INFLUENCE OF SOIL DYNAMIC PARAMETERS ON SEISMIC RESPONSE OF A SITE AT FABRIANO, ITALY

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SUMMARY

Dynamic parameters play an important role in evaluating the seismic response of soil deposits. However, they are affected by many uncertainties associated with the natural variability of soils, sampling techniques, laboratory testing procedures, sensing devices, instrumental biases, operators, etc. Therefore, choosing the 'best values' is generally a complex problem of decision-making. In this paper, the problem of the influence on seismic response of the uncertainties connected with laboratory testing equipment and procedures used in measurement of damping ratio is considered. Reference is made to the evaluation of site effects in a cohesive deposit in the town of Fabriano, Italy, which was damaged by the Umbro-Marchigian seismic sequence that initiated on September 26, 1997

INTRODUCTION

The analytical evaluation of seismic response in a given site is inevitably approximate, since many simplifications must be made in order to produce a solution. Moreover, no matter what method of analysis is used, the uncertainties due to the idealisation of soil behaviour and to the choice of design dynamic parameters always influence the results of the model.

In the last twenty years, many researchers (e.g. Lumb, 1974, Harr, 1987) have attempted to quantify the influence of the major sources of uncertainty in estimating the main geotechnical properties: i.e. natural soil variability, sampling patterns and size, sampling disturbance, testing procedures, machines, operators. Some of these uncertainties have been recognised as being of prevalently random nature; others are of a systematic type. But, in practice, it is not easy to separate the design 'errors' due to the different components. In principle, the random errors due to the natural variability of a uniform soil could be reduced through a large number of replicate tests on numerous representative soil specimens, by assuming an 'absolute' or 'standard' method of measurement for a given property. This must be performed with specific equipment under specified conditions. Especially for the dynamic characterisation of soils, however, it is rather difficult to pursue this approach. In fact, the elevated cost of high-quality sampling and laboratory testing means that the amount of available samples and tests is always extremely small, and generally below the minimum sample size indispensable for estimating the mean value (and *a fortiori* for estimating the standard deviation) with a confidence level of at least 90%. In addition, to Authors knowledge, no sufficiently reliable research on the natural variability of dynamic parameters has been conducted up until now. Therefore, the coefficients of variation to be recommended for precision estimates are generally unknown. So, for a precise estimate of dynamic parameters, the statistical and probabilistic methods that express uncertainty in terms of probability cannot be of much help. Lastly, as regards the possibility of assuming laboratory 'absolute' or 'standard' testing methods, it must be pointed out that the research in the field of soil dynamics is still largely in progress and that the assessment of the more reliable method deserves further investigation.

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As concerns the values of the shear modulus, G, and damping ratio, D, measured in the laboratory, it is well known that they are strongly dependent on testing procedures and apparatus (Zavoral & Campanella, 1994; Shibuya et al., 1995; Tatsuoka et al., 1995; Lo Presti et al., 1998a and 1998b; Stokoe et al., 1999). Extensive research carried out by these Authors showed that, in addition to cyclic strain, damping ratios are mainly influenced by strain rate effects, and that cyclic loading torsional shear tests (CLTST) provide values of the damping ratio that are more reliable than those from the resonant column (RC). Moreover, a comprehensive comparison between the results from the two tests showed that the small strain damping ratios from the RC are systematically larger than those obtained from CLTST.
However, it must be noted that, in the practice of most countries, RC is the most diffuse apparatus for assessing dynamic parameters. Moreover, even if restricted to a limited number of experiments, some researchers (Jamiolkowski et al. 1998; Stokoe et al., 1999) have recently shown that small strain damping ratios from seismic tests are larger than the values from RC.
Thus, at present, in order to reduce the uncertainties of design parameters for the analytical evaluation of the seismic response, the following two main geotechnical problems exist. The first problem consists of obtaining measurements of soil parameters as precise and representative of the property under consideration as possible. From a geotechnical point of view, this implies reducing sampling disturbance, machine effects, unskilled operator effects, etc.; from a statistical point of view, it signifies determining reliable estimates of means, variances and probability density functions, by means of proper techniques of sampling and identification of the population. In particular, this last point involves studies of the natural variability of soils and the performance of repeated tests on a number of samples consistent with the desired precision of the estimate. The second problem is to identify the 'best' procedure, that is, the procedure that leads as closely as possible to the 'true' value of the property. From a statistical point of view, this implies the identification of a procedure that can be considered as 'absolute' for a given soil property, and, possibly, an evaluation of the bias, or lack of accuracy, associated with the other testing procedures available.
Within the perspective of the present research, the uncertainty of the design parameters which are utilised in the seismic response analysis and which can lead to erroneous results is attributable to two main causes: soil variability (random errors) and testing techniques (systematic errors). The other sources of uncertainty have been considered of minor importance and have therefore been disregarded.
In the research, only the incidence of the testing procedures on the seismic response is considered. The approach followed consisted of determining how much the results of an analytical response model, obtained by choosing design parameters from different procedures, will differ each other.

SITE CHARACTERISTICS

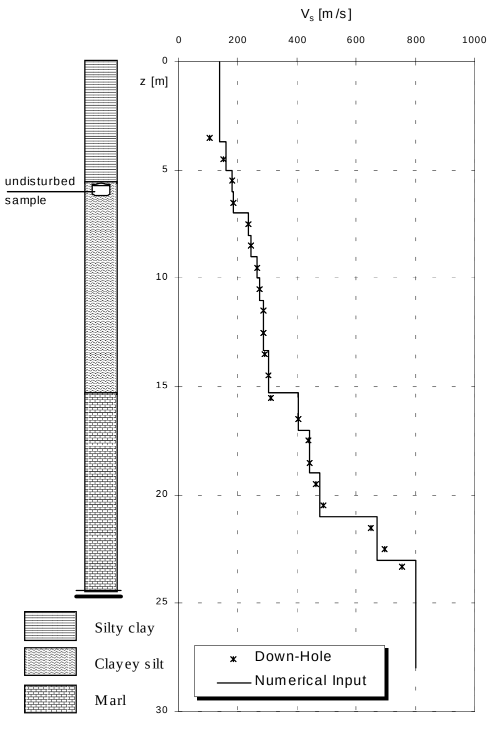

Figure 1 – Simplified stratigraphy and Vs profile

The site selected for analysing the influence of the uncertainties due to laboratory procedures on the seismic response was the zone of Borgo at Fabriano, in the Marches region of Italy. Although Fabriano was, on the whole, lightly injured by the seismic events that struck central Italy between September 1997 and June 1998, severe damage was observed (Marcellini, 1998) in the zone of Borgo. Immediately after the two main shocks of September 26, it was evident that site effects were the main source of the building damage. Therefore, for a better understanding, quick surveys aimed at estimating site amplification with Nakamura's method were undertaken (Mucciarelli & Monachesi, 1998); moreover, in order to capture the aftershocks, an horizontal array of more than 30 seismological instruments (accelerometers and velocimeters) was installed (Marcellini, 1998). Also, a down-hole accelerometric experiment has been in operation since April 1998 (De Franco et al., 1998; Crespellani et al., 1999). The system recorded many aftershocks characterised by magnitude M_D ranging from 2.5 to 4. Furthermore, a program of field investigation and laboratory testing (Crespellani et al., 1998; Ciulli, 1998) was carried out.

The Borgo site is located in a very gentle slope. Schematically, in the explored vertical, the soil profile consists of three main layers (Figure 1):
layer 1 (up to 6.00 ÷7.00 m) composed of silty inorganic clays of high plasticity with small percentiles of gravelly sands; layer 2 (between 6.00÷7.00 to about 15.00 m) consisting of clayey silts; bedrock (at a depth of about 15.00 m) composed of the marls of the Gessoso - Solfifera formation.
Two different techniques were used to determine the shear wave velocities, V_S: the down hole (DH) and Spectral Analysis of Surface Waves (SASW). The resulting data were very similar. In Figure 1, the V_S-values against depth obtained by DH testing are shown. This diagram indicates that, in layer 1, that is, in the inorganic clays, the shear velocity had a mean value of 146 m/s; in the silty clays, it increased gradually up to 300 m/s, and lastly in the marls, it had an average value of about 500 m/s. In the latter it passed from values of 400 m/s of the weathered levels to 800 m/s in correspondence with the wealthy rock.
The laboratory tests included also dynamic tests performed with the RC and the CLTST apparatus.
The field and laboratory investigations performed showed that the geotechnical properties of the deposit could be considered relatively uniform. Therefore, the zone of Borgo seemed to be a good site for highlighting the importance of testing procedures on the response of geotechnical sites during earthquakes. As a complement to the dynamic testing, an interlaboratory experiment was carried out in order to reduce as much as possible the uncertainties due to natural variability,

INTERLABORATORY TESTING PROGRAM

Three specimens, drawn from the same undisturbed sample and taken at a depth of 6.00-6.70 m, were tested in two Italian geotechnical laboratories, by using the RC (University of Florence) and CLTST equipment combined with RC (Polytechnic School of Turin). The geotechnical properties of the sample are resumed in Table 1.

Table 1 - Average geotechnical parameters

Parameters	Mean value
Water content [%]	26
Liquid limit [%]	56
Plasticity index [%]	29
Specific gravity [-]	2.76
Unit wheight [kN/m^3]	20
Void ratio [-]	0.715
Undrained strength [kPa]	100
Compression index [-]	0.24
Overconsolidation ratio [-]	6

In the Florence laboratory, the specimen, named in the following RC-F, was tested with the RC equipment by using the multistage procedure with three levels of effective confining pressure (65, 80 and 120 kPa) and adopting, for the measurement of the damping ratio, the Amplitude Decay Method (ADM). In the Turin Laboratory, the two specimens, both subjected to a confining pressure of 98 kPa, were tested in two different

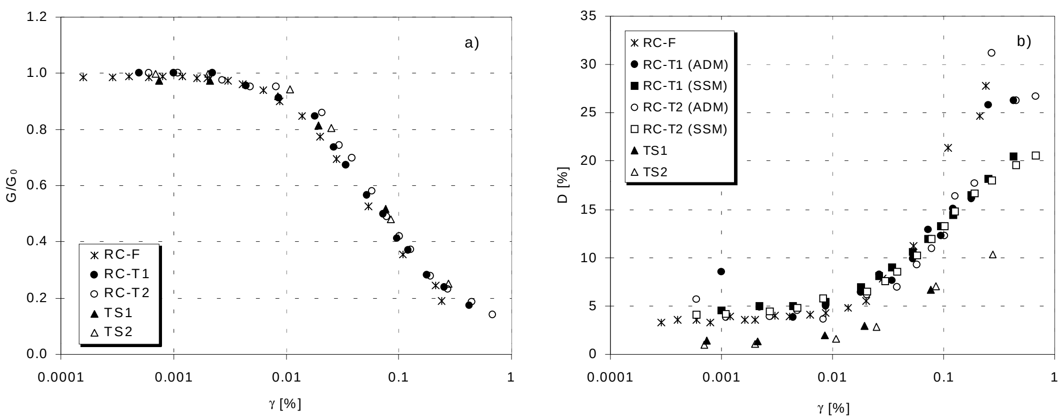

Figure 2 – Experimental data

modes. In one specimen, the RC test was performed prior to commencing the cyclic torsional tests; in the second, on the contrary, the RC test was performed after the CLTST. Both the ADM and SSM (Steady State Method) were employed in the RC tests. Here as follows, those tests are named RC-T1$_{ADM}$, RC-T1$_{SSM}$, RC-T2$_{ADM}$ RC-T2$_{SSM}$, respectively. The results obtained are shown in Figure 2.

SEISMIC RESPONSE ANALYSIS

The analytical evaluation of the seismic response was carried out by using the SHAKE code (Schnabel, 1972). The selected profile is shown in Figure 1. The values adopted of the small strain shear modulus, G_0 at the various depths were those derived from the V_S values of the down-hole tests (Figure 1).

To the data of Figures 2a and 2b were adapted, respectively, the Ramberg & Osgood model (1943), defined by:

$\gamma = G\cdot\gamma /Go + C\cdot(G\cdot\gamma /Go)^R$ and the $D = D_{max} \cdot \exp(-\lambda\cdot G /G_0)$ equation associated with the respective curves of R&O. These curves are represented in Figure 3. The good fit of the theoretical curves to experimental data can be observed (r^2 is the coefficient of determination). The parameters of the model and the relative coefficients are shown in Table 2.

Table 2 – Initial damping, equation coefficients and coefficient of determination of the models fitting experimental data

	TS1	TS2	RC-T1$_{SSM}$	RC-T1$_{ADM}$	RC-T2$_{SSM}$	RC-T2$_{ADM}$	RC-F
D_0 [%]	1.52	1.22	4.80	4.85	4.62	4.01	3.51
C	186.66	466.05	235.26	235.26	225.70	225.70	433.07
R	2.67	2.92	2.62	2.62	2.63	2.63	2.77
r^2	1.000	0.996	0.989	0.989	0.985	0.985	0.963
D_{max} [%]	39.67	26.15	28.14	31.42	28.06	37.16	48.56
λ	-3.26	-3.06	-1.77	-1.87	-1.80	-2.23	-2.63
r^2	0.973	0.963	0.991	0.867	0.986	0.951	0.989

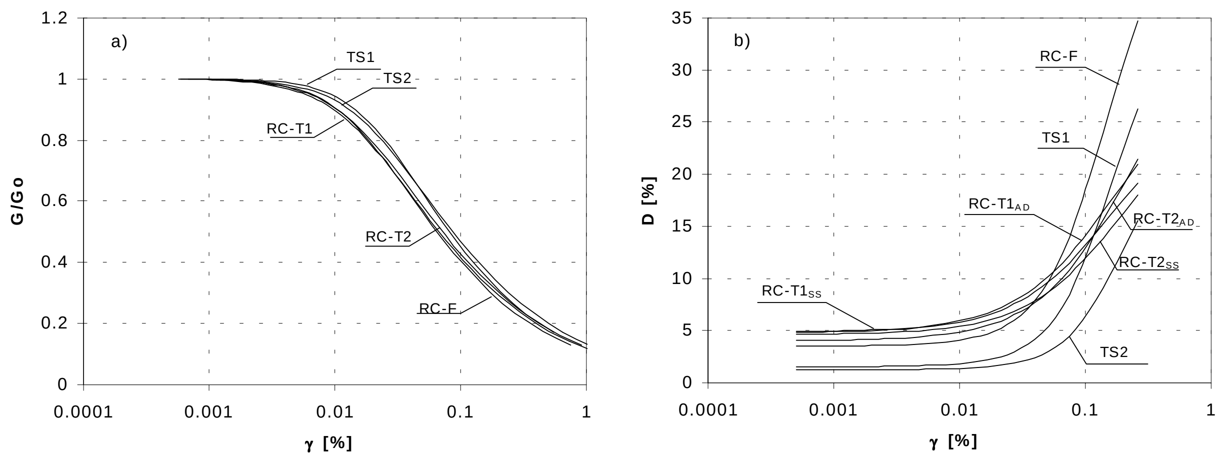

Figure 3 – Theoretical curves adapted to the experimental data

Figure 3 shows that, no matter what equipment is used, the normalised shear modulus degradation curves G/G_0 vs γ (Fig. 3a) are all very similar, while there are considerable differences among the D vs γ curves (Fig. 3b). In particular, the curve referring to the TS2 test represents the lower limit for any shearing strain level, the curves of the RC-T1$_{SSM}$ and RC-F tests represent the upper limit for shearing strain values, respectively lesser and greater, respectively, than $\gamma = 0.03\%$. Therefore, for the purposes of the numerical analyses, reference was made to the G/G_0 vs γ and D vs γ curves of the three above mentioned tests.

A two-fold modelling of the seismic ground response was performed with these data. First, the response to a design earthquake with a return period of 475 years was determined, by adopting two accelerograms representative for the site obtained by Marcellini & Tento (1998) as seismic input. An hazard analysis conducted by these Authors led to defining, at Fabriano, a uniform probability spectrum with an expected peak ground acceleration value of 0.3 g on the rock. As no accelerometric stations of the Italian network exist in the area, by assembling actual records on analogous soil conditions, these Authors suggested two possible accelerograms, the spectra of which have a good fit with the spectrum obtained for the site. These accelerograms, named here as follows IRRS1 and IRRS2, were assumed for the outcropping rock, and were utilised for computing the ground response at Borgo. The accelerograms and their Fourier spectra are shown in Figures 4 and 5.

Then, site effects due to the same design earthquakes, but scaled to a peak value of 0.1 g, were evaluated. The aim of this particular analysis was to evaluate the incidence of the soil behaviour (linear or non linear in relation to the level of shaking), in the different responses obtained with the dynamic parameters from the different testing techniques. In Table 3, the main seismic parameters of the four input accelerograms are shown.

Table 3 - Main synthetical parameters of the input earthquakes considered

	IRRS1	IRRS1-S	IRRS2	IRRS2-S
Peak Acceleration [g]	0.32	0.10	0.31	0.10
Arias Intensity [cm/s]	107.0	10.5	119.0	12.49
Response Spectrum Intensity [g^2]	1.76	0.55	1.51	0.49
Predominant Period [s]	0.32	0.32	0.17	0.17
Bracketed Duration [s]	13.85	2.37	19.31	5.59
Trifunac Duration [s]	11.11	11.11	12.52	12.52

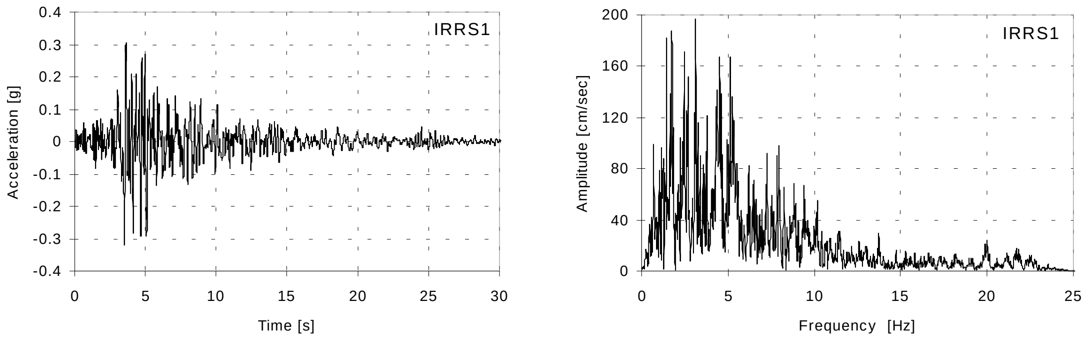

Figure 4 – Time history and Fourier spectrum of the IRRS1 accelerogram

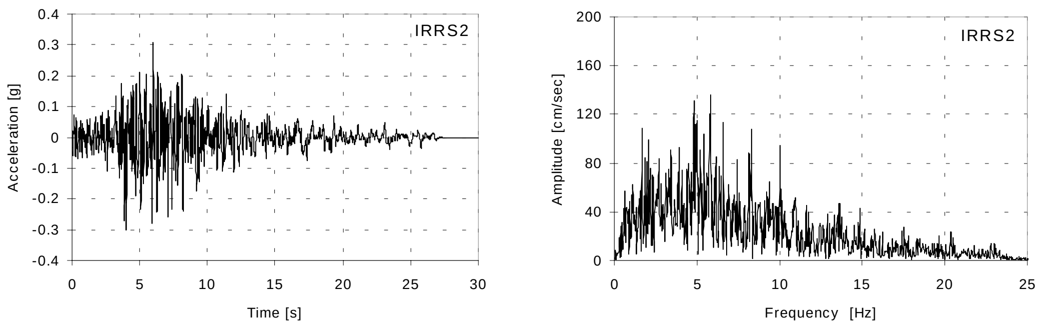

Figure 5 – Time history and Fourier spectrum of the IRRS2 accelerogram

RESULTS OF THE ANALYSIS

Figures 6 and 7, which refer to the results obtained by adopting the IRRS1 accelerogram, each show four diagrams: the first one shows, the profiles of the maximum shearing strains calculated by employing the G(γ) and D(γ) of the three aforesaid laboratory curves; the second diagram illustrates the initial shear modulus profile and the profile of the shear modulus corresponding to the maximum shearing strain are represented; in the third one, in order to point out the decay level with depth, the profiles of the normalised shear modulus are shown. Lastly, in the fourth diagram, the profiles of the damping ratios are represented. As for Figure 6a, relatively small differences can be noted (if the logarithmic scale was adopted for the shear strain γ, the said differences appeared be even smaller), especially for the RC tests, that obviously indicate maximum shearing strains always lower than those obtained with the TS2 curves. In Figures 6b and 6c, the differences are negligible, whereas Figure 6d shows profiles of the damping ratios that are very different, not only between RC and CLTST but also between the two RC tests. In the analysis conducted using the accelerogram with PGA=0.3 g, this difference is restricted to the band of high strains; in the analysis with the accelerogram scaled to a value of PGA=0.1g, also to bands of small strains (Fig. 7d). Qualitatively analogous results are obtained with the IRRS2 accelerogram.

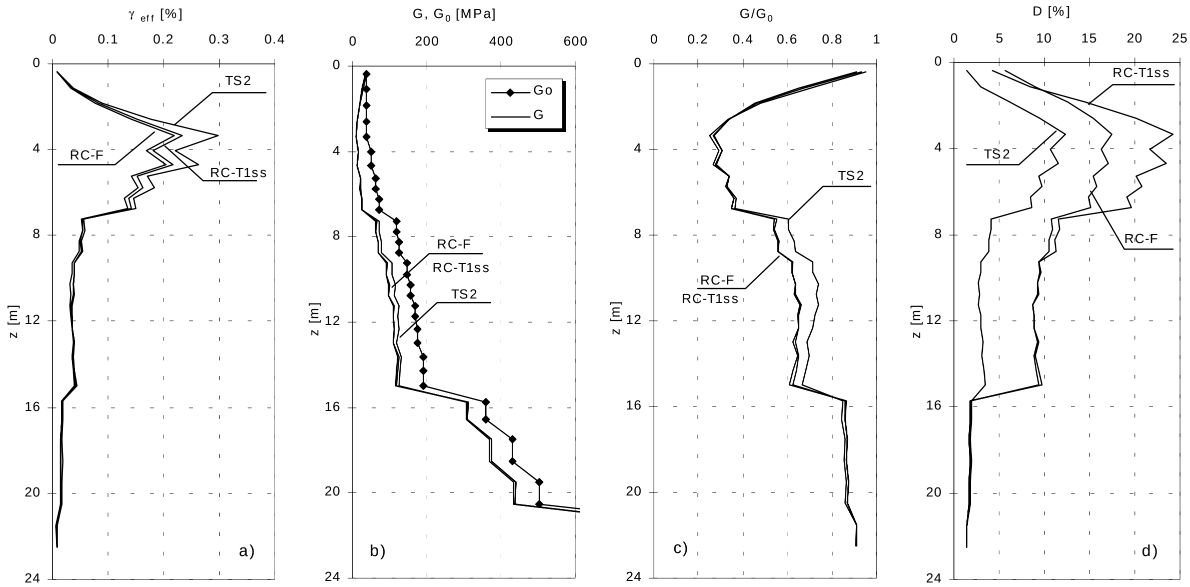

Figure 6 - Results obtained by adopting the IRRS1 accelerogram with PGA = 0.3 g

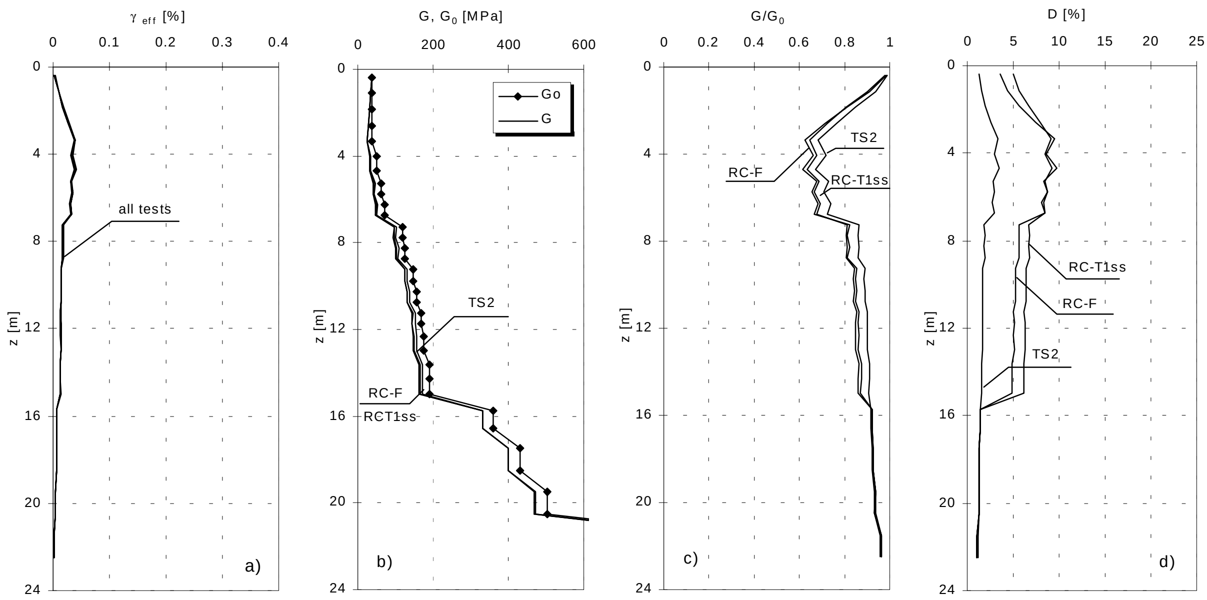

Figure 7 - Results obtained by adopting the IRRS1 accelerogram scaled to PGA = 0.1 g

Figure 8, that refers to the computations with IRRS1, shows the response spectra obtained with the data of the three laboratory tests. In Figure 8a, for PGA = 0.3g, the spectra have a similar shape; but the curve of CLTST has the larger ordinates, the differences have the maximum value in the interval between 0.2 s and 0.3 s, and are negligible for a period greater than 0.6 s. However, in Figure 8b, with the input scaled to PGA = 0.1 g, the differences among the spectra are small, even for low periods.

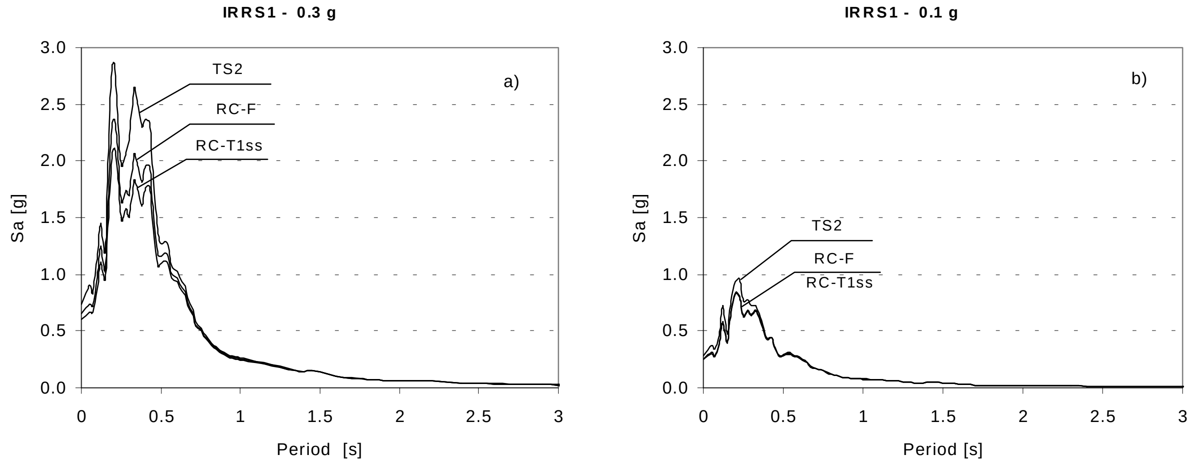

Figure 8 - Response spectra at the ground level

CONCLUSIONS

The frequency content of the earthquake affects seismic soil response, especially in highly-plastic cohesive deposits. The range of interest for most earthquakes is between 0.1 Hz and 10 Hz. Nowadays, a wide scientific literature exists in which the results of different laboratory geotechnical tests for the experimental determination of mechanical parameters of cohesive and cohesionless soils under dynamic and cyclic loading conditions are compared and critically discussed. In particular, the CLTST make it possible to investigate in detail the effect of the loading rate on the dynamic response of soils, while the combined use of the CLTST and RC on the same specimen allows one to test the response of the specimen in a wide range of strains, from very small to very large, by using the same equipment and without changing the boundary conditions. In this way, it is possible to join the mechanical properties that can be determined by static tests under large strain to those that are determined with small strains with dynamic loading.

As far as normally consolidated cohesive soils are concerned, it has been observed that the decay curves of shear modulus obtained by RC and CLTST at a frequency of 0.1 Hz, are in good agreement for low confining pressures. Their distance progressively increases with the increase in the consolidation pressure, and therefore with the initial modulus. In fact, the resonance frequency in RC tests increases with confining pressure, and so, the difference of the vibrating frequency of the two tests increases in consequence.

If the soil is overconsolidated, the differences between the results of the two tests are always relevant and less dependent on the confining pressure. As a consequence, for the analysis of shallow NC cohesive deposits, the results of CLTST and RC testing are substantially equivalent, as the in situ soil has been consolidated and confined at low effective pressures, whereas if the NC cohesive deposit is deep, the differences increase with the depth. Finally, in the OC cohesive deposits, the experimentation with CLSTS and RC leads to results that are considerably different and almost independent of the depth.

In cohesive soils, the damping ratio at small strains, D_0, is generally less than 2% and is very sensitive to the exciting frequency, f. In particular, it has been observed that, for a given level of deformation, D_0, decreases with f, for $f<0.1$ Hz, is almost constant in the $0.1<f<10$ Hz range and increases for $f>10$ Hz. Thus, the RC tests overestimate D for fine-grained soils, especially at small strains.

In the case of Borgo, the deposit is shallow and consists of OC cohesive soils. Therefore, the analytical evaluation of the seismic response, conducted first by employing the results from RC and then the results of the CLTST, represents a case in which the differences in the input data are particularly marked and, consequently, the differences between the results obtained with the two equipment are greater, especially for earthquakes associated with medium-size deformations. Moreover, the shapes of the spectra obtained are very similar, so that there is practically no difference in employing one spectrum or the other.

Therefore, to conclude: even if the expected differences qualitatively confirmed, the results of the numerical experimentation show that – for practical purposes and also in consideration of the uncertainties associated with both to the seismic input and the soil characterisation – also in principle the CLTST are more reliable, it is also permissible to use the RC results.

ACKNOWLEGMENTS

The research was supported by the CNR/GNDT (contracts N. 97.00508.54 and 98.03213.54) . The authors are grateful to Dr. B. Ciulli for her help in performing the numerical analysis.

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