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Fourier Amplitudes of the Foundation Motion connected with Soil-Structure Interaction

A. Hayir[1] and V. Gicev[2]
[1]Civil Engineering Faculty, Istanbul Technical University, Turkey
[2]Division of Computer Science, Goce Delcev University, Macedonia

Abstract

The main objective of this study is to understand the phenomena connected with the interaction and to give directions for improvement of the design of the earthquake resistant structures. Fourier amplitudes (amplitudes versus frequencies) of the foundation motion connected with the soil-structure interaction and differential motions of the foundation-structure contact due to the wave passage are considered. We expect that the motion of the flexible foundation will be larger than the existing solutions for the motion of the rigid foundation. As the foundation becomes stiffer, the Fourier amplitudes (transfer functions) on the building-foundation contact will decay and will approach the solution for the case of rigid foundation. The wave propagation approach in solving the problem is taken into account. The wave equation will be solved numerically in the domain consisting of the soil, foundation, and superstructure using the explicit Lax-Wendroff numerical scheme. Because in the real world, the problem is defined in infinite domain in the soil, we will incorporate an artificial boundary in the truncated domain which corresponds with the Sommerfeld radiation boundary condition at infinity. The velocities and the displacements at the points of the stress-free boundaries will be updated in each time step using the vacuum formalism approach. The stresses, the velocities, and the displacements at the points of soil-foundation and foundation-building contacts will be computed from the continuity of stresses and the continuity of displacements at those points. In present time the soil-structure interaction is underestimated as a factor in the codes and the standards for design of earthquake resistant structures.

Keywords: dynamic soil structure-interaction, Fourier amplitudes, Lax-Wendroff numerical scheme, flexible foundation, rigid foundation, differential motions.

1 Introduction

The dynamic soil-structure interaction problem is of much interest to engineers. It takes place during the passage of seismic waves through the soil. The interaction has a bearing on the stresses in the structure and varies in the time history of the incident earthquake waves[1]. It is well known that soil–structure interaction (SSI) is a critical factor influencing response and damage in structures during earthquakes. The main effects of SSI on response are that SSI decreases the dominant frequency of the structure's vibrations, filters the high frequencies in the excitation, and increases damping. Due to the frequency content of the ground shaking, SSI can be detrimental or beneficial for structures. SSI becomes detrimental if, because of SSI, the dominant frequency of vibrations becomes closer to the dominant frequency of ground shaking[2].

The common approach to calculating the seismic response of tall buildings has been to formulate the response as a vibration problem in terms of the mass, damping and stiffness matrices of the building, and to solve the resulting equations by using modal analysis or time integration techniques[3]. Although seismic vibrations are caused by the propagation of seismic waves into the structure, the wave propagation approach has not been widely used to calculate the seismic response[4]. The amount of seismic energy transmitted from the ground to a structure and its propagation in the structure provide critical insight into how the structure will respond during an earthquake. Few studies of wave-propagation methods have been used frequently to analyze continuous structures, such as beams and walls [5].

A rare application to non-continuous structures has been given by Uzgider and Aydogan [6] and Aydogan and Uzgider[7] where a simplified wave propagation methodology is used to calculate the undamped response of frame and masonry structures.

In this study, one numerical solution, with Lax-Wendroff numerical scheme using Finite difference method, of the interaction of an infinitely long shear wall supported by a rijid semi-circular foundation embedded in linear homogeneous half space and excited by plane SH waves with arbitrary incident are derived using wave propagation approach. The absolute amplitudes of the foundation motion Δ are calculated both rijid and flexible foundation. They are called Fourier amplitudes of the foundation.

2 The Model and Problem Formulation

The model considered here consist of an infinitely long elastic shear wall height H_b and thickness $2a$(Figure1). The model is the same as one studied by Trifunac[1], but the foundation is deformable. The rigidity and the velocity of the shear waves in the isotropic and homogeneous wall are given by ρ_b and μ_b respectively. The shear wall is resting on a rigid infinitely long semi-cylindrical foundation of radius a. The soil is assumed to be elastic, isotropic and homogeneous with rigidity μ, velocity of shear waves $\tilde{\beta}$. The contact between the soil and foundation is assumed to be welded.

The governing equation of the problem

$$v_t = \frac{1}{\rho}(\sigma)_x \tag{1}$$

and the relation between the derivative of strain and the velocity is

$$\varepsilon_t = v_x \tag{2}$$

where υ, ρ, σ and ε are particle velocity, density, shear stress and shear strain respectively, and the subscripts t and x represent derivatives with respect to time and space

$$\varepsilon = \frac{\partial u}{\partial x} \qquad v = \frac{\partial u}{\partial t} \tag{3}$$

The displacement u is the anti-plane displacement of a particle perpendicular to the propagation of the wave.

The boundary conditions of the problem are as below:

On the structure

$$\sigma_y = 0 \qquad y = H_b \tag{4a}$$

$$\sigma_x = 0 \qquad 0 < y < H_b \qquad x = \mp a \tag{4b}$$

On the free-field :at $y = 0$ and $x > \mp a$

$$\sigma_y = 0 \tag{5}$$

Between the structure and the foundation: at $y = 0$ and $x > \mp a$

$$\sigma_y^b = \sigma_y^f \qquad u^b = u^f \tag{6}$$

Between the foundation and soil : at $r=a$

$$\sigma_y^f = \sigma_y^S \qquad u^f = u^s \tag{7}$$

3 Numerical Example

The model given in figure2 is taken into account. This model might be description of six story building commonly constructed in Turkey. The steady-state response of the model is obtained. The model with the geometry and the material properties of the constitutive parts is shown on the figure.

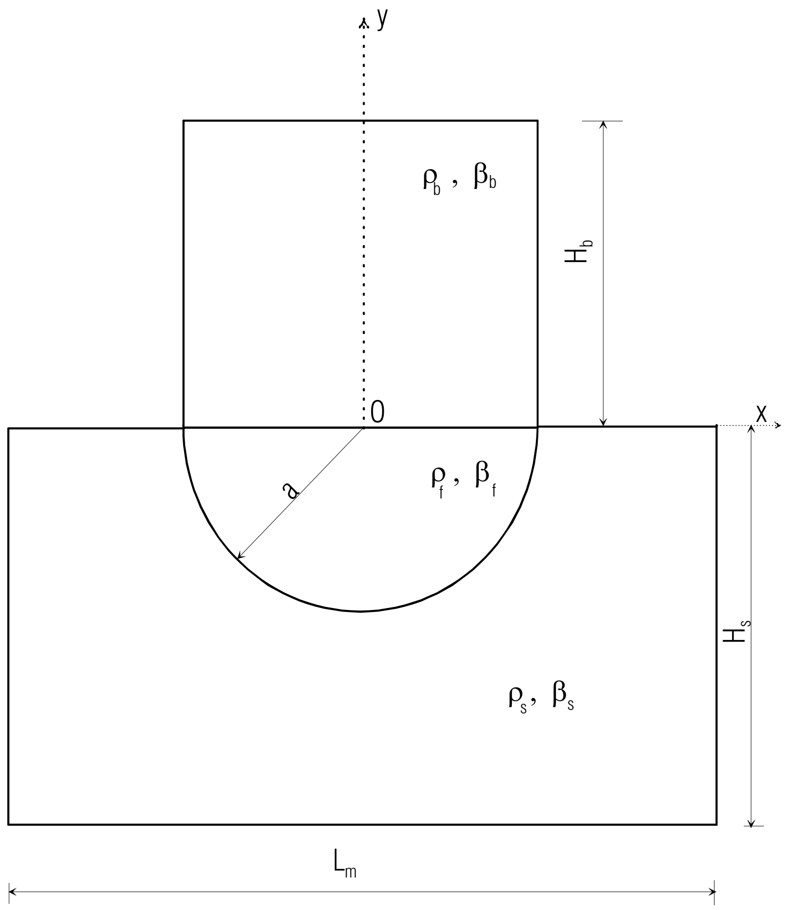

Figure 1. The model

The excitation is a monochromatic sinusoidal plane wave of the form:

$$u(x_i, y_j, t) = A[\sin(\Omega(t-t_1))H(t-t_1) + \sin(\Omega(t-t_2))H(t-t_2)]$$
$$\forall(x_i, y_j) \in \Gamma_1, \forall(x_i, y_j) \in \Gamma_2, \tag{8}$$

where $A = 0.5m$ is amplitude of the wave, $t_1 = x_i \sin\gamma/\beta + y_j \cos\gamma/\beta$ is travel time of the wave from left bottom corner(Figure1) to the considered point, $t_2 = x_i \sin\gamma/\beta + (H_s + y_j)\cos\gamma/\beta$ is travel time of the wave from left bottom corner to the free surface and from the free surface to the considered point, γ is the angle of incidence, and H() is the Heaviside function.

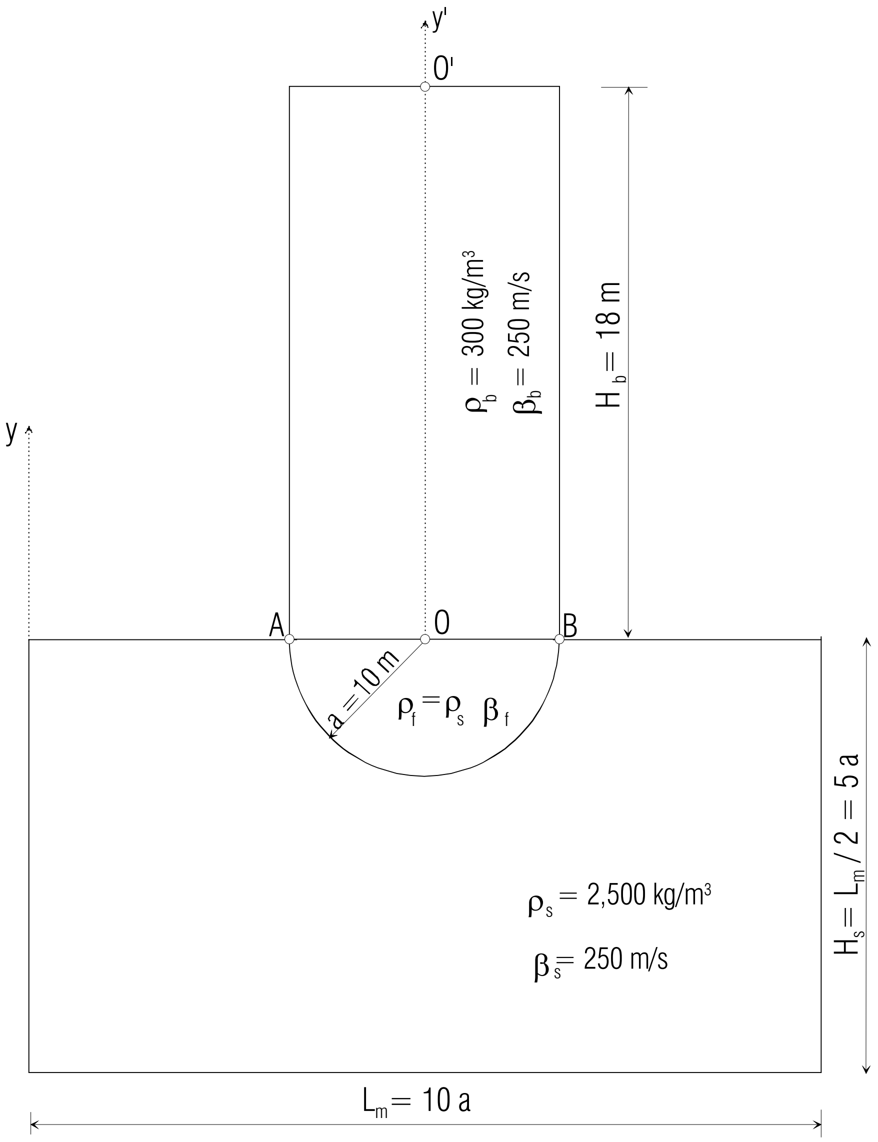

Figure2. Numerical Model showing the material properties and the sizes of the model.

The natural frequencies of the fixed-base building for the elastic modes in the y direction are:

$$f_n = (2n-1)\beta/4H_b \quad (\mathrm{Hz}) \qquad \mathrm{n} = 1,2,3,... \tag{9}$$

The first three natural frequencies in the y direction are $f_{0,1} = 3.47\ Hz$, $f_{0,2} = 10.42\ Hz$, and $f_{0,3} = 17.36\ Hz$

The modes in the x direction represent the torsional response of the structure. From the solution of the linear wave equation and the boundary condition in the x direction, the characteristic numbers in the x direction are $k_{xm} = m\pi/L_b$ (m = 0,1,2,…)

with corresponding angular natural frequencies $\omega_m = k_{xm}\beta_b$. The lowest elastic mode m = 1 (m = 0 corresponds to rigid mode) has natural frequency $\omega_{1,0} = 13.88 \cdot \pi$. The first subscript in the natural frequencies denotes the number of the mode (including the rigid-body mode 0) in the x direction, and the second one, the number of the mode in the y direction.

The analysis is performed for the frequency range of the input motion $0.5\ Hz \le \Omega \le 11.0\ Hz$, the amplitude A = 0.5 m, and for incident angles $\gamma = 30^0$ and $\gamma = 60^0$.The minimum wavelength is $\lambda_{\min} = \lambda_{\Omega=6Hz} = 2\pi\beta_s / 12\pi \approx 42\ m$ $> L_b = 2a = 20\ m$, and so Δx is chosen from the criterion for proper modeling of the foundation

4 Numerical Results

We generate motion in the system by the eq (1). A point is in quite until the wave arrives, and then experiences a sudden change in velocity from 0 to $A\Omega$. The situation is similar at all of the points in the model. The transition of the regime from rest to harmonic steady state is present in the first phase of the analysis (Figure3) until the transients go out of the system. For our numerical example, the steady-state regime for any frequency is established after 3-4 s from the beginning of the analysis, when the amplitudes of the motion become constant. In Figure 3, the time histories of the displacement at point O are shown for several input frequencies for the stiffer foundation $\beta_f = 500\, m/s$ and for the angle of incidence $\gamma = 30^0$.

In Figures 4 to 7, the dynamic amplification factor versus input frequency is shown for three points on the foundation-structure contact (Figure 2) for angles of incidence $\gamma = 30^0$ and $\gamma = 60^0$, and for foundation stiffnesses $\beta_f = 300\, m/s$ and $\beta_f = 500\, m/s$. For comparison, amplitude Δ plots for the rigid foundation are also shown. For all cases, at small frequencies, the amplitudes of all three points (A, O, B) are the same and approach 1 as f approaches 0. This can be explained by the fact that for an input wave much longer than the foundation dimension and of such small frequency that the inertial forces become negligible the whole system just rides on the wave and the effect of the interaction is negligible. As the input wavelength becomes smaller, and as the input frequency approaches the natural frequency of the building, the amplitudes at the three different contact points begin to differ. This difference is partially caused by the horizontal wave passage through the foundation (differential motion as discussed in Trifunac and Todorovska, [8]), which causes a torsional response by the foundation, and partially by the soil-structure interaction. The smaller the incident angle, the larger is the relative contribution of the soil-structure interaction to the response, and the larger the incident angle, the larger is the relative contribution of the wave passage. In extreme cases, when the incident angle $\gamma = 0$ there is no effect of the differential motion, and when the incident angle $\gamma = \dfrac{\pi}{2}$ the effect of wave passage is most prominent.

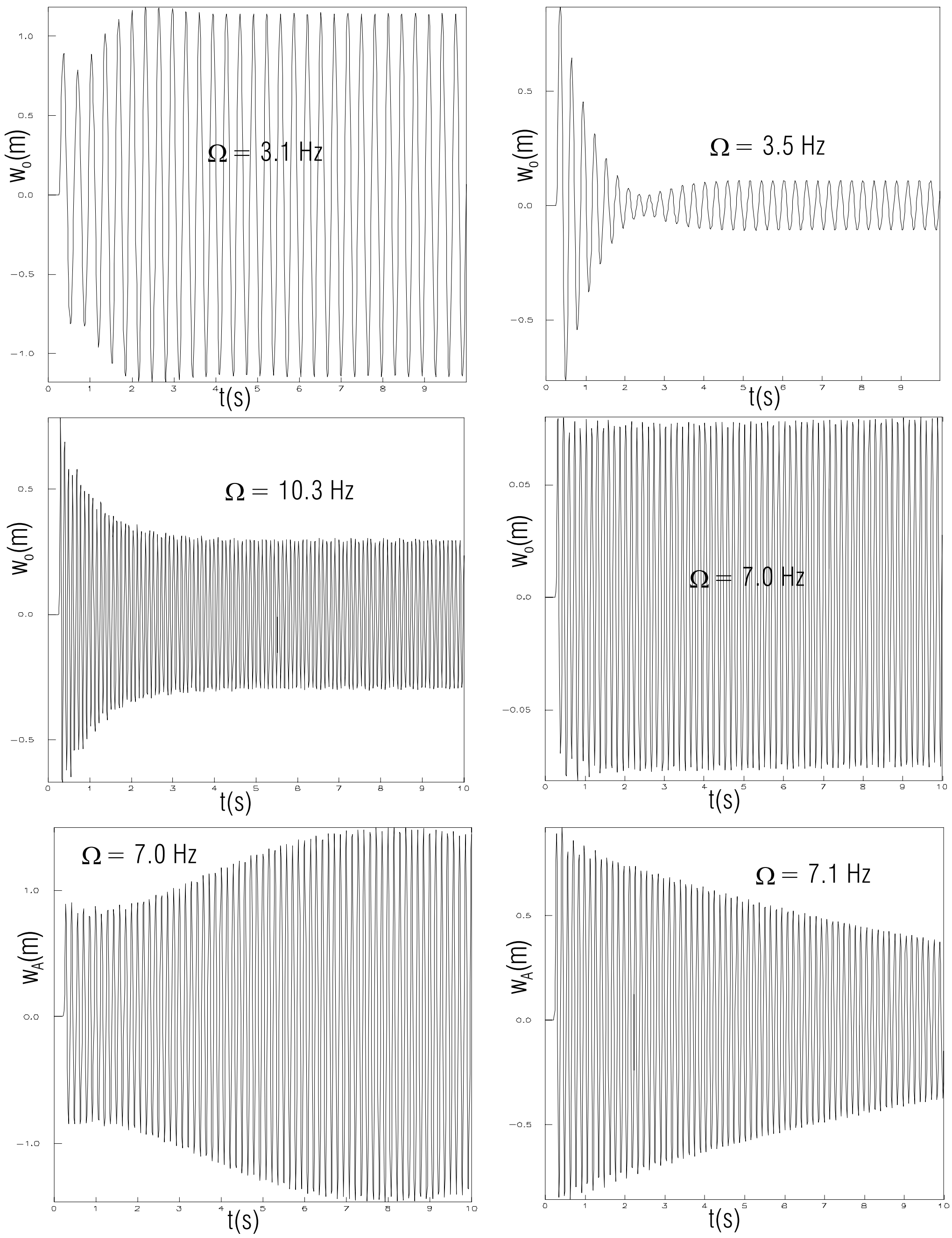

Figure 3. Time history of the displacement at point O for some frequencies β_f=500 m/s and γ=30^0.

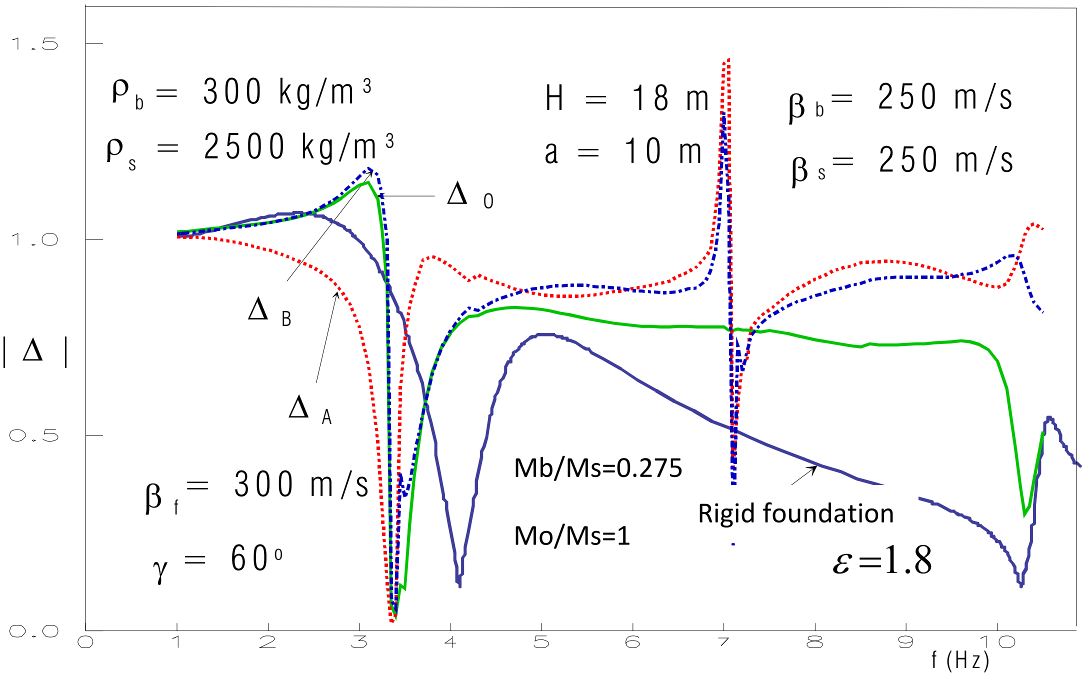

Figure4. Response at the building-foundation contact normalized by free surface response for β_f=300 m/s and γ=60^0.

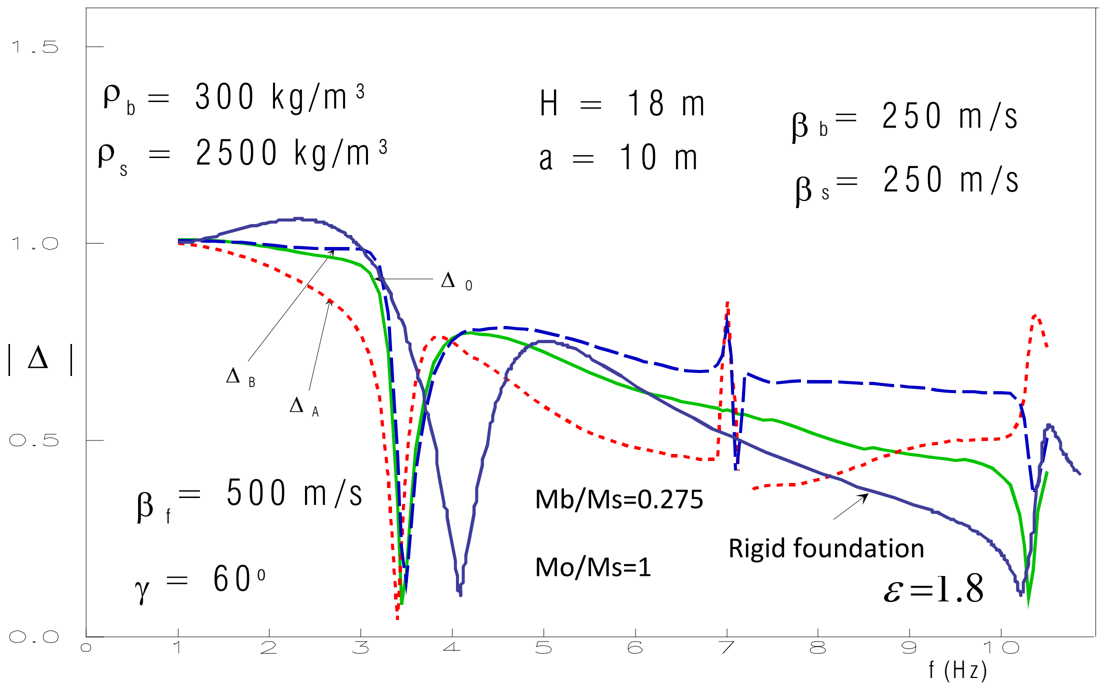

Figure5. Response at the building-foundation contact normalized by free surface response for β_f=500 m/s and γ=60^0.

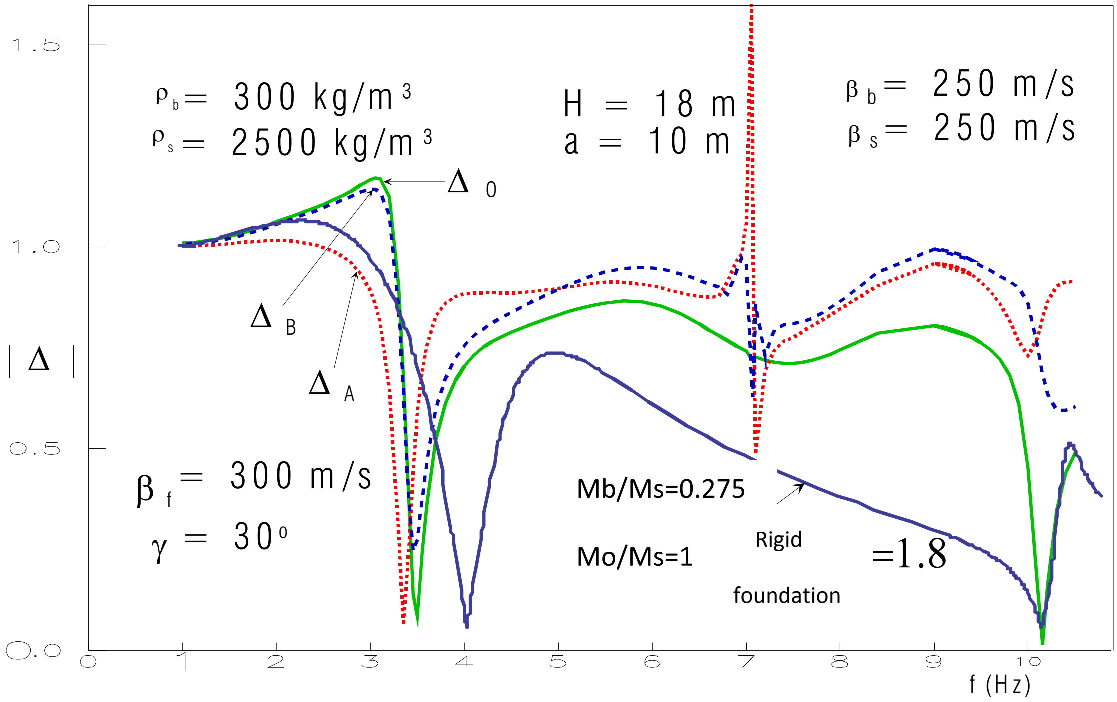

Figure6. Response at the building-foundation contact normalized by free surface response for β_f=300 m/s and γ=30^0.

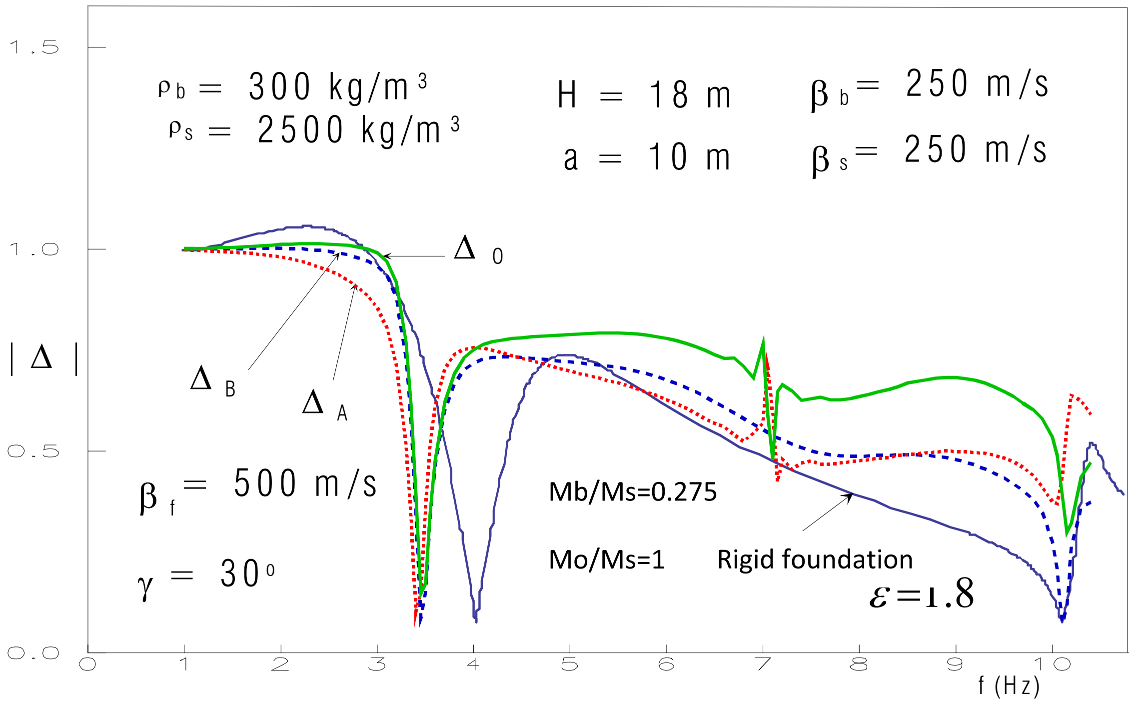

Figure7. Response at the building-foundation contact normalized by free surface response for β_f=500 m/s and γ=30^0.

5 Conclusions

Lax-Wendroff numerical scheme is performed for the solutions. It is concluded that the responses of the foundation change depending on the incident angle of the waves and characteristics of the structure, soil and foundation respectively. In this study, the foundation, structure and soil are all deformed with respect to frequency of the waves during the excitation. This case might give us in which cases of the rigidity of the foundation will be accepted as an rigid. In the approach of rigid foundation, the foundation are not deform and responses are the same in whole foundation, and the responses are independent of the incident angles. It can be analyzed more easier than the flexible foundation approach.

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References

[1] M.D. Trifunac, "Interaction of a shear wall with the soil for incident plane SH waves" Bull. Seism. Soc. of America, 62 (1), 63 – 83, 1972.

[2] E. Safak, "Time-domain representation of frequency-dependent foundation impedance functions" Soil Dynamics and Earthquake Engineering 26, 65–70, 2006.

[3] E. Safak, "Propagation of seismic waves in tall buildings" Struct. Design Tall Build. 7, 295–306, 1998.

[4] E. Safak, "New approach to analyzing soil-building systems" Soil Dynamics and Earthquake Engineering 17, 509–517, 1998.

[5] M.D. Trifunac, M.I. Todorovska, " Antiplane earthquake waves in long structures." J. Engrg. Mech., ASCE, 115(12) 2687-2708, 1989.

[6] E. Uzgider, M. Aydoğan, " Simple and efficient method for the dynamic response of 2D frames subject to groung motions." Proc., 8 th Eur. Conf. On Earthquake Engrg., Vol. 3, Laboratorio Nacional de Engenharia Civil, Lisboa, Portugal, 6.2/17, 1986.

[7] M. Aydoğan, E. Uzgider, " A numerical method for the earthquake response of brick masonry structures." Bull., Technical University of Istanbul, Istanbul, Turkey, 41(3), 415-431, 1988.

[8] M.D. Trifunac, M.I. Todorovska, "Response spectra for differential motion of columns" Earthquake Engrg and Struct Dynam 26(2), 251-68,1997.