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Dynamic Response of Chimneys Interacting with Soil

Réponse dynamique d'une cheminée et du sol de fondation

Dynamisches Verhalten eines Schornsteines bei Zusammenwirken mit dem Baugrund

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SUMMARY
In the present paper, some results obtained by the authors are presented concerning dynamic soil-structure interaction in wind and earthquake analysis of slender chimneys. In particular, the influence of the soil flexibility is taken into account for both alongwind and crosswind response, and the characteristics of the earthquake response are discussed in general terms.

RESUME
La communication présente les résultats de l'analyse d'interaction sol-structure de hautes cheminées pour les actions du vent et des tremblements de terre. En particulier, l'influence de la flexibilité du sol a été prise en compte, soit pour la réponse dans le plan du vent soit pour la réponse dans le plan normal. Les caractéristiques générales de la réponse au séisme ont été également considérées.

ZUSAMMENFASSUNG
Der vorliegende Artikel behandelt das Problem der Wechselwirkung des Bodens und eines schlanken Schornsteins unter dynamischen Belastungen wie Wind und Erdbeben. Speziell wird auf den Einfluss der Bodenflexibilität bei Längs- und Seitenwind eingegangen. Das dynamische Verhalten bei Erdbebenerregung wird kurz erörtert.

1. INTRODUCTION

The recent tendency of realizing more and more slender chimneys, is due to the need of ejecting the smokes in the higher atmosphere, so that the smoke dispersion and the ash fall-out will take place as far as possible from the source. The reason of this need is based on the fact that industrial plants are often poles of aggregation of minor activities and it is therefore desirable to prevent pollution in the neighbouring areas.
By the other hand, progressing design and construction technologies allow very light structural solutions thus making dynamic analyses more and more important. The Authors did some research activities in this field, dealing especially with soil-structure interaction effects when wind and earthquake loads are of concern. The present paper can be regarded as a synthesis of such researches and also contains some additional considerations.

2. SOIL-STRUCTURE INTERACTION EFFECTS

In the context of dynamic analyses of slender structures, the presence of the soil is usually taken into account by applying, at the base of the structure itself, frequency dependent springs and dash-pots. This idealization is based, of course, upon the assumption that the structures have direct foundations; that is, the effects of moderately flexible soils only can be modelled in this way. The stiffness and damping values can be computed in the frequency domain according to a variety of techniques. Analythical methods are available, indeed, for rigid circular or elliptical footings, embedded or not, resting on half-spaces or strata, of a homogeneous linearly visco-elastic nature. Some of the most popular solutions are available in Refs [1-3] .
Semianalytical procedures can be derived, for instance, from Ref. [4], in which, by means of one and two-dimensional Fourier transforms, combined with a transfer matrices solution of the wave propagation problem [5], stiffness and damping functions can be obtained for strip or rectangular rigid surface footings, respectively, on layered soils.
Rectangular foundations can be treated, under the same conditions, by an ad hoc finite element technique [6].
It should be pointed out that the two latter methods can also be applied to the stiffness analysis of foundation systems, composed by two or more footings of the same kind [7].
More general cases can obviously be treated by means of general pourpose finite element procedures.
Several alternative techniques are also available to analyze the structural behaviour, including soil flexibility.
Continuum and discrete approaches can be used to this pourpose both in time and in frequency domain.
Some more comments are due to the extension of modal analysis to soil-structure interaction problems because, as already mentioned, the foundation stiffness and damping coefficients are frequency dependent and therefore the soil-foundation-structure system does not possess classical normal modes. Indeed, it has been shown in Refs. [8,9] that modal superposition can still be used giving rise to rather small errors but relevant simplifications.
Substantially the presence of the soil induces the following three main effects:

a) rigid-body motions are added to the state of displacement of the structure
b) the natural frequencies of the coupled system are different from the ones belonging to the structure alone
c) the energy dissipation in the coupled system is the sum of the energy dissipation within the structure plus the energy dissipation in the soil due to the mechanisms of hysteresis and radiation.

The consequences on the dynamic behaviour of the structure can be different with respect to the kinds of external loads; in the next paragraphs these consequences will be discussed in more detail.

3. WIND LOADS

It is well known that, when the wind is acting on a chimney, the resultant force can be split in two components: a drag force, in the plane of the wind, and a lift force, in the normal plane. Usually, the dynamic analysis is performed separately for the two kinds of forces and the corresponding results are termed alongwind response and crosswind response, respectively.

This traditional approach is justified by the fact that the strongest vibrations in the wind plane and in the cross plane are due to winds of different characteristics. The former ones are consequence of strong gusty winds while the second ones are produced by winds characterized by moderate velocities.

3.1 Alongwind Response

A general formulation of the problem of the dynamic alongwind response of structural systems including soil flexibility is given in Ref.[10]. Essentially, the theoretical formulation can be summarized as follows.

When the wind is idealized as a stationary Gaussian random process, the alongwind displacement, at height z, may be written as:

$$Y(z) = \overline{Y}(z) + g_y(z)\ \sigma_y(z) \tag{1}$$

where $\overline{Y}(z)$ is the mean displacement, $g_y(z)$ is the peak factor and $\sigma_y(z)$ is the root mean square of the fluctuating displacement, expressed by the integral over the frequency domain of the spectral density of the displacement S_y:

$$\sigma_y^{\,2}(z) = \int_0^\infty S_y(z;n)\ dn \tag{2}$$

In Ref.[10], by utilizing modal analysis, it was shown that, also when soil-structure interaction is to be taken into account, the contribution of the higher modes than the first can be neglected as it is already usual in the analysis of clamped structures.

In Ref.[11] it was also proved that a linear shape can be assumed for the first mode still getting good approximations in the analysis of soil-structure systems.

By the above considerations, together with the hypothesis of small damping, eq. (2) can be simplified and takes at the top of the structure, that is at the height H, the following form:

$$\sigma_y^{\,2}(H) = \frac{1}{(2\pi n_1)^4\ m_1^{\,2}}\ \left\{\int_0^\infty S_p(n)\ dn + \frac{\pi n_1}{4\xi(n_1)}\ S_p(n_1)\right\} \tag{3}$$

in which n_1 is the first natural frequency; m_1 is the first modal mass; S_p is the spectral density of the alongwind excitation; ξ is the damping computed at the frequency n_1.
Simple expressions to calculate n_1 and $\xi(n_1)$ are contained in Ref.[11].
In Ref.[10] it was shown that the key parameter for understanding the correct influence of soil flexibility is the ratio between the energy dissipated internally in the structure and the energy dissipated in the soil by both radiation and hysteretic damping. More precisely, when the former energy is prevailing on the latter energy, as in the case of large structural damping, the amplification of the response due to rigid-body motion and to the decrease of n_1, is prevailing on the damping effects. In all the other cases, these effects are the most relevant.
The first situation is typical of reinforced concrete chimneys, while the second one is typical of unlined welded steel stacks.
Within the practical limits of direct foundations, it should be observed that in absence of large rigid-body motions, the variation of the fundamental frequency does not originate significant effects on the response.
By the other hand, when the structural damping is small, also relatively good soils can give rise to an energy dissipation which can be very large. This does not mean, however, that the structural response will be significantly modified as the resonant part only of the response (second term in brackets in eq.(3)) will be affected by such a large soil damping.
For instance, being 0.2% a typical value of the hysteretic damping of a 100 m height unlined welded steel stack and assuming a 5% hysteretic damping in a soil with G=1000 daN/cm^2, it will be found that the equivalent damping is 0.43% while the reduction in the top displacement will be about 10%.

3.2 Crosswind response

It is well known that a bluff chimney sheds alternating vortices whose primary frequency n_s is, according to the Strohual relation:

$$\frac{n_s D}{V} = \mathcal{S} \tag{4}$$

in which the Strohual number $\mathcal{S}$ depends on the Reynolds number, while D is the diameter of the chimney and V is the velocity of the wind.
Under the action of vortex shedding the structure oscillates in a plane normal to the wind direction; in particular, when the frequency n_s approaches the fundamental frequency of the chimney n_1, a typical resonant excitation arises. In this case the wind velocity V_c is defined critical velocity:

$$V_c = \frac{n_1 D}{\mathcal{S}} \tag{5}$$

It is easy and immediate to observe that, when taking the soil flexibility into account, the decrease of the fundamental frequency n_1 determines a decrease of V_c, leading to a reduction of the dynamic external loads.
It is well known that the key parameter of this dynamic behaviour is the damping. When damping is large, which is the case of reinforced concrete chimneys, it is possible to notice that the crosswind response of the structure is a random

process, and that the displacement is proportional to $\xi^{-1/2}$.
This kind of behaviour can be correctly estimated by utilizing a prediction procedure based on a random excitation model. Then the maximum displacement in the crosswind direction may be expressed in the usual form

$$X(z) = g_x(z)\ \sigma_x(z) \tag{6}$$

A general formulation to predict the standard deviation σ_x is contained in [12]. However, by introducing the same hypotheses already used in the evaluation of the alongwind response, the top value of σ_x may be expressed in the following form:

$$\sigma_x^2(H) = \frac{1}{(2\pi n_1)^4\ m_1^2}\ \frac{\pi n_1}{4\xi(n_1)}\ S_c(n_1) \tag{7}$$

S_c being the spectral density of wake excitation.
By the other hand, at low values of damping, which are typical of unlined welded steel stacks, displacement dependent lock-in excitation become significant, giving rise to a large increase in cross wind excitation forces. In other words the increase in crosswind response causes an essential interdipendence between the crosswind excitation and the response process. In this case two main effects can be observed: the crosswind forces become well-correlated along the chimney, while the response becomes proportional to ξ^{-1} ; consequently a sinusoidal excitation model must replace the random model to predict the response.
Giving to the lift forces the following expression:

$$F_L(z,t) = C_L\ \frac{\rho V_c^2 D}{2}\ \sin 2\pi n_1 t \tag{8}$$

the maximum crosswind top displacement becomes:

$$X(H) = \frac{C_L \rho D^3 H}{2\xi(n_1)\ m_1\ (4\pi\mathscr{S})^2} \tag{9}$$

in which C_L is the lift coefficient and ρ the air density.
In conclusion it should be pointed out that the crosswind response is completely controlled by the amount of damping present in the system.
From eqs. (7) and (9) indeed it can be seen that the resonant term is the only one appearing in the expression of the response.
In the same example mentioned while analyzing the alongwind response, the same damping increase will now produce a 50% reduction in the top displacement.

4. EARTHQUAKE LOADS

The major difference between earthquake and wind analysis is that in the former one also modes higher than the first are to be taken into account. Indeed the first natural frequency of such structures is generally very low and subsequent frequencies only fall into the most significant range of frequency content of an earthquake. Due to this fact the influence of soil flexibility has different

implications as in the wind case.
First of all, if the first natural frequency is not greatly modified by the presence of the soil, higher natural frequencies can differ very significantly from the clamped situation. For instance, still referring to the already mentioned example, the third natural frequency is reduced by a 20%.
Secundly the variation of the system damping associated with higher modes may be essential but quite impredictable by simple formulas.
In conclusion, it can be affirmed that soil-structure interaction effects are important for a realistic evaluation of the dynamic structural behaviour, although they cannot be described and categorized as simply as in the wind case.
However, a phenomenon which is significant in earthquake excitation and which should be mentioned is the following. Let us consider the presence of a large heavy building in the neighborouds of a chimney. As shown in Ref. [13] this condition can significatly alter the resonant amplification of the chimney itself.
For values of the structural parameters typical of conventional power plants the second natural frequency of the chimney is very close to the first frequency of the other building, thus generating a very high mutual interaction effect.

REFERENCES

1. VELETSOS, A.F. and WEY, G.T.: Lateral and Rocking Vibration of Footings, J. Soil Mech. Found. Div., ASCE, 97, 1971.
2. BYCROFT, G.N.: Forced Vibrations of a Rigid Circular Plate on a Semi Infinite Elastic Space and on an Elastic Stratum, Phil. Trans. Roy. Soc. London, Series A, 248, 1965.
3. NOVAK, M. and SACHS, K.: Torsional and Coupled Vibrations of Embedded Footings, Earthq. Engng. Struct. Dyn., 2, 1973.
4. GAZETAS, G.C.: Dynamic Stiffness Functions of Strip and Rectangular Footings on Layered Media, S.M.Thesis, Dept. Civ. Eng., M.I.T., 1975.
5. ROESSET, J.M.: Soil Amplification, I.C.C.A.D. Lecture Series n°2/76, Genova, 1976.
6. KAUSEL, E.: Forced Vibrations of Circular Foundations on Layered Media, Ph.D. Thesis, Dept. Civ. Eng., M.I.T., 1974.
7. DEL GROSSO, A., STURA, D. and VARDANEGA, C.: Building-Soil-Building Interaction in Seismic Analysis of Nuclear Power Plants, 5th S.M.I.R.T., Berlin, 1979.
8. JENNINGS, P.C. and BIELAK, J.: Dynamics of Building-Soil Interaction, Bull. Seism. Soc. Am., 63, 9, 1973.
9. ROESSET, J.M., WHITMAN, R.V. and DOBRY, R.: Modal Analysis for Structures with Foundation Interaction, J. Struct. Div., ASCE, 99, 1973.
10. SOLARI, G. and STURA, D.: Dynamic Alongwind Response of a Structural System Including Soil Flexibility, Proc. 5th Int. Conf. on Wind Engineering, Fort Collins, Colorado, 1979.
11. SOLARI, G. and STURA, D.: Effects of Soil-Structure Interaction on the Dynamic Along-Wind Response of Structures, Proc. Int. Conf. on Environmental Forces on Engineering Structures, London, 1979.
12. VICKERY, B.J. and CLARK, A.W.: Lift or Across-Wind Response of Tapered Stacks, J. Struct. Div., ASCE, 98, 1972.
13. DEL GROSSO, A., SOLARI, G. and STURA, D.: Heavy Building-Tall Building Interaction under Wind and Earthquake Effects, Int. Conf. on Engineering for Protection from Natural Disasters, Bangkok, Thailand, 1980.