LARGE SCALE SOIL-STRUCTURE INTERACTION EXPERIMENTS ON SAND UNDER CYCLIC LOADING

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SUMMARY

Large-scale specimens of sand were constructed and tested under the cyclic loading imposed on a shallow foundation model, in the framework of the activities of the EC-funded project TRISEE (3D Site Effects of Soil-Foundation Interaction in Earthquake and Vibration Risk Evaluation). The tests were designed to provide validation data for the calibration of new and existing constitutive soil models, and to improve the assessment of the permanent deformations and bearing capacity of the soil-foundation systems. Two tests were performed, with relative densities 45% and 85%. The set-up consisted of a model of shallow foundation (1m x 1m in plan) resting on a large volume (4.6m x 4.6m, 3 m deep) of saturated sand of uniform properties, and well known geo-mechanical characteristics (Ticino sand). The specimens were subjected to the same loading sequence. After application of the vertical load and stabilization of the settlement, a series of small-amplitude force cycles of increasing level were applied, to identify the onset of non-linear behaviour. A realistic time-history of horizontal force and overturning moment, representative of the seismic actions transmitted by the super-structure to the foundation during an earthquake was then applied on top of the foundation. This resulted in non-linear behaviour, with development of significant permanent settlements and rotations. Finally, a series of displacement cycles of increasing amplitude were applied, up to the ultimate capacity of the soil-foundation system.

The paper provides a description of the experimental activity and of the global results. In particular, the different behaviour of the high and low-density specimens is discussed. The limitations of the set-up are critically analized. Some relevant engineering results, especially the permanent deformations developed during the earthquake-like loading phase, are illustrated and emphasis is given to the need for improving the current predictions of earthquake-induced foundation settlements and rocking.

INTRODUCTION

The seismic behaviour of shallow foundations has been mainly investigated through pseudo-static analysis of the bearing-capacity reduction due to seismic forces [Sarma and Iossifelis, 1990; Pecker and Salençon, 1991; Paolucci and Pecker, 1997a and 1997b], and the evaluation of the earthquake-induced settlements [Richards et al., 1993; Paolucci, 1997]. However, these investigations have been scarcely supported by parallel experimental investigations, essential to check the analytical procedures.

Laboratory tests encounter several major difficulties for a sound experimental analysis of this complex, nonlinear dynamic soil-structure interaction problem, such as:

careful control of soil properties: the deposition procedure and the saturation (if required) of the soil specimen must be carefully conducted and checked;

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boundary conditions: the boundaries of the testing apparatus should be enough removed from the foundation so to prevent any constraint on the development of failure mechanisms. Besides, flexible boundaries should be used, with well calibrated properties to reproduce free-field boundary conditions;

scale problems: large-scale tests are more expensive, involve a very large amount of material, and cannot be repeated easily, while the use of scaling laws in small-scale tests is questionable when applied to the grain size of soil materials, especially for strongly non-linear problems with pore-pressure build up;

seismic loads: both seismic actions transmitted by the superstructure (vertical and shear force, plus overturning moment) and soil inertia effects should be taken into account simultaneously.

It is impossible to cope with all of these requirements with the same testing apparatus. Centrifuge testing has encountered a notable success in the recent years. An interesting description of a centrifuge test setup for validation of innovative concepts in foundation engineering is reported by [Garnier and Pecker, 1999]. Another potentially useful apparatus for testing geotechnical structures is the shear stack mounted on the shaking table of the University of Bristol [Taylor et al., 1994], that allows to perform large-scale experiments and to closely simulate free-field boundary conditions.

A programme of large-size, cyclic loading experiments has been designed in the framework of the TRISEE Project (3D Site Effects and Soil-Foundation Interaction in Earthquake and Vibration Risk Evaluation), funded by the European Commission, to investigate the non-linear interaction between shallow foundations and the supporting soil under seismic loading. The basic set-up of the experiments consists of a shallow foundation lying on a saturated sand of known properties, and excited by a time-varying horizontal force and moment, which simulate the inertial forces transmitted to the foundation by the superstructure. The soil mass is at rest, so that the wave propagation and inertia effects in the soil are neglected with respect to the dynamic structural inertia forces transmitted by the foundation. In fact, theoretical work on seismic bearing capacity of shallow foundations [Pecker and Salençon, 1991; Paolucci and Pecker, 1997a and 1997b] has shown the soil inertia has a negligible influence on the failure loads.

The tests have been carried out with two different soil relative densities (Dr ≈ 85% and DR ≈ 45%), that are representative of high density (HD) and low density (LD) soil conditions. The latter can be considered as a lower bound for design of shallow foundations in practice, since the presence of sands at lower density generally leads the engineer to other design solutions.

DESCRIPTION OF THE EXPERIMENTAL SETUP

The experimental prototype consists of a stiff concrete caisson filled with sand (Ticino sand, [Bellotti et al, 1996]), and of a steel mock-up, representative of a concrete shallow foundation (Fig. 1). The caisson has dimensions 4.60 m by 4.60 m in plan and 4 m in height, while the foundation is 1 m by 1 m in plan. The lateral boundaries of the caisson are rigid and waterproof. While the bottom boundary is far enough from the foundation to avoid any interference with the possible failure mechanisms, the rigid lateral boundaries may have a significant influence on the bearing capacity of the foundation on dense sand, that should be taken into account in the interpretation of experimental results. On the contrary, the effect of the lateral constraints on the development of permanent displacements and rotations is less important, except at failure.

The foundation is made of steel, and has a concrete interface with the underlying soil that ensures a high friction resistance to horizontal loads. As shown in Fig. 1, the foundation is embedded 1 m in the sand, corresponding to a lateral overburden of about 20 kPa. A 1 m high steel formwork was placed around the foundation to retain the sand.

The vertical load is transmitted by an air cushion system designed to keep the force constant throughout the test. A hydraulic actuator, acting 0.9 m above the foundation level, transmits to the foundation the prescribed time-varying horizontal force or displacement.

Details on the reconstitution and saturation of the soil samples, on the assessment of soil properties and on the instrumentation are reported elsewhere [Jamiolkowski et al., 1999]. Full saturation of the soil mass could not be attained.

TEST SEQUENCE

The HD and LD specimens were subjected to a similar test sequence, consisting of the application of the design-level vertical load (which was kept constant throughout the whole loading sequence), and of three subsequent loading phases reproducing different levels of horizontal excitation. The design values for the vertical load were 300 kN and 100 kN for HD and LD specimens, corresponding to design pressures of 300 kPa and 100 kPa, respectively. These are typical design values for foundations on medium to dense sands, and are governed by admissible settlement requirements. The resulting static safety factor was found to be about 5 in both conditions.

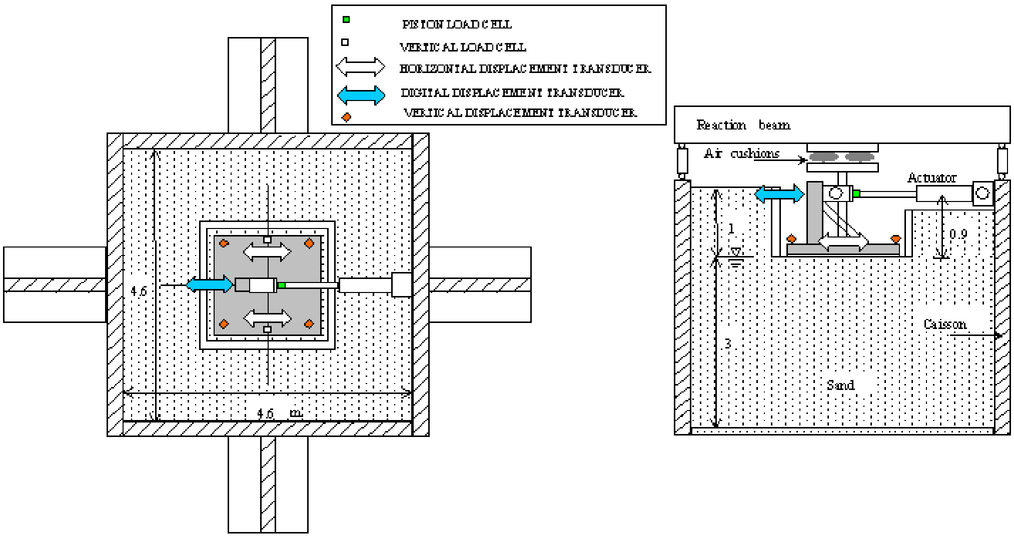


Fig. 1: Scheme of the experimental setup.

After the static loading phase, the final vertical settlement experienced by the foundation was about 7 mm for HD, and about 16 mm for the LD soil conditions. Details on the settlements resulting from the application of the static load alone are provided elsewhere [Jamiolkowski et al., 1999].

After completion of the static loading, the horizontal cyclic loading was applied in three phases, as follows.

Phase I

A series of small-amplitude force-controlled cycles was applied first, to identify the onset of significant non-linear behaviour in the soil. The cycles were sine-shaped, with frequency f=0.5 Hz. Their amplitude was gradually increased up to about 5% of the vertical load, to obtain evidence of stiffness degradation and development of hysteresis loops.

Phase II

The foundation was then subjected to an earthquake-like time history of horizontal force and overturning moment transmitted by the hydraulic actuator at 0.9 m height. The horizontal force was adapted from the base-shear time history measured on a four-story RC building, designed according to EC8 and tested at the ELSA laboratory [Negro et al., 1996]. The peak of the seismic input was scaled to a seismic coefficient (horizontal force divided by vertical force) of about 0.2 (Fig. 2). The combination of seismic coefficient and height of application of the horizontal force (h=0.9m) was such that a compressive stress was maintained everywhere on the foundation interface. The absolute value of the force peak was of about 60 kN and 20 kN for the HD and LD tests respectively. To preserve the accuracy in the force-control system, the original time scale was expanded. For the first (HD) test, the time scale was expanded by a factor of 6, whereas for the second one (LD) the original time scale was expanded by a factor of 3. The original time history of horizontal force had a fundamental frequency of about 0.8 Hz. The resulting diagram of horizontal force was, instead, characterized by a fundamental frequency of 0.13 Hz and of 0.27 for the HD and the LD test respectively.

Phase III

Finally, sine-shaped displacement cycles of increasing amplitude were imposed to the top of the structure, up to the attainment of a limit threshold of the foundation resistance. The test was displacement-controlled in order to avoid excessive movement of the system close to its ultimate capacity. Pairs of cycles (f=1/6 Hz) were used for HD test and single cycles (f=1/3 Hz) for LD test.

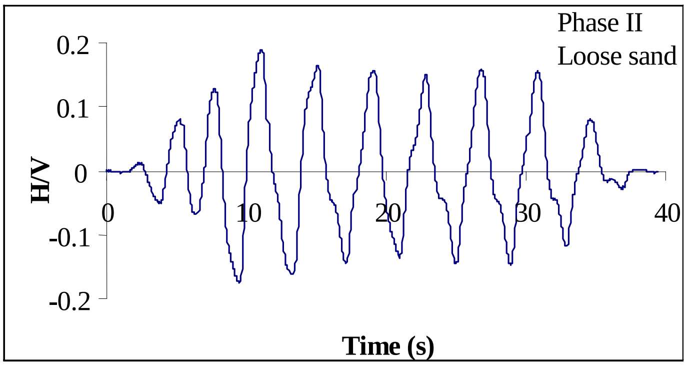
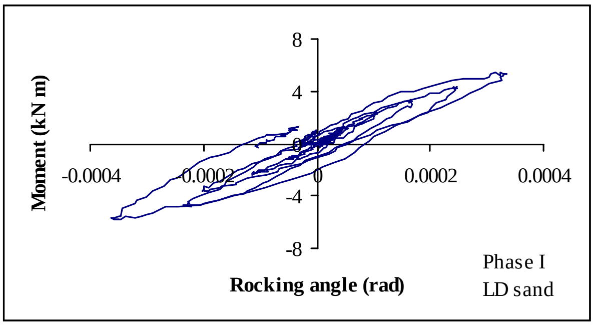


Fig. 2: Phase II: time-history of horizontal force

TEST RESULTS

Phase I

The application of force-controlled cycles of small amplitude resulted in substantially similar behaviour in the two tests. As shown in Fig. 3 for the overturning moment vs. rocking, hysteresis loops are rather stable and denote a limited amount of dissipation. The rocking stiffness for the HD case is more than twice that in the LD case. The final settlement of the foundation after this loading phase was about 0.15 mm in both HD and LD cases, denoting that for low values of the seismic coefficient (up to about 0.05 g) the non-recoverable part of foundation displacement and rocking is negligible.



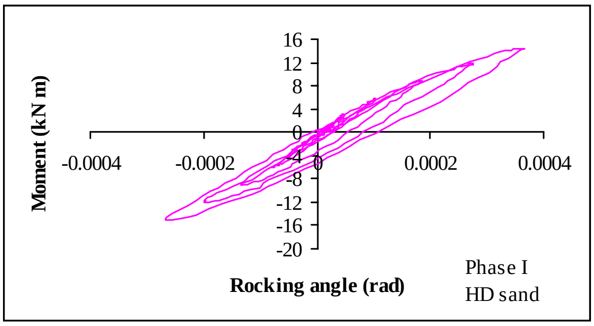


Fig. 3: Phase I: overturning moment vs. rocking for LD (left) and HD (right) soil conditions

Phase II

This is the most meaningful loading phase for analysing the foundation behaviour under earthquake loading. Some representative results are illustrated in Figures 4 and 5, namely the overturning moment vs. rocking diagrams and the vertical settlements, respectively. In both HD and LD cases, the largest cycle corresponds to the peak of horizontal force, while the subsequent cycles are essentially contained inside this loop. During the most severe loading cycle, stiffness reduces to about 30% of the initial value for the HD case and to about 20% for the LD case. However, as shown in Fig. 4, the initial stiffness is gradually recovered during subsequent cycles.

We note that even though in this loading phase the seismic coefficient did not exceed 0.2, the permanent deformations of the foundation are significant, especially in terms of rocking. Recalling that a foundation rotation of 2 mrad is considered as a threshold value for the onset of cracking on the superstructure (e.g. [Lambe and Whitman, 1969]), this value is slightly exceeded during several cycles in the HD case, while in the LD case the peak rocking reaches 6 mrad. According to Eurocode7 [EC7, 1994], the latter value is the relative rotation likely to cause an ultimate limit state. The permanent value in the LD case at the end of the loading phase is about 2 mrad. Vertical settlements experienced by the foundation are less severe than rocking in terms of serviceability limit state. However, for LD conditions, the final settlement is about 10 mm, that is about 60% of the vertical settlement under the static loading phase. For HD conditions the increment with respect to static loading is about 30%.

These results stress the need of improving the accuracy of current predictions of foundation settlements and rocking during earthquakes; the indication is that the movements may attain significant values, possibly beyond serviceability limit states, even under a moderate seismic excitation as the one considered in these tests.

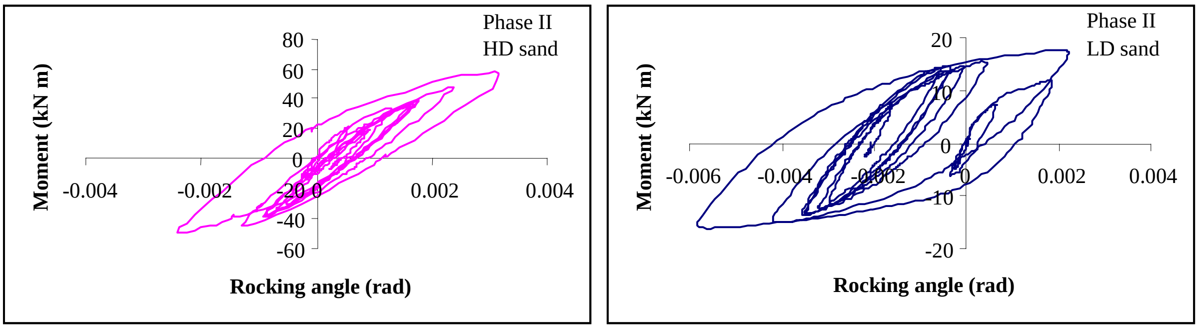


Fig. 4: Phase II: Overturning moment vs. rocking for HD and LD conditions.

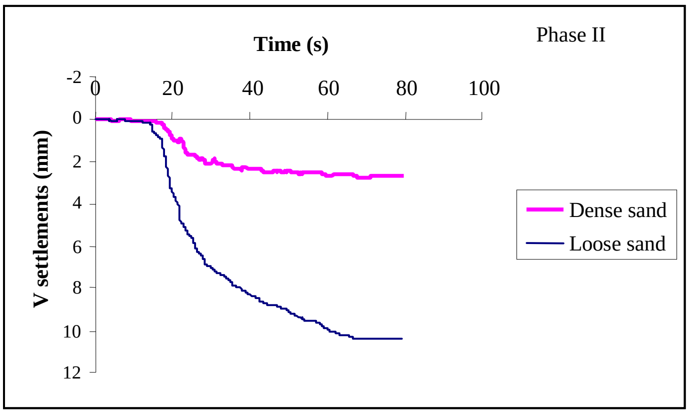


Fig. 5: Phase II: Vertical displacement of the foundation.

Phase III

During this phase, displacement-controlled cycles with increasing amplitude were applied to the foundation with the aim of reaching the ultimate foundation resistance.

The loops described by the curves of overturning moment vs. rocking (Fig. 6) are remarkably regular, with a characteristic s-shape for the HD case. Theoretical modelling of the experiment [Pedretti, 1998], has shown that such shape can be explained in terms of foundation uplift under eccentric loading. During uplift, the stiffness of the system degrades significantly but, as soon as the eccentric load decreases, the contact at the soil-foundation interface increases correspondingly and the rocking stiffness recovers. This effect does not appear for LD conditions, since "punching" is the prevailing failure mode of the foundation in low to medium dense conditions [Vesic, 1973]: the foundation sinks into the sand and uplift effects are prevented.

Foundation punching in LD conditions is well illustrated in Fig. 7, with settlements that tend to increase linearly, probably due to the progressive expulsion of sand from underneath the plate toward the sides during the sinking of the foundation. A linear increase of settlements is seen to occur also for HD conditions, but final values in this case do not exceed 20 mm.

In Fig. 8 foundation settlements are plotted as a function of the seismic coefficient k_h. A limit value of k_h slightly lower than 0.4 is found, both for HD and LD soil conditions. However, the interpretation of such value in terms of seismic bearing capacity should be considered with care, even in the HD case. First, the lateral boundaries of the concrete caisson are too close to the foundation for a shear failure mechanism to completely develop, so that the resulting bearing capacity should increase with respect to the theoretical value. Second, the experiments were carried out in dynamic conditions, that generally leads to an increase of the bearing capacity with respect to the conventional monotonic loading (Vesic, 1973).

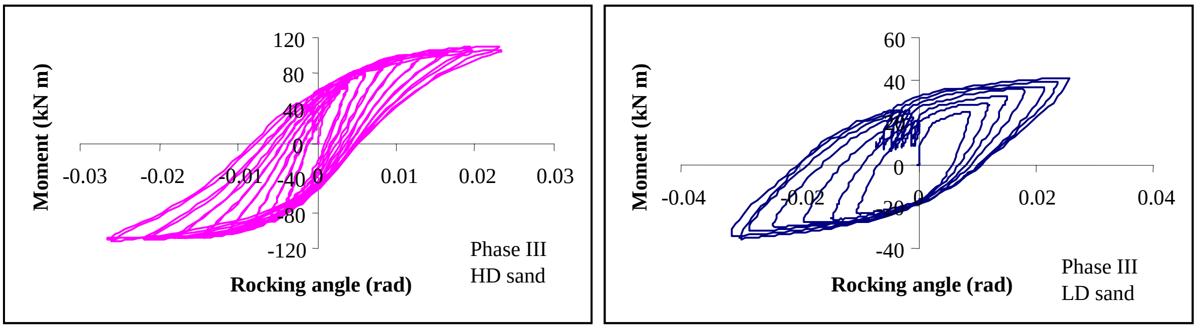


Fig. 6: Phase III: Overturning moment vs. rocking for HD and LD conditions.

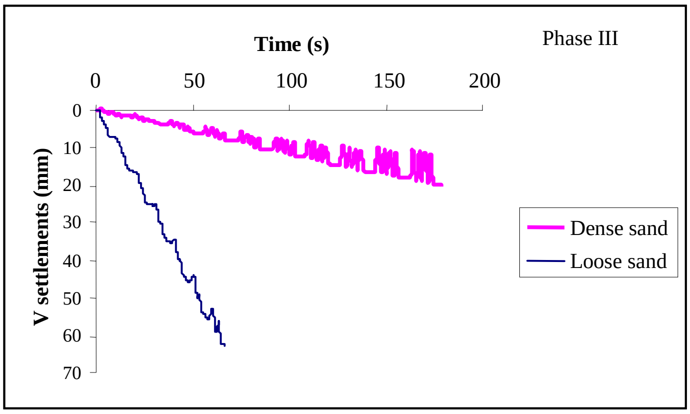


Fig. 7: Phase III: Foundation settlements.

CONCLUSIONS

We have explored the capabilities of a new experimental approach for the analysis of soil-structure interaction effects during seismic loading. The main advantages are the following: a) full-scale modelling; b) accurate determination of soil properties; c) application of realistic time histories of horizontal force and overturning moment. On the other side, a) soil inertia forces are not taken into account, b) lateral and bottom boundaries are close to the foundation and cannot reproduce completely free-field conditions, c) repetition of the experiment involves the treatment of a large amount of soil material.

The results illustrated herein have been obtained with two almost completely saturated soil specimens, characterized by relative densities Dr ≈ 45% (LD) and Dr ≈ 85% (HD), subjected to the same loading phases: 1) small amplitude cycles; 2) earthquake-like force-controlled excitation; 3) cyclic displacements up to foundation failure.

While a thorough investigation of these results is still under way, one important indication of the tests is the relevance of permanent deformations of shallow foundations even during the moderate seismic excitation used in the earthquake-like loading phase (seismic coefficient k_h=0.2). The vertical settlements observed are an important fraction (from 30% to 60%) of the static ones, and may play a relevant role in the development of differential settlements between adjacent points of the same structure. Rocking values are the most significant ones. Permanent values at the end of excitation reach 2 mrad for LD conditions, while peak values in the transient phase reach about 2 mrad for HD and 6 mrad for LD conditions. The attainment of such values can affect significantly the serviceability of the structure.

The results are considered to be very accurate and provide a useful basis for a number of investigations on dynamic soil-structure interaction, such as: a) validation of non-linear constitutive models for soil-structure interaction analyses; b) validation of current methods for assessing the seismic bearing capacity of shallow foundations and of the simplified approaches for calculation of permanent deformations; c) analysis of the effect of uplift of shallow foundations; d) check of the currently used formulas for spring and dashpot coefficients of shallow foundations.

A more complete description of the experimental results can be found in [Negro et al., 1998] or can be accessed in the TRISEE Project web site http://www.crs4.it/trisee.

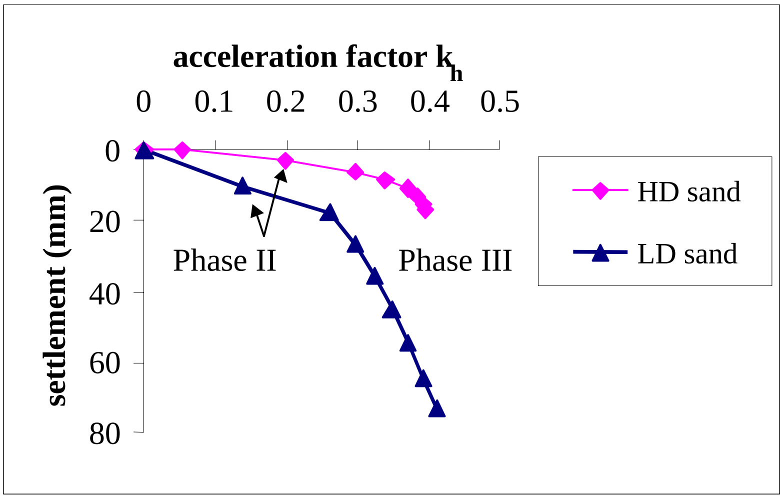


Fig. 8 - Comparison of foundation settlements in HD and LD soil conditions as a function of the seismic coefficient (k_h=H_{max}/V).

ACKNOWLEDGEMENTS

The experimental activity presented herein was part of the research programme TRISEE funded by the European Commission under contract ENV-CT96-0254. Many individuals and organisations contributed to the successful execution of the tests. G. Verzeletti, head of the European Laboratory for Structural Assessment (ELSA), designed the sand spreader, and was responsible for the execution of the tests. J. Molina managed to overcome the speed limitations initially assumed for the apparatus. The enthusiasm of the whole ELSA staff made possible the execution of the tests. M. Jamiolkowski and D.C.F. Lo Presti from the Politecnico di Torino made an invaluable contribution in the conception and execution of the tests. S. Pedroni and P. Morabito contributed with the expertise of ENEL/CRIS, and provided part of the instrumentation. The testing activity would not have been possible without the participation of all of them.

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