

# An original dynamic interpretation of acceleration time-histories recorded during undrained Triaxial Cyclic Tests

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**Abstract** – The paper presents the results of laboratory tests carried out on undisturbed specimens of a gravelly soil sampled by in situ freezing. Because of the heterogeneity of the natural deposit, the grain size distribution of the specimens ranged from uniform sand with a very low percentage of gravel to well-graded sandy gravel, and three different soil groups could be defined according to grain size (respectively named A, B and C). The testing programme comprised drained and undrained monotonic triaxial tests and undrained cyclic triaxial tests, carried out on both isotropically and anisotropically ( $K_0$ ) consolidated specimens. Because of the coarse gradations of the soils, a special large triaxial cell was used. This cell was set up with a wave generator, to apply both shear and compression wave at the base of the specimen, and a devoted acquisition system was set up as well. With these devices, measurements of the soil shear waves velocity ( $V_s$ ) were achieved, even during the undrained cyclic tests. Thanks to this system it was possible to carry out an original dynamic interpretation of the acceleration time-histories recorded during undrained Triaxial Cyclic Tests.

## I. INTRODUCTION

Cyclic triaxial test is one of the most used laboratory tests to evaluate the cyclic behaviour of coarse grained soils. In this test, a cyclic axial load is applied to a specimen of soil over a several number of cycles, measuring axial strain and/or pore pressure development. Cyclic triaxial tests may be used to investigate some aspects of the dynamic/cyclic behaviour of soils, such as cyclic strength under undrained conditions. As a matter of fact, the soil failure can occur under a cyclic deviatoric stress lower than the one that leads to failure in static conditions. This kind of failure condition can be related to an excess of pore pressure and/or can be highlighted by large strain amplitudes. Hence, the failure condition can be identified by either a critical pore pressure ratio,  $r_{u,critic} = \Delta u / \sigma'_c$ , between the cyclically induced pore pressure increment

$\Delta u$  and the confining stress  $\sigma'_c$ , or a critical threshold of double amplitude axial strain  $\varepsilon_{DA,critic}$ .

This test procedure was adopted to investigate the mechanical behaviour of the foundation soil of the Straits of Messina bridge.

## II. TESTED SOIL

A wide laboratory experimental investigation was performed on undisturbed samples obtained freezing the soil on site (Cannitello, Reggio Calabria, about 400 kilometers SE of Rome). The tested samples belong to the same lithological unit with a wide grain size distribution, ranging from sand to gravel.

Three different soil groups were identified (Fig. 1):

- Soil A: Uniform sand with a very low percentage of gravel;
- Soil B: Sandy matrix with gravel particles inside for 30%
- Soil C: the coarser material (a well-graded sandy gravel).

Tests were performed on all the soil group specimens.

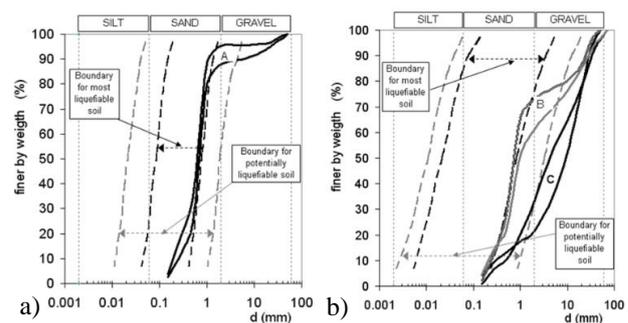


Fig. 1. Ranges of grain size distribution of soil A (a), and soils B and C (b), along with the boundaries of liquefaction susceptible soils suggested by the Ministry of Transportation of Japan (1999), respectively for uniform (a) and non uniform (b) gradation.

The laboratory investigation showed that the uniform sand A and the gap-gravelly sand B exhibited a similar mechanical behaviour in monotonic drained and undrained tests. They can be considered as a single material from a mechanical point of view. The monotonic tests showed that the coarser and the well-graded soil C has an ultimate strength ( $\phi'_u = 43.2^\circ$ ) larger than the one of the sandy soil A-B ( $\phi'_u = 39^\circ$ ).

The cyclic resistance curves obtained imposing initial isotropic stress conditions indicate that, for a given value of  $\psi$ , the gravelly soil C has a resistance to liquefaction larger than the sandy soils A and B. On the other hand, for a given soil, cyclic resistance generally increases with  $\psi$ , confirming that this parameter may be well suited to represent initial state conditions for the analysis of the susceptibility to the liquefaction, also for natural soils coarser than the reconstituted sands investigated in literature[1].

### III. WAVE PROPAGATION TESTS

In addition to traditional measurements, records of wave propagation in the soil during the triaxial tests were performed. At this purpose, the cyclic triaxial cell was set up with a wave generator to apply both shear and compression wave at the base of the specimen. An acquisition system to record data from a pair of accelerometers located both at the base and the top of the specimen was set up as well.

With these devices, measurements of the soil shear waves velocity (Vs) can be achieved, even during the undrained cyclic tests.

Shear waves velocity was obtained comparing the acceleration time-histories recorded at the base and the top of specimen.

To derive the source-receiver transfer function, the Fourier spectra of the accelerometer time-histories recorded close to the source (input) and near the receiver (output) were analyzed (Fig. 2). By the ratio between the Fourier spectra amplitudes at the receiver and the source, the relationship plotted in Figure 3 can be obtained and the frequency of the vibration first mode of the system can be identified.

Assuming the specimen as a visco-elastic solid beam, we back-analyzed the experimental main periods of the system with the theoretical ones. For an elastic beam of length L, that vibrates at the n<sup>th</sup>-mode, the characteristic wavelength,  $\lambda$ , can be expressed as below:

$$\lambda = \beta L/n \quad (1)$$

where  $\beta$  is a coefficient depending of degree constraint, and  $\lambda$  can also be obtained from experimental data, as below:

$$\lambda = V_s/f \quad (2)$$

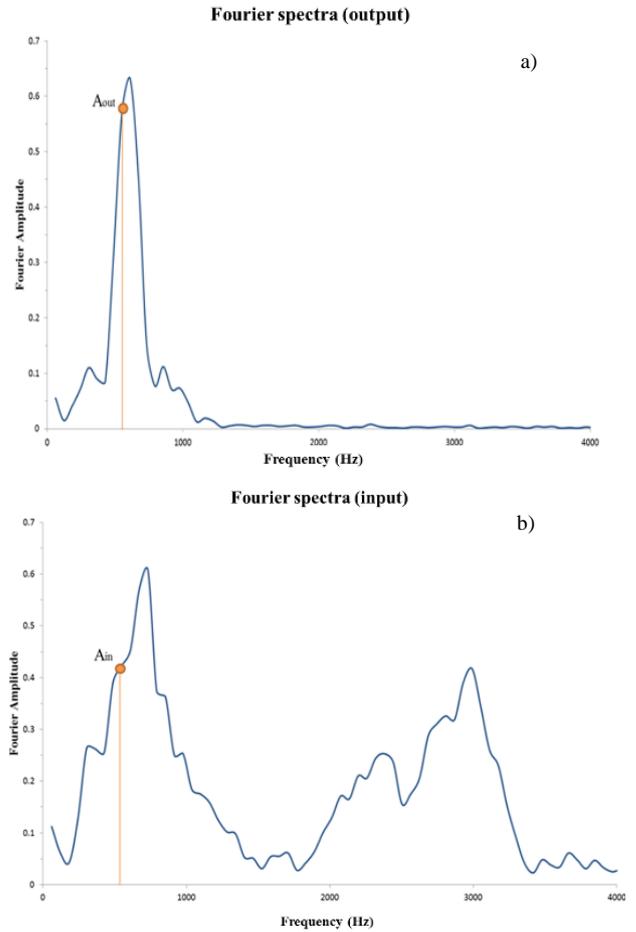


Fig. 2. Fourier spectra of the a) output recorded signal; b) input recorded signal.

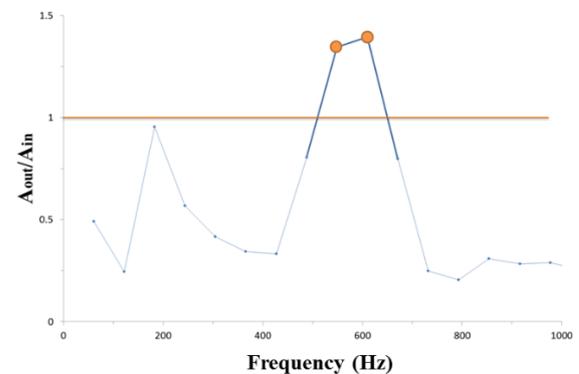


Fig.3. Identification of the frequency of the first mode vibration of the system

#### IV. ORIGINAL INTERPRETATION

The analysis of the data, while the specimen was moving to liquefaction, led to the different degrees of constraint.

In detail, at low strain conditions the free-body configuration of specimen looks like a cantilever ( $\beta=4$ ). Then, the more the specimen get close to liquefaction, the more the free body configuration became close to a double fixed beam ( $\beta=1$ ) (Fig. 4).

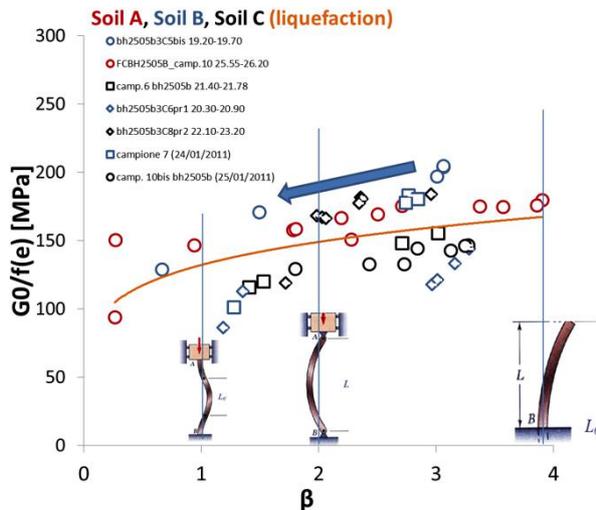


Fig. 4. Evolution of constraint conditions of the specimen in the triaxial cell, while the specimen was moving to liquefaction. At low strain conditions the free-body configuration of specimens looks like a cantilever ( $\beta=4$ ). Then, the more the specimen get close to liquefaction, the more the free body configuration became close to a double fixed beam ( $\beta=1$ )

This phenomenon can be read looking at the evolution of shear modulus by increasing pore pressure. When the pore pressure rises up, the shear modulus goes down, then the ratio between soil stiffness and cell bases stiffness falls down, and both base-soil interfaces become points of maximum stiffness, such as clamps.

On the other hand, when cyclic mobility has been observed during the tests, the free-body system configuration moved toward a simple supported beam ( $\beta=2$ ) (Fig. 5).

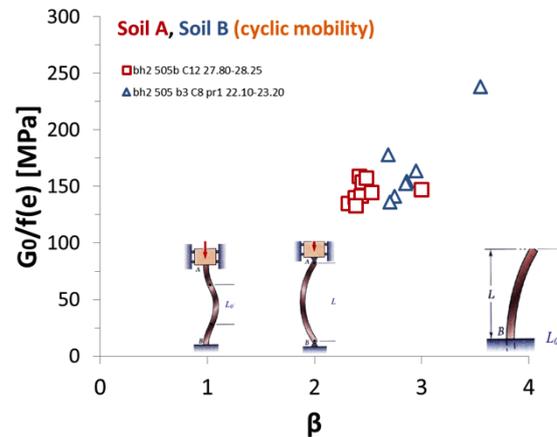


Fig. 5. Evolution of constraint conditions of the specimen in the triaxial cell when cyclic mobility has been observed during the tests, in this case the free-body system configuration moved toward a simple supported beam ( $\beta=2$ )

This experience shows that the experimental/theoretical method here described takes into account several phenomena occurring in the soil during cyclic undrained triaxial tests, and allows to identify the liquefaction-prone soils and the cyclic mobility-prone soils in a very original way.

#### REFERENCES

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