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CENTRIFUGE MODELLING OF A GEOGRID REINFORCED FOUNDATION ON SOFT COHESIVE SOIL

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ABSTRACT

The paper is concerned with the physical modelling carried out in the geotechnical centrifuge operated at ISMES (Italy) in order to investigate the stability of a geogrid reinforced foundation on soft cohesive soil.

The foundation consists of a 3 m high bank of coarse granular material supporting a breakwater for oil tankers docking to be built at an Electrical Power Plant site of the Italian National Electricity Board (ENEL). The breakwater stability has been studied taking into account both the vertical and the horizontal loads acting on the breakwater structure. The results obtained show that the geogrid reinforcement yelds a significant enhancement of the ultimate limit loads acting on the breakwater. However the values are much more increased for the vertical loads than for the horizontal ones. On the opposite the reinforcement has a poor influence on the settlements and the rotations undergone by the breakwater under the service loads. Possible scale effects influencing the interface behaviour in the centrifuge tests have been taken into account and an appropriate design of the model geogrid has been adopted to try to minimise them.

1 INTRODUCTION

The paper is concerned with the physical modelling carried out in the geotechnical centrifuge operated at ISMES (Italy) [Baldi et al, 1988] in order to evaluate the stability of a geogrid reinforced foundation on soft cohesive soil. The foundation consists of a 3 m high bank of coarse granular material placed below a breakwater for oil tankers docking to be built at an Electrical Power Plant site of the Italian National Electricity Board (ENEL). The breakwater is a reinforced concrete gravity caissons dike whose submerged part is 15 m in height and 23m in width. A schematic cross section of the breakwater is shown in Fig. 1. The foundation soil consists of a 13m thick layer of overconsolidated silty-clay of medium consistency underlain by a very deep deposit of gravel and sand. The stratigraphic profile is shown in Fig. 1. A preliminary analysis, which is outside of the scope of this paper, was performed in order to determine the horizontal pseudo-static force induced by the maximum expected waves storm. The following maximum design forces were assumed in this study:

a) vertical load V=3100kN/m. The above value is applied at the base of the caisson and corresponds to an uniformly distributed vertical pressure p≅150kPa;

b) horizontal load H=1200kN/m. H acts at approximately 10.4 m from the base of the caisson.

The poor characteristics of the cohesive soil layer associated to the severe loading conditions pose serious problems both in terms of foundation stability and settlements. For this reason it was decided to reinforce the soil by placing a high strength geogrid tensile reinforcement at the foundation base. Such

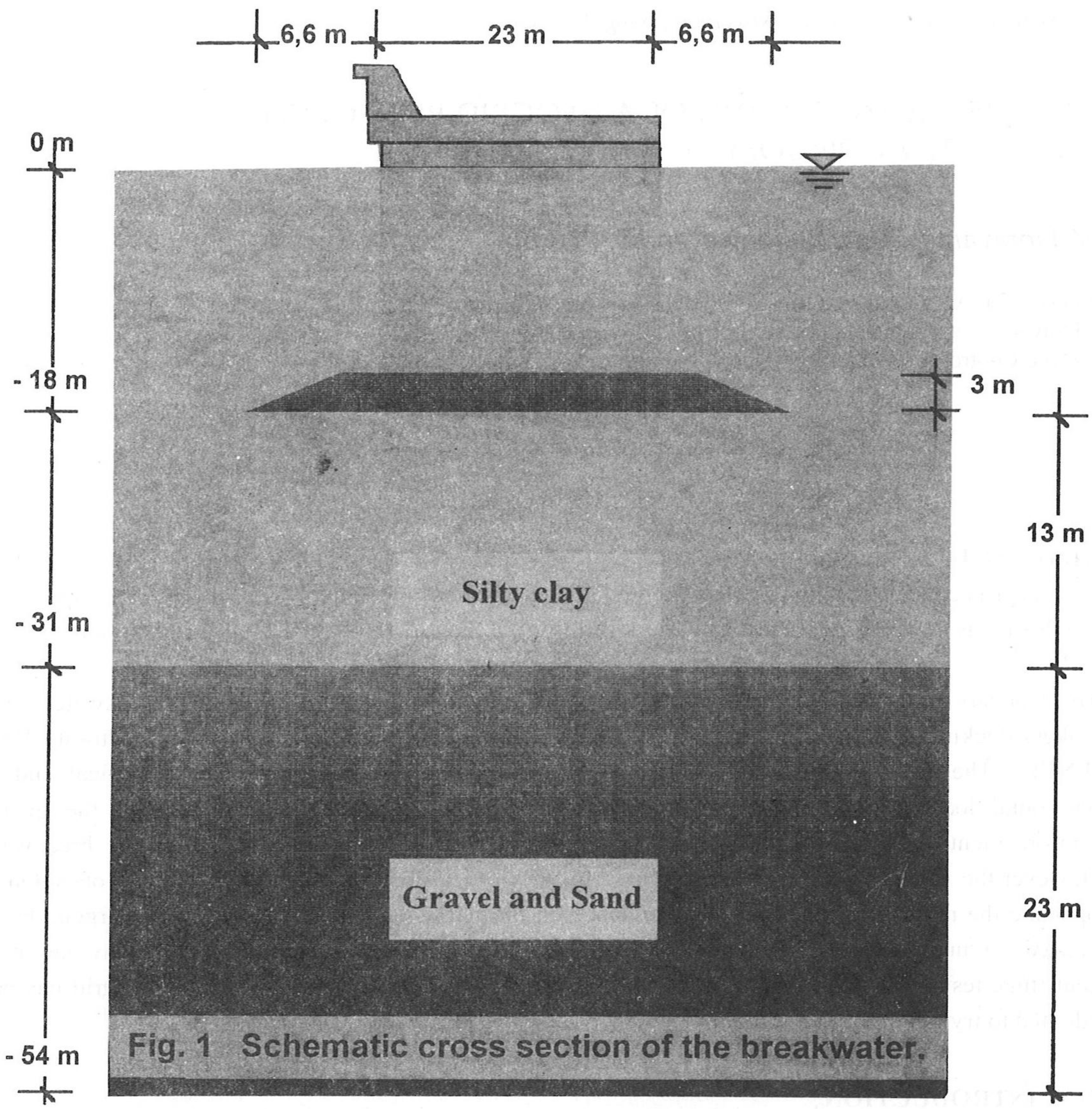


Fig. 1 Schematic cross section of the breakwater.

technique has received in the past several applications mainly related to the construction of embankments on soft soils [(Bergado et al (1994), Holtz (1990), Gourc (1992), Rowe at al (1984), Watari et al (1986), Dabke et al (1992)]. It has also been extensively studied from a theoretical point of view [Jewell (1988), Hird and Kwok (1990), Hird and Jewell (1989), Rowe (1984), Rowe and Mylleville (1990), Rowe and Soderman (1986), Milligan and La Rochelle (1981), Schaefer and Duncan (1988) Wu et al (1992), Oikawa (1988), Leshchinsky and Smith (1988), Sabhahit et al (1994)]. Such studies have evidentiated that the tensile reinforcement yelds a significant enhancement of the critical height of the embankments but only a limited reduction of the total settlements (5 to 10%). The improvement of the bearing capacity is strongly dependent on the rigidity and strength of the fabric. The above results cant be completely extended to the considered breakwater because the concrete gravity caisson imposes a rigid boundary condition on top of the bank. On the other band, solutions based on numerical modelling suffer several uncertainties mainly because of the lack of reliable interface models. For this reason it was considered more appropriate to study the problem through a physical modelling in the geotechnical centrifuge [Blivet et al (1986), Bolton (1990), Gourc et al (1992), Taniguchi et al (1988), Sprigman et al (1992)]. As it is well known, provided that one fulfils appropriate scaling requirements correlating single or groups of the influencing parameters, the centrifuge allows a good duplication of the full scale stresses and strains. This enables one to take into account the most important reological features of the real

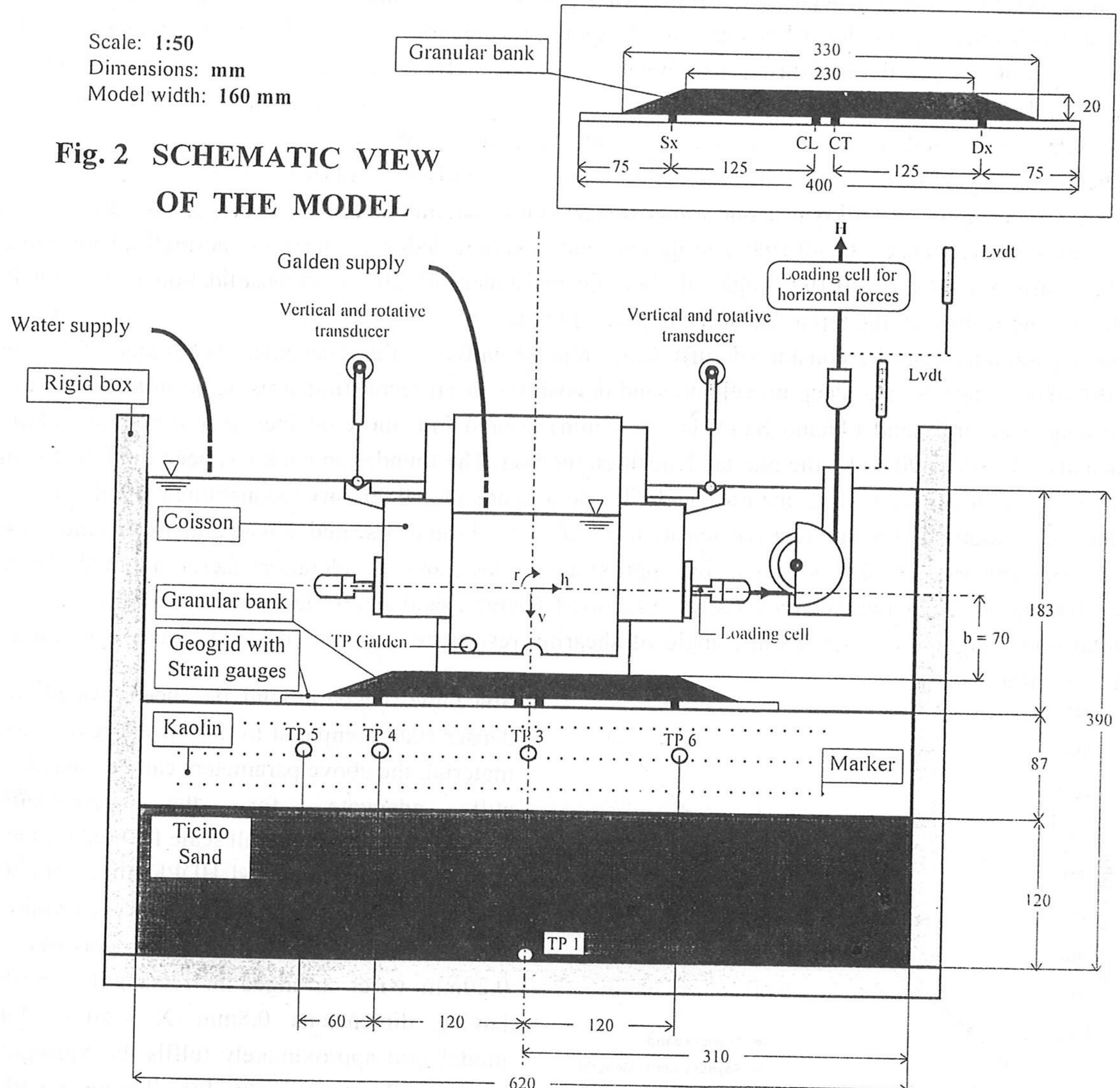


Fig. 2 SCHEMATIC VIEW OF THE MODEL

materials such as non linear stress-strain response, curved failure envelope, dilatancy etc. Furthermore it should be taken into account that the frictional bond between the soil and the textile grid depends on the relative movements between the longitudinai strands of the meshes and the soil which is inside the apertures. As pointed out by Springman at al (1992) an ideal model should adopt identical materials with all significant dimensions scaled down by a factor N, being N the ratio between the centrifuge acceleration (a) and the earth gravity acceleration (g). Alternatively materials having the same unit weight and stress-strain constitutive relationships could be adopted maintaining the same geometrical scaling factor N. When applying the above statements to the reinforced earth, the textile capacity (tensile resistance or tensile modulus per unit width of the sheet) of the model grid ("microgrid") is reduced by a factor N. If the meshes of the grid are characterised by a limited spacing between two adjacent transverse ribs and by a sufficient width of such ribs (compared to the average grain size of the soil) then the grid acts as a "sand-encrusted sheet". When this happens, the interface behaviour is mainly governed by the soil to soil relative movement, and no further scaling factor is required. The model grid adopted in this research has been specifically designed in order to fulfil such requirements.

2 MODEL DESCRIPTION

All tests have been performed in a rectangular cross section rigid walls container having internal dimensions 0,62m X 0,40m at an acceleration ratio $N=a/g=150$. This value has been selected as a compromise between the needs to fulfil the scaling requirements stated above and to avoid undesirable boundary effects on model behaviour. Fig. 2 reports a schematic view of the model evidentiating the loading systems and the measuring instrumentation adopted. The cohesive soil layer has been modelled using a Kaolin mud preliminary consolidated in the container up to a maximum vertical pressure of 200kPa and then unloaded before housing the container in the centrifuge.

The most relevant properties of the used Kaolin can be summarised as follows:

total unit weight : γ_l=17kN/m^3; one dimensional compressibility index : Cc=0,198; one dimensional compressibility index : Cc=0,198; one dimensional swelling index : Cs=0,044; normalised undrained shear strength : Cu/σ'_p=0,197; angle of shearing resistance :ϕ'=20°; overconsolidation ratio : OCR= decreasing from 8 at the top to 2,5 at the bottom of the layer.

Such parameters can be considered sufficiently representative of the mechanical behaviour of the real silty-clay materials. The deep gravel and sand deposit has been reconstituted using an uniform medium to coarse natural sand (Ticino Sand, D_{50}=0,47mm) poured into the container at a very high relative density (Dr=80 to 90%) by the pluvial deposition method. The foundation bank has been modelled using an uniform medium to fine natural sand (Toyoura sand, D_{50}=0,20mm) reconstituted by the pluvial deposition method at a high relative density (Dr=70%). Such sand is scaled down compared to the coarse granular material of the prototype by approximately the same acceleration factor adopted for the centrifuge tests. The main characteristics of the used Toyoura sand can be described as follows:

total unit weight : γ_l=19.5 kN/m^3; angle of shearing resistance : ϕ'=40°; drained Young's modulus : E'=40MPa.

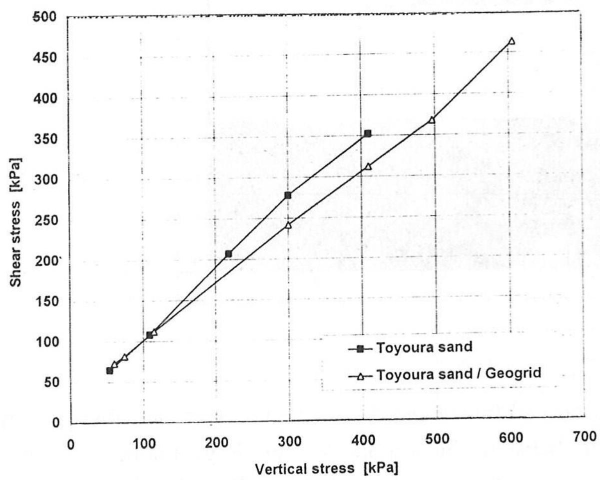


Fig. 3 Peak strength envelopes from direct shear box tests

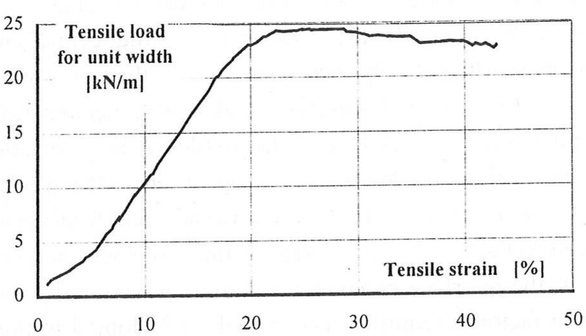


Fig. 4 Tensile stress-strain curve of the model grid

Since the Toyoura sand has been placed in a denser state compared to that of the real coarser material, the above parameters can be considered still adeguate for the geotechnical characterisation of the full scale foundation bank. An expressly constructed HDPE microgrid was used to model the full scale geosynthetic reinforcement. The microgrid consists of 0,20mm wires arranged in rectangular meshes having dimensions 0,8mm X 0,5mm. Such model grid approximately fulfils the Springman et al (1992) requirements for allowing a soil to soil relative movement like behaviour. However it should be expected that the above requirements are better fulfilled at low sliding stresses than at high ones. This arised because of the more pronounced deformability of the transverse ribs at high sliding stresses which increases the relative freedom between the mesh wires and the entrapped soil grains. Fig 3 shows a comparison between the peak strength envelopes measured in a modified direct shear box both on Toyoura sand and on the microgrid Toyoura sand interface [Campagnoli (1994)]. It can be seen that in the range of the vertical stresses induced on the foundation by the maximum service load ($\sigma_n \cong$ 150kPa) the interfacial friction angle (δ) approximately equals the soil to soil angle of shearing resistance (φ'). The tensile stress-strain behaviour of the model grid is reported in Fig. 4. The microgrid exhibits a tensile capacity T=24kN/m and a tensile modulus evaluated at 10% longitudinal strain which is approximately J=10 kN/m. Taking into account the scaling rules described in previous chapter, the above values are indicative of a very high capacity special type full scale geogrid. Consequently the results obtained should be considered as an upper limit for the proposed reinforcement technique. The purpose to investigate such an extremely performing geogrid was just to verify the full potentiality of the technique rather than to give an indication about the type of the geosynthetic reinforcement to be used. As shown in Fig. 2 the position of the water table has been taken 0,183m above the top surface of the soil which corresponds to a full scale sounding depth of 27m.

3. LOADING SYSTEM AND INSTRUMENTATION

A schematic view of the loading systems adopted is shown in Fig. 2. The vertical loads have been applied in steps through the letting in of a high density liquid ("Galden") into the caisson. The horizontal loads have been applied by two servo-controlled hydraulic jacks capable of continuously adjusting the direction and the point of application of the thrust in order to counteract the effects of the settlements and the rotations undergone by the caisson. One loading cell has been used for measuring the horizontal loads while the vertical force has been measured by monitoring the volume of the liquid sent to the caisson. The other instrumentation consists of: n. 2 LVDT transducers for the measurement of the vertical settlements at the top of the caisson; n.2 rotative transducers for measuring the horizontal displacements and the rotations at the top of the caisson; n. 4 miniature pore pressure transducers (TP) inside the cohesive layer and n. 1 inside the bottom layer; n. 3 horizontal alignments of lead shots inside the cohesive layer; n. 4 strain gauge extensometers installed on the model grid.

4 TEST PROCEDURE AND RESULTS

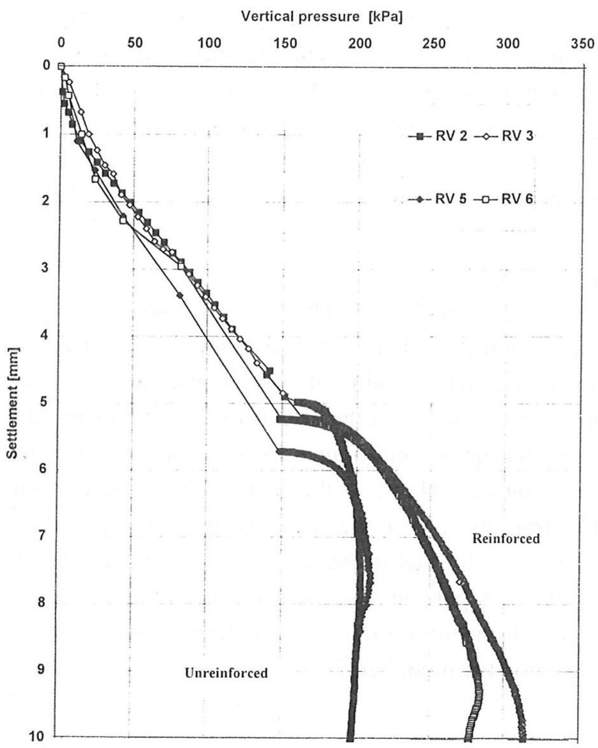
All tests were performed in two stages. In the first one the soil was firstly reconsolidated in fly with the granular bank already in place and then the caisson was lowered on the top of the bank. In the second stage the foundation was preliminary loaded in the vertical direction up to the maximum service load (p=150kPa) and then brought to its ultimate state by increasing the vertical force (RV tests) or by applying subsequent steps of the horizontal thrust (RIZ tests). Each series of tests was performed both on reinforced (R) and unreinforced (U) foundations. Finally each test was duplicated in order to check their repeatability. A complete list of the tests performed is reported below:

TABLE 1 : List of the tests

Test abbreviations	Foundation type	Loading direction at failure	Test abbreviations	Foundation type	Loading direction at failure
RV2	U	V	RIZ3	U	H
RV3	R	V	RIZ4	R	H
RV4	U	V	RIZ5	U	H
RV5	R	V	RIZ6	R	H

The results of the RV tests have been expressed in terms of pressure vs settlement curves. They are summarised in Fig. 5. All curves exhibit a clear change in trend at the vertical pressure of 150kPa. The first part of the curves refers to the preliminary loading of the caisson under the service loads. This phase is conducted in loading steps reproducing approximately drained conditions. The following stage corresponds to the collapse phase of the test. In this stage the caisson is loaded in undrained conditions by a rapid increase of the vertical pressure. The loading is continued up to the development of very large settlements. The results reported in Fig. 5 show a good repeatability of the tests. Furthermore the total drained settlements observed in the first phase of the tests are approximately the same for both the unreinforced and the reinforced foundations. On the opposite the ultimate limit vertical pressure is significantly influenced by the presence of the reinforcement increasing from 200kPa observed on the unreinforced foundation to 280kPa observed on the reinforced one. As far the RIZ tests are concerned

Fig. 5 Vertical pressure vs settlement curves in RV tests



only the results referring to the failure phase of the tests have been reported here. They have been expressed in terms of the following parameters:

- horizontal thrust vs. rigid vertical settlement of the caisson (Fig.6)
- horizontal thrust vs. rigid horizontal displacement of the caisson (Fig.7)
- horizontal thrust vs. rigid angular rotation of the caisson (Fig.8)

As it can be seen in all tests there is a clear evidence of the attainment of an ultimate limit peak value of the horizontal thrust followed by a softening phase. The repeatability of the tests seems to be still quite good even if it is better for the unreinforced foundation than for the reinforced one. Comparing the load-displacement curves obtained from the same series of tests performed on reinforced and unreinforced foundations, it can be seen that they follow an approximately indistinct trend up to a value of the horizontal thrust which is quite close to the failure. Only when the ultimate state is approached the corresponding curves start to differ reaching a peak failure values of the horizontal force which, for the reinforced foundation, are approximately 15-20% higher those for the unreinforced one.

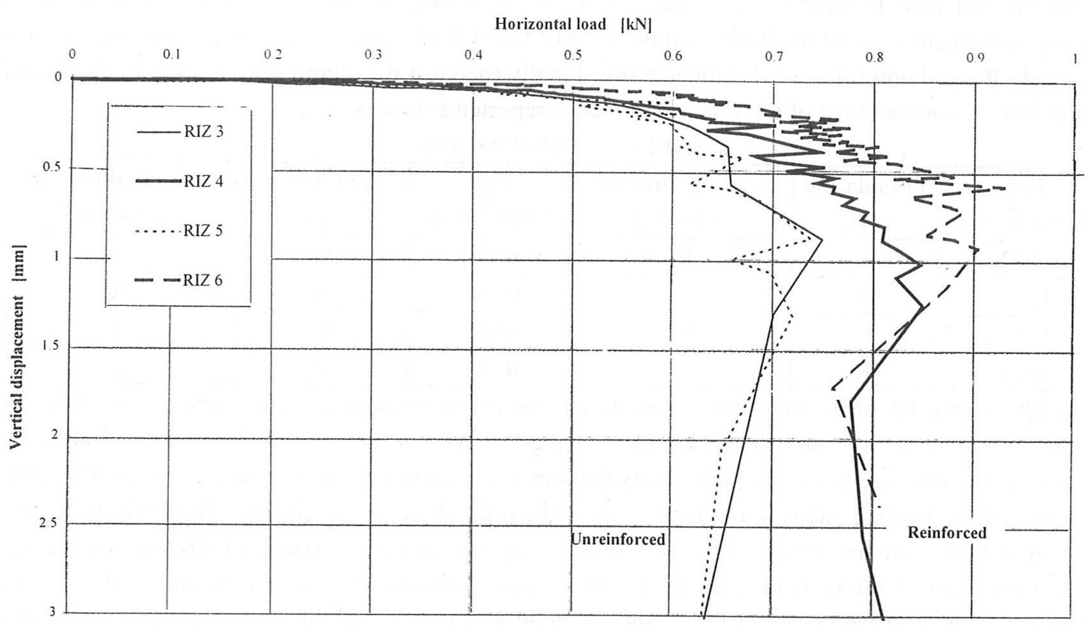


Fig. 6 Horizontal load vs average vertical settlement in RIZ tests

5 CONCLUSIONS

The research is concerned with the physical modelling in the geotechnical centrifuge operated at ISMES (Italy) of a geogrid reinforced foundation on soft cohesive soil. The foundation consists of a coarse

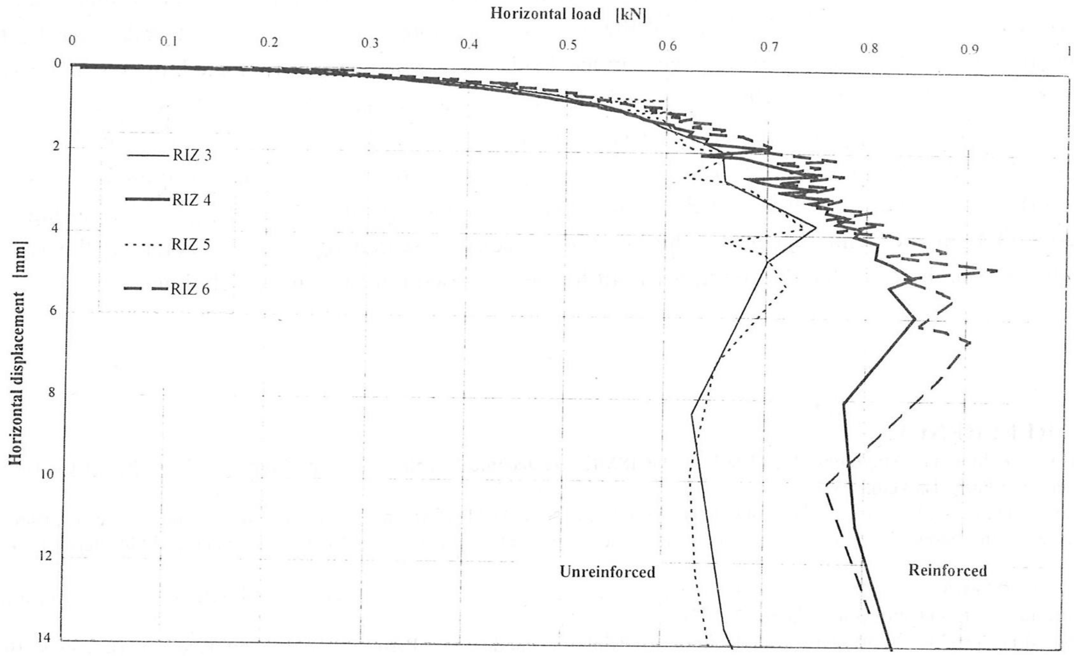


Fig. 7 Horizontal load vs average horizontal displacement in RIZ tests

granular bank supporting a concrete gravity caisson breakwater to be built at an Eletrical Power Plant site of the Italian National Electricity Board (ENEL). The aim of the research was to investigate the effect of a high strength geogrid reinforcement sheet on the settlements and the stability of the foundation. All centrifuge tests have been performed at an acceleration ratio N=a/g=150. Two series of tests have been carried out respectively referred to as RV and RIZ tests. In the RV tests, the foundation, once consolidated under the maximum vertical service load (p=150kPa) has been brought to collapse in undrained condition by a rapid increase of the vertical load. In the RIZ tests the foundation, preliminarly consolidated as above, has been brought to failure by subjecting the caisson to increasing steps of the horizontal thrust simulating the effects of the maximum assume design waves storm. The tests have been performed both on reinforced and unreinforced foundation in order to evaluate the effect of the

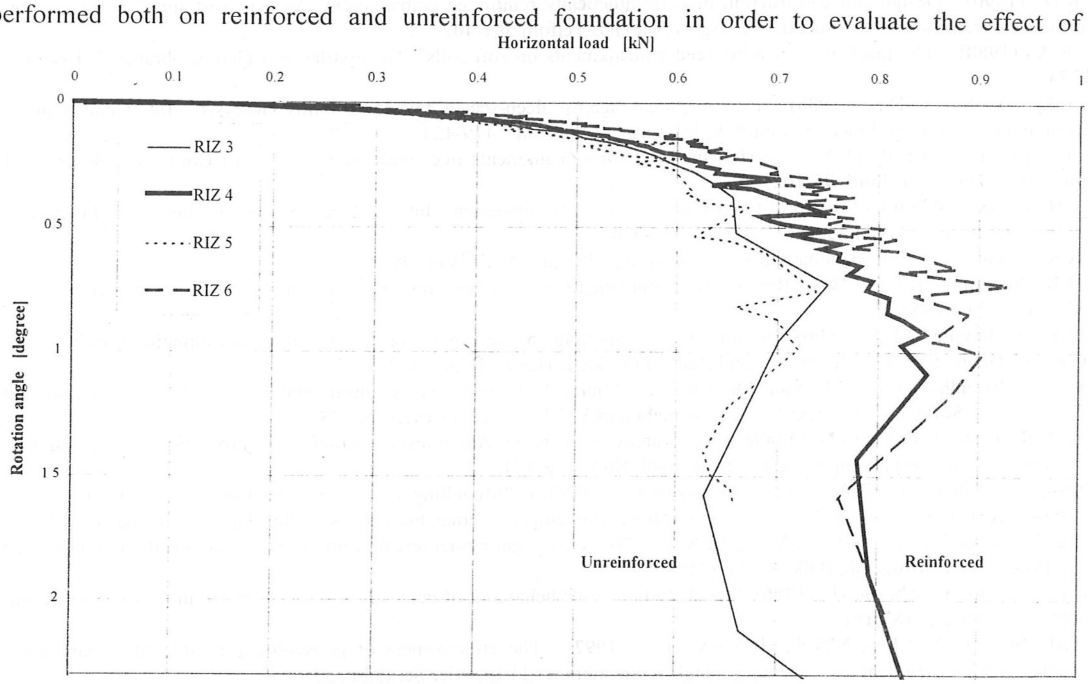


Fig. 8 Horizontal load vs rotation angle in RIZ tests

reinforcement. Each test has been duplicated in order to check their repeatability. The results obtained has evidentiated a good reproducibility of the tests. Furthermore the total settlement undergone by the foundation under the application of the service loads were practically unaffected by the geotextile reinforcement. On the opposite the ultimate limit vertical load was significantly enhanced by the use of the reinforcement (approximately 40-45%) The benefict was lower for the horizontal ultimate limit load (15%-20%). The model geogrid ("microgrid") was specifically designed in order to minimise possible scale effects affecting the interface behaviour between soil and grid in the centrifuge tests. Taking into account that, in the centrifuge tests, the geogrid capacity is scaled down by a factor N, the results obtained should be considered as un upper limit for the proposed reinforcement technique.

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