 4(1 - 2\nu_i)(\pi A^{1/2})(1 - \nu_i)^2$ is the sedimentation attenuation coefficient for the soil layer $i$; $A$ is the actual area of foundation contact area on the subsoil; $K_i$ is a constant equal to unity for the first layer ($B_1 = 1$), and for the second and subsequent layers it changes exponentially $B_i = B_{i-1} e^{-\gamma (h_i - h_{i-1})}$; $\delta_i = \sum_{i=1}^{n} \frac{h_i}{h}$ is base compliance coefficient calculated for the total depth of compressible thickness $H_c$ (all layers from 1 to $n$) and inversely proportional to the bed coefficient under compression $C_1 = \frac{1}{\delta_i}$; $\delta_i = \sum_{j=1}^{n} \frac{h_j}{h}$ is the compliance coefficient from the second or subsequent layers of the subsoil to the depth of the compressible thickness, which is assumed to be zero at $i = n + 1$.

The first of the methods in Table 1 describes the subsoil model, often called the Winkler-Pasternak one, since the first coefficient is similar to the Fuss-Winkler model, and the second was obtained by PL Pasternak, who in 1954 proposed a solution for a multilayer bound base and generalized the homogeneous subsoil models of Filonenko-Borodich (1945) and MI Gorbunov-Podavos (1949). In the SCAD FEA software the Pasternak model is taken into account with the modifications introduced by VZ Vlasov and NN Leontiev.

It should be noted that the Pasternak model contains a contradiction in the derivation of formulas, which admits a discontinuity in the diagram of tangential stresses. This contradiction was removed in the work of IA Mednikov (1967). He took into account the stepwise decrease in the shear strain modulus over the height of the soil column, and also proposed a correction factor $K_i$ for the reduced shear modulus, which allows an overestimated shear bedding ratio $C_2$ to be compared with the actual soil laboratory tests. Mednikov’s mathematical dependences were generalized by KG Shashkin [11] and developed to a complete model of the surface of the elastic layer with complementary formulas for the reverse “unfolding” of stresses under the bottom of the foundation to normal, tangential stresses and displacements at any point in the half-space.

In the third technique, implemented in SCAD, the model of the elastic layer surface was developed by VG Piskunov to a hardened half-space [12]. The attenuation coefficient makes it possible to evaluate the subsoil stiffness not only at the operational stage, but also to take into account temporary or special dynamic effects with automatic limitation of the compressible thickness. This allows one to use the two-parameter model as the underlying layer by analogy with SASSI.
4. Geophysical model identification and monitoring
The complication of mathematical models, including in the proposed combined method under conditions of a heterogeneous layered subsoil, requires validation of the calculation results. As part of the preliminary examination of the model, the method modified by the authors according to SP 26.13330 or the analytical method ASCE 4-16 can be used [5]. However, both of these methods are simplified models of homogeneous inertia-free Near-Field Soil media with a structure represented by a single-mass oscillator. Therefore, additional validation of the dynamic parameters of the foundation and structure model is required in the process of pre-design surveys and construction monitoring. This could be solved using the non-destructive method of microseismic measurements at low financial and time costs.

The second aspect for improving the quality of a heterogeneous subsoil model is the study of the upper part of geological sections by the seismic method of wave refraction, which allows one to get a picture of the velocity profile in the soil layers to a depth of 50-100 m. Besides clarifying the geometry of the bedding, this method solves another problem of standard geological surveys by determining the dynamic elastic moduli for seismic calculations using measured wave propagation velocities.

4.1. Identification of the dynamic characteristics of structures and soil media
In one of the previous publications [13], the authors clearly described and demonstrated the advantages of the technology of using the REGISTR-SD three-channel seismic signal recorder and the Reg3MSD software package developed by the Seismometry Laboratory of the Institute of Geophysics, Ural Branch of the Russian Academy of Sciences [14]. On the example of a pre-emergency multi-storey residential building in Kemerovo with a precast-monolithic reinforced concrete frame, a fairly close coincidence of the amplitude-frequency characteristics according to the measurement data and according to the design calculations according to the modified dependencies in SP 26.13330.2012 “Foundations of machines with dynamic loads” (figure 5.a).

The instrumental determination of relative damping in the upper layer of the Near-Field Soil $\xi_3$ and in the center of the foundation slab $\xi_4$, made it possible to clarify the most optimal analytical method for calculating damping according to SP 26.13330 as for steady-state oscillations $\xi_2$ (figure 5.b). It also made it possible to derive the conversion coefficient of instrumentally measured damping by the microseismic method to the value of relative damping $\xi_1$ under seismic conditions. As a result of the geophysical refinement of the dynamic characteristics of the model in SCAD FEA software, more plausible results of modal analysis were achieved (Figure 5.a). This study demonstrated the importance of identifying the SSI subsoil model for civil engineering purposes, as well as the possibility of numerical dynamic design monitoring during the construction process.

Figure 5. Instrumental amplitude-frequency spectra of the surveyed building from 1 to 30 Hz with eigenfrequency points of the equivalent subsoil model according to SP 26.13330 (a) and the relative damping diagram $\xi$ according to SP 26.13330 and microseismic measurements (b)
4.2. Seismic exploration of the upper part of the geological section

Figure 6 shows a comparison of two geophysical methods for studying the upper part of a geological section. The object of additional studies carried out in August 2019 is the zone of tectonic contact of granite and gabbro in the city of Yekaterinburg, described in detail in a previous publication [15].

The upper profile was obtained by the method of refracted waves of non-explosive shallow seismic exploration, which proved itself in determining the depth of bedrock. The recording of seismic vibrations was carried out using the 24-channel digital seismic station “SINUS” [15], designed and manufactured at the Seismometry Laboratory of the IGF UB RAS. When processing and interpreting seismograms, the Sin24M software packages developed by the IGF UB RAS and WinSism 9.0 by the Swedish company GeoSoft were used. Based on the traced hodographs of the first arrivals, a velocity and deep soil layered compaction 2D model was obtained. This model allows one to refine the engineering-geological sections in the framework of standard tests, as well as go from the speed of wave propagation to the dynamic stiffness of the base layers [3, 4].

The lower profile was obtained by the electromagnetic method using the SIR-3000 (GSSI) GPR with a 270 MHz antenna and post-processing in the RADAN 7 software. This method has been successfully used in archaeological excavations and survey of the grounds for the presence of karst formations or artificial inclusions of high density. The conditions of increased humidity after precipitation increased the dielectric constant of the soil, reducing the penetration depth of the electromagnetic pulse from 15 m to 6 m. Therefore, the georadar study did not solve the problem of clarifying the configuration of the layered base, which must be performed to a depth of compressible thickness, usually exceeding 6 m for critical structures. A qualitative visual picture of the layered structure of soils does not allow us to determine the physical density of soils. The main task of determining the dynamic stiffness of soils is also not solved by this method.

Figure 6. The soil velocity profile of the upper part of the geological section with a hot colour palette and the profile of the ground penetrating radar survey along the same 50 m long studied area
5. Multi-component damping

The damping parameters are determined by the REGISTER-SD seismic recorder using the standard method for calculating the attenuation decrement (figure 7.a). The quality of measurements can be improved by analysing the amplitude-frequency spectrum (figure 6.a) and the power spectrum using an instrumentally fixed accelerogram for a fragment with random dynamic action (figure 2). Such a refinement of damping is useful both at the early stages of the project, when SSI effects are analyzed by modal calculation methods, and at the stages of a detailed SSI analysis in the mode of direct integration of equations of motion in time domain. Most programs, including LS-DYNA, in the direct dynamics mode use proportional attenuation of the system in the form of a Rayleigh dissipation matrix:

\[
[C] = \alpha [M] + \beta [K]
\]

where \([M]\) is the mass matrix, \([K]\) is the stiffness matrix, \(\alpha\) is the constant inertial coefficient for describing the dependence of the damping of the system on the nodal masses, \(\beta\) is the constant coefficient of damping of the system in the structural materials in proportion to the stiffness.

Since Rayleigh's publication of Theory of Sound in 1878, despite his "unphysical" assumption of the influence of the mass matrix, his method continues to be universally used due to its simplicity. For two a priori given frequencies, large systems of equations with many degrees of freedom are quickly solved. The problem has so far been solved by the fact that for systems with one predominant structural material, the dissipation matrix may contain off-diagonal "non-Rayleigh" parameters of viscous dispersion in single-node dampers modelling the dissipative properties of soils at the contact with the base of the foundation. However, at the present stage of development of computing systems, one cannot but take into account that in real structures damping should be inhomogeneous, especially for direct physical SSI models with solid finite elements. In systems with a large number of degrees of freedom, especially using shell finite elements, there may not be two clearly distinguishable frequencies. The sagging curve underestimates the damping between the selected frequencies (figure 7.b), while instead of the expected solution "to the reserve", an "anomalous" understatement of seismic reactions can be observed, which is eliminated by multiple sorting of a priori specified frequencies [16].

In the combined SSI model proposed by the authors, the main advantage is the possibility of analysing a single design scheme with a foundation and aerial parts. The microseismic method for validating dissipative properties also does not make sense if it is impossible to assign unique dynamic characteristics to different soils and structural materials in the general model. This problem was solved by S Yu Fialko in a new numerical approach for accounting for multicomponent damping [17]. He proposed to present the dissipation matrix in the form of damping by the type of material, regardless of the vibration frequencies:

\[
[C] = \sum_e \gamma_e [P_e]^T[K_e][P_e]
\]

where \([K_e]\) is finite element stiffness matrix \(e\), \([P_e]\) is permutation matrix, \(\gamma_e\) – coefficient of inelastic resistance of a material from Design Codes and reference literature.

Figure 7. Graphs for the determination of damping by seismogram (a) and by the Rayleigh method (b)
6. Conclusion
Modern urban construction conditions in seismic regions are often disadvantageous undeveloped areas that are located on slopes or in floodplains of the river, have heterogeneous soil layers’ compaction, and require reinforcing the foundation with piles. For SSI analysis in such conditions, the authors propose a combined model with a Near-Field Soil of solid finite elements on the underlying elastic half-space. The model can be implemented by this method in SCAD FEA software with multicomponent damping and its instrumental refinement.

In order to exclude time-consuming calculations, the authors give a criterion for assessing the presence of SSI effects in the form of the inverse effect of building vibrations on the Near-Field Soil. Then, at the design stage, the correctness of the combined model can be verified by a simplified model with equivalent stiffness and damping of the subsoil according to the modified by the authors formulas of the actual Design Code SP 26.13330.

Geophysical identification of the dynamic characteristics of the models and the survey of the high-speed geological profile of the base complement standard geological surveys. The considered methods using the REGISTER and SINUS devices provide a comprehensive solution to the problem of creating a plausible dynamic subsoil model for civil engineering design. The microseismic method is also suitable for further computational construction support of technically complex structures by checking the floor-level amplitude-frequency characteristics and early detection of deviations from the project.

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