International Journal of Bridge Engineering (IJBE), Vol. 3, No. 1, (2015), pp. 1-9

ASPECTS OF THE SOIL-STRUCTURE INTERACTION

The dynamic response of bridge piers on caisson foundations

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ABSTRACT: Site conditions and soil flexibility play an important role in determining the seismic response of bridges. In particular, the motion imposed at the foundation can differ from that in the free-field and relevant rocking component may be induced. Using simple analytical methods, the dynamic response of a bridge-soil system excited by harmonic S-waves is investigated.

KEY WORDS: Kinematic interaction; Inertial interaction; Caisson foundation.

1 INTRODUCTION

Under strong earthquake, several highway structures such as viaducts, bridge piers and abutments exhibited damage in the past owing to excessive displacements and deflections, in general. Most part of these lateral movements were induced by soil liquefaction even if lateral spreadings generated by non-liquefied crust were also detected.

As summarized by Sextos et al. [1], other important features that showed their influence in the seismic behavior of extended structures such as bridges are: 1) the spatial and temporal variations of seismic motion at the support points; 2) the local site conditions which can strongly modify the surface motion; 3) the deformability of the soil that may lead to erroneous evaluation of structural displacements. In particular, the key role of the dynamic soil-structure interaction (SSI) on the design of structures with massive or deep foundation is well documented and recognized in the literature [2].

A first kinematic consequence of the interaction between soil and foundation derives from the propagation of the seismic waves which makes the soil motion at any given instant generally different from point to point. A relatively stiff foundation produces an averaging effect in which the overall motion at the foundation interface, u_{ko}, is less than the maximum displacement, u_{ffo}, that would have occurred in the free-field soil, i.e. in the absence of the structure. Moreover, the distribution of soil displacements along depth is incompatible with the rigid lateral movement of the side-walls of the foundation, so that a rotation (rocking) component, θ_{ko}, develops.

Both base-slab averaging and embedment effects cause foundation motion (usually called *foundation input motion*, or FIM) to deviate from free-field motion in a manner that is independent of the superstructure. These occurrences are strongly frequency-dependent as they are influenced by the wavelength of seismic waves compared to the dimension of the foundation elements. This type of interaction is known as *kinematic interaction*. In the case of a shallow foundation, the kinematic components u_{ko} and θ_{ko} are usually negligible respect to the motion (u_o and θ_o) which is in turn generated at the foundation level by the oscillation of the superstructure. This latter phenomenon is called *inertial interaction*. Kinematic interaction should be in principle more relevant for drilled shafts and caissons, owing to the foundation size and embedment.

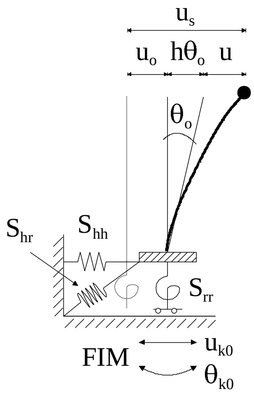
Although earthquake response of bridges should be evaluated with a direct analysis capable of modeling the entire system composed of the superstructure, foundation and the supporting soil, to date the state of practice is usually restricted to a multistep approach, which makes use of the superposition theorem. It consists of: (a) evaluating the free-field response of the site; (b) solving the kinematic interaction, i.e. the response to incident seismic waves of the soil-foundation system with the mass of the superstructure set equal to zero; (c) determining the inertial interaction, that is the response of the overall soil-foundation-superstructure system to forces associated with accelerations arising from the kinematic interaction.

As a further simplification, kinematic interaction can be reduced to the evaluation of the foundation input motion, disregarding the presence of the superstructure. The FIM is subsequently applied at the base of the superstructure (Fig. 1). In the inertial interaction, the soil-foundation system is conveniently modeled with springs and dashpots (dynamic impedances) associated with each mode of vibrations. The response of the structure is thus determined.

In this paper, some aspects of the dynamic behavior of a bridge pier founded on caisson subjected to harmonic shear waves are analysed. The bridge is idealized as a single-degree-of-freedom (SDOF) structure and the foundation is represented by a rigid block connected to the surrounding soil by a series of springs and dashpots according to the Winkler-type model developed by Gerolymos and Gazetas [3].

2 OUTLINES OF THE METHOD USED

Although the seismic analysis of bridge structures has received considerable attention in recent years, some aspects concerning the effects of soil-structure interaction are not yet completely investigated and clarified. Bridge piers, together with the abutments, constitute one of the most critical elements in securing the safety of bridges during earthquakes. They are widely supported by drilled shafts and caissons embedded in soft soil or founded directly on a firm stratum.



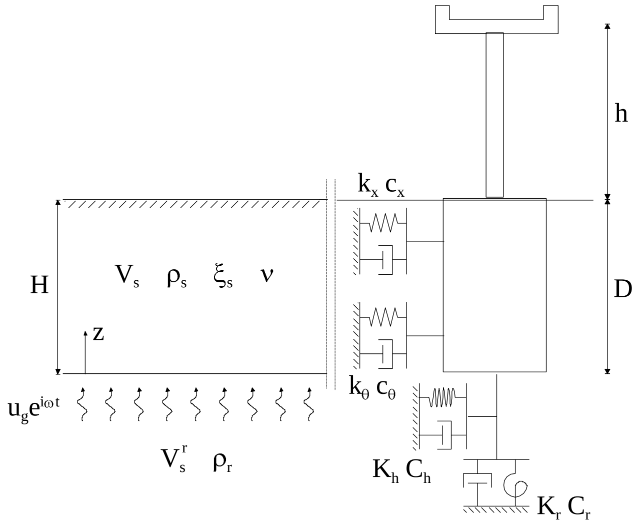


*Figure 1.*Idealized model studied *Figure 2.* Soil deposit and bridge-pier on Winkler foundation.

A Winkler-type model for the dynamic response of rigid caisson foundations has been developed by Gerolymos and Gazetas [3]. It consists of four types of linear springs and dashpots (impedances): two distributed translational and rotational impedances on the shaft of the caisson, two concentrated translational and rotational impedances, indicated with capital letter, at the base (Fig. 2). The spring coefficient k (K) takes into account the stiffness and inertia of the supporting soil and is therefore termed dynamic stiffness; c (C) is the dashpot coefficient which reflects the radiation and material damping generated in the system. As known, in the frequency domain each dynamic impedance component can be written in complex notation as

$$\bar{k} = k + i a_0 c \tag{1}$$

where $i = -1^{0.5}$; $a_0 = \omega B / V_s$ is the dimensionless frequency being ω the circular frequency of the harmonic excitation, B the diameter or width of the caisson, V_s the shear wave velocity of the soil. Both k and c are functions of ω.

The impedance matrix of the soil-foundation model, referred to the base of the caisson, takes the following form [3]

$$[S] = \begin{bmatrix} \bar{K}_h + \bar{k}_x D & \bar{k}_x D^2/2 \\ \bar{k}_x D^2/2 & \bar{K}_r + \bar{k}_\theta D + \bar{k}_x D^3/3 \end{bmatrix} \tag{2}$$

In general, the dynamic impedances of rigid caissons come indirectly from studies of the dynamic response of embedded footings [3,4]. According to Tsigginos et al. [4], the impedances of the base of the caisson can be selected as the springs, K, and dashpots, C, coefficients of a surface foundation on halfspace or on a soil stratum underlain by a homogeneous halfspace. The lateral springs, k, and dashpots, c, of the equivalent Winkler model can be derived by equating the translational and rocking components of the impedance matrix in Eq. (2) with the aforementioned elastodynamic solution [3,4] or by curve-fitting with 3D finite element simulations [2]. In this study, the simple relations of the dynamic spring and dashpot coefficients furnished by Varun et al. [2] are used.

3 BRIDGE-SOIL SYSTEM STUDIED

In this section the dynamic response of a bridge pier founded on rigid caisson is studied (Fig. 2). The superstructure is idealized as a SDOF system with lumped mass m=450 Mg, height h=10 m, elastic stiffness k=5.9×10^4 kN/m and damping ratio ξ=5%. For fixed-base response, the structure has undamped natural frequency $\omega_s=(k/m)^{0.5}$=11.45 rad/s that corresponds to 1.8 Hz. The base is assumed to be a square prismatic block of height D=8 m, width B=4 m, mass m_o=320 Mg, embedded in a uniform elastic soil stratum overlying a homogeneous halfspace. The system is subjected to vertically propagating harmonic S-waves. The soil-foundation interaction is solved by the aforementioned Winkler-type model for both kinematic and inertial interaction.

3.1 Kinematic interaction

The first step of the kinematic analysis is to evaluate the free-field response of the soil deposit. To this end, a soil layer resting on bedrock is considered (Fig. 2). The soil has thickness H=8 m, shear wave velocity V_s=100 m/s, damping ratio ξ_s=0.10, mass density ρ_s=2 Mg/m^3 and Poisson's ratio ν=0.4. The bedrock is represented by a homogeneous halfspace with shear wave velocity V_s^r and mass density ρ_r. Assuming a harmonic motion with circular frequency ω and amplitude u_g at the bedrock level, the displacement of the soil with the depth z can be evaluated [3].

This wave pattern imposes forces and moments at the supports of the distributed springs and dashpots along the caisson height and at the concentrated impedances of the base. By solving the dynamic equilibrium of the caisson in the absence of the superstructure, the horizontal displacement u_{ko} and the rotation θ_{ko} (foundation input motion) of the caisson are calculated.

Figure 3 plots the components of the FIM as a function of the ratio of the excitation frequency ω to the fixed-base natural frequency ω_s of the super-structure. Two cases are examined: one in which the caisson is embedded in a homogeneous halfspace $(V_s^r = V_s)$, the other, termed layered, with the caisson embedded in a soft soil overlying a firm stratum $(V_s^r = 4V_s)$. The amplitudes (absolute values) of the input motion are normalized by the amplitude of the surface free-field motion (*kinematic factors*). The solutions obtained with the mass of the caisson m_o set equal to zero are also shown.

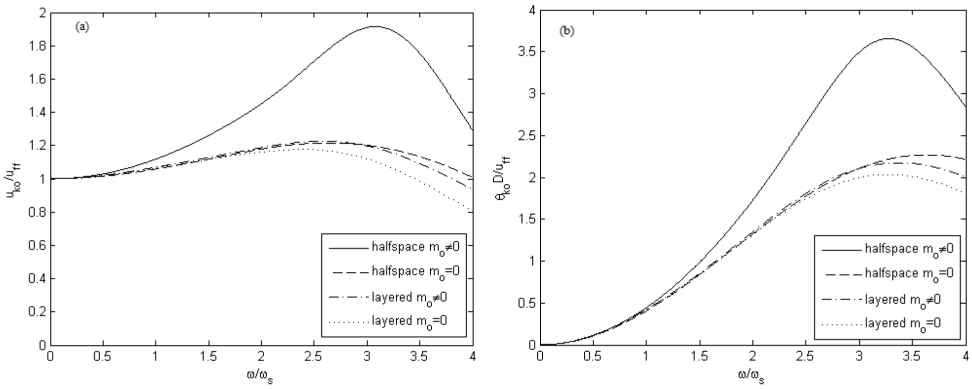


Figure 3. Kinematic response of the foundation as a function of frequency ratio

As can be noticed, for a frequency ratio ω/ω_s<3 the kinematic response of the caisson u_{ko} is greater than the surface free-field displacement (Fig. 3a). This is caused by the rapid development of the rocking component θ_{ko} in the same frequency range (Fig. 3b). At higher frequencies, u_{ko} tends to attenuate rapidly and becomes minor than the movement of the ground, although significant values of the rocking motion remain. The more pronounced kinematic effects are found when the foundation is embedded in a homogenous halfspace. In this case, the mass of the caisson determines an increase of the kinematic factors. When the caisson is founded directly on the bedrock, the kinematic response reduces and the influence of mass m_o is practically negligible.

3.2 Inertial interaction

In the inertial interaction, the soil-foundation system is modeled through the dynamic impedances computed at the top of the caisson (Fig. 1), obtained by means of coordinate transformation of the impedance matrix of Eq. (2) [3].

The response of the bridge pier to the foundation input motion with amplitude u_{ko} and θ_{ko} is thus evaluated solving the dynamic equilibrium of the idealized structural system sketched in Fig. 1. The foundation has two degrees of freedom consisting of the horizontal displacement with amplitude u_o and rocking with amplitude θ_o. The elastic horizontal displacement of the top mass relative to the base mass has amplitude u. The latter is representative of the shear force Q_o and overturning moment M_o at the base. The total displacement amplitude of the superstructure results $u_s = u_0 + h\theta_0 + u$.

Formulating dynamic equilibrium of the mass of the superstructure and the translational and rotational equilibrium of the entire system yields

$$\begin{bmatrix} -1+\dfrac{1+2i\xi}{(\omega/\omega_s)^2} & -1 & -1 \\ -1 & \dfrac{S_{hh}}{\omega^2 m}-(1+\overline{m}) & \dfrac{S_{hr}}{\omega^2 mh}-1 \\ -1 & \dfrac{S_{hr}}{\omega^2 mh}-1 & \dfrac{S_{rr}}{\omega^2 mh^2}-\left(1+\dfrac{I_t}{mh^2}\right) \end{bmatrix} \begin{Bmatrix} u \\ u_0 \\ h\theta_0 \end{Bmatrix} = \begin{Bmatrix} 1 \\ 1+\overline{m} \\ 1 \end{Bmatrix} u_{k0} + \begin{Bmatrix} 1 \\ 1 \\ 1+\dfrac{I_t}{mh^2} \end{Bmatrix} h\theta_{k0} \qquad (3)$$

being $\overline{m} = m_0 / m$ and I_t the total centroidal moment of inertia of the masses.

The total displacement amplitude of the structure normalized by the amplitude of the surface free-field displacement is presented in Figs. 4-5. The solution disregarding the soil, i.e. for fixed-base structure, is also presented. This corresponds to the well-known frequency response curve with the peak of $1/2\xi$ occurring at ω/ω_s=1.

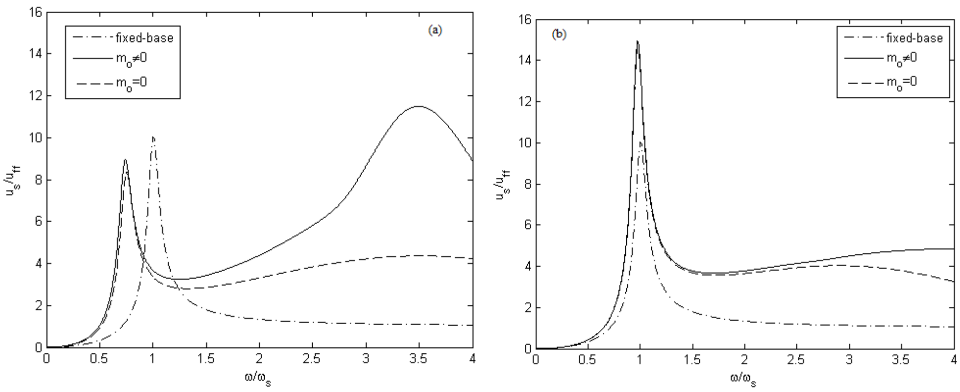


Figure 4. Frequency response curve of the structure: homogeneous (a) and layered (b) soil

When the soil is assimilated to a homogeneous halfspace (Fig. 4a), the peak response of the coupled soil-structure system results smaller than that of the same structure on a rigid base and occurs at a lower frequency, corresponding to a more flexible system. This curve is also broader, indicating that the damping is larger due to radiation in the surrounding soil. A different behavior is exhibited when the caisson is based on the bedrock $(V_s^r = 4V_s)$, as shown in Fig. 4b. The peak response occurs nearly at the fixed-base natural frequency, but it is considerably greater than the displacement of the structure when the presence of the soil is neglected. In this case, the movement of the caisson is restricted by the presence of the bedrock in such a way that the fundamental natural frequency of the system is not affected by the flexibility of the soil. However, the displacement of the structure tends to be larger because the amplitude of the motion imposed at the foundation is higher than that of the free-field (Fig. 3a). Furthermore, radiation damping reduces because of the presence of the bedrock.

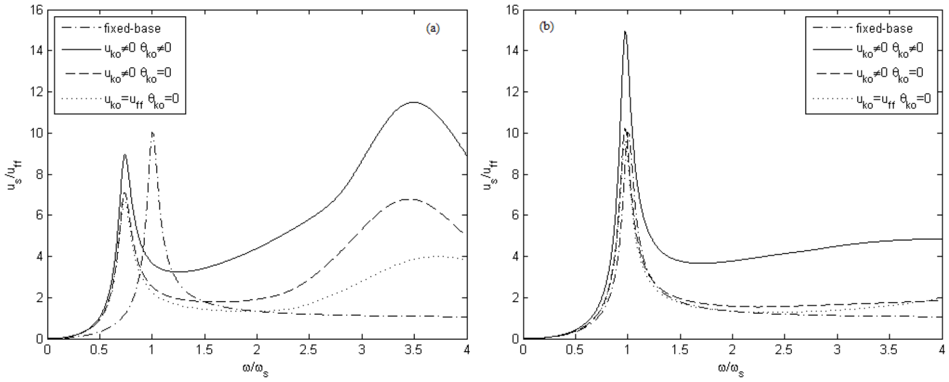


Figure 5. Frequency response curve of the structure for different foundation motion

In the high frequency range, the foundation compliance influences significantly the response of the bridge (Fig. 4). This interaction effect can be correlated to the large values of the rocking component of the FIM, especially when the foundation is embedded in homogeneous halfspace (Fig. 4a). In Fig. 5 the importance of kinematic interaction on the frequency response curves is depicted. As can be seen, large amplitudes of u_s at high frequencies are essentially due to the presence of the rocking component θ_{ko} of the foundation input motion (Fig. 5a). This effect is evident when the base of the caisson rests on the bedrock (Fig. 5b). Moreover, it is worth noting that the assumption of the foundation input motion does not modify the natural frequency of the soil-structure system.

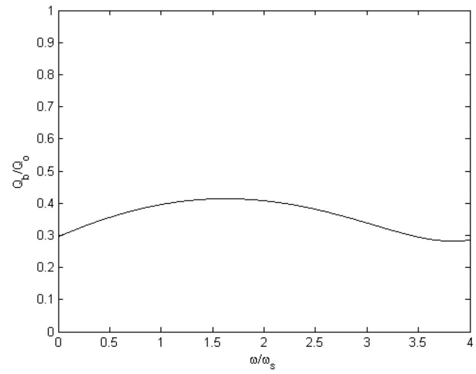


Figure 6. Shear forces acting on the caisson as a function of frequency

Finally, the ratio of the shear force at the base of the caisson Q_b to that at the top Q_o as a function of the frequency ratio is displayed in Fig. 6 in the case of layered soil. It can be observed that a small part of the force induced by the oscillation of the bridge and generated by the mass of the caisson is transmitted at the foundation as a consequence of soil-foundation interaction. The overturning moment at the base M_b is set equal to zero since the corresponding rocking spring K_r and dashpot C_r coefficients have been ignored for the aspect ratio D/B considered [2]. More accurate analyses have shown that M_b is almost negligible compared with the overturning moment M_o induced at the top (<10%) in the whole frequency range [5].

4 CONCLUSIONS

From the presented results, the following remarks can be drawn:

a) the kinematic interaction for caisson foundations is characterized by a significant rotational component, which leads to a lateral displacement greater than the surface free-field motion in a wide frequency range;
b) the soil profile can play an important role both in kinematic and inertial interaction: in the case of a homogeneous soil, the fundamental natural frequency of the soil-structure system is reduced with respect to that calculated ignoring SSI and the peak response becomes smaller, owing to radiation damping; when the base of the caisson is embedded in a soil layer underlain by a stiffer halfspace, the peak response occurs at the fixed-base natural frequency, but may be considerably greater than the displacement of the structure determined neglecting the soil. Similar amplification of structural response is attained in the high frequency range, owing to the large values of the rocking component of the foundation input motion;
c) the overturning moment induced at the base of the caisson is negligible respect to that acting at the top.

Although the used approaches are based on some approximate assumptions, the conclusions drawn in the present study can be of help in predicting the seismic response of bridges on deep foundations, or in interpreting the results of more rigorous numerical studies.

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Received: Aug 31, 2014 Accepted: Oct 15, 2014