COMPARISON OF SOIL-STRUCTURE-INTERACTION
IN TIME DOMAIN VERSUS FREQUENCY DOMAIN

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ABSTRACT

Soil-structure-interaction models have been developed in the 1990’s based on complex frequency response analysis. In those models the subsoil is represented by an elastic half-space and taken into account as a frequency-dependent dynamic stiffness, which depends on the size of the foundation and properties of the subsoil. The seismic demand in the frequency domain is defined by Fourier transform of artificial time histories. While present-day design codes still refer to complex frequency response analysis or general substructuring methods, also direct methods in the time domain are allowed and viable. For the present study a number of results from direct non-linear analysis in the time domain with variable soil layers are compared to results from a 2-degree-of-freedom system for foundation and superstructure in the frequency domain. A fair agreement of base shears has been found. Furthermore, a sensitivity analysis of homogenization of the subsoil parameters in complex frequency response analysis is presented.

Keywords: SSI; Complex; Half-space; Frequency; FFT

1. INTRODUCTION

The analysis of soil-structure-interaction (SSI) as part of seismic analysis can be performed in several ways. Nowadays large models of structures can be created and analysed in the time domain using the finite element method (FEM) and time integration methods. If soil elements are attached to the structural model, then soil-structure-interaction effects can directly be taken into account. In the past, when computing power was much smaller and FEM techniques not customary, indirect SSI methods have been proposed, particularly for the frequency domain. As a consequence, results from those indirect SSI methods can be obtained much faster than from direct SSI analysis. The present paper aims to bring those approaches together for some cases and compare their respective results.

Numerous solutions for the dynamic impedances of foundations of any shape have been published. In case of a rigid rectangular foundation resting on the surface of a half-space Gazetas (1990), Mylonakis et al. (2006) and NIST (2012) present equations for stiffness and damping terms. Factors are given when the foundations are embedded in the soil. The complex dynamic stiffness depends in a non-linear way on the frequency. Pecker (2007) describes modelling of indirect soil-structure interaction in a general way for the finite element method by distinguishing kinematic and inertial effects. The validity of this method relies on the superposition theorem established by Kausel (1978) and Roesset (1973). For pile foundations solutions are given by Gazetas (1990). In addition simplified methods are available in the case of pile foundations to account for group effects (Dobry and Gazetas 1988).

To be fully efficient in a substructure approach and allow the use of conventional dynamic codes, the impedance functions which are frequency dependent must be represented by frequency independent values. The simplest version of these frequency independent parameters is the spring-dashpot assembly. From published results and ASCE 4-16 (2017), it appears that only under very restricted

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soil conditions (homogeneous half-space, regular foundations) these impedances can be represented by constant springs and dashpots. Nevertheless, structural engineers still proceed using these values which, more than often, are evaluated as static component (zero frequency) of the impedance functions (Pecker 2007).

For time domain analysis, a single frequency is usually selected for the purpose of evaluating foundation spring and dashpot coefficients. This can be taken as the frequency corresponding to the period associated with the dominant response of the structure. In most cases, this will be the first-mode, flexible-base period.

2. FREQUENCY-DOMAIN FORMULATION FOR 2DOF SYSTEM

One of the simplest representations of a building is a 2-degree-of-freedom (2DOF) system for foundation (index 0) and superstructure (index 1) as shown in Figure 1. The degrees of freedom are the absolute horizontal displacements \( u_1 \) and \( u_2 \), or alternatively the relative horizontal displacements \( w_1 \) and \( w_2 \). The system properties are determined by the masses \( M_i \), stiffnesses \( K_i \), and viscous dampers \( C_i \).

![2DOF system representing a building](image)

The time domain formulation of dynamic equilibrium corresponding to the 2DOF system subjected to ground motion \( u_g \) reads:

\[
\begin{bmatrix}
M_1 & 0 \\
0 & M_0
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_1 \\
\ddot{u}_0
\end{bmatrix} + 
\begin{bmatrix}
-C_1 & -C_1 \\
-C_1 & C_1 + C_0
\end{bmatrix}
\begin{bmatrix}
\dot{u}_1 \\
\dot{u}_0
\end{bmatrix} + 
\begin{bmatrix}
K_1 & -K_1 \\
-K_1 & K_1 + K_0
\end{bmatrix}
\begin{bmatrix}
u_1 \\
u_0
\end{bmatrix} = 
\begin{bmatrix}
C_0 \dot{u}_g + K_0 u_g \\
0
\end{bmatrix}
\]

(1)

or alternatively, using relative displacements \( w_i \):

\[
\begin{bmatrix}
M_1 & 0 \\
0 & M_0
\end{bmatrix}
\begin{bmatrix}
\dot{w}_1 \\
\dot{w}_0
\end{bmatrix} + 
\begin{bmatrix}
-C_1 & 0 \\
-C_1 & C_0
\end{bmatrix}
\begin{bmatrix}
\dot{w}_1 \\
\dot{w}_0
\end{bmatrix} + 
\begin{bmatrix}
K_1 & 0 \\
-K_1 & K_0
\end{bmatrix}
\begin{bmatrix}
w_1 \\
w_0
\end{bmatrix} = 
\begin{bmatrix}
0 \\
0
\end{bmatrix}
\]

(2)

which can be rewritten as:

\[
\begin{bmatrix}
M_1 & M_1 \\
0 & M_0
\end{bmatrix}
\begin{bmatrix}
\ddot{w}_1 \\
\ddot{w}_0
\end{bmatrix} + 
\begin{bmatrix}
-C_1 & 0 \\
-C_1 & C_0
\end{bmatrix}
\begin{bmatrix}
\dot{w}_1 \\
\dot{w}_0
\end{bmatrix} + 
\begin{bmatrix}
K_1 & 0 \\
-K_1 & K_0
\end{bmatrix}
\begin{bmatrix}
w_1 \\
w_0
\end{bmatrix} = 
-\begin{bmatrix}
M_1 \\
M_0
\end{bmatrix}\dot{u}_g
\]

(3)

Upon expansion of the DOFs and ground motion to the following complex quantities:

\[
u_1 = \bar{U}_1 e^{i\omega t}, \quad u_0 = \bar{U}_0 e^{i\omega t}, \quad w_1 = \bar{W}_1 e^{i\omega t}, \quad w_0 = \bar{W}_0 e^{i\omega t}, \quad u_g = \bar{U}_g e^{i\omega t}
\]

(4)
where \( \tilde{W}_1, \tilde{W}_0, \tilde{U}_g \) and \( \tilde{A}_g \) are complex constants (representing both amplitude and phase lag), \( \omega \) is the angular frequency and \( t \) time, the following frequency domain formulation of the set of Equations 1 results, analogous to Veletsos and Tang (1990):

\[
\begin{bmatrix}
-\omega^2 M_1 + K_1 + i\omega C_1 \\
-(K_1 + i\omega C_1)
\end{bmatrix}
\begin{bmatrix}
\hat{D}_1 \\
\hat{U}_0
\end{bmatrix}
= \begin{bmatrix}
0 \\
-(K_0 + i\omega C_0)\hat{U}_g
\end{bmatrix}
\]

(5)

The frequency domain formulation of the set of Equations 3 is then:

\[
\begin{bmatrix}
-\omega^2 M_1 + K_1 + i\omega C_1 \\
-K_1 - i\omega C_1
\end{bmatrix}
\begin{bmatrix}
\tilde{W}_1 \\
\tilde{W}_0
\end{bmatrix}
= \omega^2 \begin{bmatrix}
M_1 \\
M_0
\end{bmatrix} \tilde{U}_g
\]

(6)

Using:

\[
\omega_j^2 = \frac{K_j}{M_j} \quad \zeta_j = \frac{C_j}{2M_j\omega_j} = \frac{C_j\omega_j}{2K_j}
\]

(7)

Equations 6 can be written as:

\[
\begin{bmatrix}
-\omega^2 M_1 + K_1 \left(1 + 2i\zeta_1 \frac{\omega}{\omega_1}\right) \\
-K_1 \left(1 + 2i\zeta_1 \frac{\omega}{\omega_1}\right)
\end{bmatrix}
\begin{bmatrix}
\tilde{W}_1 \\
\tilde{W}_0
\end{bmatrix}
= \omega^2 \begin{bmatrix}
M_1 \\
M_0
\end{bmatrix} \tilde{U}_g
\]

(8)

And division of Equations 8 by \( \omega^2 M_1 \) and \( \omega^2 M_0 \) respectively yields:

\[
\begin{bmatrix}
\omega_1^2 \left(1 + 2i\zeta_1 \frac{\omega}{\omega_1}\right) - 1 \\
-\frac{K_1}{M_0\omega_1^2} \left(1 + 2i\zeta_1 \frac{\omega_0}{\omega_1}\right) \frac{\omega_0^2}{\omega^2} \left(1 + 2i\zeta_0 \frac{\omega_0}{\omega_0}\right) - 1
\end{bmatrix}
\begin{bmatrix}
\tilde{W}_1 \\
\tilde{W}_0
\end{bmatrix}
= \begin{bmatrix} 1 \\ 1 \end{bmatrix} \tilde{U}_g
\]

(9)

The responses \( \tilde{W}_1 \) and \( \tilde{W}_0 \) can be readily obtained from the inverse of the set of Equations 9, or the responses \( \tilde{U}_1 \) and \( \tilde{U}_0 \) from the inverse of the set of Equations 5, where \( \tilde{U}_g \) is determined by discrete Fourier transform (i.e., fast Fourier transform, FFT) of the earthquake excitation.

The base shear \( \tilde{V} \) can be determined either from:

\[
\tilde{V} = (K_0 + i\omega C_0)(\tilde{U}_0 - \tilde{U}_g)
\]

(10)

or from:

\[
\tilde{V} = K_0 \left(1 + 2i\zeta_0 \frac{\omega}{\omega_0}\right) \tilde{W}_0
\]

(11)

The numerical result of the base shear will be compared to the ones deduced from large FEM models described in the next chapter. Note that on the basis of Equations 2 and 4, and defining \( \tilde{u}_1 = A_1 e^{i\omega t} = -\omega^2 \tilde{U}_1 e^{i\omega t} \), the absolute acceleration \( \tilde{A}_1 \) of the superstructure with mass \( M_1 \) is:

\[
\tilde{A}_1 = \frac{\omega^2}{\omega_1^2} \tilde{W}_1
\]

(12)

According to Wolf (1985, 1994) the stiffness \( K_0 \) and damping ratio \( \zeta_0 \) of the subsoil due to soil-structure-interaction can be defined as:

\[
K_0 = \frac{E}{2\gamma} GR
\]

(13)
\[ \zeta_0 = \frac{\pi}{16}(2 - v) \frac{R}{v_s} \omega_0 + \zeta_g \]  

(14)

with \(v\) as the Poisson’s ratio of the subsoil material, \(v_s\) its shear wave velocity, \(\zeta_g\) its material damping ratio, and \(R\) the equivalent radius of the foundation. Alternatively, according to ASCE 4 (2017) a frequency-independent damping coefficient of \(C_0 = 0.576K_0R/v_s\) and stiffness \(K_0 = 32(1 - v)GR/(7 - 8v)\) may be used.

3. FEM MODELS FOR DIRECT SOIL-STRUCTURE-INTERACTION

Recently non-linear time-history (NLTH) analyses of tens of buildings have been performed by Royal HaskoningDHV using DIANA FEM software. The NLTH analyses were committed because of earthquake damages to buildings due to gas extraction from the subsoil in the North of The Netherlands. A direct soil-structure-interaction approach has been followed in those analyses, as can be seen in Figure 2 and Figure 3. Those figures show the mesh of soil elements and layers extending far beyond the perimeter of the building. The properties of those soil layers are listed in Table 1 and Table 2.

Figure 2. NLTH model of building and subsoil nr. 1

Figure 3. NLTH model of building and subsoil nr. 2
In order to reduce the model size without compromising the accuracy, the subsoil is modelled by single elements per horizontal plane from a certain depth down to 30 m below grade, a “soil column”. Those elements have properties equivalent to the total horizontal extent of the fully meshed soil layers above. The FEM models analyzed here have shallow foundations. Model 1, with a total mass of 261 tons and a plan area of 180 m$^2$, consists of 84,878 finite elements in total, of which 46,513 solid elements represent the subsoil. Model 2, with a total mass of 900 tons and a plan area of 332 m$^2$, is larger and consists of 123,849 finite elements in total, of which 61,038 solid elements represent the subsoil.

Table 1. Subsoil properties of FEM model 1.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil type</th>
<th>Thickness</th>
<th>Top level</th>
<th>Bottom level</th>
<th>Specific weight</th>
<th>Shear wave velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sand</td>
<td>0.6</td>
<td>0.0</td>
<td>-0.6</td>
<td>14.3</td>
<td>101</td>
</tr>
<tr>
<td>2</td>
<td>Clay</td>
<td>2.0</td>
<td>-0.6</td>
<td>-2.6</td>
<td>14.8</td>
<td>96</td>
</tr>
<tr>
<td>3</td>
<td>Clay</td>
<td>4.0</td>
<td>-2.6</td>
<td>-6.6</td>
<td>15.2</td>
<td>112</td>
</tr>
<tr>
<td>4</td>
<td>Sand</td>
<td>3.0</td>
<td>-6.6</td>
<td>-9.6</td>
<td>17.0</td>
<td>176</td>
</tr>
<tr>
<td>5</td>
<td>Sand</td>
<td>1.0</td>
<td>-9.6</td>
<td>-10.6</td>
<td>17.3</td>
<td>172</td>
</tr>
<tr>
<td>6</td>
<td>Clay</td>
<td>1.0</td>
<td>-10.6</td>
<td>-11.6</td>
<td>17.1</td>
<td>156</td>
</tr>
<tr>
<td>7</td>
<td>Sand</td>
<td>9.0</td>
<td>-11.6</td>
<td>-20.6</td>
<td>19.5</td>
<td>293</td>
</tr>
<tr>
<td>8</td>
<td>Sand</td>
<td>1.0</td>
<td>-20.6</td>
<td>-21.6</td>
<td>19.8</td>
<td>259</td>
</tr>
<tr>
<td>9</td>
<td>Clay</td>
<td>6.0</td>
<td>-21.6</td>
<td>-27.6</td>
<td>18.0</td>
<td>252</td>
</tr>
<tr>
<td>10</td>
<td>Sand</td>
<td>2.4</td>
<td>-27.6</td>
<td>-30.0</td>
<td>20.7</td>
<td>312</td>
</tr>
</tbody>
</table>

Table 2. Subsoil properties of FEM model 2.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil type</th>
<th>Thickness</th>
<th>Top level</th>
<th>Bottom level</th>
<th>Specific weight</th>
<th>Shear wave velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sand</td>
<td>1.0</td>
<td>0.0</td>
<td>-1.0</td>
<td>16.7</td>
<td>96</td>
</tr>
<tr>
<td>2</td>
<td>Clay</td>
<td>3.5</td>
<td>-1.0</td>
<td>-4.5</td>
<td>15.4</td>
<td>108</td>
</tr>
<tr>
<td>3</td>
<td>Clay</td>
<td>5.3</td>
<td>-4.5</td>
<td>-9.8</td>
<td>14.9</td>
<td>123</td>
</tr>
<tr>
<td>4</td>
<td>Sand</td>
<td>2.5</td>
<td>-9.8</td>
<td>-12.3</td>
<td>18.3</td>
<td>231</td>
</tr>
<tr>
<td>5</td>
<td>Sand</td>
<td>0.8</td>
<td>-12.3</td>
<td>-13.0</td>
<td>17.3</td>
<td>201</td>
</tr>
<tr>
<td>6</td>
<td>Sand</td>
<td>7.0</td>
<td>-13.0</td>
<td>-20.0</td>
<td>19.2</td>
<td>288</td>
</tr>
<tr>
<td>7</td>
<td>Sand</td>
<td>10.0</td>
<td>-20.0</td>
<td>-30.0</td>
<td>20.4</td>
<td>376</td>
</tr>
</tbody>
</table>

For the seismic excitation, applied as 7 sets of artificial time-histories (each with two horizontal components and one vertical) with a sampling rate of 0.005 s at the bottom of the soil column through Lysmer dampers (NPR 1998 2015, Lysmer 1978), maximum base shears of 1059 and 2998 kN were found in FEM model 1 and 2 respectively. Those results were obtained by Newmark implicit time marching with 0.01 s steps using DIANA software and soil behaviour according to the Hardin-Drnevich model. The Hardin-Drnevich model takes into account the G/G$^0$-curves and hysteretic behaviour of the individual soil layers. The seismic displacement time-histories and corresponding response accelerations are shown in Figure 4. It is investigated here to what extent those base shears can be reproduced by the 2DOF model for complex frequency response analysis described in the previous chapter using the equivalent parameters listed in Table 3.

Figure 4 (right-hand side) shows that at grade a great reduction of the horizontal seismic excitation has occurred as compared to bedrock, which has been assumed 30 m below grade. Note that while the seismic displacement time-history $u_g$ at bedrock has been baseline-corrected, Figure 4 (left-hand side)
shows that the site responses for model 1 and 2 yield some residual displacement at the end of the seismic excitation.

Figure 5 shows the cumulative participating mass of the natural frequencies corresponding to the FEM models without soil elements prior to seismic excitation. From these graphs it is derived that the first dominant horizontal frequency of model 1 is 7 Hz and that the one for model 2 is 15 Hz. Those frequencies are initially used for the reduction of the model to a 2DOF system.

Figure 4. Horizontal seismic displacement time-histories (left-hand side) and response accelerations (right-hand side) applied to the FEM models at bedrock and to the equivalent 2DOF systems at grade. The time-histories at grade and response spectra corresponding to only one time-history at bedrock are shown

Figure 5. Cumulative participating mass of natural modes in model 1 (left-hand side) and model 2 (right-hand side). Lumped frequencies have been indicated by vertical lines

4. APPLICATION OF THE 2DOF SYSTEM

The complicated buildings shown in Figure 2 and Figure 3 are reduced to 2DOF systems, which constitute of course gross simplifications. A key step in this process is the identification of lumped masses and associated natural frequencies. The equivalent natural frequency of the superstructure $f_1$ in the 2DOF system has been assessed based on the cumulative participating mass of the structure before application of the seismic loading as shown in Figure 5. The stiffness $K_1$ follows then from $f_1$ and $M_1$. The equivalent shear wave velocity $v_s$ and the equivalent density of the soil $\rho$ have been determined by the weighted average of the values in Table 1 and Table 2 over a depth equal to the radius of the foundation $R$ as specified in Table 3. Note that the frequency-independent damping coefficient $C_{0,\text{ASCE4}}$ takes values in the case of model 1 and model 2 that are 103% and 88% of the critical damping respectively, if the critical damping is defined by the total mass $M_1 + M_0$ and the foundation spring $K_0$.

The sampling rate of the seismic time-history, equal to 0.005 s, yields a Nyquist frequency of 100 Hz. The duration of the ground displacement time-history is 10 s, so there are 2000 samples. For the FFT
of the ground displacement time-history the number of samples has been increased to 2048 (= 2^{11}) by adding trailing zeros.

The result of the complex frequency response analysis with the 2DOF system for model 1 is that a maximum base shear of 1210 kN is reached (after 2.7 s), see Figure 6. Note that the analysis has produced some numerical noise at the beginning of the seismic excitation, which can obviously be disregarded. In a time-domain analysis with the same 2DOF system, but using the equivalent frequency-independent damping coefficient \( C_{0,\text{ASCE4}} \) according to ASCE 4 (2017), a maximum base shear of 1020 kN is reached (at 2.7 s). The maximum base shear found in the FEM model with direct SSI (1059 kN) is in between the last two values, see also Table 4.

Table 3. Properties of the analyzed 2DOF systems consistent with the FEM models.

<table>
<thead>
<tr>
<th>Model</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subsoil</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Density</td>
<td>( \rho )</td>
<td>1526</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>( G )</td>
<td>2.03E+07</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>( \nu )</td>
<td>0.4</td>
</tr>
<tr>
<td>Stiffness</td>
<td>( K_0 )</td>
<td>7.68E+08</td>
</tr>
<tr>
<td>Stiffness according to ASCE 4 (2017)</td>
<td>( K_{0,\text{ASCE4}} )</td>
<td>7.76E+08</td>
</tr>
<tr>
<td>Shear wave velocity</td>
<td>( v_s )</td>
<td>115</td>
</tr>
<tr>
<td>Material damping ratio</td>
<td>( \zeta_g )</td>
<td>10%</td>
</tr>
<tr>
<td>Damping ratio</td>
<td>( \zeta_0 )</td>
<td>261%</td>
</tr>
<tr>
<td>Basic natural angular frequency</td>
<td>( \omega_0 )</td>
<td>122</td>
</tr>
<tr>
<td>Damping coefficient</td>
<td>( C_0 )</td>
<td>3.29E+07</td>
</tr>
<tr>
<td>Damping coefficient according to ASCE 4 (2017)</td>
<td>( C_{0,\text{ASCE4}} )</td>
<td>2.94E+07</td>
</tr>
<tr>
<td>Ratio of ground cone depth to radius of foundation (Wolf 1994)</td>
<td>( z_0/R )</td>
<td>0.63</td>
</tr>
<tr>
<td>Ground cone depth</td>
<td>( z_0 )</td>
<td>4.76</td>
</tr>
<tr>
<td>Foundation and superstructure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass of foundation</td>
<td>( M_0 )</td>
<td>5.20E+04</td>
</tr>
<tr>
<td>Mass of superstructure</td>
<td>( M_1 )</td>
<td>2.09E+05</td>
</tr>
<tr>
<td>Stiffness</td>
<td>( K_1 )</td>
<td>4.04E+08</td>
</tr>
<tr>
<td>Radius of foundation</td>
<td>( R )</td>
<td>7.6</td>
</tr>
<tr>
<td>Damping ratio of superstructure</td>
<td>( \zeta_1 )</td>
<td>5%</td>
</tr>
<tr>
<td>Natural angular frequency</td>
<td>( \omega_1 )</td>
<td>44</td>
</tr>
<tr>
<td>Natural frequency</td>
<td>( f_1 )</td>
<td>7</td>
</tr>
<tr>
<td>Critical damping coefficient</td>
<td>( C_{\text{cr}} )</td>
<td>1.84E+07</td>
</tr>
<tr>
<td>Damping coefficient of superstructure</td>
<td>( C_1 )</td>
<td>9.19E+05</td>
</tr>
</tbody>
</table>

With the 2DOF system for model 2 a maximum base shear of 2690 kN is found (at 1.5 s, see Figure 6), which is 10% less than the base shear resulting from the NLTH analysis with the FEM model (2998 kN). The linear analysis in the time domain with the constant damping coefficient \( C_{0,\text{ASCE4}} \) yields a base shear of 2450 kN (at 1.5 s), even 18% less than the NLTH FEM analysis result.
Figure 6. Resulting base shear from complex frequency response analysis and linear time-history analysis using a 2DOF system representing model 1 (upper) and model 2 (lower)

5. DISCUSSION

The reduction of the full FEM models to 2DOF representations obviously entails massive lumping of parameters. In order to put the previous results into perspective a sensitivity analysis has been performed with respect to model parameters that are considered critical, i.e. the shear wave velocity
\(v_s\), the Poisson’s ratio \(\nu\) and the material damping ratio \(\zeta_g\) of the subsoil. The responses to lower shear wave velocities have been determined by taking the weighted average of the shear wave velocities over a depth less than \(R\) (2.6 m in case of model 1 and 4.5 m in case of model 2). The results are shown in Table 4 and Table 5. It appears that the resulting base shears are affected by maximum 18\%, taking the base shear resulting from the full FEM models as a reference.

Table 4. Summary of SSI calculations for model 1. “N/A” means not applicable and “Var.” means variable.

<table>
<thead>
<tr>
<th>Type</th>
<th>(f_1) Hz</th>
<th>(v_s) m/s</th>
<th>(\nu)</th>
<th>(\zeta_g)</th>
<th>Max. base shear kN</th>
<th>Relative</th>
</tr>
</thead>
<tbody>
<tr>
<td>NLTH FEM direct SSI</td>
<td>N/A</td>
<td>Var.</td>
<td>Var.</td>
<td>Var.</td>
<td>1059</td>
<td>0%</td>
</tr>
<tr>
<td>2DOF frequency-domain</td>
<td>7</td>
<td>115</td>
<td>0.40</td>
<td>10%</td>
<td>1210</td>
<td>14%</td>
</tr>
<tr>
<td>2DOF time-domain</td>
<td>7</td>
<td>115</td>
<td>0.40</td>
<td>N/A</td>
<td>1020</td>
<td>-4%</td>
</tr>
<tr>
<td>2DOF frequency-domain</td>
<td>7</td>
<td>97</td>
<td>0.40</td>
<td>10%</td>
<td>1160</td>
<td>10%</td>
</tr>
<tr>
<td>2DOF frequency-domain</td>
<td>7</td>
<td>115</td>
<td>0.40</td>
<td>5%</td>
<td>1220</td>
<td>15%</td>
</tr>
<tr>
<td>2DOF frequency-domain</td>
<td>7</td>
<td>115</td>
<td>0.45</td>
<td>10%</td>
<td>1220</td>
<td>15%</td>
</tr>
</tbody>
</table>

Table 5. Summary of SSI calculations for model 2. “N/A” means not applicable and “Var.” means variable.

<table>
<thead>
<tr>
<th>Type</th>
<th>(f_1) Hz</th>
<th>(v_s) m/s</th>
<th>(\nu)</th>
<th>(\zeta_g)</th>
<th>Max. base shear kN</th>
<th>Relative</th>
</tr>
</thead>
<tbody>
<tr>
<td>NLTH FEM direct SSI</td>
<td>N/A</td>
<td>Var.</td>
<td>Var.</td>
<td>Var.</td>
<td>2998</td>
<td>0%</td>
</tr>
<tr>
<td>2DOF frequency-domain</td>
<td>15</td>
<td>111</td>
<td>0.40</td>
<td>10%</td>
<td>2690</td>
<td>-10%</td>
</tr>
<tr>
<td>2DOF time-domain</td>
<td>15</td>
<td>111</td>
<td>0.40</td>
<td>N/A</td>
<td>2680</td>
<td>-11%</td>
</tr>
<tr>
<td>2DOF frequency-domain</td>
<td>15</td>
<td>105</td>
<td>0.40</td>
<td>10%</td>
<td>2450</td>
<td>-18%</td>
</tr>
<tr>
<td>2DOF frequency-domain</td>
<td>15</td>
<td>121</td>
<td>0.40</td>
<td>5%</td>
<td>2700</td>
<td>-10%</td>
</tr>
<tr>
<td>2DOF frequency-domain</td>
<td>15</td>
<td>121</td>
<td>0.45</td>
<td>10%</td>
<td>2700</td>
<td>-10%</td>
</tr>
</tbody>
</table>

The influence SSI on the base shear could be estimated by imposing the seismic loading to a single-degree-of-freedom (SDOF) system with the same properties as the superstructure in the 2DOF system. The resulting shear should then be increased by the shear produced by the foundation mass at the PGA. Figure 7 shows that a base shear of 978 kN is caused by the superstructure of model 1 (at 2.3 s). The foundation, with a mass of 52 tons, produces an additional base shear of 138 kN at a PGA of 0.27g, which makes a total of 1116 kN, conservatively adding the amplitudes. Hence, the base shear reduction due to SSI could be estimated at only 5\% in case of model 1, in spite of the low shear wave velocity (EN 1998-5 2005).

For model 2 Figure 7 shows that with the SDOF representation of the superstructure a maximum base shear of 1220 kN is obtained (at 2.2 s), and upon addition of the base shear from the foundation (600 tons multiplied with a PGA of 0.31g) the total of 3030 kN results. So, also for this case the base shear reduction due to SSI is minor (1\%).

Figure 7. Resulting base shear from linear time-history analysis using a SDOF system representing the superstructure of model 1 (left-hand side) and model 2 (right-hand side).
It appears that the base shears resulting from time-domain analysis using a 2DOF system with frequency-independent damping coefficients can be 18% less than the base shears resulting from full NLTH FEM analysis. Depending on the case, complex frequency response analysis seems to overestimate or underestimate the latter base shears by 10% to 15%. Furthermore, complex frequency response analysis suffers from numerical noise at the beginning and end of the time-history.

6. CONCLUSION

Simple models for complex frequency response analysis including soil-structure-interaction can be used to approach the base shear resulting from huge FEM models of buildings in which the subsoil has been explicitly meshed. Approximations based on such 2DOF representations may be reasonable, however the results may deviate significantly from those of full FEM models, considering also the uncertainty of the lumped parameters. The lumped models with frequency-independent damping for time-domain analysis tend to produce smaller maximum base shears than those found in the full FEM models. These conclusions are based on a limited comparison to two large FEM models for non-linear time-history analysis set up in DIANA software using the Hardin-Drnevich soil model.

7. ACKNOWLEDGEMENTS

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