29 Mar 2001, 4:00 pm - 6:00 pm

Influence of Micro-Structure on Small-Strain Stiffness and Damping of Fine Grained Soil and Effects on Local Site Response

Anna d'Onofrio
Università degli Studi "Federico II", Italy

Francesco Silvestri
Università della Calabria, Italy

INFLUENCE OF MICRO-STRUCTURE ON SMALL-STRAIN STIFFNESS AND DAMPING OF FINE GRAINED SOILS AND EFFECTS ON LOCAL SITE RESPONSE.

Anna d'Onofrio
Dipartimento di Ingegneria Geotecnica
Università degli Studi "Federico II"
Via Claudio 21 - 80125 NAPOLI (Italy)

Francesco Silvestri
Dipartimento di Difesa del Suolo
Università della Calabria
C.da S. Antonello - 87040 Montalto Uffugo (CS) (Italy)

ABSTRACT

Various experimental studies have shown that soil micro-structure strongly influences the small strain stiffness and damping (G_0, D_0) of fine-grained soils, as well as their variation with shear strain γ. This dependency, in turn, is expected to influence the local site response. This paper firstly synthesizes the dependency of G_0 and D_0 on stress state through power relationships, expressed in terms of dimensionless parameters, obtained by fitting a large number of experimental data collected on normally consolidated natural clays. Thereafter, these parameters were correlated to soil micro-structure, by assuming the plasticity index I_P as a representative index property. Subsequently, in order to verify the effect of micro-structure on the local site response, a numerical sensitivity study was carried out on a reference subsoil by introducing in the 1D wave propagation model both non-linearity and hetherogeneity as correlated to variations of I_P.

INTRODUCTION

Experimental investigations on soil properties under dynamic loading have frequently shown the dependence of stiffness and damping on constitutive factors (such as grain size, plasticity, macro-structure and cementation) for natural, reconstituted and compacted soils (e.g. Stokoe et al., 1994; Mancuso et al., 1997). The influence of the above factors on the variation of G/G_0 and D with γ has been extensively examined in literature, since the work by Seed & Idriss (1970) until the contribution by Vucetic & Dobry (1991); these latter Authors consistently correlated the shape and position of the curves $G(\gamma)/G_0$ and $D(\gamma)$ to the plasticity index, I_P, selected as the most representative estimator of soil micro-structure. Thereafter, several efforts have been devoted to integrate and update the information on these correlations (e.g. Lanzo, 1995), as well as to assess their utility to identify non-linear equivalent parameters for seismic response analyses, especially when direct investigation is not affordable (e.g. EPRI, 1993). Conversely, analogous empirical estimates for small-strain parameters, G_0 and D_0, have not yet been proposed, because of the 'scatter' in the approaches used to analytically describe the dependence of small-strain stiffness G_0 on stress state and history variables (confining stresses, overconsolidation ratio, void ratio) and the lack of any quantitative approach to express the relationship between small-strain damping ratio D_0 and the same variables.

In this paper, an updated data base was collected from literature, selecting results of laboratory tests on natural fine-grained soils. Based on such data, it was attempted to synthesize the dependency of G_0 and D_0 on plasticity, by adopting univocal and consistent approaches to describe their variation with stress state and history. Soil plasticity is expected to influence the local site response predictions through the combined effects of both variation of small strain properties (G_0, D_0) and non-linearity (curves $G/G_0(\gamma)$ and $D(\gamma)$, e.g. Vucetic & Dobry, 1991). A sensitivity study on the local site response of a reference subsoil, evaluated through 1D wave propagation analyses, was carried out to highlight the terms of such microstructure-dependent behavior.

REVIEW OF LITERATURE DATA

In order to find out a correlation between small strain parameters and constitutive factors, small strain stiffness and damping data were carefully selected from updated literature. The data base collected was restrained to dynamic shear tests (resonant column and bender elements), to assemble experimental results obtained at comparable strain rates (d'Onofrio et al, 1999). The data base collected includes results of tests carried out in a wide range of confining pressures (0-2 MPa) on normally consolidated natural fine-grained soils (La Ferola, 1998).

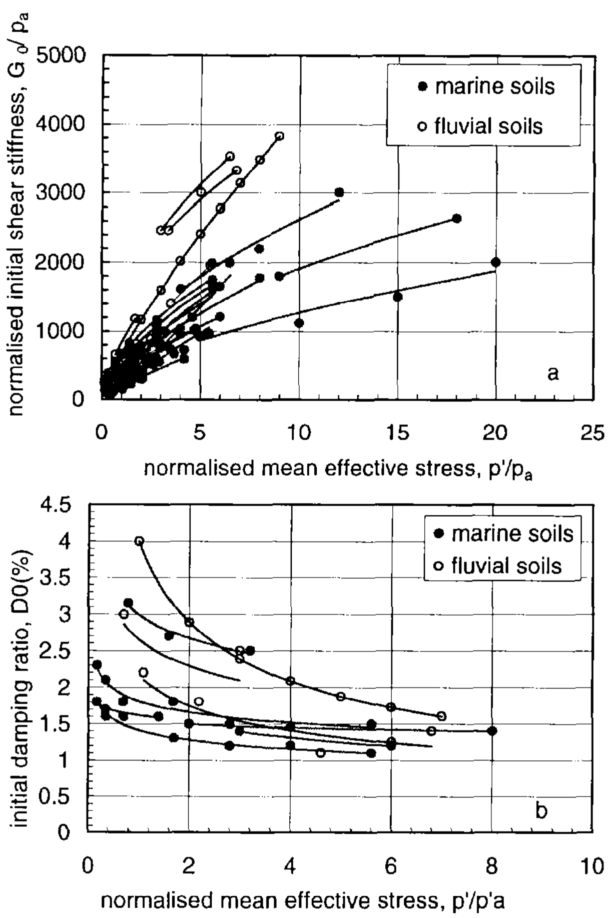

Fig. 1 Literature data: a) normalised initial shear modulus and b) initial damping ratio versus normalised mean effective stress.

Figure 1a synthesizes the dependence of initial stiffness on the mean effective stress, as resulting from literature data collected on 37 soils. The full list of references is given by La Ferola (1998). Both G_0 and p' are normalised with respect to the reference atmospheric pressure, p_a, and plotted with different symbols (open and full circles) according to the deposition environment. Each data set pertaining to a single soil was then fitted by an analytical relationship describing the variation of G_0 with confining stress level p'. The general formulation suggested by Rampello et al. (1994):

$$G_0/p_r = S\,(p'/p_a)^n\,OCR^m \qquad (1)$$

was adopted, which for n.c. soils reduces to:

$$G_0/p_r = S\,(p'/p_a)^n \qquad (2)$$

The resulting best fitting power functions are reported in the same figure with solid lines.

In eq. (2), S represents the shear stiffness in a normal consolidation state, at the reference isotropic stress p_s, while the exponent n synthesizes the dependence of shear stiffness on the current stress level.

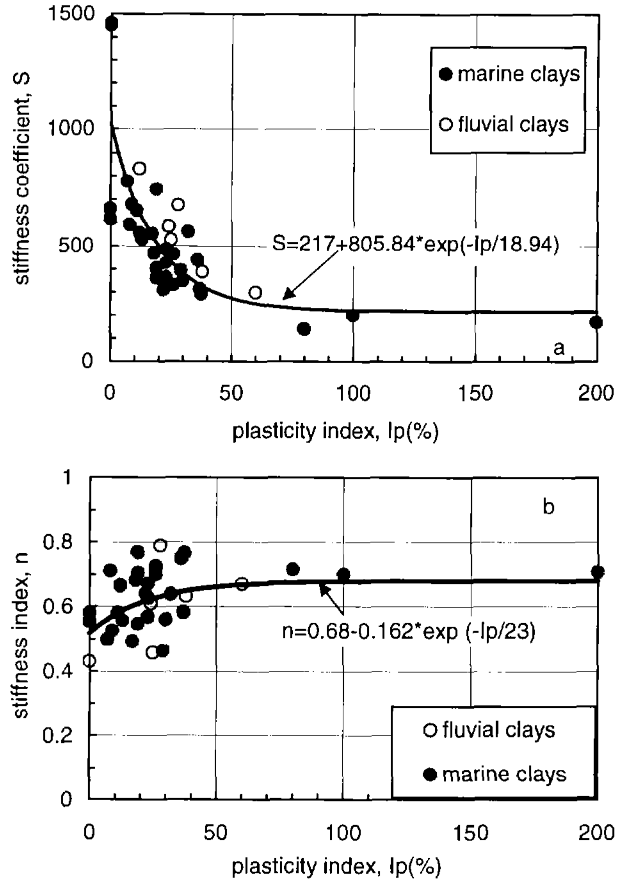

Fig. 2. a) Stiffness coefficient S, and b) stiffness index n versus plasticity index I_P.

A similar approach was adopted to describe the dependence of the small strain damping D_0 on the stress level; following the suggestions by Hardin (1965) on sands and the experience by d'Onofrio et al. (1995) on compacted sand-bentonite mixture, the small strain damping was formulated as a negative power function of mean effective stress p':

$$D_0 = Z\,(p'/p'_r)^{-d} \qquad (3)$$

expressed through two dimensionless parameters, Z and d, the meaning of which is somehow analogous to that of S and n.

The literature data on small strain damping are reported in figure 1b (open or full circles, according to the soil origin) together with the negative power functions adopted to fit each set of experimental points.

From the observation of both plots, it is apparent that soils deposited in a fluvial environment show, overall, higher stiffness and damping ratio if compared to these of marine origin. This behavior might be ascribed to the different micro-structures of marine and fluvial soils. As a matter of fact, both electrolyte concentration and soil mineralogy strongly influence soil fabric: in a marine environment, where salt content and cation valence are significant, flocculation often occurs, whereas an 'aggregated fabric' is more frequent in a

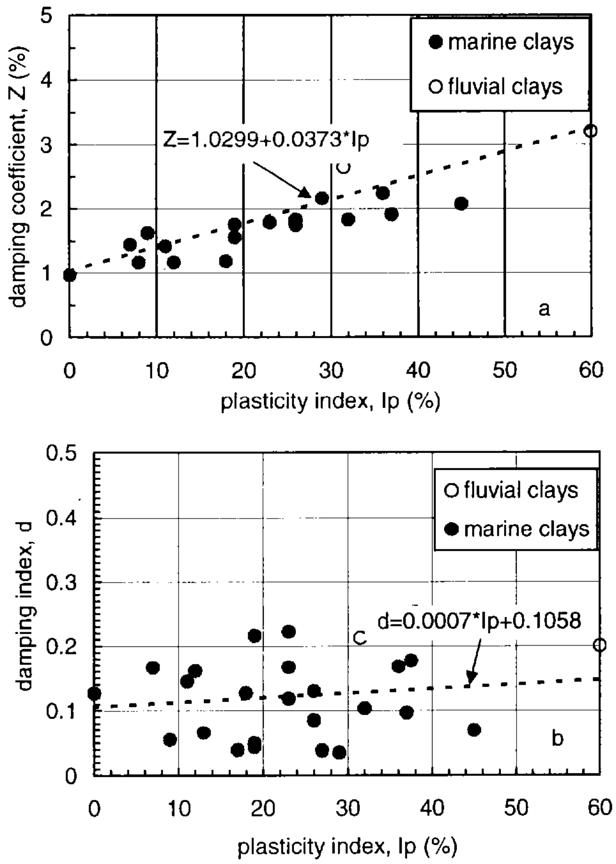

Fig. 3. a) Damping coefficient, Z and b) damping index, d versus plasticity index, I_P.

fluvial environment, because of the lower electrolyte concentration (Mitchell, 1993; Soga, 1994). The open soil fabric (edge-to-edge or edge-to-face) confers higher deformability and linearity to the flocculated soils, which may exhibit a less dissipative behavior due to the localized contacts between particles (Dobry & Vucetic, 1987; Soga, 1994). On the other hand, the face-to-face contacts of the aggregated fabric lead to a stiffer, but less linear, behavior, if compared to the flocculated fabric of marine soils; at the same time, a fabric of such a kind, due to the more diffused inter-particle contacts, is expected to lead to an increase in both viscous and frictional damping.

To correlate the small strain parameters to soil constitutive factors, the plasticity index I_P was again chosen as a key parameter, even though the above observations highlight that this index property cannot thoroughly account for the influence of fabric.

The variation of the stiffness and damping dimensionless parameters (S, n, Z, d) with plasticity index is reported in figures 2-3.

From Fig. 2 it appears that, with increasing plasticity, the stiffness coefficient S decreases, while the stiffness index n increases, both not indefinitely, as already shown by Rampello et al. (1994). At comparable plasticity index, the S values of fluvial soils are always somehow higher than those of the

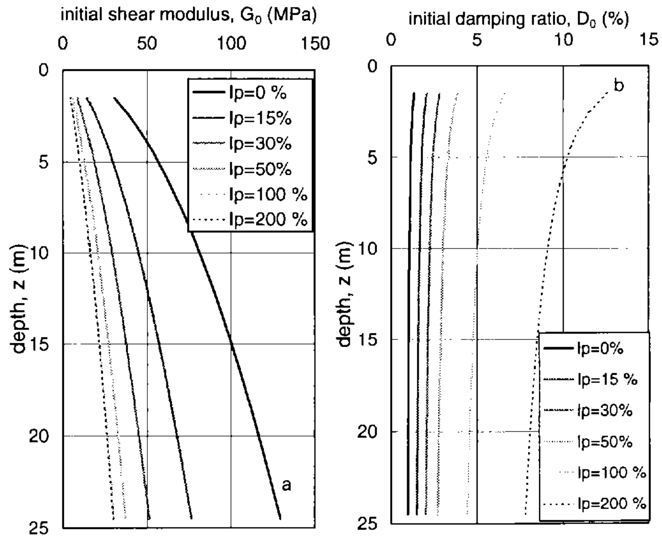

Fig. 4. G_0 and D_0 profiles for different plasticity indexes.

marine soils; nevertheless, it seemed quite reasonable to fit the data adopting the median relationships with the exponential expressions reported in the figures.

In Fig. 3, the damping coefficient Z and the damping index d are plotted versus plasticity index. The parameter Z, hence the overall small strain damping, increases with plasticity, because of the likely increase of viscous energy dissipation due to the higher interaction between solid particles and adsorbed water. The scatter in data points reported in figure 3b show that the correlation between damping index d and I_P is quite uncertain, probably because this index is very sensitive to the extent of the investigated stress range, as for the stiffness index n. However, just like this latter parameter, it seems reasonable that the damping index is expected to increase with I_P. Summarizing, for both parameters, literature data were best-fitted by the linear functions reported in Fig. 3.

SENSITIVITY ANALYSIS RESULTS

The well-known study by Dobry & Vucetic (1987) firstly correlated non-linearity, expressed in terms of shape and position of the curves $G(\gamma)/G_0$ and $D(\gamma)$, to the plasticity index I_P. The same authors (Vucetic & Dobry, 1991) analyzed the influence of the degree of non-linearity, referred to I_P, on the local site response of a reference soft subsoil (Mexico City clay). In this study, a similar sensitivity analysis has been carried out, but now also introducing the above described dependency of the small strain parameters, G_0 and D_0, on the plasticity index. An homogeneous deposit 25 m thick was assumed as reference subsoil, with a constant unit weight of 18 kN/m^3. The EW component of the accelerogram recorded on a stiff outcrop at the UNAM site during Mexico City earthquake (19.IX.1985) was selected as input motion. Six different subsoil profiles were simulated as representative of corresponding values of I_P (0%, 15%, 30%, 50%, 100%, 200%); eqs. 2-3 were adopted to describe the variation of initial

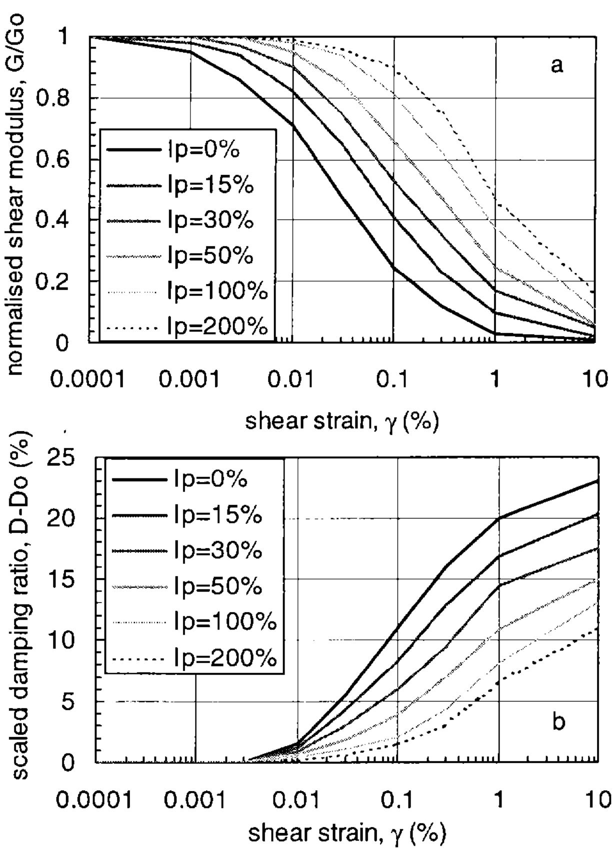

Fig. 5. a) Normalised shear modulus and b) scaled damping ratio versus shear strain, for different plasticity indexes (modified from Vucetic and Dobry, 1991)

stiffness and damping with depth, introducing the values reported in Table I for the dimensionless coefficients S, n, Z, d, as determined by the average empirical correlations with I_P reported in figs. 2 and 3.

I_P	0	15%	30%	50%	100%	200%
S	1023	582	382	275	221	217
n	0.52	0.59	0.642	0.66	0.68	0.68
Z	1.03	1.6	2.1	2.9	4.8	8.5
d	0.111	0.116	0.120	0.126	0.141	0.171

Table I. Values of small strain parameters correlated to I_P.

The resulting profiles of G_0 and D_0 are plotted in figure 4. It can be noted that, with the observed dependency of S and n on I_P, the initial deformability of the subsoil profile (Fig. 4a) overall increases and shows a lower degree of heterogeneity with increasing plasticity. On the other hand, a practically homogeneous profile of initial damping is observed until I_P trespasses 50%, after which D_0 considerably increases, with more sensible variations at the lowest depths for the highest plasticity values (Fig. 4b).

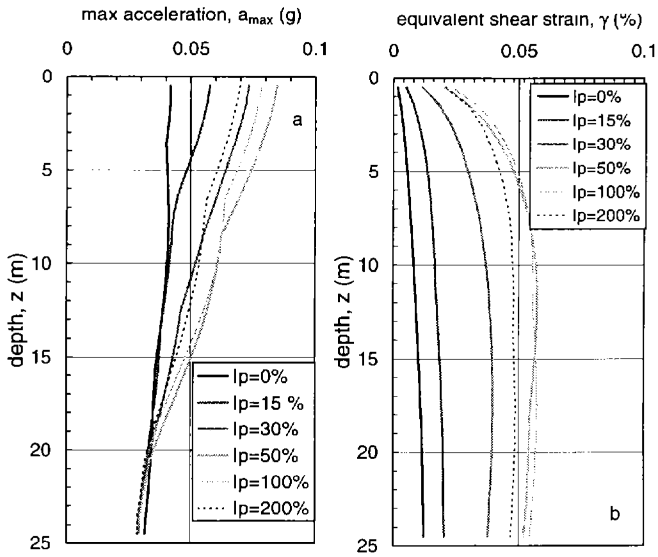

Fig. 6. Analysis results: a) maximum accelerations and b) equivalent shear strain profiles.

Numerical predictions of 1D non-linear seismic response of the reference subsoil were carried out using the SHAKE '91 code (Idriss & Sun, 1992). The influence of I_P on the strain-dependency of the equivalent parameters, $G(\gamma)$ and $D(\gamma)$, was introduced through the curves shown in figure 5. Fig. 5a reports the same average $G(\gamma)/G_0$ curves by Vucetic & Dobry (1991), while the curves in Fig. 5b represent the $D(\gamma)$ curves suggested by the same authors, this time scaled to the initial value D_0, because of their uncertainty at small strain levels in the original paper. It is worth to remind that, as above demonstrated, the average D_0 of a soil deposit is expected to increase with I_P, as already highlighted by the standard design $D(\gamma)$ curves suggested by EPRI (1993).

The results of the sensitivity analyses are reported in Fig. 6a,b in terms of profiles of maximum accelerations (a_{max}) and equivalent shear strains (γ), computed as $0.67\gamma_{max}$ consistently to the usual

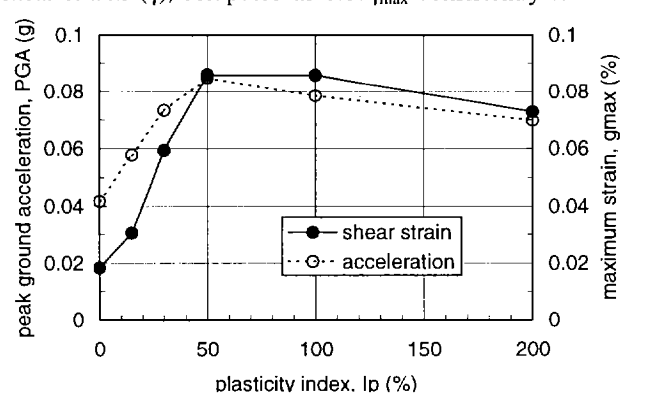

Fig.7. Analysis results: peak ground acceleration and maximum strains versus plasticity index I_P

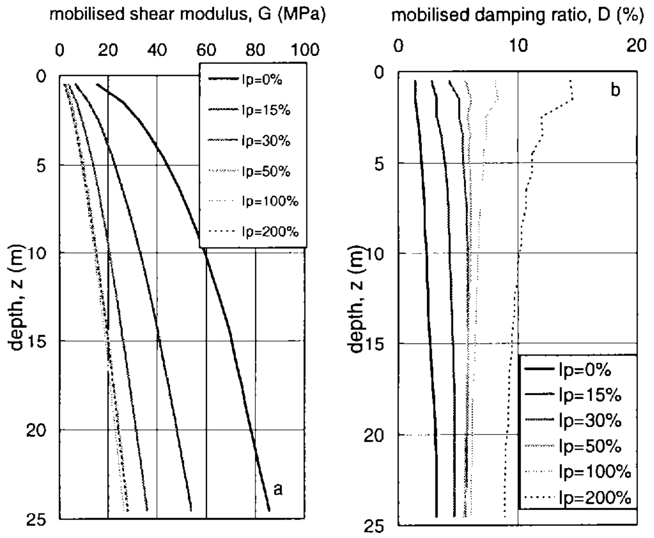

Fig 8. Analysis results: a) mobilized shear modulus and b) mobilized damping ratio.

practice. The maximum values along depth of both acceleration (always attained at surface) and shear strain (attained at depths overall decreasing with increasing I_P) are plotted in Fig. 7 against the plasticity index I_P. It can be noted that the maximum surface acceleration, PGA (hence, the amplification factor), first rapidly increases with plasticity (for I_P less then 50%), thereafter it slightly decreases for higher I_P. A similar trend can be observed for the maximum shear strain mobilized along the soil profile.

The combined effects of both heterogeneity and non-linearity related to micro-structure is shown by the profiles of mobilized shear modulus and damping ratio reported in fig. 8a,b. It can be noted that, while the mobilized stiffness profile remains quite unchanged for I_P higher than 50%, the mobilized damping ratio continuously increases with plasticity index. This behavior consistently affects the local site response, as reflected by the amplification functions plotted in figure 9a. In fact, the first natural frequency, f_1, decreases with plasticity until becoming constant for I_P>50%, as well as it occurs for the second natural frequency, f_2 (Figs. 9a,b). The first peak amplification, A_1, increases reaching a maximum for I_P=50%, and thereafter decreases; conversely, the second peak amplification, A_2, continuously decreases with I_P (Fig. 9b).

Referring to the simplified pattern of a homogeneous visco-elastic stratum with shear wave velocity V_S and thickness H, laying on a deformable bedrock (Roesset, 1970), it is possible to express the n-th natural frequency and peak value of the amplification function as follows:

$$f_n \cong \frac{(2n-1)V_S}{4H} \quad (n=1,2\ldots\infty) \tag{4}$$

$$A_n \cong \frac{1}{\mu + (2n-1)\frac{\pi}{2}D} \quad (n=1,2\ldots\infty) \tag{5}$$

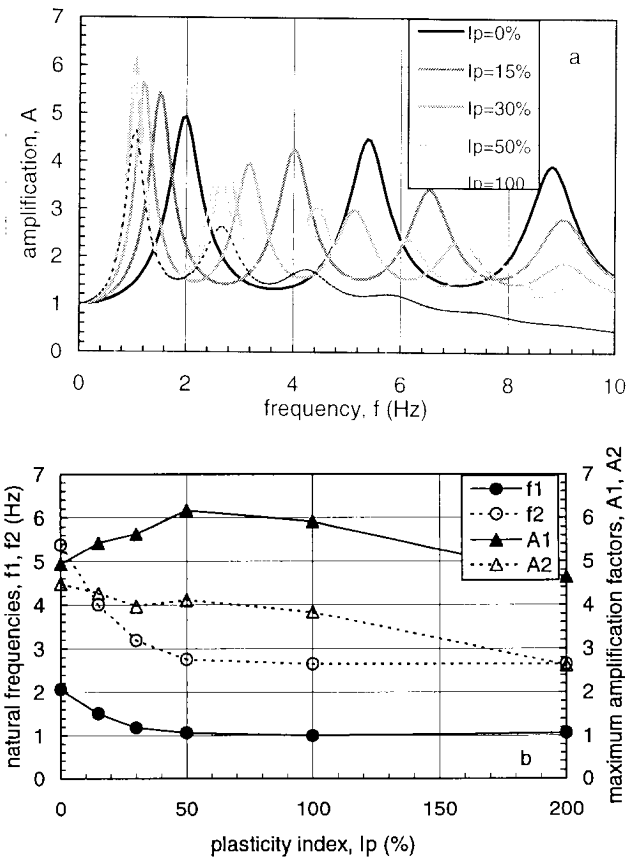

Fig.9. Analysis results: a) amplification function and b) first and second peak amplifications, A_1, A_2 and corresponding natural frequencies, f_1, f_2, vs. I_P

being μ the soil/bedrock impedance ratio and D the internal damping. Summarizing, in this example it appears that the amplification of subsoils with ordinary plasticity values (I_P<50%) is dominated by the increase of contrast of impedance (i.e. the radiation damping), while for soils with unusually high plasticity (I_P>50%) the internal damping plays a mayor role on the surface amplification, which becomes increasingly attenuated at subsequent peaks occurring at higher frequencies. On the other hand, the natural frequencies, depending on soil deformability only, continuously decrease with I_P.

The surface motions are represented in fig. 10a in terms of response spectra (structural damping ξ=5%). As for the surface amplification, the peak spectral acceleration, $S_{a,max}$ first increases with the plasticity of the deposit; for the subsoil profile characterized by I_P=50% it reaches a maximum value of about 0.5g, which corresponds to a spectral amplification ($S_{a,max}$/PGA) around 6. This is attained at a period of 0.9s, corresponding to a first natural frequency of 1.1Hz (see also Fig. 8). For I_P>50%, the peak spectral amplification reaches slightly lower values, while the corresponding period keeps constant (Fig. 9b).

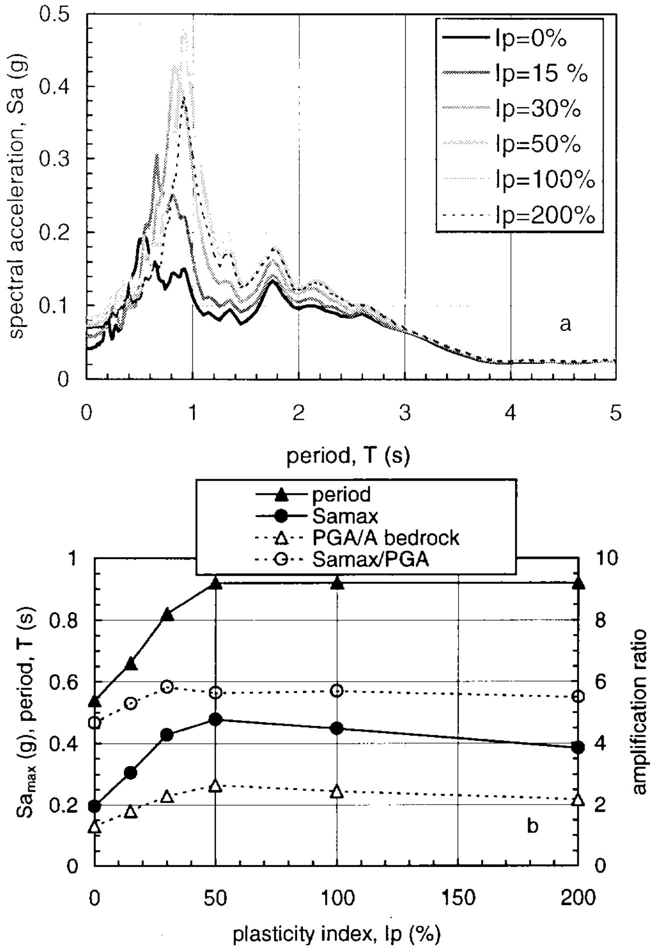

Fig. 10. Analysis results: a) response spectra for different I_P, and b) peak spectral acceleration, S_{amax}, peak period, T and amplification ratios plotted against I_P.

CONCLUSIONS

The first part of this study showed that the attempt to establish empirical correlations between small strain soil parameters relevant to site response analysis and index properties such as plasticity index is encouraging. For fine-grained soils, these correlations can be coupled with those between soil plasticity and non-linearity, as proposed by Vucetic and Dobry (1991) and Zen et al. (1987); such a framework can be helpful for rough predictions of site response with simplified methods, to be used in lack of more detailed investigations. More studies are required to extend this approach to coarse-grained soils (referring to soil properties other than plasticity index), and to empirically evaluate simplified site response estimators, such as soil amplification factors and natural periods, relevant to microzonation studies.

REFERENCES

d'Onofrio A., F. Santucci de Magistris, F. Silvestri and F. Vinale [1995]. *Behaviour of compacted sand-bentonite mixtures from small to medium strains.* Proc. I Int. Symp. on Earthq. Geotechnical Eng., Tokyo. Balkema, Rotterdam

d'Onofrio, A., F. Silvestri, and F. Vinale [1999]. *Strain rate dependent behaviour of a natural stiff clay.* Soils and Foundations, 39:2, 69-82.

Dobry R. and M. Vucetic, [1987]. *Dynamic properties and seismic response of soft clay deposits.* Proc. of the Int. Symp. on Geotechnical Engineering of Soft Soils, M.J.Mendoza y L.Montanez , Mexico City

EPRI [1993]. *Guidelines for determining design basis ground motions.* Report No. TR-102293. Electric Power Research Institute, Palo Alto (California).

Hardin B.O. [1965]. *The nature of damping in sands.* Journal of Soil Mech. and Found. Division, ASCE, Vol. 91, SM1.

Idriss, I.M. and J. I. Sun [1992]. *SHAKE91 - A computer program for conducting equivalent linear seismic response analyses of horizontally layered soils deposits.* University of California, Davis.

La Ferola, G. [1998]. *Uno studio sugli effetti della microstruttura dei terreni naturali sulla risposta sismica locale.* Master Thesis Università degli Studi di Napoli "Federico II" (in italian).

Lanzo, G. [1995] *Comportamento non lineare del terreno influente sulla risposta sismica locale.* Doctoral Thesis Università degli Studi di Roma "La Sapienza" (in italian).

Mancuso C., F. Silvestri and F. Vinale, [1997]. *Soil properties relevant to seismic microzonation.* Proc. I Japanese-Turkish Conf. Earth. Eng., Istanbul.

Mitchell, J.K. [1993]. *Fundamentals of soil behaviour.* John Wiley and Sons.

Soga, K. [1994]. *Mechanical behavior and constitutive modelling of natural structured soil,* Doctor of Philosophy thesis. University of California at Berkeley.

Rampello, S., F. Silvestri, and G. Viggiani [1994]. *The dependence of small strain stiffness on stress state and history for fine-grained soils: the example of Vallericca clay.* I Symp. Pre-failure Deformations of Geomaterials. 1:273-279. Balkema.

Seed, H.B. and I.M. Idriss [1970]. *Soil moduli and damping factors for dynamic response analyses.* EERC Report no. 70-10, University of California, Berkeley.

Roesset, J. [1970]. *Fundamentals of soil amplification. Seismic Design for Nuclear Power Plants,* ed. R.J. Hansen. M.I.T. Press.

Stokoe, K.H., J.N. Lee and S.K. Hwang [1994]. *Effects of various parameters on the stiffness and damping of soils at small to medium strains.* I Int. Symp. on Pre-Failure Deformation of Geomaterials, Sapporo (Japan), Vol. 2. Balkema, Rotterdam.

Vucetic, M. and R. Dobry [1991]. *Effect of soil plasticity on cyclic response.* Journ. Geot. Eng. ASCE. 117(1): 89-107.

Zen K., H. Yamazaki and Y. Umehara [1987]. *Experimental study on shear modulus and damping ratio of natural deposits for seismic response analysis.* Report of the Port and Harbour Research Institute, 26(1):41-113 (in Japanese).