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Evaluation of a bi-layered soil deposit dynamic properties and behavior by means of accelerometric measures

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Abstract **– The definition of a suitable geotechnical model is a primary task in performing site response analyses. Soil mechanical properties should be carefully estimated and soil profile inhomogeneities should be taken into account to obtain appropriate results. In this paper the dynamic response of a bi-layer soil profile is investigated by means of 1-g shaking table tests. A back-analysis procedure to estimate the mechanical parameters and the inhomogeneity rating of the soil profile, by means of accelerometric measures, is presented. The dynamic response of the geotechnical model obtained is simulated through a 1-D approach and compared with experimental results.**

I. INTRODUCTION

An extensive experimental activity has been performed at the BLADE laboratory of Bristol University (UK). Small scale shaking table tests have been performed on five model piles embedded in a two-layered soil profile [1].
In order to interpret the experimental evidences the definition of an accurate geotechnical model is mandatory. The dynamic parameters and non linear constitutive laws of the soils employed has to be estimated in order to well reproduce the model response by numerical simulations. To this aim an estimation technique based on accelerometric measures during white noise tests has been implemented.

II. EXPERIMENTAL LAYOUT

A two layered soil profile was pluviated into an Equivalent Shear Beam (ESB) laminar container (Figure 1). The ESB consists of 8 rectangular aluminum rings, which are stacked alternately with rubber sections to create a hollow yet flexible box of inner dimensions 1.190 m long by 0.550 m wide and 0.814 m deep [2]. The free surface of the soil deposit was 800 mm above the laminar container floor. The bottom layer was 460 mm thick and was made of LBB and LBE in a 85%-15% granular mix. For this layer a mass density of 1.78 Mg/m^3 was achieved. The upper layer was 340 mm thick, contained LBE sand, and a mass density of 1.39 Mg/m^3 was achieved.
Sixty transducers of different type have been employed to monitor the scale model. Soil response in terms of acceleration has been recorded by using 7 accelerometers (Type SETRA 141A): 5 of these were placed along a vertical line in the ESB laminar container middle plane, their distance from the nearest pile (pile 3) was 85 mm (4 pile diameter circa); the other 2 were placed out of the ESB container middle plane, near the soil surface and the layers interface. Three of these accelerometers have been employed to estimate and monitor the dynamic properties of the soil deposit (Figure 1).

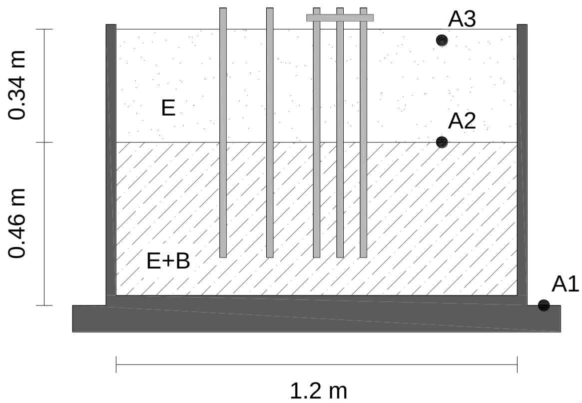

Fig. 1. Accelerometers employed in the back analysis procedure.

III. BACK ANALYSIS PROCEDURE

White noise tests have been performed to estimate and

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monitor soil properties during the experimental activity. Using these tests, soil shear wave velocity and damping ratio initial values have been back-calculated, by means of the elasto-dynamic theory. Transfer functions have been experimentally obtained, for both upper layer, $F_1(\omega)$, and whole deposit, $F(\omega)$, trough A1, A2 and A3 sensors measures.

According to Cairo et al. [3] for a two layer deposit, transfer function can be obtained analytically using Equation 1.

$$F(\omega)=\frac{1}{\cos(\kappa_1 h_1)\cos(\kappa_2 h_2)-I_r\sin(\kappa_1 h_1)\sin(\kappa_2 h_2)} \quad (1)$$

where: κ_1 is the wave number pertaining to the upper layer; κ_2 is the wave number pertaining to the bottom layer; h_1, h_2 are the upper and bottom layer thicknesses, respectively; I_r is the layers impedance ratio.

In addition, it must be noticed that the upper layer transfer function depends only on the upper layer thickness and wave number. For this reason the peak position of the experimental transfer function $F_1(\omega)$ can be used to estimate the upper layer fundamental frequency and consequently its shear wave velocity.

The bottom layer shear wave velocity, then, can be estimated from Equation 1, as the value that provides the experimental deposit transfer function best fitting .

The dynamic properties estimation procedure works as follows:

- Experimental transfer function are obtained (Fig. 2).

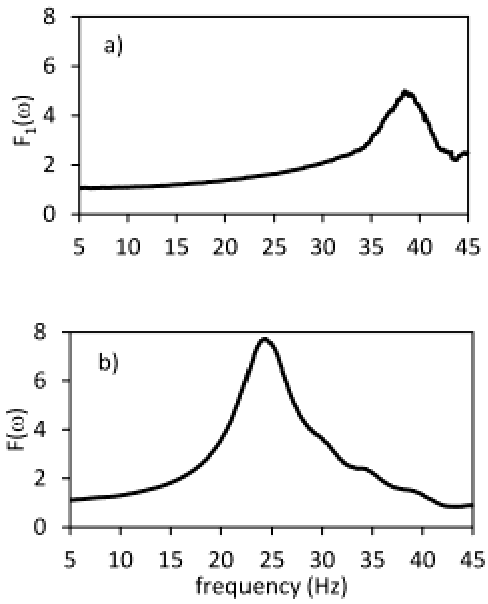

Fig. 2. Amplification functions obtained from a white noise test for a) the upper layer and b) the whole deposit.

- Upper layer first mode frequency (f_N^1) is estimated as the transfer function maximum correspondent abscissa, and upper layer shear wave velocity is calculated from Equation 2:

$$V_{S1}=4 f_N{}^1 \cdot h_1 \quad (2)$$

- Soil deposit damping ratio is estimated by means of the half-power bandwidth method [4].
- Bottom layer shear wave velocity is estimated using a recursive algorithm based on Equation 1. The bottom layer shear wave velocity is selected as the value that generates the best fitting of the deposit experimental transfer function (Figure 3).

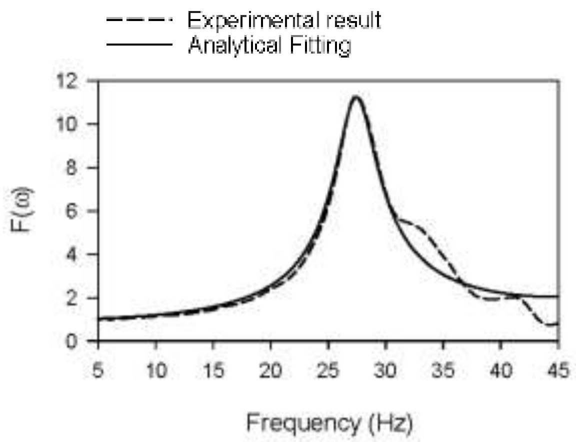

Fig. 3. Example of the deposit experimental amplification function best fitting.

IV. SOILS BEHAVIOR OBSERVED

During the tests, the shear wave velocity of the soil deposit was constantly monitored. From the experimental evidences some interesting observation can be pointed out.

A. *Upper layer behavior*

Two different stages can be recognized in the upper layer material behavior (Figure 4):

- Non-reversible stage: in the first three inspector tests a non-reversible stiffening of the upper layer occurs. In these tests, in fact, as the input motion amplitude increases a shear wave velocity increasing is detected.
- Transient stage: in this stage the upper layer material, under a global point of view, behaves as a quasi linear elastic material. Indeed, it can be

noticed that for each input motion amplitude level, the shear wave velocity value does not exhibit significant variations.

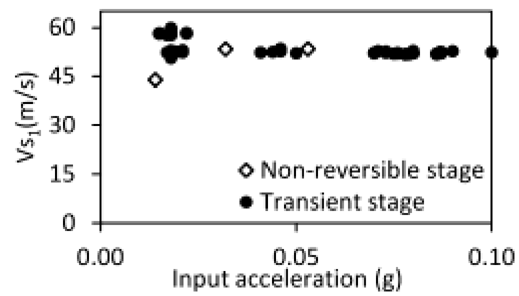

Fig. 4. Upper layer shear wave velocity variation during tests.

B. Bottom layer behavior

Two different stages can be recognized in the bottom layer material behavior (Figure 5):

- Non-reversible stage: in the first three inspector tests, a non-reversible softening of the bottom layer occurs. In these tests, in fact, as the input motion amplitude increases a permanent shear wave velocity decreasing is detected.
- After shaking stage: after the first three inspector tests, the bottom layer granular mix starts to behave as a strain-softening non-linear material (as the input motion amplitude increases the shear wave velocity decreases).

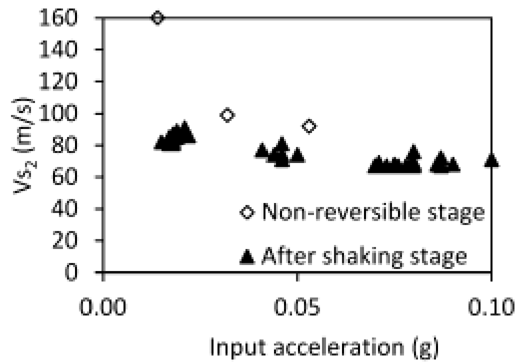

Fig. 5. Bottom layer shear wave velocity variation during tests.

C. Deposit damping ratio

As it could be foresee, the hysteretic soil behavior drives the damping ratio to rise up as the input acceleration amplitude becomes higher (Figure 6).

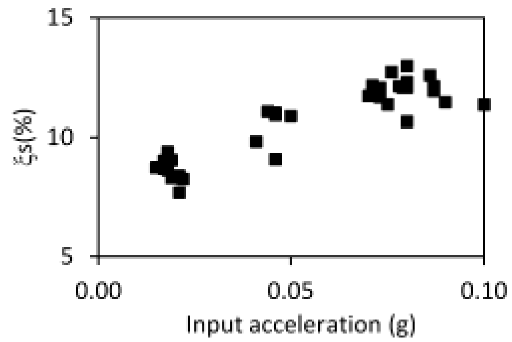

Fig. 6. Damping ratio of the soil deposit during tests.

V. GEOTECHNICAL MODEL AND SITE RESPONSE

A 1-D Parameter lumped mass model [5] has been employed to simulate the deposit response, in which a hyperbolic constitutive law [6] has been incorporated to take into account the non-linear behavior of the soil.
Two different shear wave velocity profiles against depth have been hypothesized (Figure 7): Profile (A) have a constant value of shear wave velocity for both layers; Profile (B) have a constant value of shear wave velocity for the bottom layer, and V_s varying against depth, according to Rovithis et al. [7], for the upper layer.

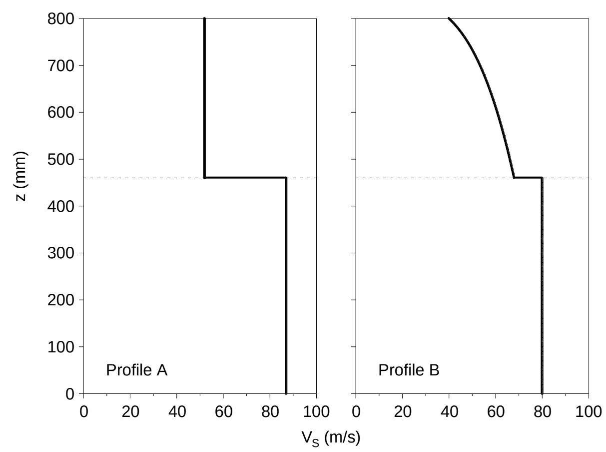

Fig. .7 Shear wave velocity profiles hypothesized

Both the soil profiles have been obtained in order to fit the low acceleration experimental amplification functions.
The dynamic response of the model deposit has been investigated by sinedwell tests (test performed exciting the model with a harmonic modulated signal).
Measured acceleration profiles are compared against the 1-D model results (for both hypothesized shear wave velocity profiles) and against an elastic analytical approach (Figure 7). Accelerations obtained through the elastic analytical approach are not in agreement with the experimental measured values. The numerical results,

obtained considered an hyperbolic back-bone curve (hyperbolic model parameters were estimated according to the experimental data by Cavallaro et al., 2001 [8]), for both profiles A and B, are satisfactory. However, better results were obtained considering an inhomogenues soil profile (Profile B).

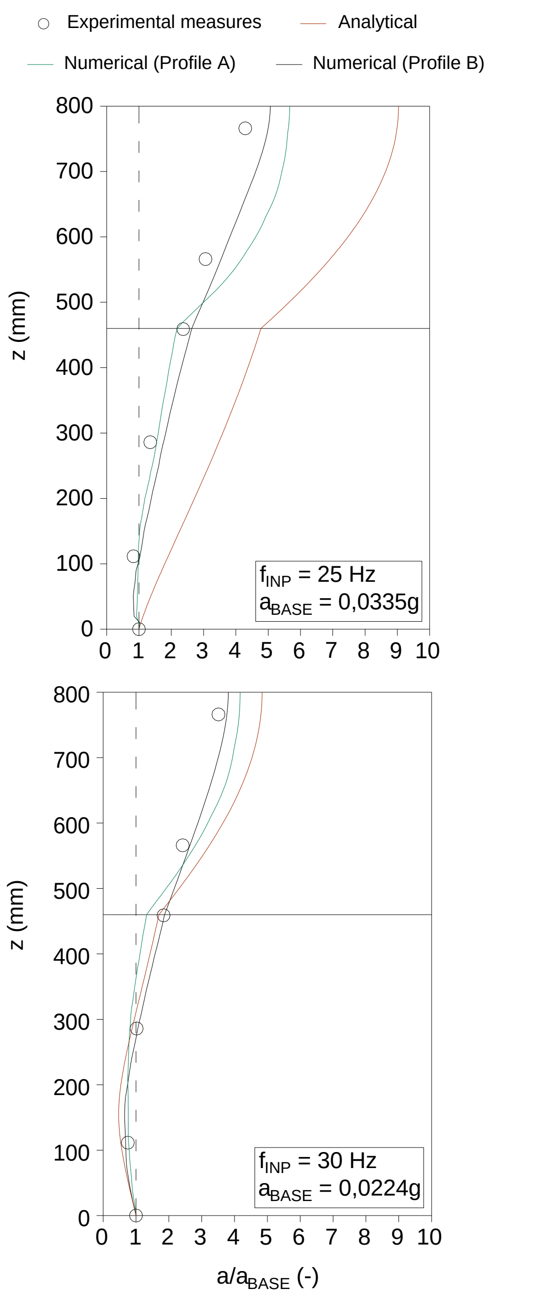

Fig. 8. Non-dimensional acceleration profiles for two different sinedwell inputs: Experimental measures vs. numerical and analytical approaches

The two considered soil profiles generate, both for linear-elastic soil and nonlinear soil, the same fundamental frequency and comparable values of damping. Using a nonlinear constitutive model a deposit de-resonance and a significant increasing of damping is observed. This phenomenon can be attributed to the shear modulus and hysteretic damping strain dependence of the bottom layer soil.

VI. CONCLUSIONS

A simple procedure has been presented to estimate dynamic properties of layered soils starting from accelerometric measures. Comparisons of the experimental measures against a simple numerical model demonstrated the effectiveness of the approach in providing reasonable values of the shear wave velocity and damping ratio of the materials. The interpretation of the experimental evidences underlined that:

- It is possible to define a simplified geotechnical model, for 1-D site response analyses, starting only from accelerometric measures.
- Soil inohomogeneity and non-linearity should be considered in an accurate dynamic analysis because it affect considerably the dynamic response.

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