2D nonlinear soil response based on the influence of vertical component in near-fault conditions

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Abstract

In this study we propose a numerical simulation based on a parametric 2D soil response analysis, in order to analyze the influence of vertical acceleration in near fault conditions. Soil nonlinearity will be considered in a real time manner from the offline construction of a parametric solution, where shear modulus and damping factor of soils are modeled as equivalent linear relations of the shear strain. The algorithm proposes an online integration that proceeds by particularizing the parametric solution for the shear modulus and damping parameters, and then update it, from the just calculated solution, regard the level of deformation [4]. The aim is to be able to compute very fast solutions to non-linear soil dynamics and to examine the dynamic response of different stratified surface configurations considering the nonlinear characteristics, in order to evaluate the role of the vertical component of the seismic movement.

Introduction

Seismic site response analysis is usually characterized by assuming that vertical acceleration of an earthquake varies between ½ and 2/3 of the horizontal acceleration. Recently, studies have found that this relation can change significantly depending on the period and distance to the fault [1 2] . The ratio of vertical to horizontal acceleration (V/H) can exceed 1.0 for short period and near fault conditions and the commonly adopted ratios of ½ and 2/3 is related to far fault condition.

Elasto-dynamic problem

Let's consider a dynamic elasticity problem in $\Im$, the displacement field $\mathbf{u}(x, y, t)$ is governed by the following equation

$$\rho \ddot{\mathbf{u}} = \nabla \cdot \sigma.$$

The weak formulation, in frequency domain writes as follows:

$$-\int_{\Omega} \mathbf{u}^* \rho \omega^2 \mathbf{u} \, d\Omega = -\int_{\Omega} \varepsilon^* \mathbf{D} \, \varepsilon \, d\Omega + \int_{\Omega} \mathbf{u}^* \mathbf{F} \, d\Omega,$$

with $\Omega = \Omega_{xy}$, where ρ is the density, ω is the frequency, $\mathbf{F}$ is the volumetric body forces in the frequency space and $\mathbf{D}$ is the linear elastic isotropic

$$\mathbf{D} = \begin{bmatrix} \frac{E}{1-\nu^2} & \frac{E\nu}{1-\nu^2} & 0 \\ \frac{E\nu}{1-\nu^2} & \frac{E}{1-\nu^2} & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix}, \qquad \varepsilon = \begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} = \begin{bmatrix} \frac{\partial u}{\partial x} \\ \frac{\partial v}{\partial y} \\ \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \end{bmatrix},$$

the displacement filed can be expressed as

$$\mathbf{u}(x, y, \omega) = \begin{pmatrix} u(x, y, \omega) \\ v(x, y, \omega) \end{pmatrix}$$

Nonlinear seismic ground response analysis

To evaluate the soil dynamic response, soil properties such as shear modulus G and damping ratio ζ need to be known. These properties present nonlinear behavior regarding the strain level (γ). Soil stiffness is normally characterized by the small-strain shear modulus G_{max} and the dissipative soil behavior is characterized by the damping ratio ζ. According to this dependence, the problem becomes highly nonlinear and an iteration method is required.
The soil model can be represented by the constitutive relationship between the stress and strain from a Kelvin-Voigt (KV) 2D model. Thus, the shear stress results

$$\sigma = \mathbf{D}\epsilon + \mathbf{D}' \frac{d\epsilon}{dt} = (\mathbf{D} + i\omega \mathbf{D}')\varepsilon,$$

where tensor $\mathbf{D}$ is defined in dependence on the Elastic modulus E and the Poisson coefficient ν, and $\mathbf{D}'$ involves the viscosity modulus $\eta = \frac{2G\zeta}{\omega}$. Now, taking into account the damping, the complex young modulus E^* can be written for soil as

$$E^* = (1 + i2\zeta)\, E,$$

Lysmer and Kuhlemeyer [4] transmitting boundary condition is implemented with the aim to avoiding reflected waves from boundaries of finite domains. These can be defined as stresses in the lateral surfaces,

$$\sigma_x = c_p \rho i \omega u$$
$$\tau_{xy} = c_s \rho i \omega v$$

Meanwhile, the seismic loading is applied at soil-bedrock, thus, the half-space (bedrock) is replaced with an equivalent shear stress time history

$$\sigma_y = c_p^* \rho_b \dot{y}_y - c_p^* \rho_b i \omega v$$
$$\tau_{yx} = c_s^* \rho_b \dot{y}_x - c_s^* \rho_b i \omega u$$

The first terms correspond to the forces, $\mathbf{F}(t) = (F_x, F_y)$ acting in the different directions (x, y) and the second are dashpots to mimic the infinite half space at bedrock.

Iterative Procedure

The equivalent linear approximation method [4]] is implemented to address the nonlinear soil behavior, approximated by a linear analysis compatible with the level of deformation. Thus, the curves proposed by [3] can be used iteratively to reach compatibility between the properties and the strain.

The procedure is summarized below:

1. Compute the displacement solution
$$\mathbf{u}(x, y, \omega)$$
2. Compute the Fourier transform of the input motion (in x and y directions),
$$\mathbf{F}(\omega) = \mathcal{F}(\mathbf{F}(\mathbf{t})).$$
3. Initialize from G_0 and ζ_0, and the constant parameters:
$$(G_0, \zeta_0).$$
4. Repeat the procedure until convergence:
   - The complex displacement amplitude for each frequency is computed from the parametric solution
   $$\mathbf{u}(x, y, \omega, G^m, \zeta^m),$$
   where m is the nonlinear iteration. Applying the principle of superposition, yields
   $$\mathbf{u}^m(x, y, \omega) = \mathbf{F}(\omega) \circ \mathbf{u}(x, y, \omega, G^m, \zeta^m).$$
   - Calculate the 3 strains component in the frequency domain at each element and the principal strains. Then, the corresponding effective shear strain is;
   $$\gamma_{eff} = \frac{2}{3}\gamma_{max},$$
   - Determine new strain values from the strain-dependent G and ζ curves and then update the new values of G^{m+1} and ζ^{m+1}
   - The convergence criteria is given by $\mathcal{E}^m$:
   $$\mathcal{G}^m = \left(\frac{|G^{m+1} - G^m|^2}{|G^1|^2} + \frac{|\zeta^{m+1} - \zeta^m|^2}{|\zeta^1|^2} \right).$$
5. Generate all outputs applying the inverse Fourier Transform.

Numerical example

To illustrate the potentialities of the algorithm here proposed, a 2D soil analysis is considered. The solution is obtained assuming the horizontal and vertical acceleration time history in the outcropping is known, which correspond to x and y directions, vertical input motion (y direction) is assumed equal to the horizontal input motion.

Case	V_s(m/s)	V_p (m/s)	G (MPa)	E(MPa)	k (MPa)
1	100	187	20	52	43
2	300	561	180	468	390
3	500	935	500	1300	1080
4	700	1309	980	2250	2120
5	1000	1870	2000	5200	4300

Table 1:Material properties of the five subsoil models.

with $\mu = 0.3$ and density $\rho = 2000Kg/m^3$. A homogeneous soil deposit, with dimensions 100 m x 300 m, has been studied (Fig. 1).

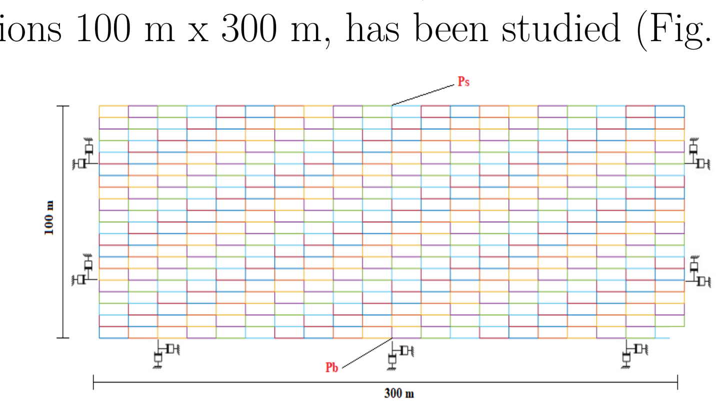

Figure 1:Soil model mesh and boundary condition

Linear Results

The input time-histories are chirp functions, as velocity time history, that is sinusoidal signal with constant amplitude and variable frequency with time (see Fig. 2)

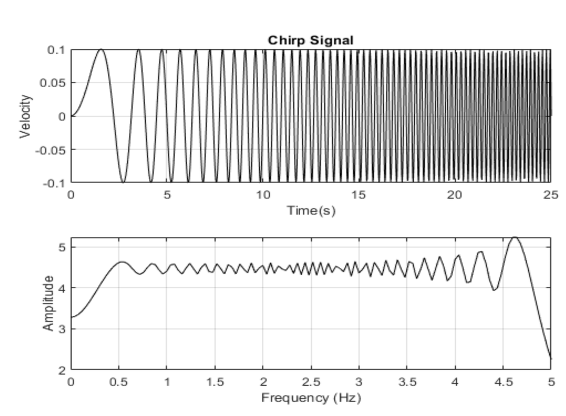

Figure 2:Dynamic input motion

Case	fH	fH*	fV	fV*
1	0.24	0.25	0.46	0.47
2	0.76	0.75	1.40	1.40
3	1.24	1.25	2.32	2.34
4	1.76	1.75	3.28	3.27
5	2.48	2.50	4.68	4.68

Table 2:

with $fH^* = \frac{V_s}{4h}$ and $fV^* = \frac{V_p}{4H}$.

The results of frequency response for all the 5 analyzed deposits are resumed in Table 2. The main horizontal frequency values of the deposits obtained by the numerical analyses (fH and fV) are compared with the ones given by the theoretical approach for a homogeneous elastic soil (fH^* and fV^*): as expected, the agreement is very good.

In figures 3 and 4 can be observed the comparison with 3 different approaches, DeepSoil Program(1D), FEM-1D and FEM-2D. Figure 5 and 6 shows the amplification function for cases 1 and 5, both for the horizontal and vertical components. Observing practically the same amplitudes for both components.

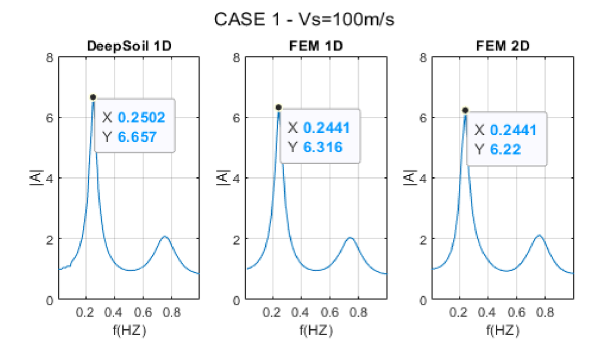

Figure 3:Case 1 for 3 different analysis.

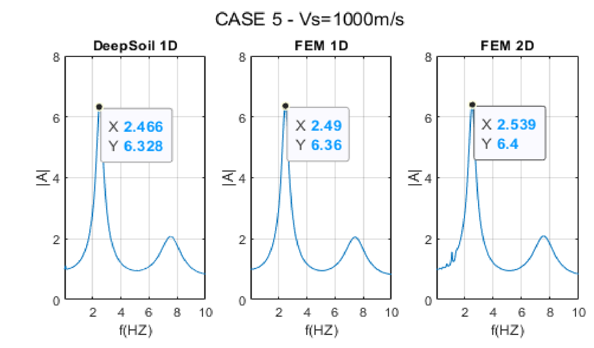

Figure 4:Case 5 for 3 different analysis.

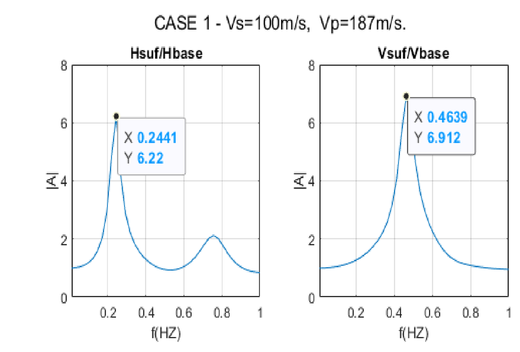

Figure 5:Amplification Function (case 1)

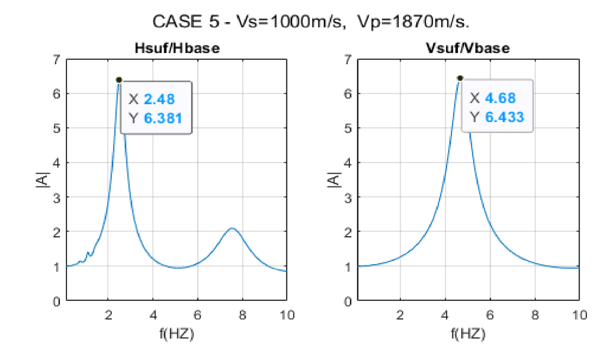

Figure 6:Amplification Function (case 5)

Nonlinear Results

Nonlinear behavior was considered by the equivalent linear method. For this approach, the first 3 cases given in Table 1 and a seismic input motion in Fig. 7 were evaluated.

Case	Mat.	fH	fV
1	Sand	1.44	2.68
2	Clay	7.12	13.50
3	Sand	12.4	23.27

Table 3:

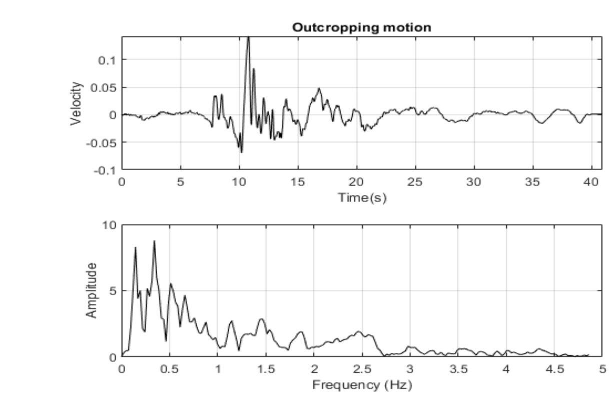

Figure 7:Dynamic input motion

Figures 8 and 9 shows the response for different analysis.Variations are very small, and may be due to the dimensional order to compute the shear strain.

The vertical motion amplification has been evaluated and compared to the horizontal one (Fig.10 and 11), revealing to be significant and not negligible.

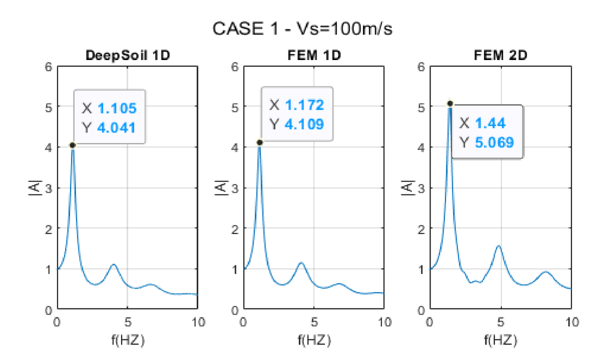

Figure 8:Case 1 for 3 different analysis.

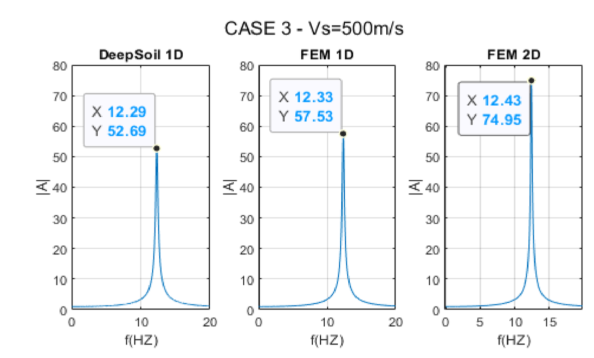

Figure 9:Case 5 for 3 different analysis.

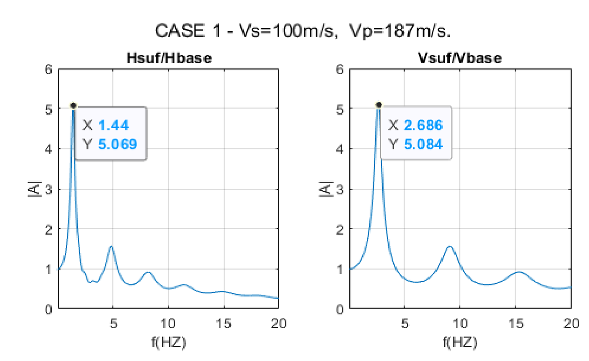

Figure 10:Amplification Function (case 1)

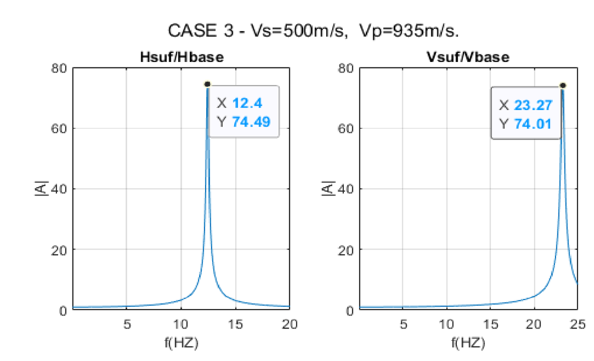

Figure 11:Amplification Function (case 3)

Conclusion

The dynamic response of simple subsoil configurations under combined horizontal and vertical input motions has been examined, for evaluating the role of vertical motion component, which revealed to be very significant in recent earthquakes in near-fault conditions. Numerical analyses allowed to define the frequency response of homogeneous soil to vertical input motion: the main frequency can be effectively correlated to the soil compression wave propagation velocity.

References

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