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MODAL ANALYSIS OF SOIL-STRUCTURE INTERACTION USING COMBINED ASYMPTOTIC METHOD

Alexander Tyapin
JSC "Atomenergoproject", Moscow, Russia
Email: atyapin@bvcp.ru

ABSTRACT

This paper demonstrates the difficulties in the modal approach to soil-structure interaction (SSI) analysis caused by heterogeneous damping in the platform soil-structure model (one damping in the structure, another one – in the "soil dashpots"). In such a system modal responses do not part (i.e., remain coupled in the generalized coordinates through the off-diagonal terms of damping matrix), so the conventional modal approach gives wrong results. The attempt to compensate missing off-diagonal damping terms by the additional cut-down of the remaining diagonal terms of damping matrix in the generalized coordinates also gives poor results. The author proposes to use combined asymptotic method (CAM) to return to the homogeneous damping in the system and thus enable modal approach.

Keywords: soil-structure interaction, impedance approach, modal analysis.

INTRODUCTION

Soil-structure interaction (SSI) controls structural seismic response in different situations - not only for heavy structures like NPP reactor buildings, but also for embedded structures, for structures on piles, etc. (Wolf, 1985; ASCE4-98).

The key attribute of soil in SSI is wave propagation: wave effects change the effective dynamic stiffness of soil foundation towards basement, including effective damping. Wave damping (i.e. wave propagation from the moving basement, taking energy away) is added to the conventional internal damping in the soil medium, but can be far greater in scale.

The simplest approach to SSI is the so-called "impedance approach" assuming the rigidity of the basement. The resulting model of SSI system is a structural model with rigid basement, placed on a rigid platform via the so-called "soil springs and dashpots". Seismic excitation is usually applied to the platform as a kinematical excitation.

Generally "soil springs and dashpots" compose in the frequency domain a frequency-dependent complex dynamic stiffness matrix 6 x 6. However, very often they simplify this matrix, leaving only diagonal complex terms and making them as simple as possible – with frequency-independent real parts and frequency-proportional imaginary parts, corresponding to six pairs of "soil springs and dashpots" in SSI mechanical model. Such a simplification can be sometimes justified for homogeneous soil half-space.

However, even for such a simple model the dynamic analysis is not obvious. Direct integration meets no difficulties with viscous dashpots; however, structural damping needs special consideration, and the most popular Rayleigh approach often leads to the excessive conservatism.

Common alternative to direct integration is modal approach. First, it often helps to save computational resources, especially with very detailed models common nowadays. Second, modal approach is the base for spectral method of seismic structural analysis, widely used in standards.

However, conventional modal approach has important limitations, not always understood by engineers. The goal of the paper is to discuss some of these limitations and proposals how to keep modal approach valid even for soil-structure models.

LIMITATIONS OF CONVENTIONAL MODAL APPROACH

Let us start with well-known definition of the modal approach. Basic equation for the seismic analysis of linear system resting on the moving rigid platform is for one-component excitation as follows (e.g., see ASCE4-98)

$$[M]\{\ddot{X}\}+[C]\{\dot{X}\}+[K]\{X\}=-[M]\{U_g\}\ddot{u}_g \tag{1}$$

where

$[M]$ = mass matrix (n x n);

$[C]$ = damping matrix (n x n);

$[K]$ = stiffness matrix (n x n);

$\{X\}$ = column vector of relative displacements (n x 1);

$\{\dot{X}\}$ = column vector of relative velocities (n x 1);

$\{\ddot{X}\}$ = column vector of relative accelerations (n x 1);

$\{U_g\}$ = column vector of influence; vector of nodal "rigid" displacements due to the static unit platform displacement in the direction of seismic excitation (n x 1);

n = number of dynamic degrees of freedom;

$\ddot{u}_g$ = platform acceleration in the selected direction of excitation.

Modal superposition is described by

$$\{X\}=[\Phi]\{Y\} \tag{2}$$

where

$[\Phi]$ = is a mode matrix of size (n x m); comments are given below;

$\{Y\}$ = vector of relative displacement in generalized coordinates (m x 1);

m = number of modes considered.

Natural modes $[\Phi]$ have two valuable special properties. First of all, different modes (i.e. columns of $[\Phi]$) are "orthogonal by masses", enabling convenient scaling to unit matrix E (the so-called "normalization by masses")

$$[\Phi]^T[M][\Phi]=E \tag{3}$$

Besides, different modes are also "orthogonal by stiffness"; as normalization (3) has been already performed, the resulting matrix in the right-hand part of the analogue of (3) will be non-unit, but carrying natural frequencies ω_j^2 (j=1,…,m) instead:

$$[\Phi]^T[K][\Phi] = diag[\omega_j^2] \tag{4}$$

Putting (2) into (1) and multiplying (1) by $[\Phi]^T$ from the left, one comes to the equation of motion in generalized coordinates:

$$[\Phi]^T[M][\Phi]\{\ddot{Y}\} + [\Phi]^T[C][\Phi]\{\dot{Y}\} + [\Phi]^T[K][\Phi]\{Y\} = -[\Phi]^T[M]\{U_g\}\ddot{u}_g \tag{5}$$

Putting (3) and (4) into (5) we come to

$$\{\ddot{Y}\} + [\Phi]^T[C][\Phi]\{\dot{Y}\} + diag[\omega_j^2]\{Y\} = -[\Phi]^T[M]\{U_g\}\ddot{u}_g \tag{6}$$

Different equations in the system (6) are coupled only through the off-diagonal terms of the second matrix $[\Phi]^T[C][\Phi]$ in the left-hand part. If these off-diagonal terms are comparatively small and may be neglected, (6) leads to the well-known set of independent 1D equations of motion

$$\ddot{Y}_j + 2\lambda_j\omega_j\dot{Y}_j + \omega_j^2 Y_j = -\Gamma_j\ddot{u}_g \tag{7}$$

where

Y_j = scalar "modal coordinate j";

λ_j = modal damping coefficient j, as part of critical damping;

ω_j = natural frequency j (rad/s);

Γ_j = modal participation factor j, equal to

$$\Gamma_j = \frac{\{\Phi_j\}^T[M]\{U_g\}}{\{\Phi_j\}^T[M]\{\Phi_j\}} = \{\Phi_j\}^T[M]\{U_g\} \tag{8}$$

These are briefly the fundamentals of conventional modal approach. 1D equations (7) are convenient to solve; spectral method is based on that.

One can note the principal limitation: to keep modal approach applicable damping matrix [*C*] must provide small off-diagonal terms in the generalized coordinates.

For homogeneous systems (e.g., made fully of steel or fully of reinforced concrete) damping is an attribute of material, thus linked to stiffness. In the frequency domain this fact is modeled by the addition of imaginary part to the material elasticity module leading to complex natural frequencies in (4) but with the same "undamped" natural modes. Off-diagonal terms of damping matrix in the generalized coordinates remain zeroes, so responses along different modes remain uncoupled. However, material damping is not viscous, i.e. force is not proportional to the velocity as in (1). Hence, it is not directly applicable in the time domain.

In the time domain with viscous damping we can also link damping matrix [*C*] to stiffness matrix [*K*] and get uncoupled modal responses, but then according to (4) and (7) modal damping coefficient λ_j appears to be proportional to natural frequency ω_j. However, experiments show that for homogeneous systems modal damping coefficients λ_j are similar, i.e. do not depend on *j*.

Such a similarity may be achieved in analysis via several options. The simplest option is just to forget about matrix [*C*] and to use modal equations (7), directly setting up similar λ_j depending only on material (and sometimes on the intensity of excitation also). Such option is proposed, for example, by ASCE4-98.

If we still need matrix [*C*] for some purpose, another common option is to use Rayleigh model

$$[C] = \alpha\,[M] + \beta\,[K] \tag{9}$$

Due to (3) and (4) we get with (9) the uncoupled modal responses, though modal damping coefficients are not similar (i.e. they depend on *j*):

$$\lambda_j = \frac{\alpha}{2\omega_j} + \frac{\beta\,\omega_j}{2} \tag{10}$$

Adjusting α and β in (9, 10) one can achieve “target values” of damping coefficient λ_j at two particular frequencies ω_j. Common approach in seismic analysis is to set up two frequencies limiting “frequency domain of seismic response” (e.g., 3 Hz and 20 Hz) and require that for these frequencies formula (10) gives damping values prescribed by material, as mentioned above. Then for all natural frequencies between the prescribed two ones formula (10) will give conservative (i.e., less than prescribed) modal damping coefficients. Note that this more or less satisfactory result is achieved by non-physical tool: the first term in (9) (the so-called “external damping”) unlike the second term has no physical meaning. This is a pure mathematics.

Now let us turn to the non-homogeneous systems, composed of several different homogeneous subsystems. Matrix [*C*] for such system must be composed of matrices [*C*] for subsystems in the same way as matrices [*M*] and [*K*]. However, this can break the uncoupling of the modal responses. Let us illustrate this effect using a very simple sample 1D model with 3 DOFs shown (without damping) in Fig.1.

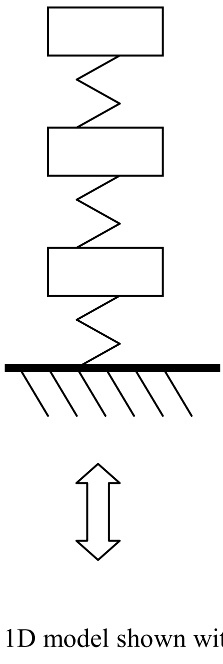

Fig.1. Sample 1D model shown without damping

Parameters of the model are given in the Table 1 together with natural frequencies and natural modes of the fixed-base system.

Table 1. Parameters of the sample model and natural frequencies/modes of the fixed-base SSI system

Number from below	Mass, tones	Stiffness of springs, kN/m	Natural frequencies, Hz	Modal displacement of the lower mass, $t^{-1/2}$	Modal displacement of the middle mass, $t^{-1/2}$	Modal displacement of the upper mass, $t^{-1/2}$
1	1	1.3E3	3.0025	0.4703	0.6069	0.6407
2	1	3.25E3	11.297	0.8535	-0.1281	-0.5051
3	1	6.75E3	20.073	-0.2245	0.7844	-0.5782

Natural frequencies in Table 1 are typical for SSI models of the NPP reactor buildings in vertical direction. Let the lower spring in Fig.1 represent "soil stiffness" and let other two springs together with three masses represent structure. Then we can use formula from ASCE4-98 linking coefficient of viscosity c_z for soil dashpot to the coefficient k_z of soil stiffness

$$c_z / k_z = 0{,}85 r / V_s \tag{11}$$

Here r is equivalent radius of the base mat, V_s is shear wave velocity in the soil. For further calculations r=40 m, V_s=400 m/s, so soil viscosity coefficient in kN/(m/s) is

$$c_z = k_z \times (0{,}85 r / V_s) = 1{,}3 \times 10^3 \times 0{,}85 \times 40 / 400 = 110{,}5 \tag{12}$$

Viscosity coefficient c_z should be directly added to the term (1,1) of matrix [C].

Let us model structural portion of damping using Rayleigh approach with two boundary frequencies f_b=3 Hz and f_e =20 Hz. Let us take "target" value of damping λ=0.05. Then Rayleigh coefficients are calculated as

$$\alpha = 4\pi\, \lambda \,\frac{f_b \; f_e}{f_b + f_e} = 1{,}639\ s^{-1}$$

$$\beta = \lambda \,\frac{1}{\pi\,(f_b + f_e)} = 6{,}91978 \times 10^{-4}\, s \tag{13}$$

Structural part [C_{str}] of damping matrix [C] is given by (9):

$$[C_{str}] = 1{,}639 \times \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix} + 6{,}91978 \times 10^{-4} \times \begin{bmatrix} 3{,}25 \times 10^3 & -3{,}25 \times 10^3 & 0 \\ -3{,}25 \times 10^3 & 10{,}0 \times 10^3 & -6{,}75 \times 10^3 \\ 0 & -6{,}75 \times 10^3 & 6{,}75 \times 10^3 \end{bmatrix} =$$

$$= \begin{bmatrix} 3{,}8879 & -2{,}2489 & 0 \\ -2{,}2489 & 8{,}5588 & -4{,}6709 \\ 0 & -4{,}6709 & 6{,}3099 \end{bmatrix} \tag{14}$$

After the addition of the soil viscosity coefficient (12) we get full damping matrix

$$[C] = \begin{bmatrix} 114{,}3879 & -2{,}2489 & 0 \\ -2{,}2489 & 8{,}5588 & -4{,}6709 \\ 0 & -4{,}6709 & 6{,}3099 \end{bmatrix} \tag{15}$$

Let us transfer it to the generalized coordinates using matrix of modes taken from Table 1:

$$[\Phi] = \begin{bmatrix} 0{,}4703 & 0{,}8535 & -0{,}2245 \\ 0{,}6069 & -0{,}1281 & 0{,}7844 \\ 0{,}6407 & -0{,}5051 & -0{,}5782 \end{bmatrix} \tag{16}$$

In generalized coordinates we get

$$[\Phi]^T\,[C]\,[\Phi] = \begin{bmatrix} 26{,}127 & 43{,}994 & -11{,}572 \\ 43{,}994 & 84{,}965 & -21{,}001 \\ -11{,}572 & -21{,}001 & 18{,}170 \end{bmatrix} \tag{17}$$

Conventional modal approach requires neglecting the off-diagonal terms of this matrix. Let us estimate the error arising from using this approach via the comparison of transfer functions $\{TF\}$ from 1D platform motion to the absolute displacements of masses in Fig.1:

$$\{TF\} = \{-\omega^2\,[M] + i\omega\,[C] + [K]\}^{-1} \begin{bmatrix} k_z + i\omega c_z \\ 0 \\ 0 \end{bmatrix} \tag{18}$$

In (14) we shall change matrix $[C]$ and look at the absolute values of the results (18). The most "physical" curve is that for material damping, where $i\omega[C_{str}\ ]$ is replaced in (18) by $2i\lambda[K]$ and soil viscosity term in the frequency domain $i\omega c_z$ is added to the term (1,1) of matrix $i\omega[C]$.

Another curve will be with "Rayleigh structure and viscous soil" damping using (15).

The third variant will be conventional "modal full" damping where instead of (18) the following formulae are used:

$$\{TF\} = [1\quad 1\quad 1]^T + [\Phi]\{Y\},\quad Y_j = \Gamma_j\omega^2[-\omega^2 + i\omega\,c_j + \omega_j^2]^{-1} \tag{19}$$

Here modal damping parameters c_j are taken from the diagonal of matrix (17). According to (7) they correspond to the following dimensionless parameters $\lambda_j = c_j/(2\omega_j)$: λ_1=0,6925; λ_2=0,5985; λ_3=0,0720. We see that the first two coefficients are far greater than "material" values of λ, usually making 4%...7%. This is typical for soil-structure interaction, where "wave" damping dominates over "material" one.

The fourth curve is called "cut-down modal damping". It was obtained in the similar way as the previous curve, but in (19) values of c_j (j=1,2) were artificially cut down to $0.4\omega_j$ (corresponding to $\lambda_1=\lambda_2=0{,}2$); c_3 and λ_3 stay the same as before. Some specialists still believe that such cutting down can provide necessary conservatism. The results will be discussed below.

The results for the lower mass are shown in Fig.2; the results for the middle mass – in Fig.3, the results for the upper mass – in Fig.4.

First of all we see that the "Rayleigh damping" results are very close to the "material damping" results. Some difference can be seen, but it is small. The reason is that soil damping is represented in full in both variants, and it is more important in our case than structural damping.

The "modal damping" results are satisfactory only for low frequencies. For upper frequencies they are considerably non-conservative.

Absolute Values of Transfer Functions From Platform to Lower Mass

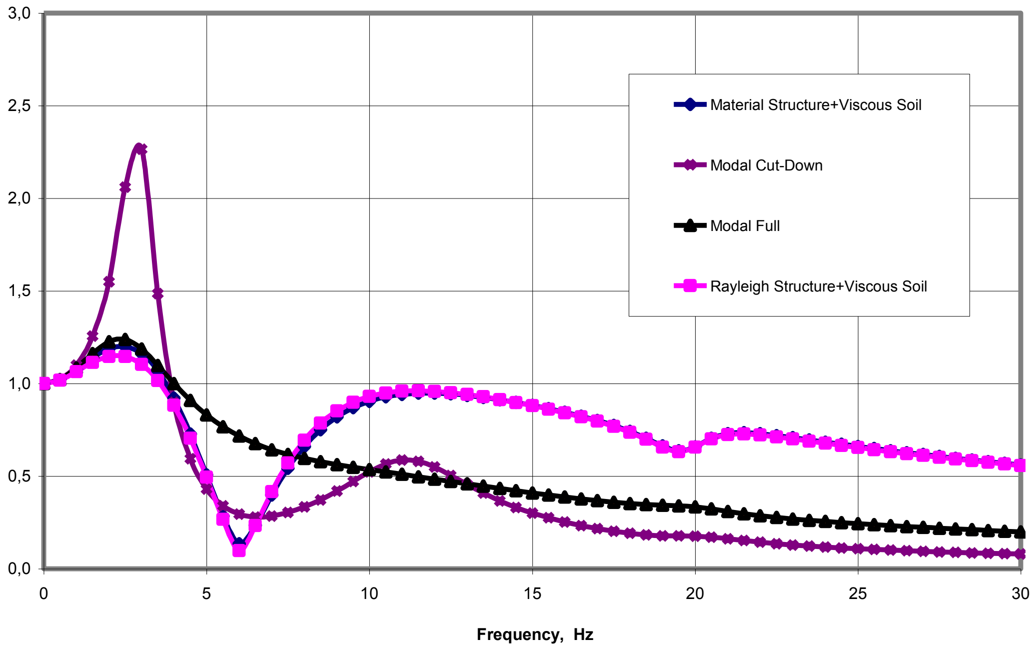

Fig.2. Absolute values of transfer functions from platform to the lower mass

Absolute Values of Transfer Functions From Platform to Middle Mass

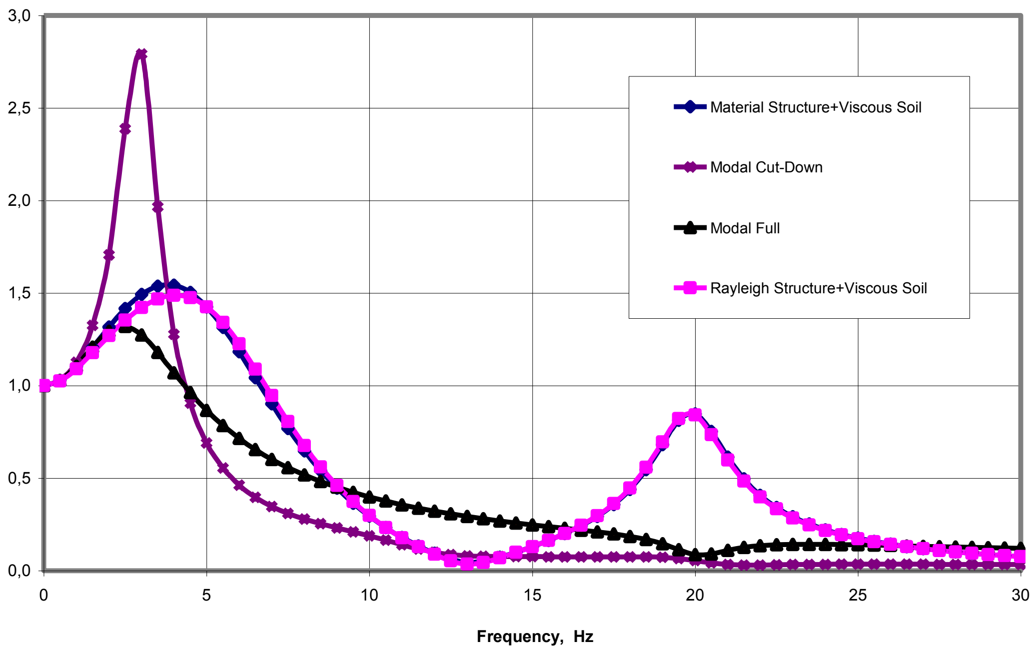

Fig.3. Absolute values of transfer functions from platform to the middle mass

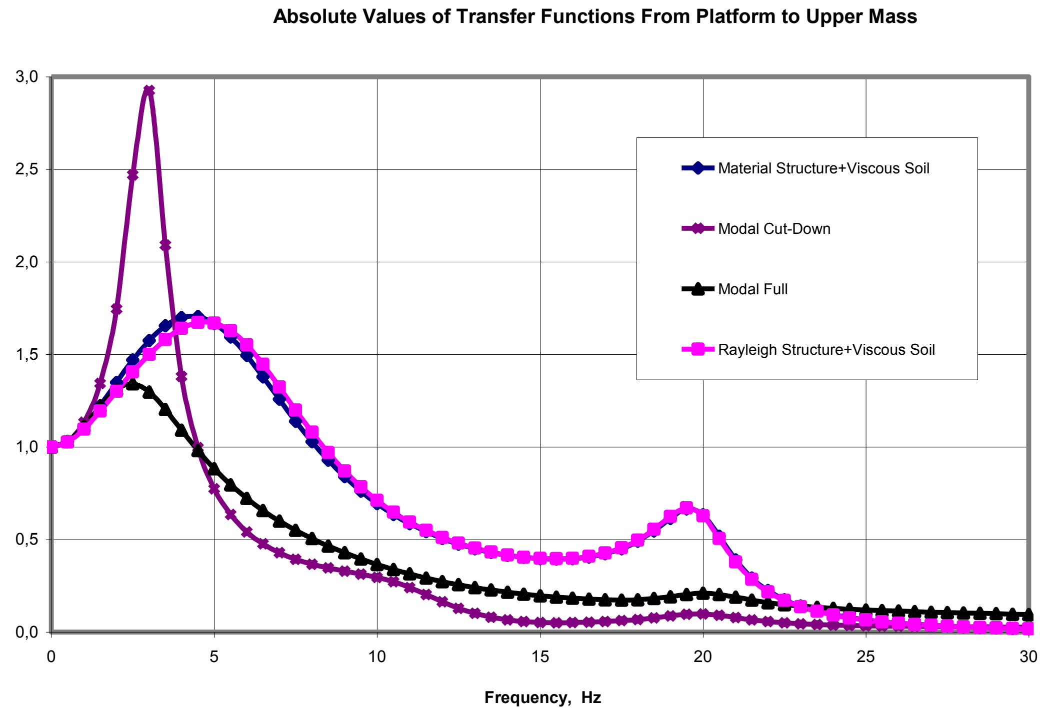

Fig.4. Absolute values of transfer functions from platform to the upper mass

The "cut down damping" results corresponding to the "limit" modal damping coefficient λ_{lim}=0.2, are also inappropriate: they are too conservative around the first peaks, but non-conservative for higher frequencies.

So, we see that for SSI platform models with heterogeneous damping the conventional modal approach gives non-conservative results. This is in line with the comments in ASCE4-98. The attempts to save conservatism by means of cutting down modal damping coefficients to "limit value" of 0.2 lead to excessive conservatism in lower frequency range and non-conservative results for the high frequencies.

3. COMBINED ASYMPTOTIC METHOD

Combined asymptotic method (CAM) was developed by the author some time ago (Tyapin, 2010; Tyapin, 2012). It is called "combined" because it combines frequency-domain and time-domain calculations. It is called "asymptotic" because the results are rigorous for rigid basements (surface and embedded ones) resting on flexible soil foundations, and they are approximate for flexible basements.

The first principal step of CAM is the evaluation of the so-called "dynamic inertia" of the upper structure resting on a rigid basement. It is a complex frequency-dependent matrix 6 x 6 linking basement motion $\{U_b\}$ in the frequency domain (different from platform motion $\{U_g\}$!) to the forces $\{R\}$ acting from the structure to the base:

$$\{R(\omega)\} = -\omega^2 [M(\omega)]\{U_b(\omega)\} \tag{20}$$

If the upper structure is completely rigid, "dynamic inertia" matrix $[M(\omega)]$ is similar to conventional "static inertia" matrix $[M_0]$: it is real and independent of frequency. For a flexible damped upper structure $[M(\omega)]$ becomes complex; it depends on frequency and structural damping. For the material damping in the upper structure the "dynamic inertia" is

$$[M(\omega)]=[M_0]+\sum_{j=1}^{n}\frac{\omega^2}{\omega_j^2-\omega^2+2i\lambda_j\omega_j^2}\{S_j\}^T\{S_j\} \qquad (21)$$

Here $[M_0]$ is the conventional real inertia matrix 6 x 6; i is an imaginary unit; ω_j is natural frequency of j-th mode for the fixed-base upper structure (different from that in (4)!); $\{S_j\}$ is a line 1 x 6 of the participation factors for the j–th mode (the inertial normalization of the natural modes is assumed); λ_j is the j-th modal damping coefficient calculated in the FEM codes along with natural frequencies/modes of the fixed-base structural model. We see that in static case (i.e. for zero frequency) dynamic inertia is similar to the conventional “static” inertia even for flexible structure.

In our sample 1D model the fixed-base structure has two degrees of freedom. The line of participation parameters for each mode consists of a single element, so in our case “dynamic inertia” is just a “dynamic mass”.

Table 2 shows basic modal parameters of the fixed-base sample structure.

Table 2. Natural frequencies/modes of the fixed-base structure

Number	Natural frequencies, Hz	Modal displacement of the lower mass, $t^{-1/2}$	Modal displacement of the middle mass, $t^{-1/2}$	Modal displacement of the upper mass, $t^{-1/2}$	Participation factor, $t^{1/2}$
1	6.023	0.0	-0.6189	-0.7855	-1.4044
2	19.700	0.0	-0.7855	0.6189	-0.1666

Changing circular frequencies ω in (21) for ordinary frequencies f in Hz, we get for our sample structure

$$M(f)=3.0+\frac{f^2*1.4044^2}{6.023^2-f^2+2*0.05*6.023^2 i}+\frac{f^2*0.1666^2}{19.7^2-f^2+2*0.05*19.7^2 i} \qquad (22)$$

Real and imaginary parts of complex dynamic inertia are shown in Fig.5.

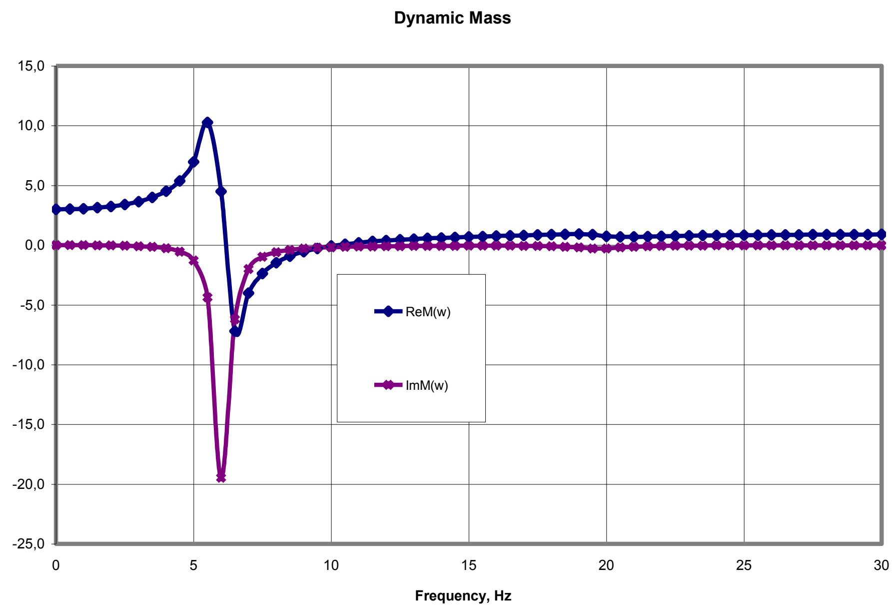

Fig.5. Real and Imaginary parts of “dynamic mass” for a sample structure

We see that the second "fixed-base" mode unlike the first one does not impact "dynamic mass" due to the comparatively small participation factor shown in Table 2.

The second step of CAM is also performed in the frequency domain. The equation of motion for weightless rigid "contact surface" is

$$\{[C_{soil}(\omega)]-\omega^2[M(\omega)]\}\{U_b(\omega)\}=[C_{soil}(\omega)][B(\omega)]\{U_0(\omega)\} \tag{23}$$

Here $[C_{soil}(\omega)]$ is an impedance (dynamic stiffness) matrix for rigid stamp on the flexible foundation (complex frequency-dependent matrix 6 x 6); $[B(\omega)]$ is a transformation matrix from free-field motion to the so-called "foundation input motion" for rigid stamp on the flexible foundation for certain type of seismic waves (complex frequency-dependent matrix 6 x 6, if there is a six-component excitation); $\{U_0(\omega)\}$ is seismic excitation displacement in the control point of free foundation in the frequency domain (complex frequency-dependent column matrix 6 x 1, if excitation is a six-component one); $\{U_b(\omega)\}$ is the response absolute displacement of the rigid base mat (complex frequency-dependent column matrix 6 x 1).

"Foundation input motion" is defined as motion of weightless rigid base of the same geometry as an actual one for the same seismic excitation. For the simplest but common case when rigid base mat is resting on the surface of soil foundation, seismic waves are vertical body waves in horizontally-layered media, and control point is in the centre of the mat in the free surface of the soil, $[B(\omega)]$ is a unit matrix (i.e., weightless rigid base moves similar to the free surface of the soil). In our sample model this is also the case; otherwise model in Fig.1 would not represent SSI.

Equation (23) enables obtaining transfer functions $TF(\omega)$ for the rigid base mat:

$$\{U_b(\omega)\}=[TF(\omega)]\{U_0(\omega)\};\quad [TF(\omega)]=\{[C_{soil}(\omega)]-\omega^2[M(\omega)]\}^{-1}[C_{soil}(\omega)][B(\omega)] \tag{24}$$

Transfer functions from the excitation displacements to the response displacements (24) are similar to those from the excitation accelerations to the response accelerations. These functions are the result of the second step in CAM. In our sample they will be exactly the same as curves for "material structure and viscous soil" damping in Fig.2. However, (24) is much more convenient than conventional general approach (18), because even for very detailed structural models we operate in (24) only with matrices 6 x 6.

After the second step of CAM one has a choice between two options. The first option is to use transfer functions (24) in the frequency domain and excitation accelerations in the time domain to get the response accelerations of rigid base mat in the time domain using Fast Fourier Transform (FFT) technique. Then one can analyze fixed-base structural model with prescribed 6D base motion, different from the initial seismic excitation or from the "foundation input motion". As structural damping is more or less homogeneous, such analysis can be done using modal approach and even spectral approach (six excitation spectra will be applied at the base instead of usual three ones).

The first concern about this variant is that fixed-base model does not allow analysis of the internal forces in the base mat itself. The second concern is that there are no means to consider flexibility of basement in this option. Nevertheless, the assumption about the rigidity of base mat is rather common.

The second option in CAM after the second step is to keep SSI model for the time-domain analysis instead of the fixed-base structural model in the previous option. The idea is to set up a new "platform" SSI model with some simplified impedance matrix $[D(\omega)]$ instead of $[C_{soil}(\omega)]$. The choice of simplified matrix will be discussed below. Then we adjust the "foundation input motion" $[B(\omega)]\{U_0(\omega)\}$ to some "modified platform excitation" $\{V_0(\omega)\}$. The most obvious requirement for such a modification is that the base mat response in the simplified platform model should be equal to the "true" one from (24):

$$U_b(\omega)=\{[C(\omega)]-\omega^2[M(\omega)]\}^{-1}[C(\omega)][B(\omega)]\{U_0(\omega)\}=$$
$$=\{[D(\omega)]-\omega^2[M(\omega)]\}^{-1}[D(\omega)]\{V_0(\omega)\} \quad (25)$$

Equation (25) gives us the modification matrix $[T(\omega)]$ from the "foundation input motion" to the "modified platform excitation":

$$\{V_0(\omega)\}=[T(\omega)][B(\omega)]\{U_0(\omega)\}$$
$$[T(\omega)]=[D(\omega)]^{-1}\{[D(\omega)]-\omega^2[M(\omega)]\}\{[C_{soil}(\omega)]-\omega^2[M(\omega)]\}^{-1}[C_{soil}(\omega)] \quad (26)$$

One can derive from (26) the alternative formula for $[T(\omega)]$

$$[T(\omega)]=[E]+[D(\omega)]^{-1}\{[D(\omega)]-[C_{soil}(\omega)]\}\{[C_{soil}(\omega)]-\omega^2[M(\omega)]\}^{-1}\omega^2[M(\omega)] \quad (27)$$

Here $[E]$ is a unit matrix 6 x 6. Formula (26) means that the excitation modification may be treated as the addition of some excitation to the initial one. This additional excitation depends on the difference between "platform" and "true" impedances.

Now let us discuss the choice of simplified impedance matrix $[D(\omega)]$. The logic of the previous section tells us that to keep modal approach applicable one must get homogeneous damping for the platform model. If we are going to use conventional modal approach, we should just take Rayleigh coefficient β previously used for structure and apply it to the soil springs in the new platform model.

For our sample model it means that 1D impedance $[D(\omega)]$ in our new platform model will be formed by the same spring as "true" impedance $[C_{soil}(\omega)]$ in the old model, but soil dashpot in this new platform model will have a new viscous coefficient in kN/(m/s)

$$c_z=k_z\times\beta=1{,}3\times10^3\times6{,}91978\times10^{-4}=0{,}89957 \quad (28)$$

As compared to the "true" viscosity coefficient (12) this makes considerable difference: "true" damping matrix element (1,1) has excessive viscosity Δc_{11}=110,5-0,89957=109,6 kN/(m/s). Returning to the previous section we can estimate the impact of this excessive viscosity to the damping matrix in the generalized coordinates:

$$[\Phi]^T[\Delta C][\Phi]=[\Phi]^T\begin{bmatrix}\Delta c_{11} & 0 & 0\\ 0 & 0 & 0\\ 0 & 0 & 0\end{bmatrix}[\Phi]=\begin{bmatrix}0{,}4703\\ 0{,}8535\\ -0{,}2245\end{bmatrix}\times109{,}6\times\begin{bmatrix}0{,}4703 & 0{,}8535 & -0{,}2245\end{bmatrix}=$$

$$=\begin{bmatrix}24{,}242 & 43{,}994 & -11{,}572\\ 43{,}994 & 79{,}840 & -21{,}001\\ -11{,}572 & -21{,}001 & 5{,}5239\end{bmatrix} \quad (29)$$

Comparing the off-diagonal terms in (29) with the off-diagonal terms in (17) we conclude that great values of the off-diagonals terms in (17) were caused by this excessive viscosity. When we took it away in a new platform model, we provided zero off-diagonal terms in damping matrix and made modal approach applicable.

So, this second option of CAM returns modal (and spectral) approach for SSI models, but platform excitation should be modified as compared to the "foundation input motion". The advantage of this option as compared to the previous one is that one can ease the rigidity of the base mat, using distributed soil springs and dashpots in the platform model. This will provide internal forces in the mat itself. Besides it will change the response of the upper structure. However, the results will be approximate. Some additional details about using CAM with flexible base mats may be found in previous publication of the author (Tyapin, 2011).

CONCLUSION

Conventional modal approach (and spectral approach based on modal one) meets serious difficulties when system has heterogeneous damping, as modal responses along natural modes do not part in such system. This is exactly the case for soil-structure interaction: one damping in the structure, another one (usually far greater in scale) – in the "soil dashpots" modelling "wave damping" in soil.

The results of the conventional modal approach neglecting the coupling between modal responses prove to be non-conservative. The attempts to cut down modal damping values lead to the excessive conservatism in one frequency region and non-conservatism in other region.

The author proposes to address these difficulties using CAM - combined asymptotic method, developed by the author some time ago.

The idea is to return to the model with homogeneous damping. It could be either a fixed-base structural model (in this case we are to obtain base motion out of the seismic excitation beforehand, which is done in CAM in the frequency domain using the "dynamic inertia" concept), or a platform model with homogeneous material damping or Rayleigh damping. In the latter case we are to account for the difference between "true" damping in soil dashpots and artificial damping in platform model. This is done in CAM by means of modification of the conventional "foundation input motion" to special "modified platform excitation". This motion is obtained in the frequency domain and always includes rotational components.

Both options of CAM help to achieve desirable result: modal (or spectral) approach can be used in soil-structure interaction analysis though with some additional preliminary procedures.

REFERENCES

Wolf J.P. Dynamic Soil-Structure Interaction. Prentice-Hall, Englewood Cliffs, NJ, USA, 1985.

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

Tyapin A. Combined Asymptotic Method for Soil-Structure Interaction Analysis. J. of Disaster Research, 2010, Vol. 5, No.4, p. 340-350.

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

Tyapin A.G. The Effects of the Base Mat's Flexibility on the Structure's Seismic Response. Part II: Platform Solutions. SMiRT21, New Delhi, 6-11 November 2011, Paper #266.