Energy exchange between piled structures through the soil

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Abstract. This work presents a study on the energy transfer between piled structures. For this analysis, two structures, supported by buried pile groups, interact with each other through the soil. A stationary external load is applied to one of the structures, and its effect is felt by the other structure solely through the energy transmitted through the soil. The soil is modeled as a three-dimensional halfspace, and the piles through the impedance matrix method. The structure is modeled via classical finite element discretizations, which enables arbitrarily-shaped structures to be considered. Coupling between the two systems is obtained by imposing continuity and equilibrium conditions at their interface. Selected results are shown for different geometric parameters of the piled structures.

Keywords: Dynamic soil-structure interaction, piled structures, energy exchange, geotechnics

1 Introduction

The technological relevance of the problem of piled structures has given rise to vibrant fields of study on pile-soil-structure interaction models and on the many parameters involved in the design of such structures [1, 2].

A common strategy to model pile group behavior in piled structures is to use Winkler-Pasternak approximations [3, 4]. While reasonable from the point of view of the behavior of the structure, these approximations are incapable of representing wave propagation between piled structures through the soil, or of quantifying the influence of the excitation by piled structures in their surroundings. On the other hand, full finite element discretizations of the soil domain and its embedded foundations such as that of Kementzetzidis et al. [5] present classical difficulties such as domain truncation problems and violation of Sommerfeld's radiation condition [6].

This article investigates the phenomenon of energy transfer between piled structures through the soil. In order to properly account for the physics of wave propagation through soil media, a boundary element-based formulation is used for the foundation part, which is based on the impedance matrix method [7]. The structure is modeled with classical finite elements, which enables consideration of arbitrarily-shaped structures, and arbitrary harmonic loads to be applied in terms of nodal equivalents. Selected numerical results are presented on the influence of geometrical and constitutive parameters of interacting piled structures on their dynamic behavior.

1.1 Problem statement

The problem analyzed in this study consists of two piled structures, which interact with each other through the foundation soil (Fig. 1). An external excitation is applied in the source structure and its effect is measured in the target structure ("Tower").

The structures are supported by a group of four piles and have a square base of sides $s = 2.5\,m$. They are at a distance S, from center to center of each other. The source structure is a 1 m-tall plate, while the tower has height H. All piles and structures are made of concrete, with mass density $\rho_c = 2500\,kg/m^3$, modulus of elasticity $E_c = 21.5\,GPa$, and Poisson's ratio $\nu_c = 0.2$. The piles have a constant section with diameter $d_p = 0.4\,m$, and length $l_p = 15\,m$. The foundation soil was defined as having shear wave velocity $c_s = 250\,m/s$, mass density $\rho_s = 1600\,kg/m^3$, internal material damping $\beta = 5\%$, and Poisson's ratio $\nu_s = 0.4$. This results in $E_s = 280\,MPa$. This study considered the influence of selected values of S and H in the response of the tower.

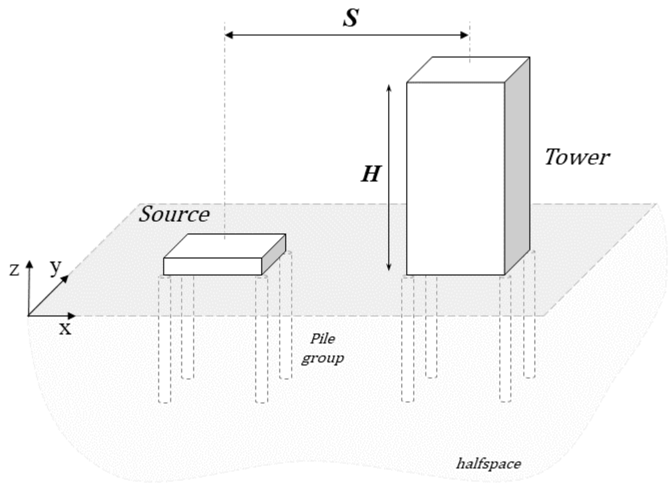

Figure 1. Representation of the analyzed case

2 Formulation

The group of piles was modeled using the formulation developed by Kaynia and Kausel [7]. This model is obtained through the impedance matrix method, in which the soil is considered an isotropic, viscoelastic, three-dimensional half-space, the solution of which is obtained by superposition of Green's functions. Piles are modeled as one-dimensional finite beam elements. Coupling between the pile elements and the soil medium is obtained by imposing equilibrium and continuity conditions in their interfaces. The dynamic stiffness matrix that relates forces and displacements at the ends of a pile in a group is given by $K_e = K_p + \Psi^T(F_s + F_p)^{-1}\Psi$, where K_p is the pile dynamic stiffness matrix, Ψ is the pile element dynamic flexibility matrix, and F_p and F_s are pile and soil flexibility matrices, respectively. For a full description of each of these terms, refer to Kaynia and Kausel [7].

The structures were modeled using eight-noded, linear-elastic, hexahedral finite elements with three translational degrees-of-freedom per node. The structure's dynamic stiffness matrix is given by $K^s = K_g - \omega^2 M_g$, where ω is the frequency of excitation, and K_g and M_g are the global stiffness and mass matrices, assembled from the elementary stiffness and mass matrices according to the classical assembly procedure of the finite element method.

The coupling between the structures and the group of piles is obtained by establishing equilibrium and continuity conditions in the nodes of the structure that are connect with pile heads. In a structure with N nodes, where nodes n and m are connected to piles i and j, the coupled dynamic stiffness matrix results in:

$$\bar{K_G} = \begin{bmatrix} K^s_{11} & \cdots & K^s_{1m} & \cdots & K^s_{1n} & \cdots & K^s_{1N} \\ \vdots & \ddots & \vdots & & \vdots & & \vdots \\ K^s_{m1} & \cdots & K^s_{mm} + K^p_{ii} & \cdots & K^s_{mn} + K^p_{ij} & \cdots & K^s_{mN} \\ \vdots & & \vdots & \ddots & \vdots & & \vdots \\ K^s_{n1} & \cdots & K^s_{nm} + K^p_{ji} & \cdots & K^s_{nn} + K^p_{jj} & \cdots & K^s_{nN} \\ \vdots & & \vdots & & \vdots & \ddots & \vdots \\ K^s_{N1} & \cdots & K^s_{Nm} & \cdots & K^s_{Nn} & \cdots & K^s_{NN} \end{bmatrix}, \quad (1)$$

where K^s and K^p consist of 3x3 matrices that contain the stiffness of the structures and the pile group in the $x-$, $y-$ and $z-$directions, respectively. This coupling scheme is shown in detail in Vasconcelos [8].

3 Results

The analyses consisted of measuring the influence of S and H in the dynamic response of the tower. The analysis of the influence of soil parameters in the response of the system is outside the scope of this article, but can be obtained by analysts with the proposed method. The results are presented in terms of the normalized frequency of excitation $a_0 = \omega \cdot d_p/c_s$ and of the normalized displacement $U_{ij} = u_{ij}/F$ of points A and B at the middle of the bottom and top surfaces of the tower, respectively, in which u_{ij} indicates the displacements of points A and B in the $i-$direction due to loads of magnitude F applied on the source structure in the $j-$direction (see Fig. 1).

3.1 Influence of distance S on tower response

In this analysis, the height of the tower was $H = 20\,m$. The horizontal response of the tower (Figs. 2 and 3), show that the distance S does not affect the natural frequencies of the system. This shows that the vibrating modes of the coupled system are governed by the vibrating modes of the tower alone. The decrease in vibration amplitude with increasing S is physically consistent, since the energy from the source is quickly dissipated through the soil radially from the source.

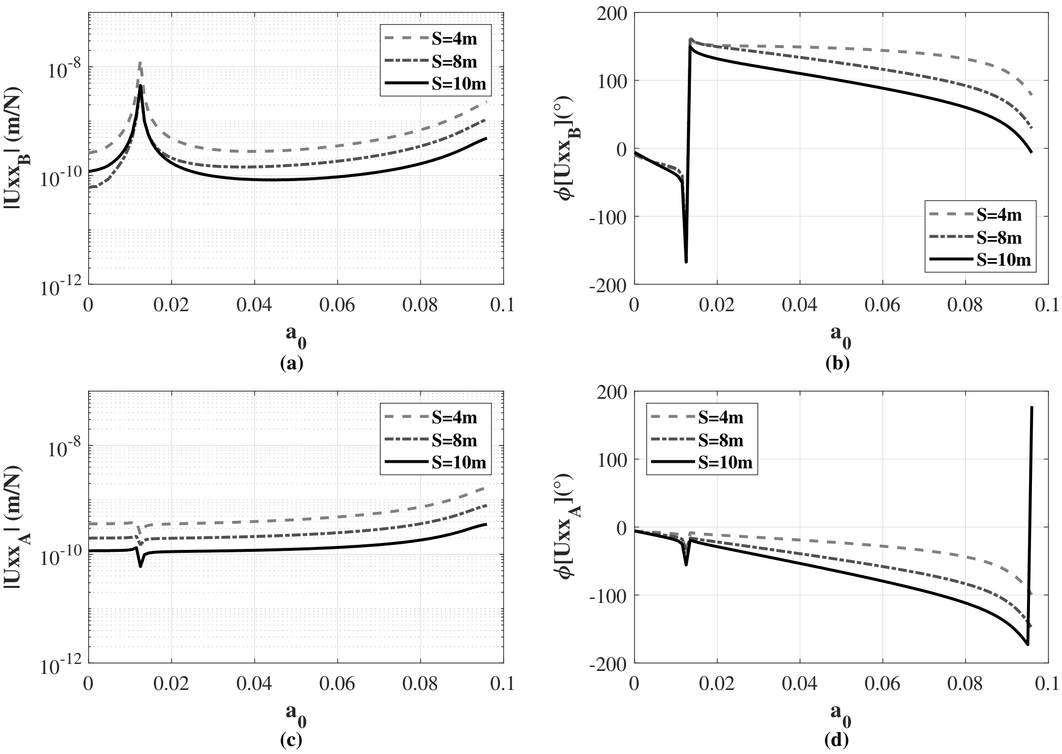

Figure 2. Horizontal response of the tower due to horizontal loading at the source.

It is also physically consistent that the cross displacement components (U_{ij} for $i \neq j$) have overall smaller magnitudes than their direct displacement counterparts. Additionally, both cross and direct vertical components have overall smaller magnitudes than their horizontal counterparts. This is due to the fact that the tower is more flexible in the horizontal direction than in the vertical direction. The compressional stiffness of the tower also explains the negligible difference between U_{zj_A} and U_{zj_B} $(j = x, z)$. The larger horizontal flexibility can also be seen in the fact that one of the natural frequencies of horizontal vibration of the tower fall within the considered $0 \leq a_0 \leq 0.1$ range, while its first natural frequency of compression is larger than $a_0 = 0.1$.

3.2 Influence of height H on tower response

This analysis considered a distance of $S = 20\,m$ between the source and the tower. Figures 6 and 7 show that an increase in the height of the tower causes its natural frequencies of horizontal vibration to be reduced. Figures 8 and 9 show that, in addition to a reduction of natural frequencies of compression of the tower for taller towers, overall larger magnitudes of displacement are observed, which in consistent with the increased flexibility of the tower in the longitudinal direction for taller towers.

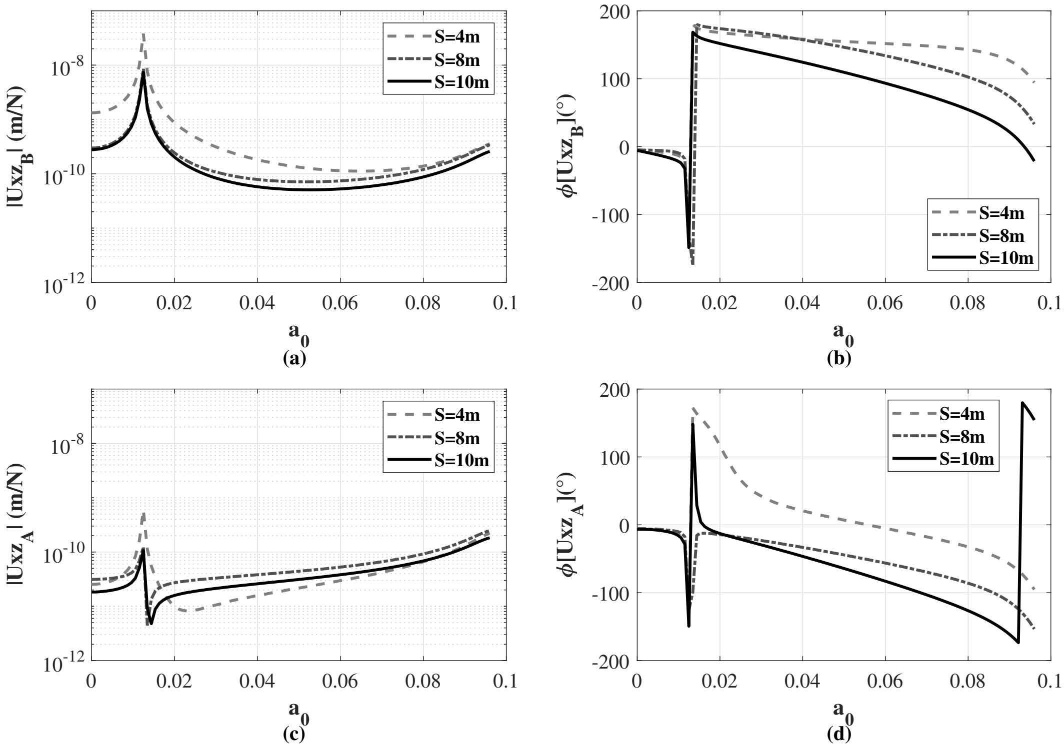

Figure 3. Horizontal response of the tower due to vertical loading at the source.

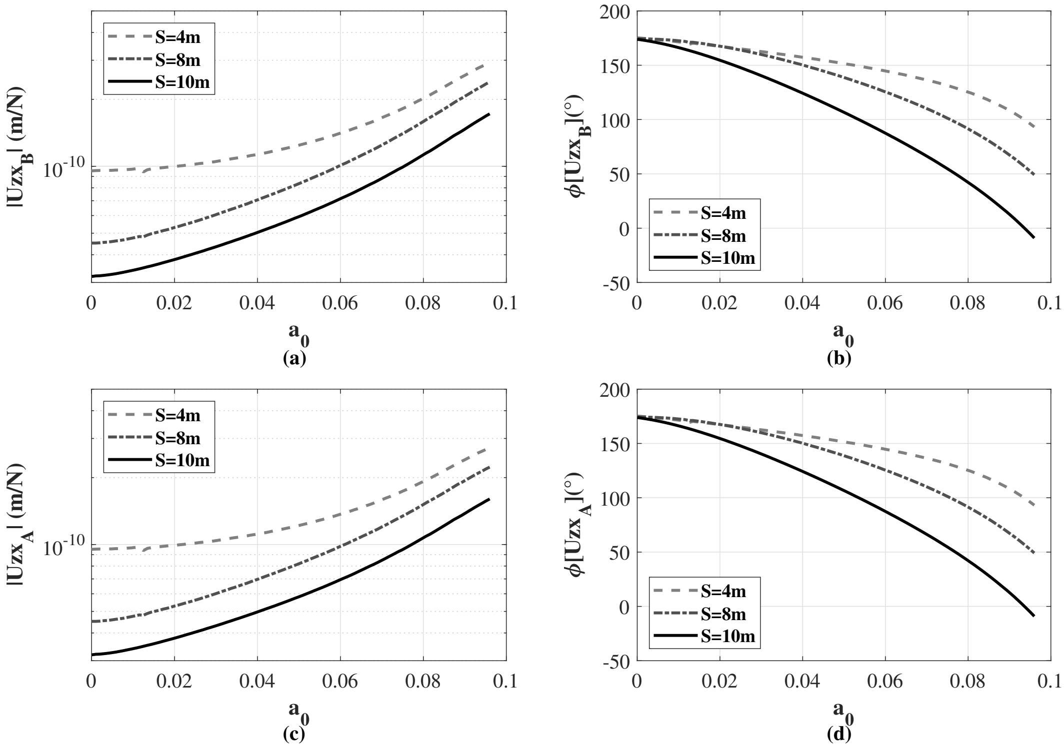

Figure 4. Vertical response of the tower due to horizontal loading at the source.

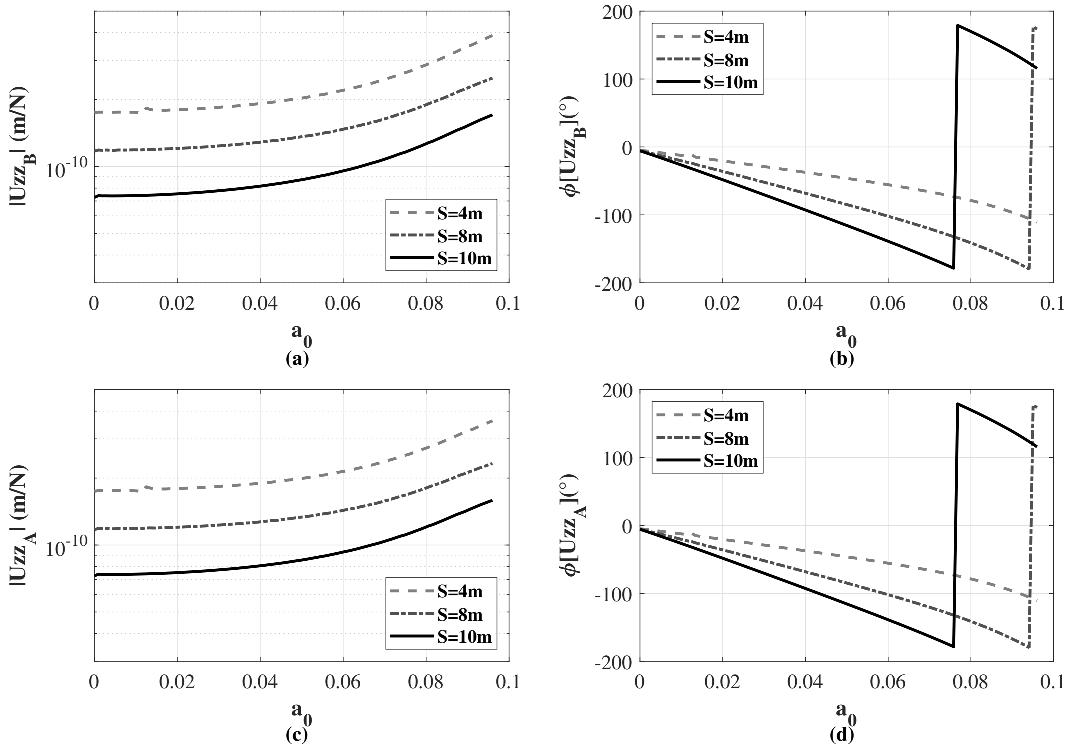

Figure 5. Vertical response of the tower due to vertical loading at the source.

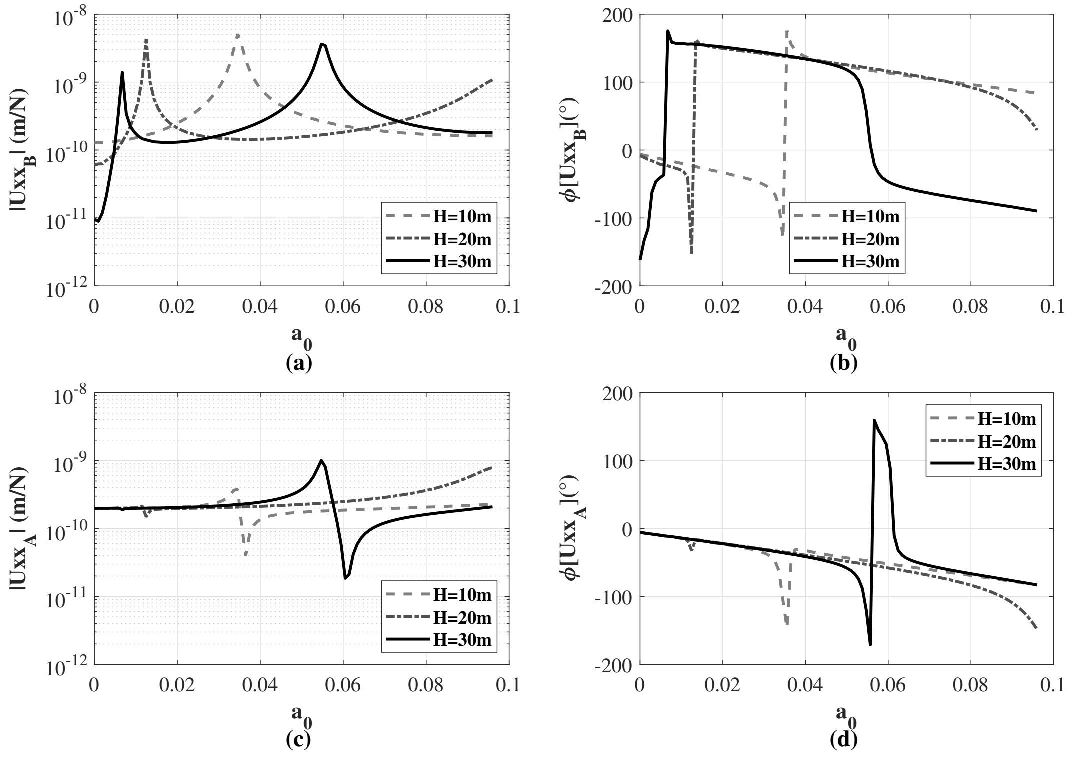

Figure 6. Horizontal response of the tower due to horizontal loading at the source.

4 Conclusions

This paper presented a study of the energy exchange between piled structures through the soil. The case of two interacting structures was considered. Results showed that the vibratory response of the target structure is

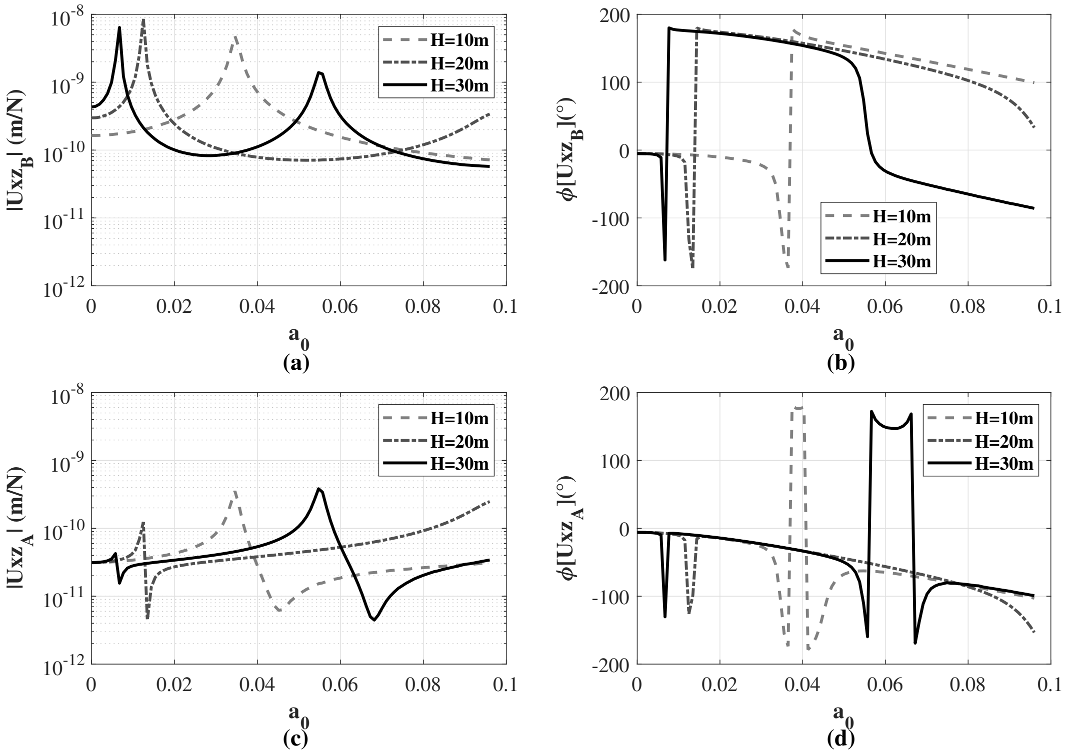

Figure 7. Horizontal response of the tower due to vertical loading at the source.

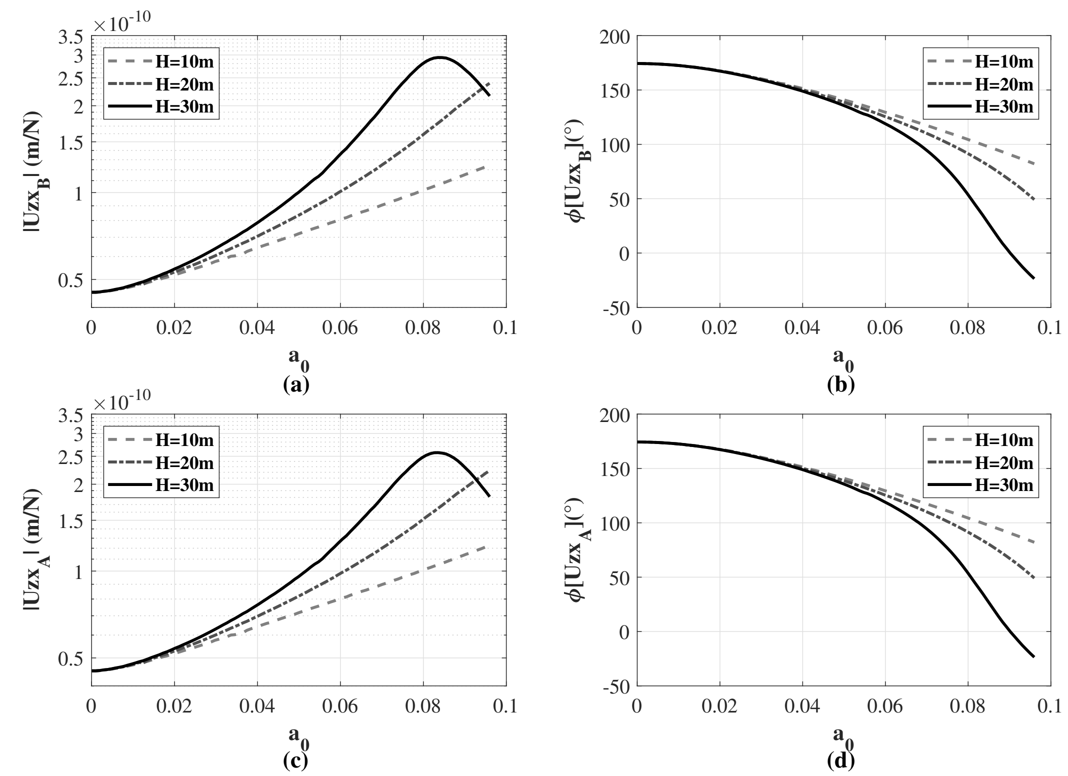

Figure 8. Vertical response of the tower due to horizontal loading at the source.

strongly dependent on its height, but also strongly dependent on its distance from the source of vibration.

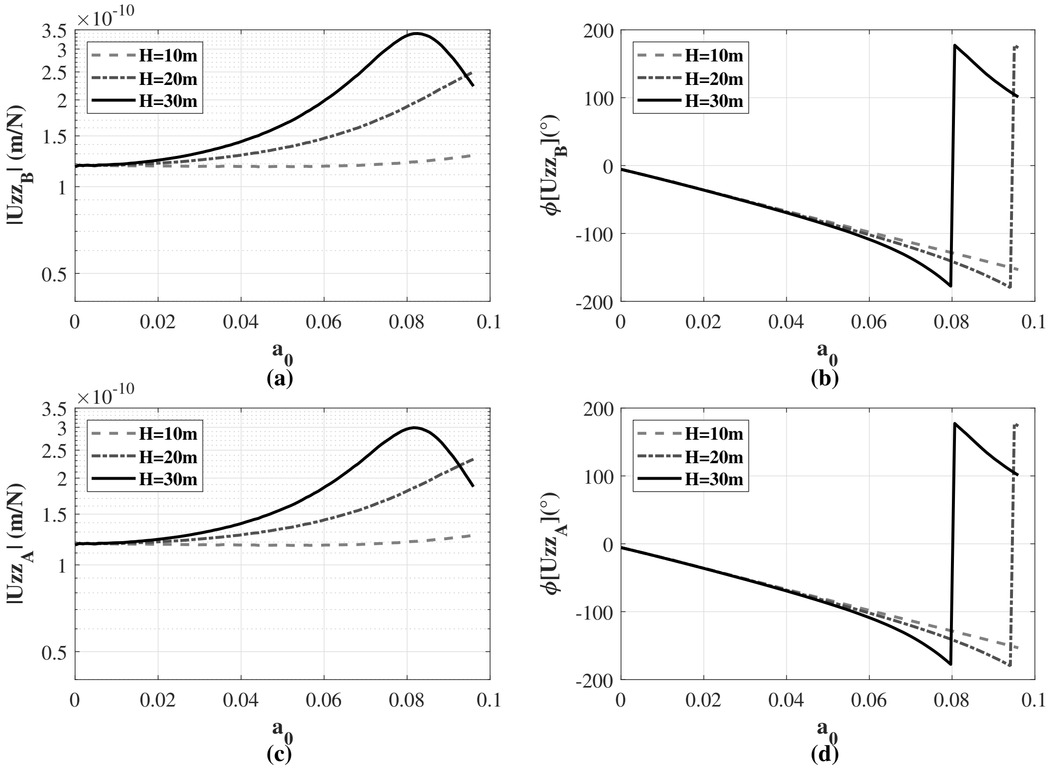

Figure 9. Vertical response of the tower due to vertical loading at the source.

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