17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

SOIL-STRUCTURE INTERACTION IN THE SEISMIC RESPONSE OF HIGH-RISE BUILDING

A. Tyapin[(1)]

[(1)] *Chief Specialist, JSC "Atomenergoproject", atyapin@bvcp.ru*

Abstract

The goal of the paper is to apply the soil-structure interaction (SSI) procedure used for the NPP design (namely Combined Asymptotic Method) to the high-rise civil structure. Soil is modeled by homogeneous half-space with Vs=400 m/s. Three-component excitation time-history is used to provide six-component response base motion. The results of the SSI analysis are compared to the results of the fixed-base analysis. Besides, the behavior of the high-rise building is compared to the behavior of the typical NPP structures.

The first conclusion is that the soil flexibility impacts the structural response differently in different directions. In the vertical direction SSI considerably changes the response base motion as compared to the free-field motion (which is common for the NPP buildings). However, the horizontal response base motion is almost similar to the free-field motion (unlike the NPP buildings). Surely, rocking response base motion appears (no rocking in the free-field motion). The SSI criteria used in the nuclear field confirm that SSI is important in the vertical direction, but not in the horizontal direction.

The second conclusion is that the first fixed-base horizontally-rocking modes have very small eigenfrequencies (about 0.1 Hz), which are far less than the dominant frequencies of seismic excitation (about 2 Hz). Relative response motion in the first modes goes in a counter-phase with excitation, which makes absolute modal displacements very small. As a result, modal masses of these modes do not contribute to the integral forces under the base. Horizontal integral forces are far less than the vertical forces, while horizontal base accelerations are greater that vertical ones.

The third conclusion is that in the linear-spectral method the first horizontal modes contribute less than subsequent modes, because great modal participation factors for them are multiplied by comparatively small spectral accelerations of excitation at low frequencies. However, these low-frequency spectral responses should be combined over different modes directly (and not by SSRS rule), as inter-correlation proves to be high.

From all the components of the soil flexibility the rocking flexibility proves to be the most important for high-rise building. Rocking response base motion goes in the counter-phase with translational horizontal response base motion, decreasing horizontal integral forces under the base. Generally, all integral forces decreased as compared to the fixed-base analysis - so, the fixed-base analysis is conservative in terms of the integral forces.

Keywords: soil-structure interaction, combined asymptotic method, high-rise building, fixed-base model

1. Introduction

Soil-structure interaction (SSI) changes seismic response motion of the base mat as compared to the free-field motion [1]. This effect is more pronounced for the heavy structures resting on medium and soft soils. In the design of the Nuclear Power Plants (NPP) the importance of SSI was recognized long ago; so, the corresponding Standards have had the requirements on SSI since 1980-s [2,3,4].

In the civil engineering up to the recent time SSI was ignored. Partial explanation was that civil structures are usually light, so they cannot change the motion of the soil. However, in recent decades there have appeared civil structures so sizeable and heavy, that they are comparable to the main structures of NPP. The goal of this paper is to apply the procedures initially developed for the SSI analysis of the NPP structures to the heavy civil structure – high-rise building. The SSI effects are numerous; let us see which of them stay the same for the civil structure as for the NPP structure.

High-rise building is analyzed using Combined Asymptotic Method (CAM) originally developed for the SSI analyses of the NPP structures [5, 6]. It is a variant of sub-structuring method based on the rigid base assumption: the upper structure and the soil are analyzed separately and later combined in the format of 6 x 6 dynamic stiffness matrices to provide the response motion of the rigid base. CAM enables a lot of additional checks and helps better understanding of the different factors in SSI [7].

2. Sample soil-structure system and basic CAM equations

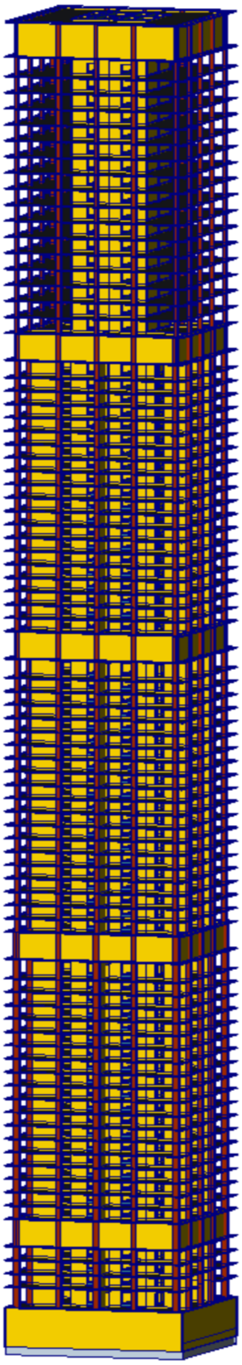
Fig. 1 - Fixed-base model

The sample building is 327 m high and has 76 floors. Base mat is 4 m thick and 40 x 40 m in plan. Total mass is 224804.6 tones (note that the typical NPP reactor building has mass about 360000 tones – so, the mass of the civil structure is comparable to the mass of the NPP structure). Center of mass is 144 m above the medium plane of the base mat. The model is shown in Fig. 1.

Initially the fixed-base model was created in the SCAD program [8] and then converted into ABAQUS [9] for the modal analysis. There were 214887 nodes and 1289322 DOFs in the model. The number of finite elements made 245759. It turned out that the first eigenfrequencies of the fixed-base model are about 0.1 Hz. Modal analysis was carried out for the first 7500 eigenmodes; the last calculated eigenfrequency proved to be only 25.6 Hz. Here we see the first considerable difference between our civil structure and NPP reactor building, having the first fixed-base eigenfrequencies about 3-4 Hz, and about 1000 eigenfrequencies below 50 Hz. частот.

Soil in this paper is modeled by the homogeneous half-space under the base mat. Of course, this is a simplification, ignoring the embedment, possible piles, etc. In the NPP design layered soils and the underground part are modeled in more details, but here we make the first step for the civil structure. Mass density of the soil is 2 t/m^3; shear wave velocity is Vs=400 m/s; primary wave velocity is Vp=1300 m/s; the internal material damping is 4%.

CAM is based on the assumption of the rigid base mat. If this assumption is valid, the basic equation of motion for the rigid base mat is written in the frequency domain as

$$[C_s(\omega) - \omega^2 M(\omega)]\, U(\omega) = B_s(\omega)\, U_0(\omega) \quad (1)$$

Here ω is a current circular frequency. $C_s(\omega)$ is a complex matrix 6 x 6 of the dynamic stiffness of the rigid stamp in the given soil (this matrix is often called “the impedance matrix”). This matrix is independent of the upper structure; it ”condenses” the soil properties and the geometry of the contact surface between soil and structure. $M(\omega)$ is a “dynamic inertia” matrix. It is also complex symmetrical matrix 6 x 6, “condensing” the properties of the fixed-base upper structure and independent of the soil properties. This matrix is controlled by the “rigid” inertia matrix M_0 (frequency-independent real matrix 6 x 6), but the additional data about the flexibility of the upper structure are added – in the format of eigenfrequencies, composite modal damping and modal participation factors [5, 6]:

$$M(\omega) = M_0 + \sum_{j=1}^{n} \frac{\omega^2}{\Omega_j^2 - \omega^2 + 2\, i\, \omega \Omega_j\, \gamma_j} S_j^T\, S_j \quad (2)$$

Here Ω_i is an eigenfrequency number j; γ_j is a composite modal damping coefficient for this mode; S_j is a line of six real participation factors for this mode (normalized by mass); n is a number of modes considered; i is an imaginary unit. As we see from Eq. (2), modal damping makes the result complex.

Now let us further describe the terms, participating in Eq. (1). $U(\omega)$ is a six-component vector, describing the base mat displacements in the frequency domain. $B_s(\omega)$ is a matrix of the seismic loads acting on the fixed base mat due to the seismic wave providing unit displacements in the control point of the free field (usually at the free surface). This matrix 6 x 3 is complex and frequency-dependent. Like the impedance matrix, it is independent of the upper structure. Unlike the impedance matrix, it depends also of the type of the seismic wave. The common assumption is that seismic waves (two S-waves and one P-wave) spread vertically in the horizontally-layered soil, providing three-component motion at the free surface. If we have a surface base mat (like in our case), matrix $B_s(\omega)$ consists just of the first three columns of the impedance matrix $C_s(\omega)$.

Finally, $U_0(\omega)$ in Eq. (1) is a three-component vector describing translational motion of the control point in the free field. It is convenient to replace three-component U_0 by six-component U_1, putting zeroes into three last lines. Then Eq. (1) may be re-written as

$$[C_s(\omega) - \omega^2 M(\omega)]\, U(\omega) = C_s(\omega)\, U_1(\omega) \tag{3}$$

3. Implementation of CAM for the sample soil-structure system

Let us return to our fixed-base model of the high-rise building. Fig. 2 shows the accumulation of the modal masses along six degrees of freedom of the base mat over the current frequency.

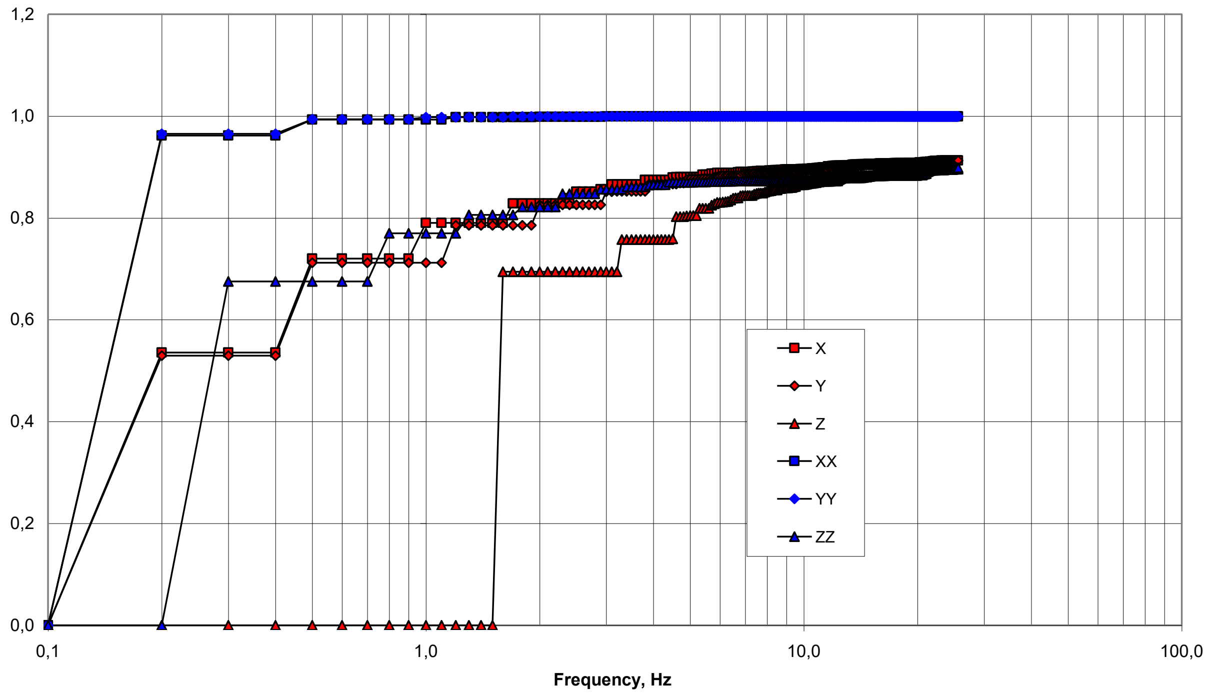

Fig. 2 - Accumulation of modal masses in six directions over the frequency

The accumulated modal mass is divided by the full mass in the given direction. This ratio goes not to the unit but to the somewhat less value, because the part of the total mass linked to the fixed base does not participate in any of the modes. In our case the base mat has volume 40 x 40 x 4 = 6400 m^3. Given mass density of the concrete 2.75 t/m^3 it makes the base mat mass 17600 t – 7.83% of the total mass. Thus, maximal achievable ratio in the translational directions makes 92.17%. At the frequency 25.6 Hz we see already 90.7 and 90.8% in the two horizontal directions. The mass in the vertical direction is the slowest one in terms of the accumulation. The fastest accumulating ones are rocking inertia moments.

Fig. 3 shows the dynamic inertia in vertical direction calculated using Eq. (2) – the diagonal term of the matrix $M(\omega)$. The behavior in Fig.3 is quite common for the NPP structures [6] (except the low value of the first eigenfrequency). We see the principal resonance at 1.5 Hz and two smaller resonances around 3.3 and 4.5 Hz. One can compare these frequencies to the Fig. 2. Note that after each resonance the real part in Fig. 3 becomes smaller.

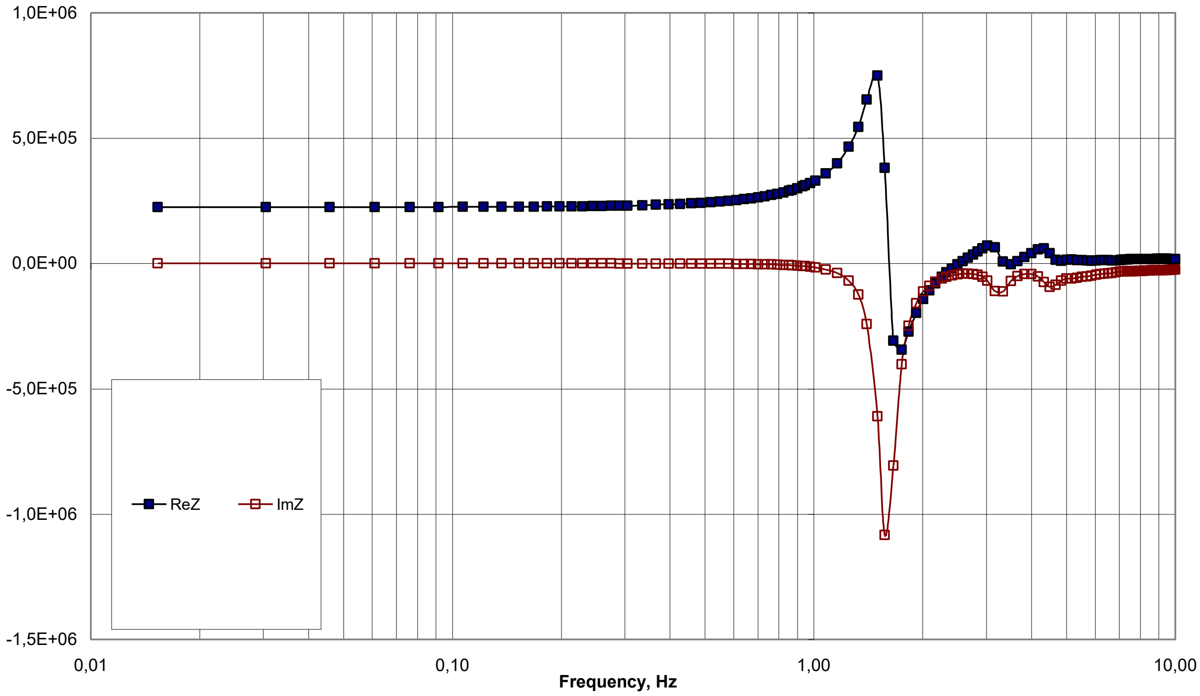

Fig. 3 - Dynamic inertia in vertical direction

Fig. 4 shows dynamic inertia in the two horizontal directions. Generally, the behavior is the same as in Fig. 3, but the principal resonant frequencies are lower. Fig. 5 shows dynamic inertia in the two rocking directions. Generally, the behavior is the same as in Fig. 3, but the principal resonances are more important as compared to the further resonances.

Now let us calculate soil impedances. This was done by SASSI program [10]. Fig. 6 shows horizontal impedances (they are similar in the two horizontal directions, as our base mat has quadratic shape) and vertical impedance. These are three first diagonal terms of matrix $C_s(\omega)$. Fig. 7 shows rotational impedances – rocking (again similar in two rocking planes) and torsion. These are the last three diagonal terms of the impedance matrix.

The next step in CAM is the calculation of the transfer functions from U_0 to U in the frequency domain from Eq. (3). In the NPP design they use not only the basic soil profile, but also two additional profiles to account for the uncertainty in soil data. Elasticity modules in the additional profiles differ from those in the basic profile 1.5 times up and down (so, the additional profiles are called “soft” and “hard” ones). In terms of the wave velocities the difference is 1.225 times. Fig. 8 shows absolute values of the transfer functions X(X) – i.e. both one-component excitation and response are along X-axis.

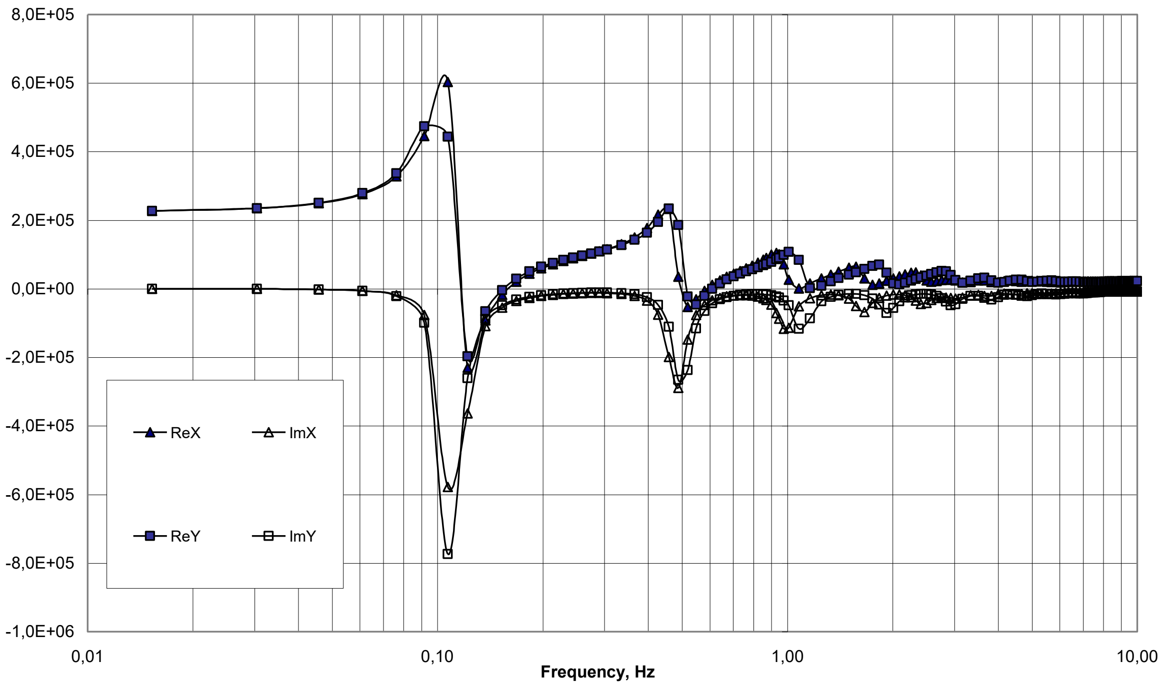

Fig. 4 - Dynamic inertia in horizontal directions

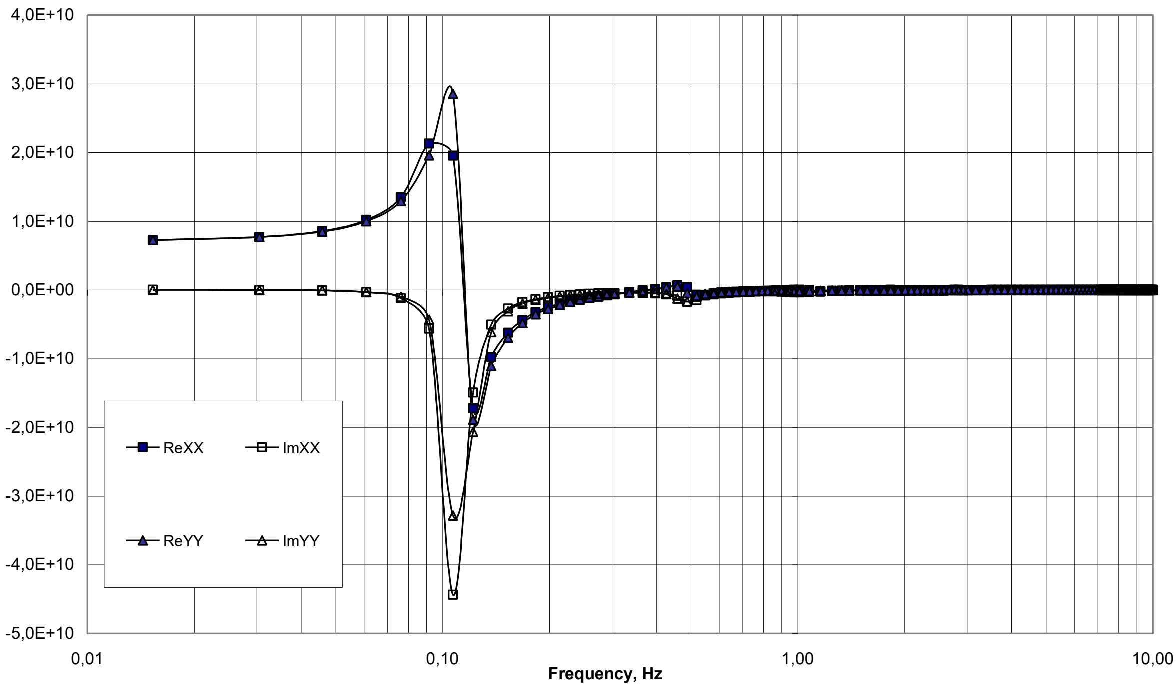

Fig. 5 - Dynamic inertia in rocking planes

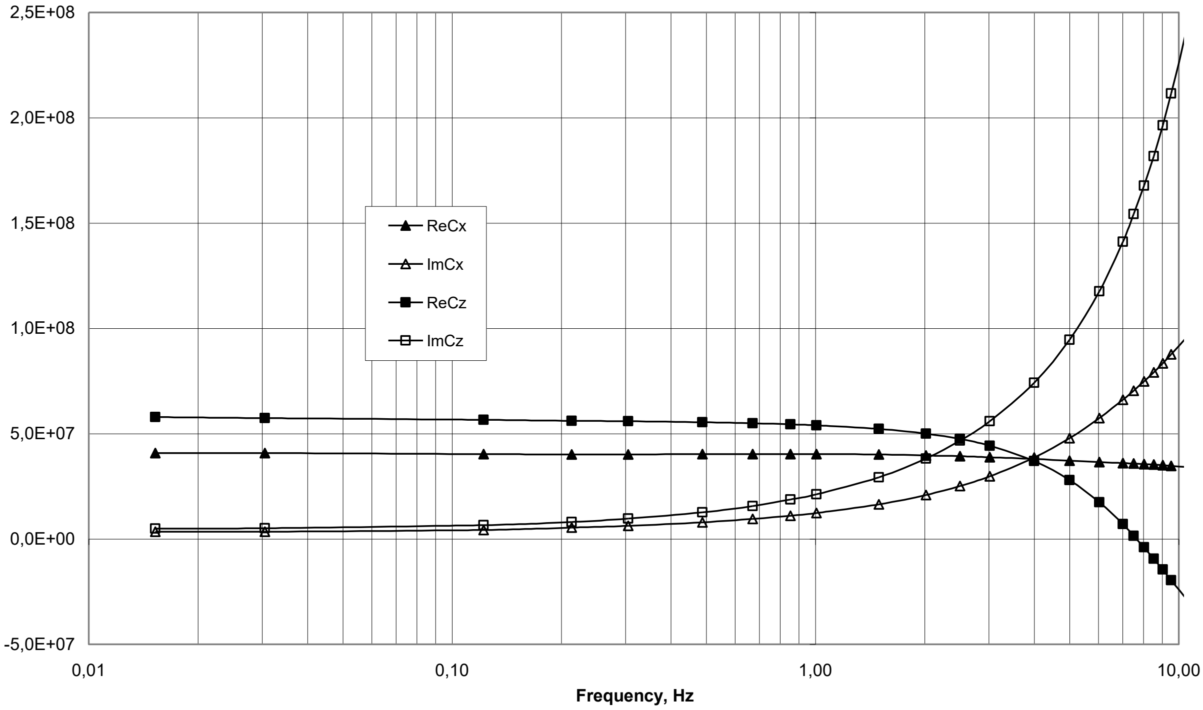

Fig. 6 - Translational impedances in two directions

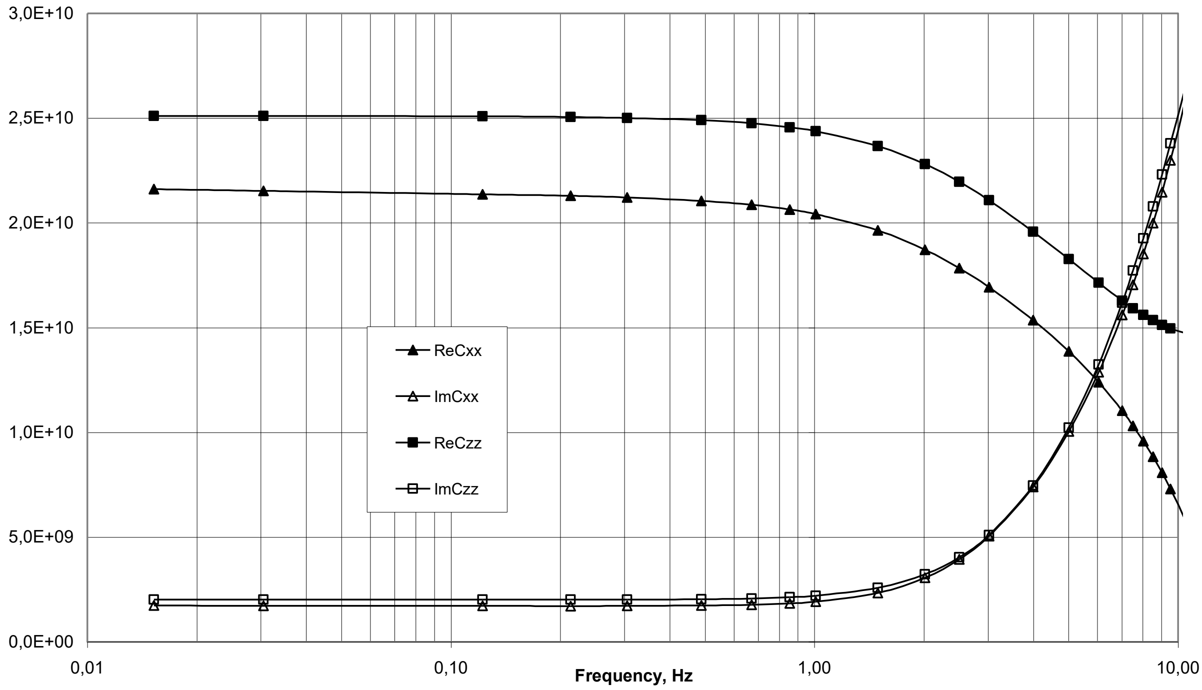

Fig. 7 - Rotational impedances – rocking and torsion

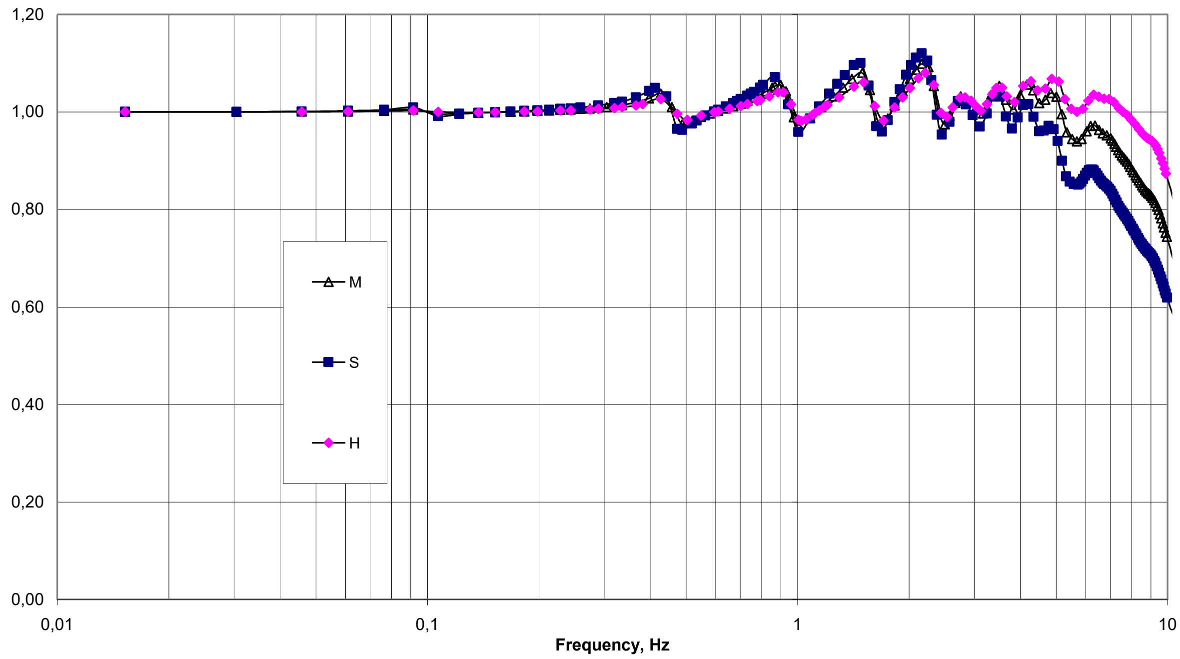

Fig. 8 - Absolute values of the transfer functions X(X)

Fig. 9 shows the transfer functions YY(X) – from the translational motion in the free-field along X-axis to the rocking around Y-axis (i.e. in the vertical plane OXZ).

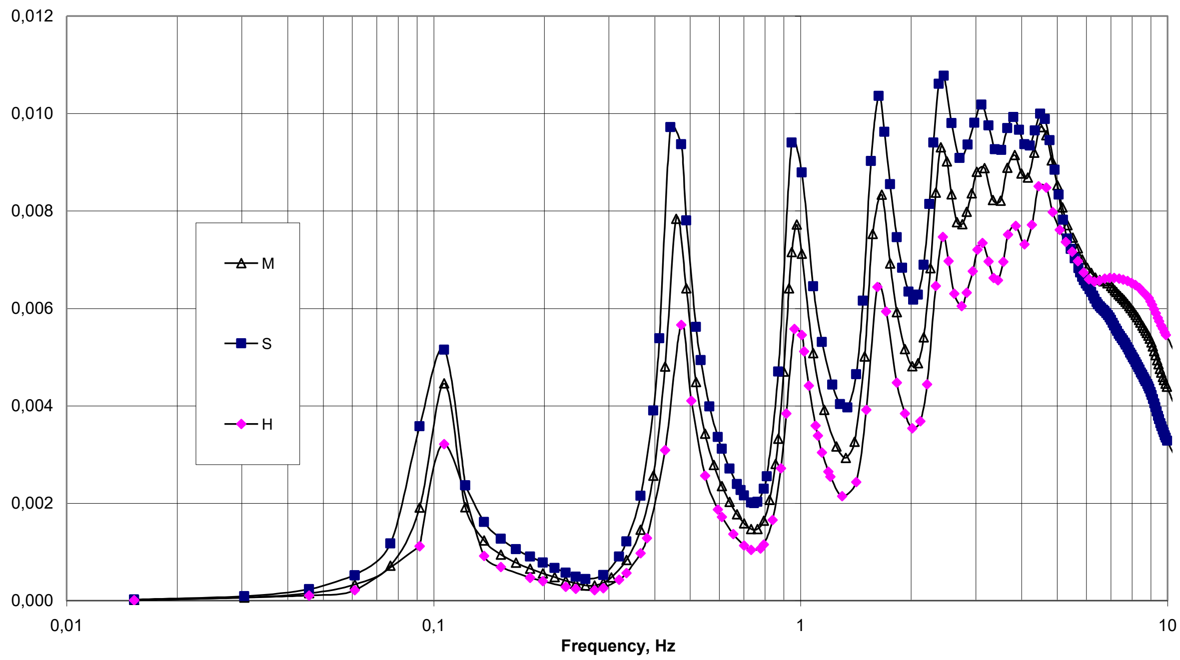

Fig. 9 - Absolute values of the transfer functions YY(X)

Without SSI these transfer functions would be unit for X(X) and zero for YY(X). We see that the five first SSI resonances impact the response in the increasing scale, despite the decreasing impact in Fig. 2. The most "influential" resonance is the fifth one at 2.4 Hz. This is different from what we used to see in the NPP design. The softer is the soil, the more pronounced is the SSI effect – this is common for the NPP design.

Fig. 10 shows the transfer functions Z(Z). Here we see the picture we always see for the NPP design: the first resonance is a dominant one.

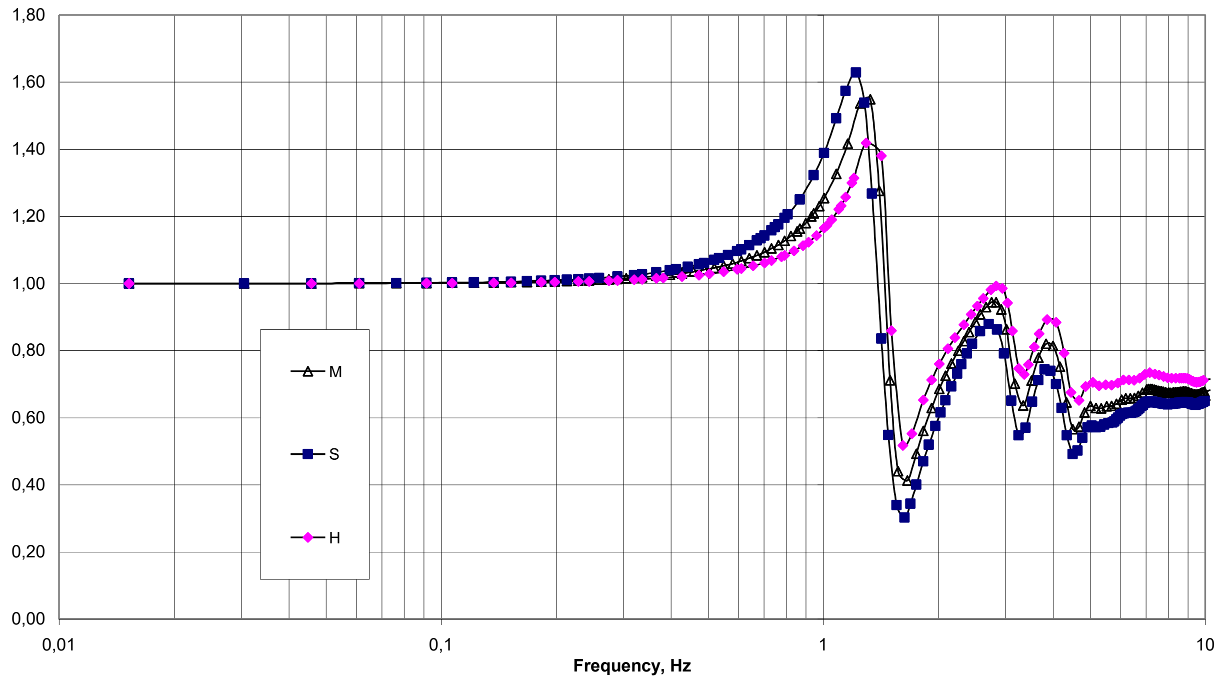

Fig. 10 - Absolute values of the transfer functions Z(Z)

The next stage in the SSI analysis by CAM is the base mat response calculation in the time domain. We will use three-component excitation time-history for the deep soil site. Fast Fourier Transform (FFT) is used to calculate the response from the excitation and the transfer functions. Fig. 11 shows response spectra (RS) for the 7% damping in oscillators along X-axis in the center of the base mat for three soil profiles and for the excitation. Along Y-axis the picture is the same. As we see in Fig. 11, RS at the mat are almost similar to those of the excitation, which means no SSI in the horizontal directions. However, in vertical direction we see another picture shown in Fig. 12. This picture is typical for the NPP designers. Fig. 13 shows rotational RS for rocking in the OXZ plane. At the edges vertical accelerations due to this rocking may be estimated as a product of the angular accelerations shown in Fig. 13 and half of the mat dimensions (i.e. 20 m in our case). So, the impact is considerable.

Table 1 shows maximal response accelerations in the center of the base mat.

Table 1 – Maximal response accelerations in the center of the base mat

Soil	X	Y	Z	XX	YY	ZZ
Medium	4,4672	4,4764	2,7123	0,33152E-1	0,28545E-1	0,90462E-3
Soft	4,3615	4,4306	2,3787	0,35942E-1	0,32532E-1	0,12283E-2
Hard	4,5110	4,5362	3,0297	0,28479E-1	0,24027E-1	0,61282E-3
Fixed-base	4,4136	4,5192	3,4364	0	0	0

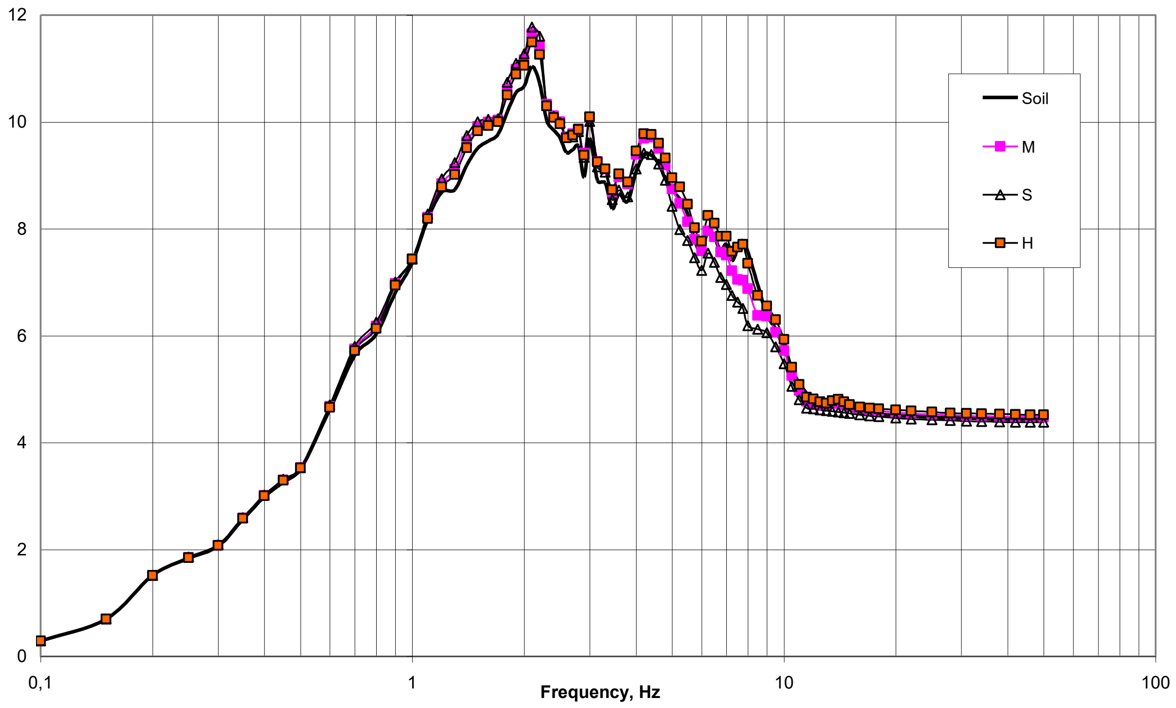

Fig. 11 – Response spectra along X-axis in the center of the base mat for medium (M), soft (S) and hard (H) soils as compared to the input RS (Soil)

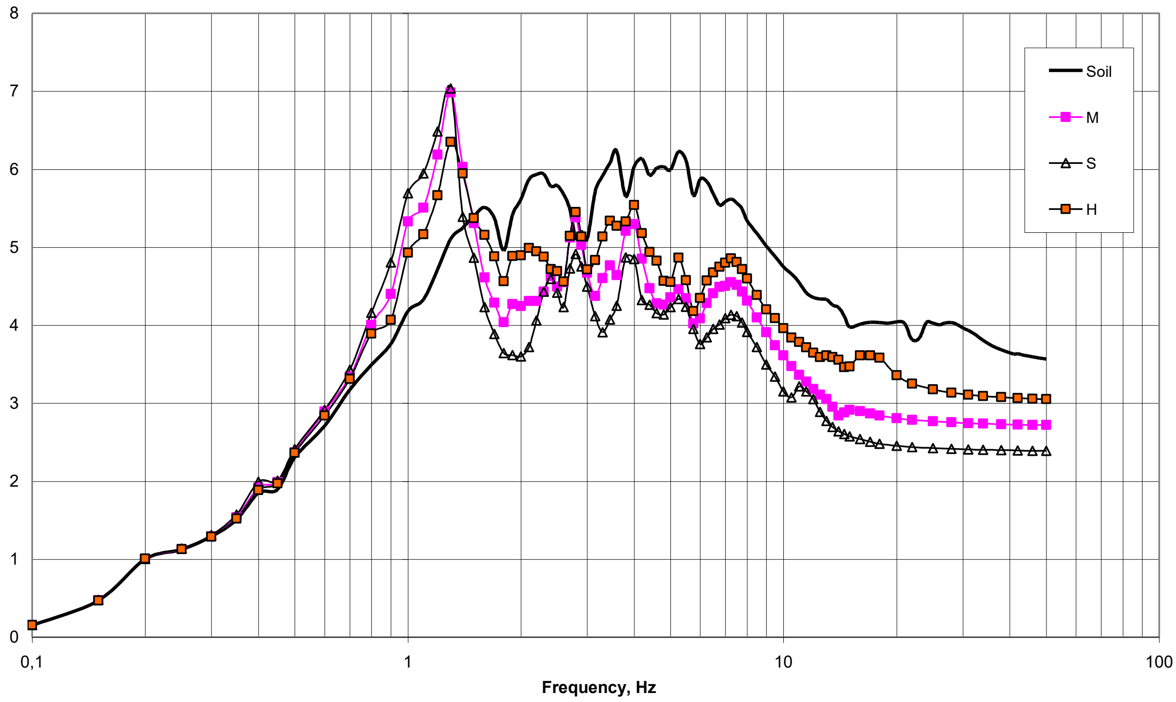

Fig. 12 – Response spectra along Z-axis in the center of the base mat for medium (M), soft (S) and hard (H) soils as compared to the input RS (Soil)

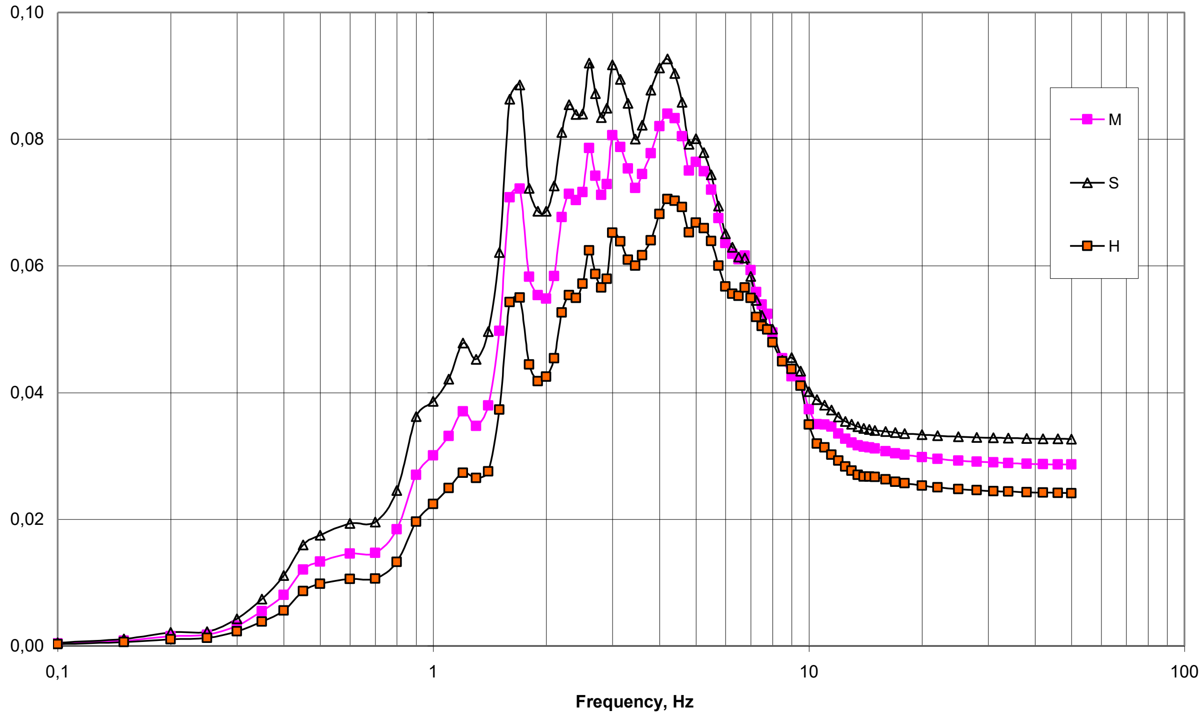

Fig. 13 – Response spectra around Y-axis at the base mat for medium (M), soft (S) and hard (H) soils

In parallel with base motion CAM enables calculation of the integral forces under the base mat. They are shown in Table 2.

Table 2 – Maximal integral forces under the center of the base mat

Soil	X	Y	Z	XX	YY	ZZ
Medium	0,20713E6	0,24391E6	0,78674E6	0,10259E8	0,10500E8	0,68797E5
Soft	0,20232E6	0,23229E6	0,74783E6	0,91761E7	0,99954E7	0,66400E5
Hard	0,22761E6	0,25129E6	0,85553E6	0,10978E8	0,11307E8	0,70847E5
Fixed-base	0,27400E6	0,26882E6	0,84750E6	0,11914E8	0,12147E8	0,72057E5

4. Discussion

The first remarkable difference between NPP and civil building is that the soil flexibility impacts the structural response of the civil building differently in different directions. In the vertical direction SSI considerably changes the response base motion as compared to the free-field motion (which is common for the NPP buildings). However, the horizontal response base motion is almost similar to the free-field motion (unlike the NPP buildings). Surely, rocking response base motion appears (no rocking in the free-field motion).

In the nuclear Standards [3] there is a criterion to estimate the importance of SSI. Two different models are considered. The first one is the fixed-base model discussed above. The second one is the same model, but rigid and resting on the so-called “soil” springs (six springs in six directions: three translational and three rotational ones). The first eigenfrequency of the first model is compared to the first eigenfrequency of the second model: if the “fixed-base” eigenfrequency is less than the “rigid” eigenfrequency over two times, the SSI effects are of no importance.

This criterion was checked for the sample high-rise building, but separately for the vertical and for horizontally-rocking modes. Stiffness of the soil springs was calculated for the quadratic rigid stamp resting on the surface of the homogeneous half-space according to the formulae given in [2, 3, 4]. Omitting the details the first “vertical rigid” eigenfrequency proved to be 2.38 Hz. One should compare it to the first “vertical fixed-base” eigenfrequency equal to 1.587 Hz (see Fig. 2). Though the latter is less than the former, the difference is just about 1.5 times – less than 2.0. It means that SSI is important according to this criterion. We can check it ourselves – looking at Fig. 12.

For horizontally-rocking modes the first “rigid” frequency proved to be 0.26295 Hz (a bit less than the partial “rocking rigid” eigenfrequency, accounting to the elevation of the gravity center of the high-rise building). But the first “fixed-base” eigenfrequency was only 0.11179 Hz (see Fig. 2). The ratio is 0.26295/0.11179=2.352 which is greater than 2.0. It means that in the horizontal direction SSI is not so important. Again we can check it looking at Fig. 11.

The second remarkable fact is that the first fixed-base horizontally-rocking modes of the high-rise building have very small eigenfrequencies (about 0.1 Hz), which are far less than the dominant frequencies of seismic excitation (about 2 Hz). Relative response motion in the first modes goes in a counter-phase with excitation, which makes absolute modal displacements very small. As a result, modal masses of these modes do not contribute to the integral forces under the base. Horizontal integral forces are far less than the vertical forces (see Table 2), while horizontal base accelerations are greater that vertical ones (see Table 1).

The same note about the modest role of the first horizontally-rocking modes can be seen also in the linear-spectral method: the first horizontal modes contribute less than subsequent modes, because great modal participation factors for them are multiplied by comparatively small spectral accelerations of excitation at low frequencies.

However, the comparison of tome-domain calculations with linear-spectral results shows that these low-frequency spectral responses should be combined over different modes directly (and not by SSRS rule), as inter-correlation between them proves to be high.

From all the components of the soil flexibility the rocking flexibility proves to be the most important for high-rise building. Rocking response base motion goes in the counter-phase with translational horizontal response base motion, decreasing horizontal integral forces under the base. Generally, all integral forces decreased as compared to the fixed-base analysis - so, the fixed-base analysis is conservative in terms of the integral forces.

However, the author would like to stress that the sample system considered had surface base mat, which is unrealistic for high-rise buildings. For the embedded buildings soil springs would be stiffer. The above mentioned remarks for the embedded buildings would refer to the “inertial SSI” – the difference between the weightless base motion without the upper structure and the base motion with the upper structure. As to the “kinematical SSI” (i.e. the difference between the free-field motion and the weightless base motion), it should be calculated separately.

5. Conclusions

The first conclusion is that the soil flexibility impacts the structural response differently in different directions. In the vertical direction SSI considerably changes the response base motion as compared to the free-field motion (which is common for the NPP buildings). However, the horizontal response base motion is almost similar to the free-field motion (unlike the NPP buildings). Surely, rocking response base motion appears (no rocking in the free-field motion). The SSI criteria used in the nuclear field confirm that SSI is important in the vertical direction, but not in the horizontal direction.

The second conclusion is that the first fixed-base horizontally-rocking modes have very small eigenfrequencies (about 0.1 Hz), which are far less than the dominant frequencies of seismic excitation (about 2 Hz). Relative response motion in the first modes goes in a counter-phase with excitation, which makes absolute modal displacements very small. As a result, modal masses of these modes do not contribute to the integral forces under the base. Horizontal integral forces are far less than the vertical forces, while horizontal base accelerations are greater that vertical ones.

The third conclusion is that in the linear-spectral method the first horizontal modes contribute less than subsequent modes, because great modal participation factors for them are multiplied by comparatively small spectral accelerations of excitation at low frequencies. However, these low-frequency spectral responses should be combined over different modes directly (and not by SSRS rule), as inter-correlation proves to be high.

From all the components of the soil flexibility the rocking flexibility proves to be the most important for high-rise building. Rocking response base motion goes in the counter-phase with translational horizontal response base motion, decreasing horizontal integral forces under the base. Generally, all integral forces decreased as compared to the fixed-base analysis - so, the fixed-base analysis is conservative in terms of the integral forces.

The sample system considered had surface base mat, which is unrealistic for high-rise buildings. For the embedded buildings soil springs would be stiffer. The above mentioned conclusions for the embedded buildings would refer to the “inertial SSI” – the difference between the weightless base motion without the upper structure and the base motion with the upper structure. As to the “kinematical SSI” (i.e. the difference between the free-field motion and the weightless base motion), it should be calculated separately.

6. Acknowledgements

The author is grateful to V.S. Mikhailov (SCAD SOFT) and A.S. Toporkov (Atomenergoproject) for the preparation and modal analysis of the fixed-base model of the sample high-rise building.

7. References

[1] Alexander Tyapin (2012): *Soil-Structure Interaction*, Earthquake Engineering, Halil Sezen (Ed.), ISBN: 978-953-51-0694-4, InTech, Available from: http://www.intechopen.com/books/earthquake-engineering/soil-structure-interaction. Chapter 6. Pp.145-178.

[2] *Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Analysis of Safety-Related Nuclear Structures. ASCE4-86*. Reston, Virginia, USA. 1987

[3] *Seismic Analysis of Safety-Related Nuclear Structures and Commentary. ASCE4-98.* Reston, Virginia, USA. 1999

[4] *Seismic Analysis of Safety-Related Nuclear Structures and Commentary. ASCE/SEI 4-16.* Reston, Virginia, USA. 2017

[5] Tyapin AG (2010): Combined Asymptotic Method for Soil-Structure Interaction Analysis // *Journal of Disaster Research.* Vol. 5. No.4. Pp.340-350. Aug.2010.

[6] Tyapin AG (2019): *Soil-Structure Interaction In Seismic Analysis*. Stockholm, ASV Construction. 2019. https://play.google.com/store/books/details?id=Q8acDwAAQBAJ

[7] Tyapin A (2019): Check Points for Verification of SSI Analysis in Combined Asymptotic Method// *SMiRT25*, Charlotte, NC, USA, 4-9 August 2019.

[8] Karpilovsky VS et al. (2018): *SCAD++. Version 21.1.7.1. Assembling 27.03.2018*.

[9] *ABAQUS. Version 6.8*. Dassault Systèmes Simulia Corp., Providence, RI, USA. 2008.

[10] Lysmer J., Tabatabaie RM, Tajirian F, Vahdani S, Ostadan F (1981): SASSI - A System for Analysis of Soil-Structure Interaction. *Research Report GT 81-02.* University of California, Berkeley.

