ISSN 1980-9743

Soils and Rocks

Soils and Rocks

An International Journal of Geotechnical and Geoenvironmental Engineering

ABMS

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Volume 32, N. 1
January-April 2009

Soils and Rocks is an International Journal of Geotechnical and Geoenvironmental Engineering published by

ABMS - Brazilian Association for Soil Mechanics and Geotechnical Engineering
Av. Prof. Almeida Prado, 532, IPT/DEC-Prédio 54
05508-901 São Paulo, SP
Brazil

ABGE - Brazilian Association for Engineering Geology and the Environment
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05508-901 São Paulo, SP
Brazil

SPG – Portuguese Geotechnical Society
LNEC, Avenida do Brasil, 101
1700-066 Lisboa
Portugal

Issue Date: March 2009

Issue: 3,200 copies

Manuscript Submission: For review criteria and manuscript submission information, see Instructions for Authors at the end.

SOILS and ROCKS

An International Journal of Geotechnical and Geoenvironmental Engineering

Soils and Rocks publishes papers in English in the broad fields of Geotechnical Engineering, Engineering Geology and Geo-environmental Engineering. The Journal is published in April, August and December. Subscription price is US$ 90.00 per year. The journal, with the name "Solos e Rochas", was first published in 1978 by the Graduate School of Engineering-Federal University of Rio de Janeiro (COPPE-UFRJ). In 1980 it became the official magazine of the Brazilian Association for Soil Mechanics and Geotechnical Engineering (ABMS), acquiring the national character that had been the intention of its founders. In 1986 it also became the official Journal of the Brazilian Association for Engineering Geology and the Environment (ABGE) and in 1999 became the Latin American Geotechnical Journal, following the support of Latin-American representatives gathered for the Pan-American Conference of Guadalajara (1996). In 2007 the journal acquired the status of an international journal under the name of Soils and Rocks, published by the Brazilian Association for Soil Mechanics and Geotechnical Engineering (ABMS), Brazilian Association for Engineering Geology and the Environment (ABGE) and Portuguese Geotechnical Society (SPG).

Soils and Rocks

1978,	1 (1, 2)
1979,	1 (3), 2 (1,2)
1980-1983,	3-6 (1, 2, 3)
1984,	7 (single number)
1985-1987,	8-10 (1, 2, 3)
1988-1990,	11-13 (single number)
1991-1992,	14-15 (1, 2)
1993,	16 (1, 2, 3, 4)
1994-2007,	17-31 (1, 2, 3)
2009,	**32 (1,**

ISSN 1980-9743 CDU 624.131.1

ISSN 1980-9743

SOILS and ROCKS
An International Journal of Geotechnical and Geoenvironmental Engineering

Publication of
ABMS - Brazilian Association for Soil Mechanics and Geotechnical Engineering
ABGE - Brazilian Association for Engineering Geology and the Environment
SPG - Portuguese Geotechnical Society

Volume 32, N. 1, January-April 2009

Table of Contents

Articles

Three Dimensional Finite Element Analysis and Back-analysis of CFA Standard Pile Groups and Piled Rafts Founded on Tropical Soil

Tomás Janda, Renato Pinto da Cunha, Pavel Kuklík, Gérson Miranda dos Anjos

Abstract. This paper deals with Plaxis 3D finite element simulations of the mechanical response of deep foundations founded in a collapsible tropical soil. Main attention is initially paid to differences between single continuous flight auger (CFA) pile behavior and the behavior of CFA piles in standard groups. The numerically computed load-settlement curves are compared to field load test data obtained at the experimental research site of the University of Brasília (UnB), leading to conclusions about the appropriateness of adopting laboratory, *in situ* or back calculated parameters as input of numerical programs that simulate 3D foundation systems. Further, the contribution of the contact surficial soil/top raft is numerically examined by simulating the behavior of identical "piled raft" systems founded in the same site. The numerical simulated results of "piled raft" and standard pile group systems are then compared in terms of load capacity, system stiffness, load share between pile tip, shaft and raft, and mean developed lateral pile shaft friction. Having the results at distinct loading stages, as at working and failure levels, the analyses show the differential behavior, and design obtained responses, one may expect from conventional pile groups and "piled rafts" of CFA floating piles when founded in tropical soils. It is a mixed theoretical/experimental paper with practical interest for foundation designers and constructors.

Key words: pile group, piled raft, numerical analysis, finite element method, settlement, collapsible soil, load distribution, Mohr-Coulomb model.

1. Introduction

Local practice in the Federal District of Brazil shows that one of the most economical types of foundations that can be used to sustain loads from elements founded on tropical unsaturated, or saturated, soils is the continuous flight auger (CFA) pile. Hence, CFA piles are frequently used in foundation systems within the city of Brasília as well as adjacent areas (even in other cities as Goiânia, for instance). Due to their relatively small diameter when compared to traditional large scale bored piles the CFA foundations are, almost in all cases, constructed in groups with a relatively small spacing pile to pile (2 to 3 diameters, in general). The understanding of the entire foundation system requires knowledge not only about the single pile interaction with the soil environment, but also the mutual influence of individual piles within the group. The complexity of the problem does not end here, especially when the pile group supports a top raft, or capped block, which is in close contact with the surficial soil. Since both structural parts of the foundation - piles and raft - need to be considered for a proper understanding of the problem, major attention must be given to numerical techniques which are capable of properly simulating the behavior of the whole foundation system, taking on account the real geometry and individual characteristics, plus the complex interactions between structural and geotechnical elements of the foundation. It is, basically, a question related to the understanding of the behavior of a "piled raft" system, rather than a traditional pile group.

In the past decade several papers have been published with emphasis on what are now called "piled-rafts", *i.e.*, pile groups in which the raft connecting the pile heads positively contributes to the overall foundation behavior (for example Ottaviani, 1975; Randolph, 1994, Mandolini & Viggiani, 1997, Poulos, 1998; Cunha & Sales, 1998 and Sales *et al.*, 1999). Other more recent papers have expanded upon these initial ideas, such as those by Cunha *et al.* (2000a and b, 2001, 2004, 2006), Sales *et al.* (2005), and Cunha & Zhang (2006). One should however realize that the term "piled raft" is expressed in the present and at all aforementioned papers as a "foundation system in which both structural components (piles and top raft) interact with each other and with the surrounding soil to sustain vertical, horizontal or moment loads coming from supported superstructures". Independently if the piles are designed as "settlement reducers" or not, piled rafts will be defined herein (and in future papers) by the basic statement previously cited. In fact, according to Mandolini (2003), piled rafts can designed under a "capacity and settlement based design", "capacity based design" or as "differential settlement based design". It is then important to mention that aforementioned definition is unquestionable valid to describe piled

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Submitted on November 29, 2007; Final Acceptance on July 3, 2008; Discussion open until August 31, 2009.

rafts designed in any of the possible criteria established by this latter author.

On the other hand, standard or traditional pile groups are those in which, by design considerations or simply by physical aspects (as the experimental tests of this paper), no superstructure load is supported by the soil underneath the raft, or by the raft itself. In other words, the entire vertical or horizontal load is solely supported by the structural pile elements and their surrounding soil at shaft and base. In this case the raft merely serves as a structural element to distribute the load between pile caps, rather than to help them in sustaining the superstructure load.

Therefore, this paper continues on this particular topic, exploring the numerical evaluation of standard pile groups vs. piled rafts, and the latter system advantages for real case designs. This particular exercise was incorporated as part of a jointed research project between the University of Brasília and the Czech Technical University, which has allowed one student of this latter University (first author) to develop his "sandwich" doctorate in the formerly cited institution. The data presented herein is the main outcome from this fruitful international partnership.

Thus, the main objective of the paper is to numerically analyze the behavior of small (traditional) pile groups and to compare them to the behavior of a single pile founded in an identical soil horizon. In addition, the article is also aimed at the foundation raft constructed on the top of the pile group and its influence on the overall bearing capacity, stiffness and final settlement of the overall foundation system (piled raft problem). Some attention is also paid to the distribution of the vertical reaction forces acting on the pile tip, pile shaft and top raft (in cases of piled raft systems). It is organized as follows: The introduction of the subject is followed by the review of the site's tropical soil properties and the description of the performed field load tests. The numerical method and its application to the solved problem are also described in the second section. Further, in the results section, the numerical load-settlement curves obtained for distinct input parameters are presented and compared to existing experimental data. This set of results is followed by back calculations done with the same foundation systems and site characteristics. Finally, numerical simulations of similar systems designed with piled raft concepts are carried out, and compared to previous results in which solely traditional pile groups were simulated. Practical aspects for using numerical tools for solving problems of complex foundation systems are concluded in the final section, and the advantages of designing such systems as piled rafts are also outlined and encouraged.

2. Site and Pile Characteristics

With increasing building density in the Brazilian capital Brasília and its neighboring area (Federal district) the civil engineers have to deal with the problem of designing foundations on the tropical porous clay, which, by the way, is commonly found throughout the Central Plateau of Brazil. This material can be geologically classified as weathered latosol of the tertiary and quaternary age. The latosol has been extensively subjected to a laterization and leaching process during the rainy seasons causing its high porosity. Throughout the district, the thickness of the latosol varies from few centimeters to more than 40 m. The clay mineral kaolinite, and oxides and hydroxides of iron and aluminum predominate in this reddish tropical soil. The variability of the properties of the Brazilian clay depends on several local factors such as topography, the vegetal cover or the parent rock (Cunha *et al.*, 1999).

Due to the leaching process and weathering, the tropical porous clay shows low unit weight and high void ratio. This properties result in the tendency of the soil to fail not only under shear loading but also by volumetric collapse. Such a presumption is confirmed by the extreme values of the coefficient of collapse which can reach up to around +12%. At the UnB experimental site, the latosol overlays saprolitic/residual soil with a significant anisotropic mechanical property and a high (SPT) penetration resistance. This underlying soil originates from a weathered, folded and foliate slate, a typical parent rock of the region. The material in the surficial layer is locally known as the Brasília "porous" clay, being geotechnically classified as sandy clay with traces of silt. All material data presented in this article refer to the geotechnical experimental research Foundation and *In situ* Investigation site of the Univ. of Brasília. The location of UnB campus within Brasilia city together with the location of the UnB experimental site is shown in Fig. 1. General soil characteristics and parameters of the soil at UnB experimental site obtained in previous research (Cunha *et al.*, 1999) are listed in Table 1.

The provided geotechnical parameters were obtained in a comprehensive laboratory and in-situ testing project carried out as a part of the postgraduate research program at UnB. Conventional classification was performed together with more sophisticated laboratory tests such as double oedometer and collapse tests, triaxial K_0 and triaxial CK_0D tests, permeability tests and direct shear tests with samples

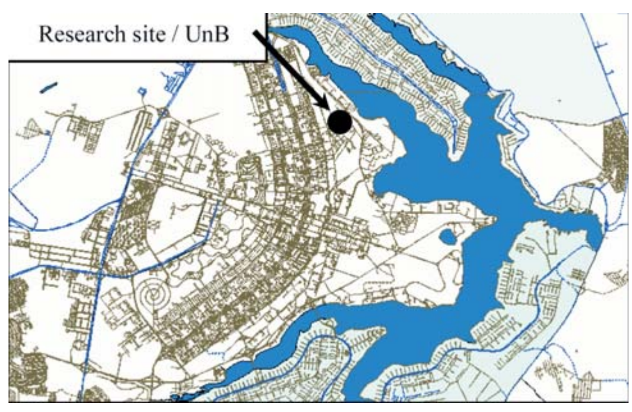

Figure 1 - Location of the University of Brasilia and UnB experimental geotechnical site.

Table 1 - General geotechnical properties of porous clay found at UnB experimental site (Cunha *et al.*, 1999).

Parameter	Unit	Range of values
Sand percentage	%	12-27
Silt percentage	%	8-36
Clay percentage	%	37-80
Dry unit weight	kN/m^3	10-17
Natural unit weight (γ_n)	kN/m^3	17-19
Moisture content	%	20-34
Degree of saturation	%	50-86
Void ratio	-	1.0-2.0
Liquid limit	%	25-78
Plastic limit	%	20-34
Plasticity index	%	5-44
Drained cohesion (c)	kPa	10-34
Drained friction angle (φ)	°	26-34
Young's modulus (E)	MPa	1-8
Coefficient of collapse	%	0-12
Coefficient of earth pressure	-	0.44-0.54
Coefficient of permeability	m/s	10^{-8}-10^{-5}
Poisson's ratio (estimation)	-	0.2-0.35

Table 2 - Geotechnical profile deduced from laboratory and *in situ* tests (Mota, 2003).

Layer	Depth (m)	γ_n (kN/m^3)	E (kPa)	ν (-)	c (kPa)	φ (°)
I	0-3	14.0	900.0	0.2	10.0	27.0
II	3-6	15.0	2200.0	0.2	10.0	27.0
III	6-9	16.0	7300.0	0.2	25.0	27.0
IV	9-12	17.5	10000.0	0.2	40.0	27.0
V	12-15	19.0	10000.0	0.2	40.0	27.0

ν = Poisson's ratio.

Table 3 - Geotechnical profile obtained via backward analysis performed in FINE Geo4 (Anjos 2006).

Layer	Depth (m)	γ_n (kN/m^3)	E (MPa)	ν (-)	c (kPa)	φ (°)
I	0-2	13.5	23	0.29	4	36.6
II	2-6	14.4	20	0.33	10	29.8
III	6-8	15.0	22	0.32	9	31.4
IV	8-9	18.0	23	0.31	7	33.1
V	9-12	17.8	24	0.31	7	33.2
VI	12-15	18.5	35	0.28	3	37.1

under distinct orientations. The geotechnical profile of the UnB experimental site has been examined by a number of researchers and recently described by Mota (2003) and Anjos (2006) to name a few. Authors characterize the profile of UnB experimental site and provide material parameters for the Mohr-Coulomb model.

The design parameters were obtained in different ways. Mota (2003) used a combination of parameters obtained in laboratory (triaxial and direct shear tests) and *in situ* tests (dilatometer and cone penetration tests). See Table 2 with the UnB experimental site profile details consisting of five layers up to the depth of 15 m where the stiff bedrock is found. Since the utilization of strain sensor along the pile reinforcement provided the distribution of shear stress acting on the pile shaft it was also possible by this author to identify distinct soil layers and to determine their approximate material properties. Parameters in the geotechnical profile published by Anjos (2006) were obtained via back calculation analysis with the results from an isolated bored pile field loaded in the UnB experimental site. This backward analysis was performed with the Geo4 foundation software (Fine, 2007), which is based on a semi analytic method as described in Anjos *et al.* (2006). The resulting geotechnical profile is shown in Table 3, based on a layering sequence defined with local experience plus the results of cone penetration tests in this same site.

Three field load tests of deep foundations constructed at the UnB experimental site are analyzed in this study. The single pile test, the group of two piles and the group of three piles are labeled as EHC1, EHC2 and EHC3, respectively in accordance with Anjos (2006) nomenclature. The Continuous Flight Auger technology was adopted to construct the piles and no injection pressure was used during the construction phase, given very soft characteristics of the surficial clay of the experimental site. Hence, during the last construction phase the auger was gradually removed with simultaneous casting of concrete under only atmospheric pressure. Although this technical simplification may cause decrease in the final bearing capacity of the entire foundation system, it allows for a straightforward numerical analysis which is also applicable to traditional bored piles. The arrangement of piles within the UnB experimental site is shown in Fig. 2 together with other tested piles and *in situ* tests carried out there by this as well as previous studies. Figure 3 shows the CFA drilling machine used to bore and cast the piles.

All piles used in the tested foundations were built with the same dimensions. The nominal diameter was 0.3 m and the nominal length 8.0 m. In the case of the pile groups EHC2 and EHC3 the axial distance between the piles was 0.9 m. The top three meters of all piles were reinforced with four steel bars with 16 mm diameter and 6.3 mm stirrups with distance of 0.15 m. A concrete block without physical contact with the underlying soil was constructed on the top of the EHC1 single pile as well as on top of EHC2 and EHC3 pile groups. See photos in Figs. 4 and 5.

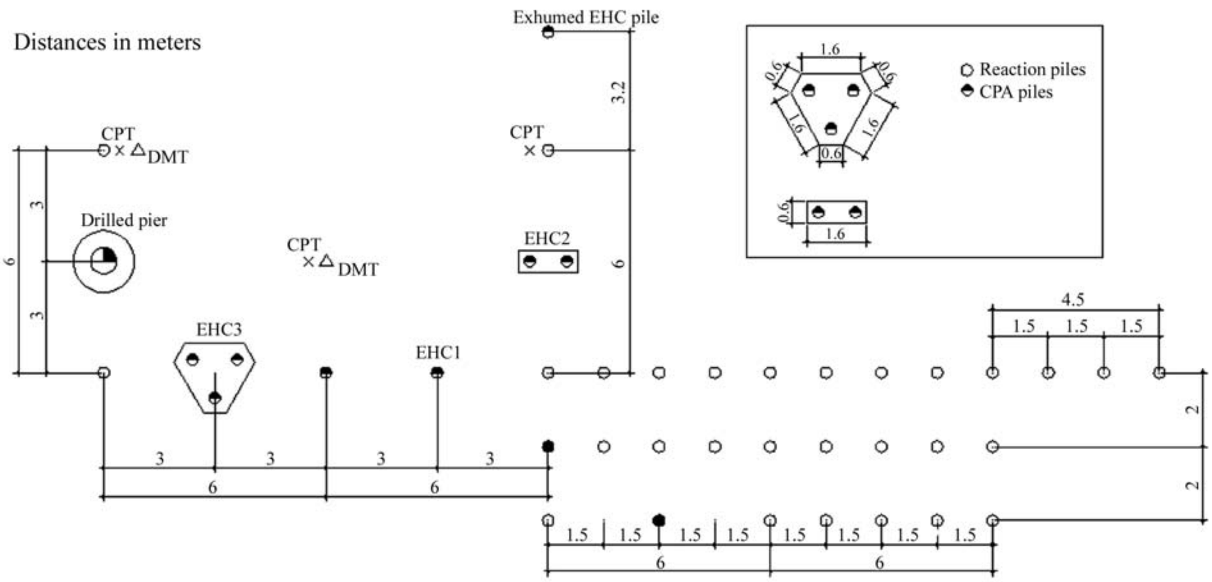

Figure 2 - Plan of UnB experimental geotechnical site (after Anjos, 2006).

Figure 3 - CFA drilling machine.

Apart from the six piles used for load tests, a testing pile of 2.8 m length was also constructed. Later, this pile was exhumed to give a general idea about how the real pile geometry differs from the nominal values. The real diameter of the testing pile varied from minimal value 0.280 m to maximum 0.360 m with the mean of 0.303 m. Although there was evident increase of diameter closer to the base, the mean diameter did not differ significantly from the nominal value of 0.3 m. The location of the exhumed pile is presented in Fig. 2 and the real geometry of it is visible in Fig. 6. All foundations were constructed in the rainy season of Brasília city (October to March) while the load tests were carried out in April, which follows dry season months.

3. Numerical Finite Element Analyses

Since a 3D effect is essential to understand the pile-soil interaction, the software Plaxis 3D Foundation was used in the numerical analysis. This software is based on displacement based finite element method (Plaxis 2007) and allows for using standard geotechnical material models based on the theory of plasticity. The outcome of the analysis is graphically represented by the distribution of displacements, strains and stresses or the load-settlement curve.

3.1. Geometrical model

The geometrical model described bellow adopted dimensions of each tested foundation. In the horizontal layout, the analyzed area was a 12 m x 12 m square with the foundation system in the middle. Equally to the real experimental foundations, the top raft in the geometrical model was separated from the surrounding lateral surface by a narrow gap as plotted in Fig. 7.

In Plaxis 3D Foundation the vertical layout of the construction is determined by horizontal planes also called working planes. Two main working planes at ±0 m and -15 m levels form the surface and bedrock levels, creating the vertical boundaries of the analyzed area. Another vertical plane bounds the bottom of the floating piles at -8.6 m. Two additional working planes were added to create the bottom surface of the surficial raft (-0.4 m) and the bottom of the excavation under the surficial raft (-0.6 m). This pair of working planes allowed to create free space under the raft as plotted in Fig. 8 (a), exactly as field loaded by Anjos (2006).

Once the geometry of the traditional pile group was created, it could be easily changed in order to simulate the response of a piled raft system which could be constructed in contact with the subsoil. This was achieved by assigning the soil to the region bellow the top raft of the pile groups. The difference in the geometry of the piled raft and the pile group can be respectively noticed by comparing Figs. 8b and a.

3.2. Material model

The majority of the computations were carried out using the Mohr-Coulomb failure criterion as a material model for the soil. This standard model for soil materials exhibits linear behavior followed by a perfectly plastic response after the plasticity condition is reached. No hardening or softening of the material was assumed. Besides the Mohr-Coulomb there is also the so called Hardening Soil model which allows increasing the failure stress according to plas-

Figure 4 - Piles of EHC1, EHC2 and EHC3 foundations before construction of the top block.

Figure 5 - Top blocks of EHC2 and EHC3 foundations.

Figure 6 - Exhumed testing CFA pile.

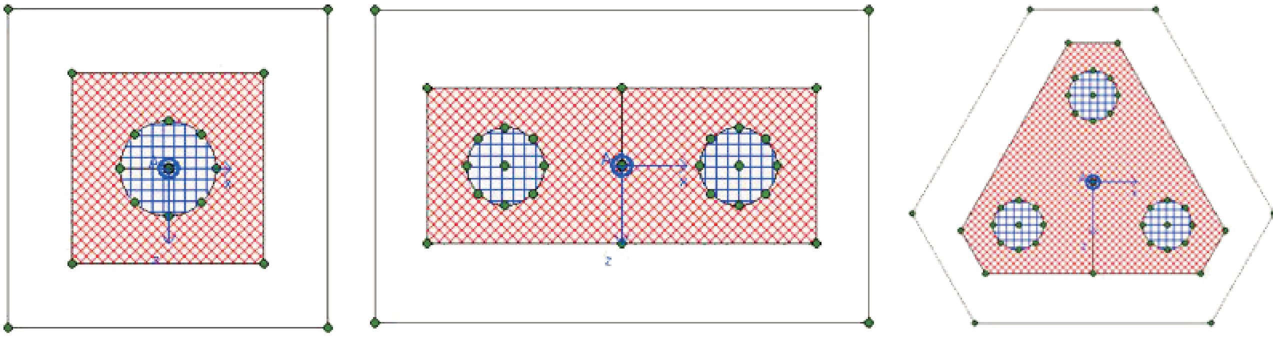

Figure 7 - Horizontal layout of EHC1, EHC2 and EHC3 models.

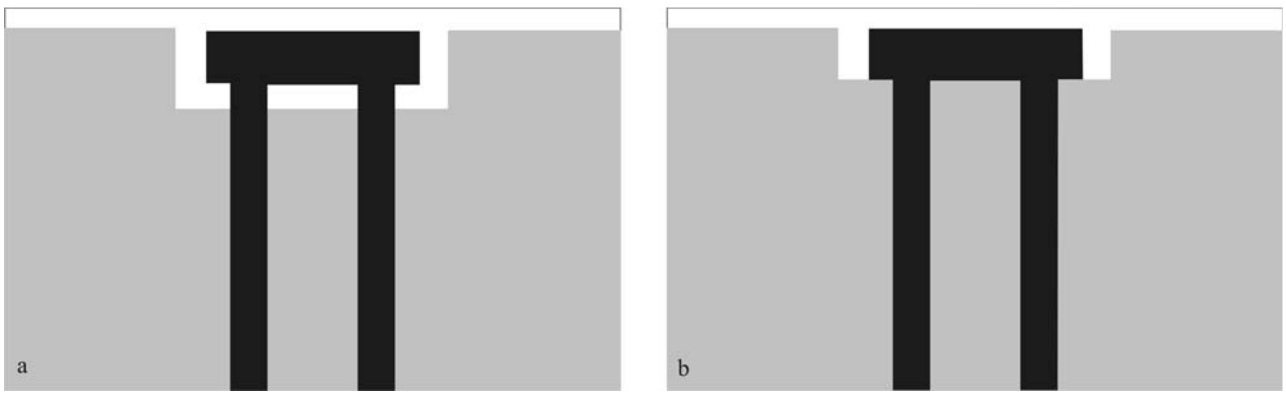

Figure 8 - Vertical layout of the geometrical model - detail of the EHC2 system.

tic shear stress. Despite the fact that such a property gives better precision to many types of soils it does not surpass the standard Mohr-Coulomb model when modeling this particular type of porous clay. The tendency of the material to collapse, *i.e.* to exhibit sudden large irreversible volumetric and shear strains, would need a material model that allows not only for strain hardening but also for strain softening. Owning to the absence of underground water in the geotechnical profile at the testing period, no pore pressure was assumed during the analysis. The soil environment was modeled by five horizontal layers when employing laboratory and *in situ* parameters (Table 2) or by six layers when using the back analyzed parameters published by Anjos (2006) (Table 3).

The concrete reinforced piles and the top rafts were modeled as homogenous nonporous linear elastic material with Young's modulus $E = 20$ GPa, Poisson's ratio $\nu = 0.2$ and a unit weight $\gamma = 24$ kN/m^3. Although in the real construction the top part of the pile, together with the top raft, was reinforced with projecting bars and stirrups, no additional reinforcement was incorporated in the numerical model.

3.3. Finite element mesh

An automatic mesh generator built in Plaxis 3D Foundation code was used to create the three-dimensional finite element mesh. The 3D mesh was generated in two steps. In the first step, the two dimensional mesh consisting of six node triangular elements was automatically created. The triangular mesh was then refined in the area surrounding the pile in order to eliminate long narrow triangles which the generator produced in this region. See the refined 2D mesh in Fig. 9. The global mesh refinement was not used in the analysis since it does not influence the resulting load-settlement curve but, as proved in a benchmark test, consumes more computation time. In the second step, this 2D triangular mesh was extended into 3D mesh compounded of 15-node wedge elements with two horizontal triangular faces and three vertical rectangular faces. In this type of three dimensional elements, three nodes are found along each edge allowing for quadratic approximation of displacement field within the volume of the element.

A relative vertical displacement along the interface between the pile shaft and the surrounding soil was allowed by means of interface elements. Hence, 16-node interface elements of zero thickness were inserted along the contact between wedge elements representing the solid construction and wedge elements representing the surrounding soil. The mechanical properties of these elements were derived from the material parameters of the neighboring soil. Thus, in the solved problem, the interface elements also followed Mohr-Coulomb failure criterion deriving the ultimate shear

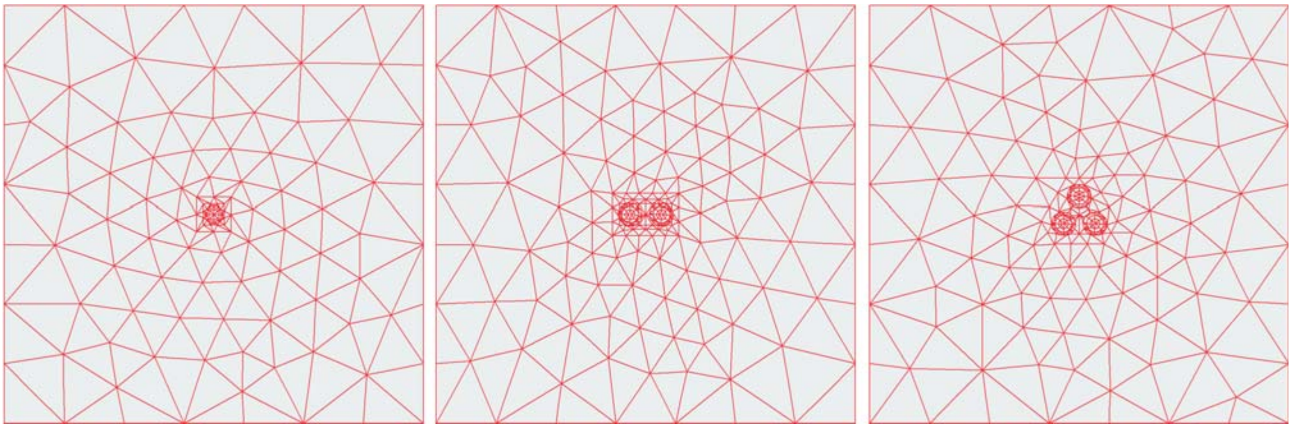

Figure 9 - 2D triangular finite element mesh extended to 3D wedge elements.

stress from the actual normal stress acting perpendicularly to the pile shaft.

3.4. Computation stages

The computation stages were defined in order to follow the phases of construction of the foundation system and the load test itself. In each computation stage the system was loaded with self weight and, if present, external forces. Subsequently, the incremental solver built in Plaxis was used to compute the changes in displacements and stresses. The problem of the field load test was modeled in the following four stages:

• Initial stage - The initial stress state of the soil before construction is created in the initial stage. The initial stage is often referred to as K_0-procedure. The displacements are set to zero after this initial phase;

• Construction stage - During the second stage a small excavation on the surface is created and the entire foundation system (pile with the concrete raft on the top) is constructed;

• Loading stage - The third stage is the loading stage. The geometry of the model is inherited from the second stage and the vertical load is applied. Data of the loading branch of the load-settlement curve are obtained in this stage;

• Unloading stage - The last fourth stage refers to the unloading. The geometry and materials are the same as in the loading stage, only the vertical load is removed. The unloading branch is computed in this last phase.

Only the original soil at natural water content is found in the geotechnical profile during the initial stages. Conventionally, the vertical stress along the depth is in the unsaturated condition, computed by using the natural specific bulk weight of the soil.

A simple relationship in the form

$$\sigma'_v = \gamma_n h \tag{1}$$

was used to generate the initial vertical stress in the soil. The symbols σ'_v, h and γ_n respectively denote effective vertical stress, depth and natural unit weight of the soil. The effective horizontal stress σ'_h then follows from:

$$\sigma'_h = K_0 \sigma'_v \tag{2}$$

where K_0 is the coefficient of earth pressure at rest computed using constant value of Poisson's ratio:

$$K_0 = \frac{\nu}{1-\nu} \tag{3}$$

For the present values of ν, the coefficient of earth pressure varied in the range 0.25-0.5, which reasonably agrees with *in situ* test measurements at the UnB experimental site (Table 1).

In the second computational stage the soil material is replaced with elastic material in order to model the concrete piles and the concrete raft on the top. The soil surrounding the raft is also excavated. The forces acting in the system refer only to loading or unloading caused by changes in the unit weight of the newly introduced materials, or by the excavation. No additional external load is added to the system in this stage.

The main computations which provide the load-settlement curves are carried out in the third stage. Here the vertical load is applied on top of the raft, and the loading branch of the load-settlement curve of a previously chosen monitoring point is computed. In all models the monitoring point was placed in the middle and on the top of the raft.

In Plaxis 3D Foundation software, the entire load defined at the beginning of the computation stage is automatically divided into load increments. The size of the increments respects the degree of nonlinear behavior. Generally spoken, the increments size decreases when the plastic zone in the pile neighborhood propagates but it remains quite large during elastic response. In the presented computations the entire applied load varied with each solved problem, as shown in Table 4. These particular values of the vertical load allowed for reaching full mobilization of the piles until the onset of the foundation failure. On the other hand, such values of the load are small enough to reach equilibrium at the end of each loading stage, and do not cause nu-

Table 4 - Values of vertical loading used in the numerical analyses (kN).

Loading case	EHC1	EHC2	EHC3
Laboratory and *in situ* parameters	300	600	900
Back analyzed profile (Geo4)	300	600	900
Pile group	400	800	1200
Piled raft	600	1200	1800
Analyses before failure	200	400	600

merical instability during computations. The differences in the load levels presented in Table 4 also reflect the different bearing capacity and settlement of the distinct foundations systems and analyzed geotechnical profiles, in order to reasonably draw both the elastic pre-failure and elasto-plastic post-failure branches of the particular load settlement curves.

The last stage (fourth) corresponds to unloading. The load added in the previous computational stage (third) is removed and the foundation heaves. Similarly as in the previous stage, the load is removed in several steps allowing the unloading branch of the load settlement curve to be plotted. The same monitoring point on the top of the foundation systems was used here. Unlike the loading stage, no significant plastic deformation usually occurs during unloading, and the automatically determined load steps are larger.

As mentioned before, the behavior within particular loading/unloading stages is examined by using the so called monitoring points. If a monitoring point is defined prior to the computational stage, the load scaling factor, and corresponding displacement of that point for each loading step is stored and can be displayed within "Plaxis Curves" module, or simply exported as a list of data to form the load-settlement curve.

4. Experimental Data and Discussion of the Numerical Results

4.1. Experimental results

The experimental results from the pile load tests EHC1, 2 and 3 are presented in Fig. 10. From this one, it is evident that the load-settlement curves present an initial stiff and relatively linear response, beyond which all curves fail abruptly due to uncontrolled measured settlements. The linear branch reaches the failure point which is at the level of 360 kN of vertical load for EHC1, 700 kN for EHC2 and 800 kN for EHC3. The vertical settlement measured prior to the failure reached 4.8 mm, 3.6 mm and 3.5 mm for EHC1, EHC2 and EHC3 respectively. After increasing the vertical load to 390 kN (EHC1), 850 kN (EHC2) and 900 kN (EHC3), hence by approximately 8%, the foundations showed large irreversible settlements reaching respective values of 35.9 mm, 30.4 mm and 37.8 mm. The unloading branch of the load-settlement curve was mea-

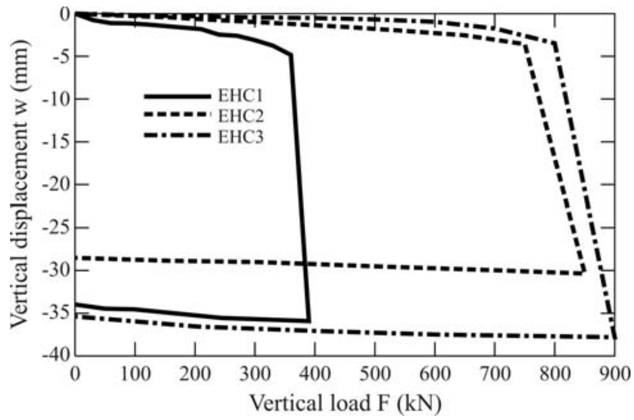

Figure 10 - Experimental results from the pile load tests carried out on EHC1, EHC2 and EHC3 foundations (after Anjos, 2006).

sured from these final points, as shown in this same figure. The unloading branches of the load-settlement curves are relatively parallel to the initial (elastic) branches of the loading curves, preserving a close to irreversible settlement constant.

It is obvious from the results that the sudden failure, accompanied by a rapid increase of the settlement of the system, was caused by a collapse of the soil around the foundation elements, given the well known collapsibility and meta-stable structure of the porous clay of Brasília. Nevertheless, even with these particular features, the experimental curves were used to be compared to numerical simulations of all pile group systems with Plaxis 3D Foundation software. After all, the analyses must refer to a realistic foundation behavior in this particular tropical clay, by using loaded systems as close as possible to normal field conditions.

4.2. Numerical results

4.2.1. Results obtained using laboratory and in situ parameters as input

The first numerical simulation was carried out with a set of parameters obtained via laboratory and *in situ* tests, as interpreted by Mota (2003) and stored in Table 2. The resulting load-settlement curves computed in loading and unloading stages are displayed in Fig. 11. It is evident from this figure that the computed failure level, *i.e.* the load level by which the settlement rate starts to increase and the loading branch starts to bend down, reaches only approximately 60% of the measured failure load in the case of EHC1 and EHC2 and about 80% for EHC3. It should also be remarked that the differences in the initial numerically derived stiffnesses of the foundations systems are even more visible. It was also observed that the numerical displacements, at failure level, varied from 20 mm (EHC1) to 30 mm (EHC3), while the measured displacements before failure point were in the range of about 4 mm. Besides, regardless of the inaccuracies when using this set of parameters

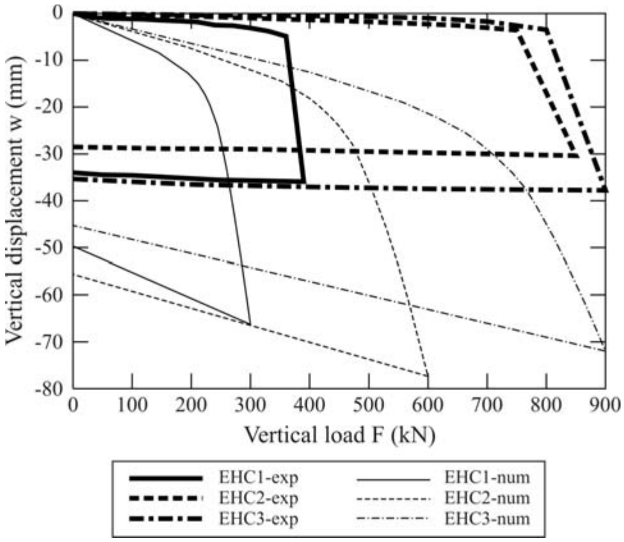

Figure 11 - Measured load-settlement curves and numerical results obtained for material parameters from laboratory and *in situ* tests (data from Mota, 2003).

(Table 2), the adopted model generates irreversible settlements which remain approximately constant during the unloading stage.

If follows from the present series of analyses that, to properly simulate pile load experiments on isolated or traditional pile groups founded on this particular soil, it does not seem to be possible to directly use geotechnical parameters deducted from conventional laboratory or *in situ* tests, as those presented in Table 2 for the UnB experimental site. Perhaps, either the modeling technique is not appropriate to capture the detailed nuances of the real site phenomenon, or the used parameters do not properly represent system interactions (foundations/soil/external factors) that take place during pile construction and loading, or both cases hinder the analyses simultaneously.

As examples of neglected aspects (in the present numerical analyses) that may have influenced the group behavior in such soil, one can mention the suction variations of the subsoil, the complex interactions between foundation elements and the surrounding soil, the minor changes or variability of construction techniques from pile to pile, the distinct technological influences on the surrounding soil by pile excavation and casting at distinct dates, the different and unknown stress paths along soil elements surrounding the foundation systems, the complex stress strain curves of heterogeneous soil elements around the piles, and so on.

Finally, underestimated foundation stiffness as observed for computations using the combination of laboratory and *in situ* material parameters can also be caused by the differences in the secant Young modulus E_{50} of the measurements and the elastic Young modulus E^{el} used in the Mohr-Coulomb material model. The choice of the unloading-reloading Young modulus E_{ur} would be perhaps more appropriate here, but such experimental value has not been determined with the available triaxial experiments of this porous clay.

4.2.2. Results obtained by the use of parameters via FINE Geo4

Given the aforementioned results, another series of analyses were carried out by employing back analyzed results via another numerical (Geo4) technique, as published by Anjos (2006) and summarized in Table 3. It is again noticed that the back analyzed parameters of this author were obtained for an isolated bored pile, rather than a pile group.

The result of this new analysis is presented in Fig. 12 where it is observed that a slightly better and closer agreement between simulated and measured curves is achieved, although still not a perfect match. Nevertheless, similarly as the previous case, with the idealized adopted geotechnical profile of Anjos (2006) it is noticed that the numerical analyses lead again to underestimated bearing capacity values. On the other hand, some improvement can be noticed in the initial part of the loading branch, suggesting a perhaps more realistic macroscopic stiffness of the foundation system than those generated for the previously adopted (idealized) geotechnical profile. However, in spite of the improvement, the stiffness before failure for all simulated systems is still underestimated.

It follows again that simulation of the pile groups founded in this particular subsoil should be better conditioned, as a proper solution could still not be addressed by adopting back calculated parameters from a previous series of analyses. It is indeed questionable if parameters derived from a slightly distinct numerical technique would be useful to simulate a system under the framework of another, more complex, modeling tool. It was expected that both methods would give comparable results when using the

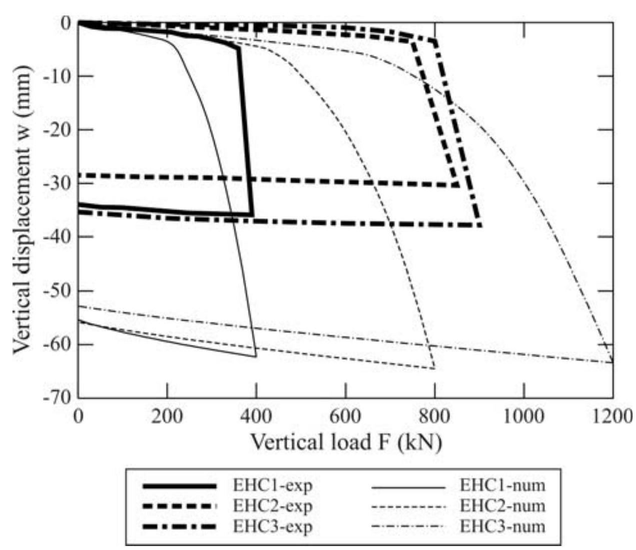

Figure 12 - Measured load-settlement curves and numerical results obtained for material parameters from backward analysis with Geo4 software (data from Anjos, 2006).

same Mohr-Coulomb material model, but this hasn't been exactly the case observed with the results.

4.2.3. Results obtained from backward calculation analysis of EHC1 single pile

The previous items demonstrated that the numerical results failed to meet the experimentally obtained load-settlement curves with sufficient engineering accuracy, either by employing parameters obtained on a combination of laboratory and *in situ* tests or by employing values from a backward calculation of a single pile with another numerical tool. Hence, in order to reach an even better agreement, a backward analysis of the EHC1 single pile was performed in Plaxis 3D Foundation, the same software adopted to simulate all other pile group systems.

In this analysis a homogeneous subsoil profile was assumed, while the predetermined geometry of a single pile EHC1 was kept unchanged. The parameters of the unique material which forms the entire soil horizon were varied using a simple trial and error method. Only a selected set of material parameters was changed during the backward analysis. In particular, it was decided to vary the elastic Young modulus, the friction angle and the cohesion while the unit weight, the Poisson's ratio and the dilation angle were kept constant. The elastic parameters of the concrete forming both pile and raft elements were also excluded from the backward analysis, and kept constant. Moreover, it was realized during the trial and error computations that the presence of the interface elements is absolutely essential for reaching the sudden collapse of the foundation system, as experimentally observed in the field.

The resulting values are presented in Table 5, while the best fitted load-settlement curve is displayed in Fig. 13. As expected, the best agreement with the measured data was provided by the computations when the back analyzed parameters were derived from the EHC1 single pile test. Both the pre-failure stiffness and the bearing capacity simulated for the single EHC1 pile exhibited a close approximation to the experimental values, validating the use of parameters presented in Table 5 for the other foundation systems. Figure 13, in addition, shows how the load-settlement curve changes when a thin layer of soil is removed from under the pile tip. In spite of that, the system with no tip resistance fails at a load level lower by only 8% than the critical failure point. Up to this stage, both back analyzed curves, with and without tip resistance, are quite identical, which indicates the predominant contribution of the shaft friction to the total capacity of a single pile in such a geotechnical profile. Indeed, such conclusion has already been experimentally shown before by Mota's (2003) instrumented pile load test results at this same site.

Table 5 - Results of backward analysis performed on EHC1 single pile with Plaxis 3D.

γ_n (kN/m^3)	E (MPa)	ν (-)	c (kPa)	φ (°)	ψ (°)
18.0	35.0	0.3	32.0	27.0	0
Assumed	B.a.	Assumed	B.a.	B.a.	Assumed

Ψ = dilation angle. B.a. = Back analyzed.

4.3. Numerical results of the other systems

The resulting load-settlement curves of all EHC foundations computed with the back analyzed material parameters from Table 5 are plotted in Fig. 14. The curves were also derived by considering interface elements around the piles and soil underneath their tips. It is clearly noticeable a much better agreement between measured and computed curves for the single (EHC1) as well as the group of two piles (EHC2). The average difference in the failure load for these previous cases is less than 5%, while the computed displacements before failure are approximately 25% higher than the measured values. Besides, the adopted model was

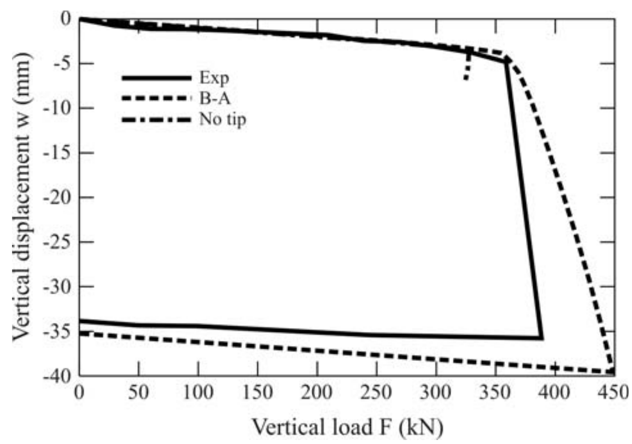

Figure 13 - Results of backward analysis on single CFA pile EHC1 performed in Plaxis 3D.

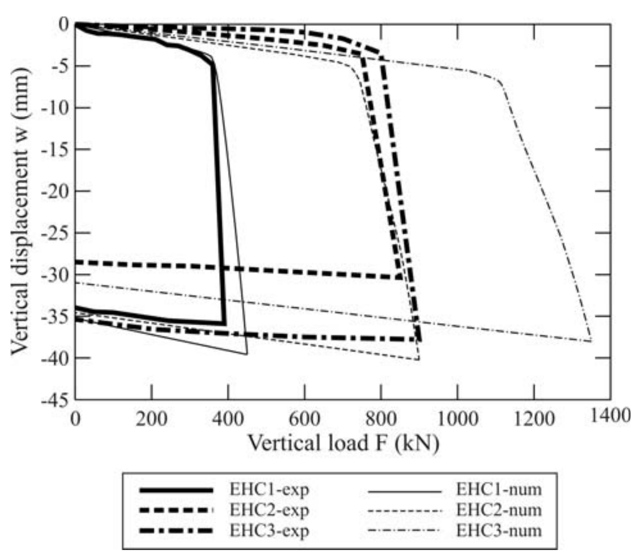

Figure 14 - Experimental and numerical load-settlement curves, using back analyzed parameters in Plaxis 3D.

also able to simulate the sudden failure with rapidly increasing irreversible settlements occurring with small load increments.

Mutual comparison of computed results obtained numerically for EHC1, EHC2 and EHC3 pile group systems also lead to the observation that the group effect, at least for the pile spacing adopted in the present research, slightly reduces the overall foundation stiffness, and may have somehow a small influence on the bearing capacity.

Nevertheless, a large difference was found between numerical and experimental results for the third, EHC3 pile group. As one may infer from Fig. 14, this particular group did not follow a predicted pattern as would be expected solely by the EHC1 and EHC2 results. For instance, Anjos (2006) results, graphically expressed in Fig. 10, indicate that the vertical loading stage just before the sudden failure was 360 kN (at 4.85 mm top displacement), 750 kN (at 3.57 mm) and 800 kN (at 3.47 mm) respectively for EHC1, 2 and 3 systems. Adopting as reference the load for the isolated pile EHC1, these numbers represent an increase of approximately 2.1 and 2.2 times respectively for EHC2 and 3 systems. Slightly differences can be found, given distinct displacement levels upon which the load values are taken, but, nevertheless, one can argue if the load obtained for the EHC3 system was indeed in the correct range it would be normally expected. According to Anjos (2006), for this particular system it was possible to estimate an efficiency factor of around 0.8 by standard empirical relationships used for floating piles in clay. This number seems to be close to the experimental measured efficiency of around 0.74, but, again, one may argue about its correctness. Actually, if the EHC3 group was submitted to the influence of pile to pile interference, which would justify the decrease in load efficiency, the same behavior would also be expected in the EHC2 system. Based on the limited available data, the authors' opinion is that only an efficiency reduction can not explain the discrepancy between EHC3 and EHC1 results. Hence, having said that, it will be assumed from this point on that the experimental results from the EHC3 system may be compromised, therefore not serving to conclude on the appropriateness of the numerical simulations of the EHC3 load-settlement curve.

4.4. Pile group *vs.* piled raft

Having the material model calibrated it was decided to examine how could the raft, in full contact with the surficial soil, positively contribute to the mechanical response of the entire foundation system for each of the studied cases. Off course, given the fact that no experimental site tests were carried out with the top raft in active contact with the soil, this subsection will entirely rely on numerical simulations of the systems. It will be assumed that the numerical load-settlement curves presented in Fig. 14, for all systems, are appropriate in engineering terms and can reasonably serve as benchmark for comparison purposes with equivalent curves from numerically derived "piled raft" simulations.

Thus, by activating the soil layer beneath the top raft, as exemplified in Fig. 8 (b), it was possible to obtain piled raft related load-settlement curves, and to compare them directly with the numerical ones from the pile groups of Fig. 14. Such comparison is depicted in Fig. 15 where it is noticed that the ultimate bearing capacity was increased by 17% for EHC1, by 12% for EHC2 and by 15% in the case of EHC3, when considering the piled raft configuration. The settlement during the initial (elastic) phase, for the all piled raft cases, shows an average slightly smaller value, decreased by approximately 7% to that equivalent of the pile group cases.

The slightly increased stiffness in the initial phase of the load-settlement curve of the piled raft systems, in comparison to the standard pile groups, is visible in Fig. 16. This figure presents a zoom of the initial part of the curves depicted in Fig. 15.

Finally, one of the most visible contributions of the foundation behavior as a piled raft is the softening of the abrupt plastic failure which has been exhibited in the simulated curves of the pile group systems. This is valid for all studied cases. Indeed, the failure of the piled raft foundations is markedly more gradual as the load is constantly shared by two elements of distinct behavior: raft and pile, plus surrounding soil. For instance, the raft has an increasing load capacity with settlement, and the pile has a limited value of shaft load capacity, which is mobilized at a low displacement range. This is combined to an increasing load capacity at tip, which generally also increases with higher displacement.

4.5. Load shared by pile shaft, pile tip and raft

The distribution of the internal forces acting in the pile element provides information about the percentage of

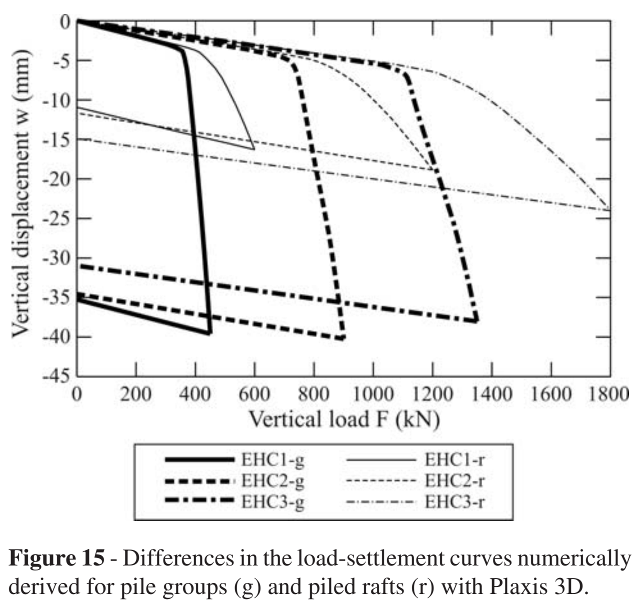

Figure 15 - Differences in the load-settlement curves numerically derived for pile groups (g) and piled rafts (r) with Plaxis 3D.

load which is attributed to the pile tip, pile shaft and possibly the raft. The distribution of the internal forces was derived from the pattern of vertical stresses displayed in the Plaxis Output module. Such a result, for illustration purposes only, is presented in Fig. 17, and has been used to determine the percentage of structural load mobilized along each of the piles of the analyzed systems.

Two sets of computations were performed in this study. The first set simulated the state of load distribution just prior to the failure, while the second examined the load distribution in already failed systems. Hence, the data as post failure refer to the state at the end of third computation stage, *i.e.* after the whole vertical loading has been applied. It corresponds to the end point of the load-settlement curves in Fig. 15. On the other hand, the data denoted as pre-failure refer to analyses in which the system was loaded with 200 kN, 400 kN and 600 kN for EHC1, EHC2 and EHC3 respectively. It can be seen in Fig. 15 that such values of the external load do not cause significant settlement and can be referred to as pre-failure state. The distribution of the vertical internal load for the state before failure is shown in Figs. 18 (a) and (b). Figure 18a shows the results for pile groups EHC1, EHC2 and EHC3 while Fig. 18 (b) provides the load distribution for the piled raft systems. The values expressed for the systems EHC2 and 3 represent the average load computed for all the piles, in all cases.

The results of this figure indicate that, for the pile group, 8% of the load is carried by the pile tip while the remaining 92% of the total load is carried by the pile shaft, as already expected, given aforementioned observations of the large contribution of the shaft friction to the total pile capacity. Indeed, the piles within the studied pile groups behaved more as floating elements than end bearing ones. The same trend was noticed for the case of the piled raft systems, with average results of 7% of the total load carried by the pile tip and 83% for the pile shaft. However, in the piled raft cases, an average value of 10% of the total applied load was absorbed by the raft.

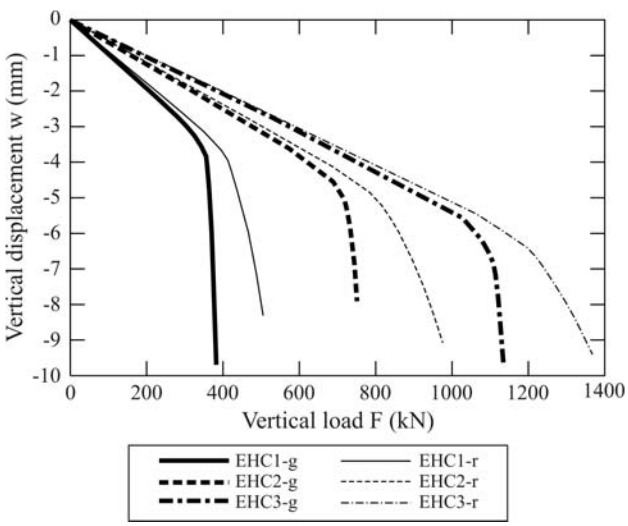

Figure 16 - Differences in the load-settlement curves numerically derived for pile groups (g) and piled rafts (r) - Zoom of initial loading stage.

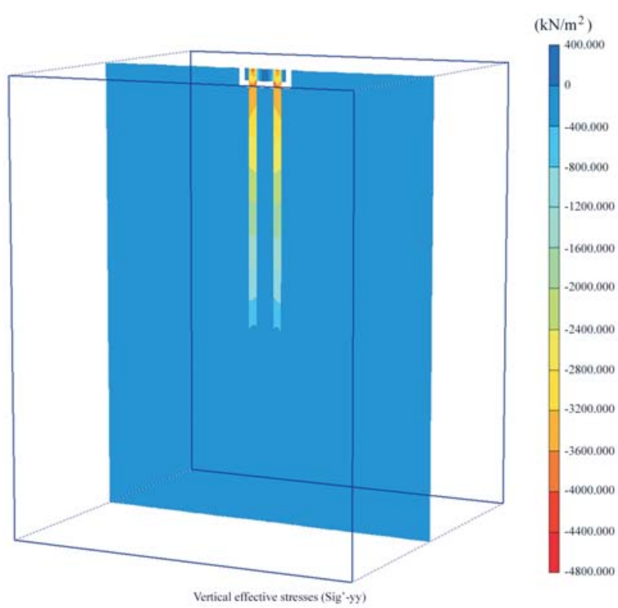

Figure 17 - Illustrative example of the output of the vertical stress distribution in the pile.

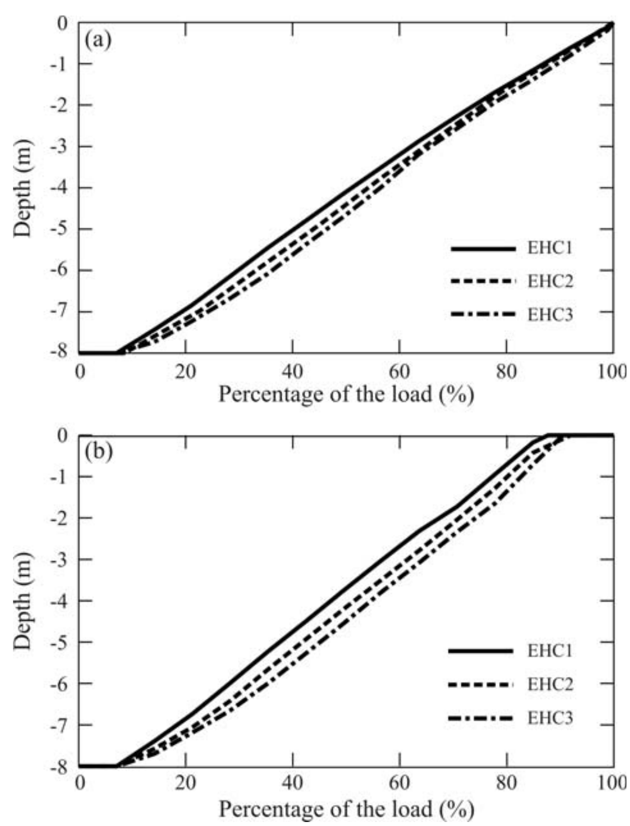

Figure 18 - Internal load distribution along the pile - stress state before failure, (a) pile group, (b) piled raft.

The distribution of the vertical internal load for the state after failure is shown in Figs. 19 (a) and (b), similarly as the previous figures. It is noticed with the results at a post failure event, for both studied systems, that the load distribution pattern did not change considerably as in the previous case. However, the magnitude of the load share has slightly shifted upwards. For instance, for the pile group 21% of the load is carried by the pile tip while the remaining 79% of the total load is carried by the pile shaft, thus giving a better end bearing performance for the piles. In the case of the piled raft systems, an average result of 11% of the total load was carried by the pile tip, 63% by the pile shaft, and 26% of the load was absorbed by the raft.

This subsection allowed the perception of the distinctive behavior of both pile group and piled raft systems when loaded at working and at failure levels. For the particular studied case, and taking on account the weak characteristics of the surficial tropical and porous Brasília clay, both systems have operated with piles with predominantly floating characteristics, where major part of the load was absorbed by lateral friction along the shaft. Nevertheless, some tip load was mobilized, respectively at ranges below 10% of the total applied load, for working levels, and beyond this range on post failure events. A beneficial absorption of the overall load by the raft, in both conditions, was clearly noticed with the piled raft systems. In working conditions, and even for low soil resistances at surface, the raft was able to sustain 10% of the total load. In failure, the piled raft systems had the load predominantly migrated from the pile shaft to the raft, rather than to the pile tip, as previously observed for the pile group systems.

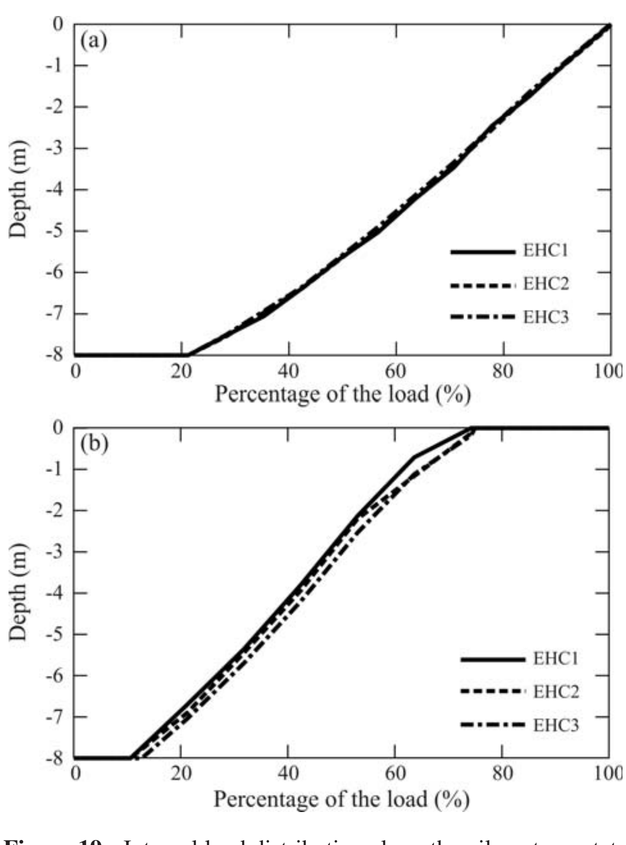

Figure 19 - Internal load distribution along the pile - stress state after failure, (a) pile group, (b) piled raft.

In other words, the raft aids in the overall behavior on pre and post failure events, being indeed an asset when designing the foundation as a piled raft system. However, the positive impact of the raft seemed to be more substantial when settlements higher than approximately 5 mm took place in the piled raft system. This is, off course, related to the weakness of this surficial porous clay layer, and may change to systems founded on rather more competent strata. As observed by the simulations, after this level of displacement the percentage of load carried by the pile tip and the raft increase more substantially with the amount of settlement.

4.6. Distribution of lateral friction resistance mobilized along pile shaft

Similarly as the previous analyses, and taking again on account Plaxis outputs as the one presented in Fig. 17, it was possible to determine the distribution of the lateral friction resistance mobilized on the pile shaft along its entire length. Again, the results for each of the piles from both EHC2 and 3 systems were averaged in order to be presented in the following figures.

Hence, Fig. 20 (a) and (b) respectively present the lateral shear stresses computed for the pile group and the piled raft systems for load levels just prior to failure. For both cases, average values in the range of 20 kPa to about 35 kPa were found, which agree with experimental results from Mota (2003) measured at equivalent load levels on a bored pile with similar dimensions loaded in this same site. Similarly as this experimental case, the numerical simulations have also shown that higher levels of lateral friction are mobilized closer to the pile tip than to the pile cap. Besides, the average results seem to be in the same magnitude for all studied systems, when cross comparing only the pile groups and only the piled rafts.

Nevertheless, when directly evaluating pile groups to piled rafts, one may notice that the presence of the raft has caused an increase of the level of the lateral friction in the vicinity of the raft, *i.e.*, within the pile shaft zone of up to 0.5 m underneath the raft, or ~1.5 diameters (d). This phenomenon becomes much more evident when comparing the results at a post failure event, as presented in Figs. 21 (a) and (b).

Again, these figures refer respectively to pile groups and piled rafts. In this case, it is noticed that the zone of increased lateral friction has extended to 0.8 m (~2.5 d), 2.2 m (~7.3 d) and 1.2 m (~4.3 d) underneath the raft, respectively for the EHC1, EHC2 and EHC3 piled raft systems. Indeed, there seems that the larger is the surficial area of the raft, the deeper will be the zone along the pile length affected by lateral friction increase.

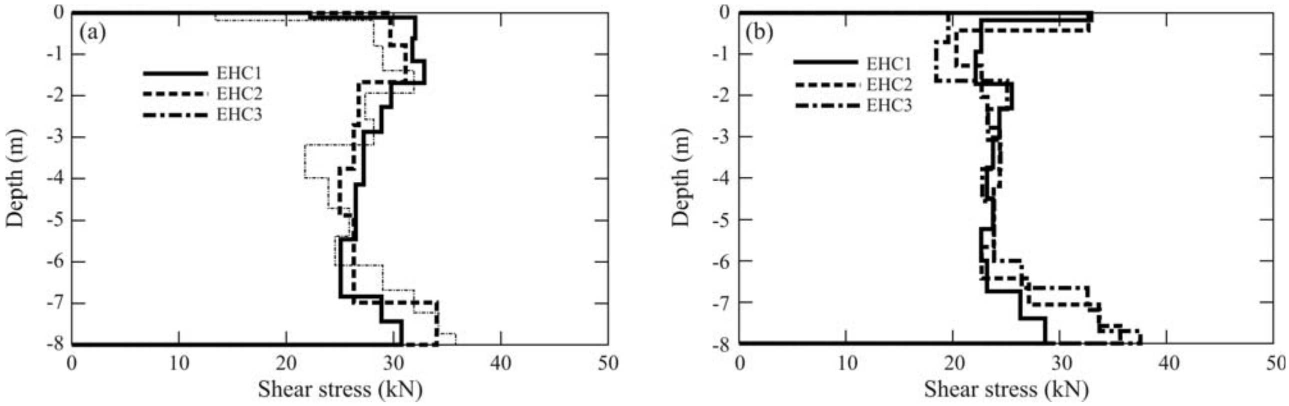

Figure 20 - Lateral friction stress distribution along the shaft - stress state before failure, (a) pile group, (b) piled raft.

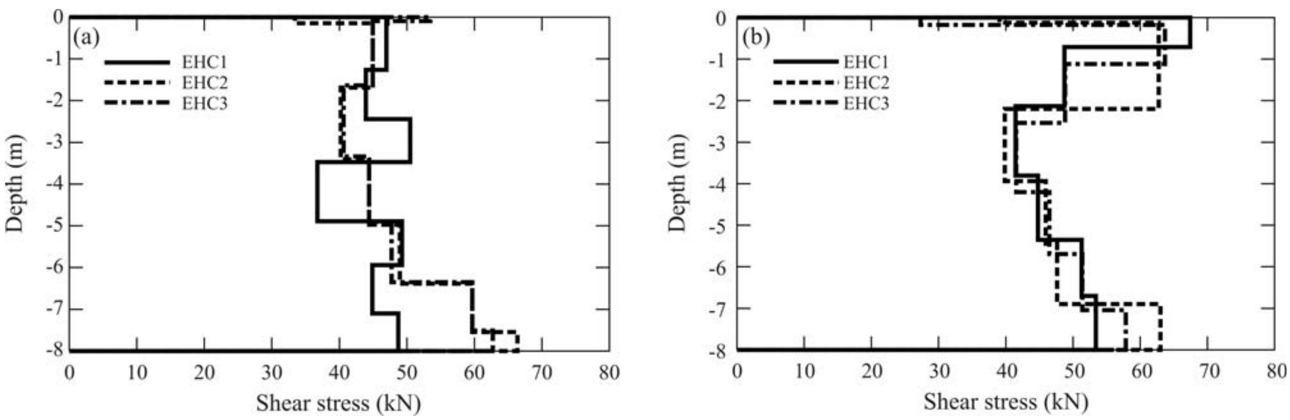

Figure 21 - Lateral friction stress distribution along the shaft - stress state after failure, (a) pile group, (b) piled raft.

The magnitude of increase of the frictional stress has exceeded 20 kN, or more than 50% of the original average (pre failure) values, for some of the simulated cases, which is another good asset of the presence of the surficial raft in piled raft systems. Besides, the level of the lateral friction stresses for the piled rafts, in all compared post failure cases, is higher than the respective level of the pile groups. On the other hand, for situations before failure, both pile groups and piled raft presented mobilized lateral friction resistances of the same range. It is finally noticed, when comparing post and pre failure events for all studied systems, that the mobilized lateral friction resistance is slightly increased when approaching failure.

This subsection enabled an envision of the differential behavior of the systems, at pre and post failure events, when considering (or not) a close contact of the raft with the surficial soil. Even with a surficial weak layer, piled raft systems will behave “better” in cases where the system eventually fails in geotechnical terms. Upon failure, the upper zone of the piles of piled raft systems will have a greater lateral friction mobilization than equivalent values from standard pile groups. On working conditions, on the other hand, both systems will operate similarly, with closer values of mobilized average lateral frictions along pile shaft. Indeed, the simulations have shown slightly higher frictions for the piles of the pile groups, since, as commented in the previous item, more load is absorbed by such piles in comparison to the piles of the piled raft systems (as the load in such systems is also shared with the raft).

On general terms, and considering the mobilized soil resistance shared by the system components (raft, pile tip and shaft, and soil) and the distribution of mobilized lateral friction at distinct load levels, it is concluded that piled raft systems will behave slightly better than standard pile groups in conditions similar to those tested in Brasília.

5. Conclusions

Three-dimensional finite element analyses of a single pile, of various types of pile groups and of piled rafts founded in typical tropical porous clay of the Federal District of Brazil were presented and discussed in this paper. The computations were performed for soil parameters obtained: a) from laboratory and *in situ* tests, b) via backward analysis performed by a semi analytical method implemented in FINE Geo4 software and c) via backward analysis of a single CFA pile performed in Plaxis 3D Foundation. In this comparison the set of laboratory and *in situ* parameters failed to model the field tests with a sufficient accuracy.

Better results were obtained by using the back analyzed values, especially those from Plaxis 3D.

Hence, a reasonable approach for choosing soil parameters as input to the FEM model should consist of two alternatives: In the first alternative, soil parameters could be estimated from geotechnical tests or obtained by a simple backward calculation by means of an adequate semi analytical method, such as the one implemented in FINE Geo4. As a better alternative, a backward analysis of a load test in an isolated pile should be made, using the same software that will perform the analysis of the entire foundation system.

The impact of the group effect of closely constructed piles on the overall bearing capacity proved to be negligible from a practical point of view, based on the numerical simulations. Nevertheless, such effect slightly influences the pre-failure stiffness of the pile group. More notable is the effect of the top raft when it is in contact with the underlying soil layer. The raft had the ability to increase the bearing capacity by approximately 15% for the presented configurations of foundation systems. The pre-failure settlement was also decreased by approximately 7% when compared to traditional pile groups. More significant contribution of the raft appears at post failure stages. Piled rafts reduce post-failure displacements more effectively and soften the abrupt fragile type failure observed in the case of traditional piled groups founded in this collapsible clay.

Lateral friction distribution along the piles, load shares between the elements of the system (raft, piles and soil), bearing capacity, and the overall behavior, are indeed improved by having the raft in close contact with the surficial soil, in other words, by designing the foundation system as a piled raft one. This is valid for load levels at working or, in extreme cases, failure conditions, and denotes the necessity that foundation practitioners have to start considering this type of design approach on daily practice. At least for more substantial foundation works, as bridges, large buildings, etc., where the raft will be, indeed, placed on top of a more competent soil stratum.

The results of this paper prove that, although not straight forward, it is possible to simulate and forecast the behavior of piled raft systems founded in rather complex soils, as the Brasília porous clay, and to compare the results directly to simulations of the same systems when behaving as standard pile groups. It also proves that a feasibility of the analysis can be reached by using readily available parameters from pile load tests or site and laboratory investigations, allied to a standard commercial software. Off course, some common sense and previous experience is desired, but this aspect is valid in all facets of the geotechnical design.

It finally emphasizes that there is a large benefit in designing with the (piled raft) approach advocated herein, especially on soils that are better suited to assist the surficial raft in sustaining part of the superstructure loads, as stiff clays, dense sands, laterized tropical soils or residual crusts. Moreover, more recent studies, as those published by Cunha *et al.* (2007) and Cordeiro *et al.* (2008), also demonstrate the large potential and beneficial aspects that exists in adopting piled raft methodology to simulate, and reinforce, foundation groups with one or more defective piles.

Although more research still needs to be done in this area, this paper also proves that the level of knowledge which exists today is more than enough to allow foundation designers to take sharp decisions in the design of foundations of any complexity, looking forward to economy allied to a better performance of such structures. In other words, as the "Star Trek" series used to mention in their openings, it is now the time "to boldly go where no one has gone before"

Acknowledgments

The financial support provided by the grant n. 1ET410430516 awarded by the Czech Academy of Sciences is gratefully acknowledged. The first author would also like to express his gratitude to all staff of the Geotechnical Post Graduation Program of the University of Brasília, and the Brazilian sponsorship organization CAPES for the support received during the period spent at this workplace. The second author thanks the sponsorship organizations CNPq, CAPES and UnB Foundation Finatec for all related support in this and all other studies carried out by him, and the last author acknowledges the support given by several engineering companies from the Federal District in carrying out all field related tasks, as the companies Embre, Soltec, OAS, WRJ, Sulamericana and Infrasolo Engineering Ltd. It is also thanked by this same author the scholarship kindly provided by CAPES during his D.Sc. studies at UnB.

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Experimental Investigation of Mechanical Damage in Geogrids

Ana Cristina C.F. Sieira, Alberto S.F.J. Sayão

Abstract. This paper presents the results of a comprehensive experimental program for investigating the influence of mechanical damage on the load-displacement behavior of geogrids. Unconfined tension tests, pullout and direct shear tests were carried out on intact and damaged specimens. Natural or artificial damages were produced either by imposing heavy compaction procedures in the laboratory or by simply cutting one or more geogrid elements. It is concluded that natural damage in the geogrid may be more pronounced when aggressive compaction methods are used with coarse grained soils. Fine grained soils did not show a significant strength reduction even when subjected to heavy compaction in the laboratory. Under pullout loading, artificial damage was also noted to be of little significance for fine soil (silty clay). Rupture of the geogrid's transverse elements led to a significant pullout strength reduction. These transverse elements are responsible for anchoring the geogrid within the soil mass. However, under unconfined tensile load, these transverse elements are responsible only for the grid's geometrical configuration and their rupture did not induce a significant strength loss. In direct shear, the position of the geogrid relative to the potential failure surface was shown to be an important factor.
Key words: geogrids, mechanical damage, laboratory shear tests.

1. Introduction

1.1. Mechanical damage regarding design

In geosynthetic reinforced soil masses, allowable tensile stress of the reinforcement is determined by reducing its characteristic strength by a global reduction factor. This characteristic strength is obtained from basic characterization tests, regardless of the geosynthetic environmental and constructional loading conditions.

The global reduction factor is usually decomposed in partial factors for considering the independent reductions of geosynthetic properties due to installation process (mechanical damage), chemical and biological degradation, connections between adjacent mats, and time dependent (creep) deformations.

In reinforced masses, the mechanical damage is the main partial factor influencing the global reduction factor. The geosynthetic material may suffer severe installation damage due to handling, contact with sharp edged soil or rock grains, compaction and traffic surcharge. These factors may induce severe reductions in the mechanical properties of the geosynthetic material.

Paulson (1990) reports on another type of mechanical damage, imposed by the initial loading characteristics, after compaction is completed. Damage occurring during the installation process may alter significantly the geosynthetic mechanical properties. The reduction factor due to mechanical damage is usually determined by the ratio between the strength magnitudes from intact and damaged specimens. Specimens with natural construction damage may be obtained by exhumation immediately after installation and compaction.

The damage intensity depends on the installation process and on the soil type in contact with the geosynthetic material. When used as pavement reinforcement, the geosynthetic may suffer intense installation damages in contact with sharp grained granular material under high compaction efforts. These damages may well be of higher magnitudes than in the case of geosynthetic reinforced fills placed under low compaction over fine grained soft soils.

Determination of reduction factor f_R due to mechanical damage is subject to controversy, due to the large number of variables to be considered. As a consequence, a variety of laboratory procedures have been proposed to simulating damage conditions observed in field installation.

A standard procedure for duplicating severe geosynthetic damage during installation in granular materials has been proposed (ISO 1998). The geosynthetic specimen is to be placed between layers of soil or aggregate. Damage is imposed by intense vibration of 200 cycles of 900 kPa compressive load, under a frequency of 1 Hz. The damaged material is then tested and its mechanical or hydraulic behavior is observed.

Christopher & Holtz (1984) tried to quantify the reduction factor f_R due to mechanical damage, relating the strength loss of the geosynthetic to its surviving capacity and to the severity of ambient conditions during installation. The authors suggest three categories (low, moderate or high) for the surviving capacity of geotextiles, according to its structural and mechanical characteristics.

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Submitted on January 7, 2008; Final Acceptance on September 23, 2008; Discussion open until August 31, 2009.

Rainey & Barsdale (1993) classify the geogrids under two main categories: flexible (woven grids) and stiff (polyethylene or polypropylene non-woven grids). Wrigley (1987) and Troost & Ploeg (1990) proposed classification criteria for the surviving capacity of geogrids based on the short term tensile strength. Allen (1991) and Azambuja (1994) suggest restricting the expression "surviving capacity" for describing only the geosynthetic's resistance against severe damage upon construction efforts and initial loading. When relating to installation conditions, these authors suggest the expression "ambient severity". The classification criterion is summarized in Table 1.

Allen (1991) also proposed a classification for severity of compaction conditions in reinforced soil retaining systems (Table 2). This classification depends on three main factors: compaction equipment, shape and dimensions of soil grains and thickness of the compacted soil layer over the geosynthetic material.

1.2. Mechanical damage regarding experimental tests

Testing programs for evaluating the effect of mechanical damage on geosynthetic behavior have been reported by several authors. In most cases, the strength loss was measured by rating the tensile strengths of intact and naturally damaged specimens. These damaged specimens were exhumed after real construction procedures (Koerner & Koerner, 1990), or after experimental field work (Bush, 1988; Wrigley, 1987; Troost & Ploeg, 1990; Koerner & Koerner, 1990; Allen, 1991; and Azambuja, 1994).

Tensile tests reported by Bush (1988) on stiff HDPE (high density polyethylene) geogrids showed a strength loss of about 4 to 8% under low severity conditions, and from 12 to 17% under moderate severity.

For tests on stiff polyester geogrids, Wrigley (1987) showed a strength loss of 5 to 10% under low severity and of 30 to 40% under high severity condition. On the other hand, Troost & Ploeg, (1990) reported that, when coated with a PVC layer, polyester geogrids exhibited a much lower strength loss (about 13%), even tested under highly severe conditions.

Tests on specimens exhumed after real construction showed that stiff HDPE geogrids did not loose strength under low severity conditions. However, both non-woven polyester and woven polypropylene geotextiles did show a significant loss of about 15%, under similar installation conditions (Koerner & Koerner, 1990).

Viezee *et al.* (1990) concluded that a localized mechanical damage does not alter significantly the average deformation of synthetic fibers. Although the strength may be reduced by the necking observed in the transverse section, the damaged fiber does maintain its stiffness.

Experimental field investigation carried out by Allen (1991) showed strength losses as high as 40% for polypropylene or polyester woven geotextiles exhumed after

Table 1 - Classification for surviving capacity of geosynthetics (Azambuja, 1994).

Surviving capacity	Geotextile		Geogrid	
	Woven	Non-woven	Flexible	Stiff
Low	$M_A \leq 135$	$M_A \leq 135$	-	-
Moderate	$135 < M_A \leq 150$	$135 < M_A \leq 200$	$T \leq 55$	$T < 55$
High	$M_A > 150$	$M_A > 200$	$T > 55$	$T \geq 55$

Legend: M_A = mass per area or gramature (g/m^2); T = tensile strength (kN/m^2).

Table 2 - Classification for ambient severity (Allen, 1991).

Compaction equipment	Filling material	Ambient severity		
		$t < 15$ cm	$15 < t < 30$ cm	$t > 30$ cm
Light	Fine to coarse sand with rounded grains	Low	Low	Low
	Well graded sand and cobbles with sub-rounded to sub-angular grains ($D_{max} < 75$ mm)	Moderate	Low	Low
	Poorly graded cobbles with angular grains ($D_{max} < 75$ mm)	Very high	High	Moderate
Heavy	Fine to coarse sand with rounded grains	Moderate	Low	Low
	Well graded sand and cobbles with sub-rounded to sub-angular grains ($D_{max} < 75$ mm)	High	Moderate	Low
	Poorly graded cobbles with angular grains ($D_{max} < 75$ mm)	-	Very high	High

Legend: D_{max} = maximum grain size. t = thickness of compacted soil layer.

construction under moderately severe conditions. Troost & Ploeg (1990) also tested woven polyester geotextiles and reported a strength loss of 7 to 15% for low severity, and of 12 to 25% for moderate severity. For strength losses under 10%, these authors observed that the exhumed material may have an initial stiffness slightly above than that of intact material. This fact may be due to the previous tensile surcharge induced by compaction.

Geosynthetic material is commonly positioned between two soil layers of similar characteristics. In uniform soils with angular particles, mechanical damage results from high contact stresses due to compaction efforts (Fig. 1).

Lopes (2000) reported pullout test results in artificially damaged geogrid specimens, under several confining stress levels. Damage was imposed by cutting selected grid elements. Nine different configurations of grid damages have been tested. Lightly damaged specimens reach the peak strength under pullout conditions. Highly damaged specimens fail by tension in a localized position of the grid. The pullout strength ratio for intact and damaged specimens was observed to increase with increasing confining stress levels.

Confined tension tests in non-woven geotextiles, immersed in sand or coarser materials, were reported by Azambuja (1999). Under low compaction energy, the confined strength value may be lower than the unconfined one. This difference is smaller for high confining levels. However, under intense compaction, the confined strength is significantly higher than the unconfined one, emphasizing the beneficial effect of confinement on the behavior of damaged specimens.

Damage reduction factors are usually defined from unconfined tension test results. However, geosynthetic materials in field applications are frequently immersed in a soil mass. Confined tests would therefore reproduce more closely the geosynthetic conditions in reinforced fills.

This paper aims at evaluating the effect of mechanical damage on the load-elongation behavior of geogrids, taking into account its interaction mechanism with the confining soil. A comprehensive testing program was carried out in the laboratory, including unconfined tension, pullout and direct shear tests in damaged specimens.

Two different types of damages were herein considered:

1) Natural damage, resulting from laboratory simulations of field installation and compaction; it may or may not cause the rupture of the grid element, depending upon the severity of the compaction process.

2) Artificial damage, imposed by physical rupture by cutting one or more grid element with a scissor.

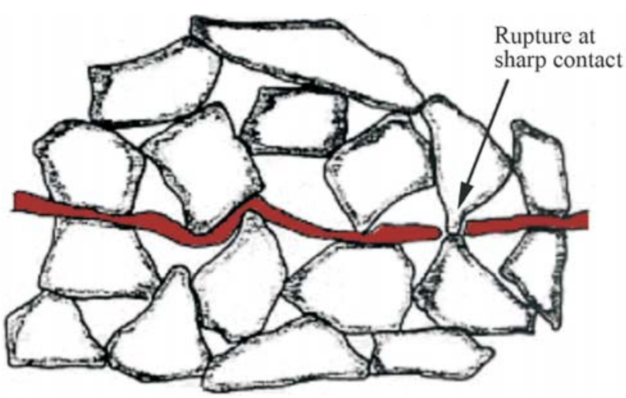


Figure 1 - Damage mechanism in geosynthetic used as reinforcement (Azambuja, 1994).

2. Materials

The experimental program made use of one specific type of geogrid and three distinct types of soil. The geogrid is commercially known as MacGrid 11/3-W and exhibits a regular woven mesh, made of stiff polyester filaments, coated by PVC for protection against installation and operational damages. The geogrid has a tensile strength of 92.4 ± 2.2 kN/m in longitudinal direction and of 29 ± 0.5 kN/m in transverse direction.

The grid geometry may be defined by a square opening of 20 mm (Fig. 2) and a solid surface area percentage of 30%, which is available for soil-geogrid friction.

The three soils had very distinct grain size distributions: silty clay, sand and cobble. The silty clayey soil is composed by 60% of clay minerals: kaolinite, chlorite and smectite. The remaining 40% is made of quartz and feldspar. The sandy soil is predominantly composed by quartz and feldspar. The coarser material (cobble) is made of basaltic rock fragments with 20 mm of average diameter (Fig. 3).

The main geotechnical characteristics of these three soils are presented in Table 3, in which G_s is the specific gravity and LL and PL are respectively the liquid and the plastic limits. Values of effective cohesion (c') and friction angle (ϕ') were obtained from direct shear tests on 300 mm x 300 mm specimens (Sieira, 2003). The sand was tested under a relative density D_r = 80%, while silty clay specimens were prepared at Proctor's optimum water content and 100% compaction degree.

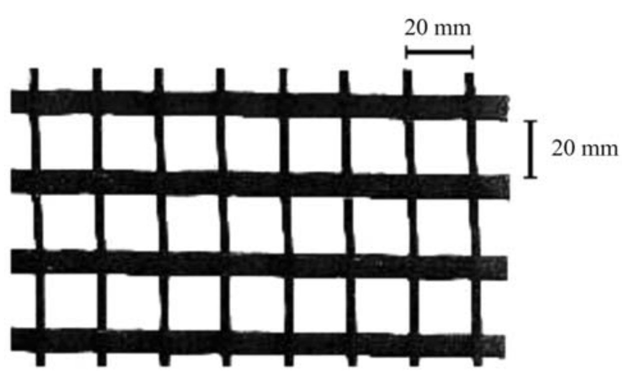


Figure 2 - Geogrid geometry.

Figure 3 - Basalt cobble.

3. Experimental Program

Unconfined tension, direct shear and pullout tests were performed on intact and damaged specimens in the CEDEX laboratory, in Spain (Sieira & Sayão, 2006). Two types of damage have been considered: natural damage, resulting from simulations of compaction procedures, and artificial damage, imposed by physically rupturing selected grid elements.

3.1. Unconfined tension tests: naturally damaged specimens

A 300 mm square metallic box, 150 mm in height, has been used for mechanical damage simulations. Initially, a 75 mm thick soil layer was compacted in the lower half of the box. The geogrid was then positioned (Fig. 4a) and the soil specimen was compacted in the upper half. Two distinct compaction procedures have been considered: a light compaction with an energy level similar to the Modified Proctor (2.63 J/cm^2), using a 4.5 kg manual hammer; and a stronger compaction (10.52 J/cm^2), using a dynamic vibrator.

After compaction, the geogrid specimens were carefully exhumed, avoiding additional damage, and then subjected to detailed microscopic inspection before taken to tension tests in the laboratory.

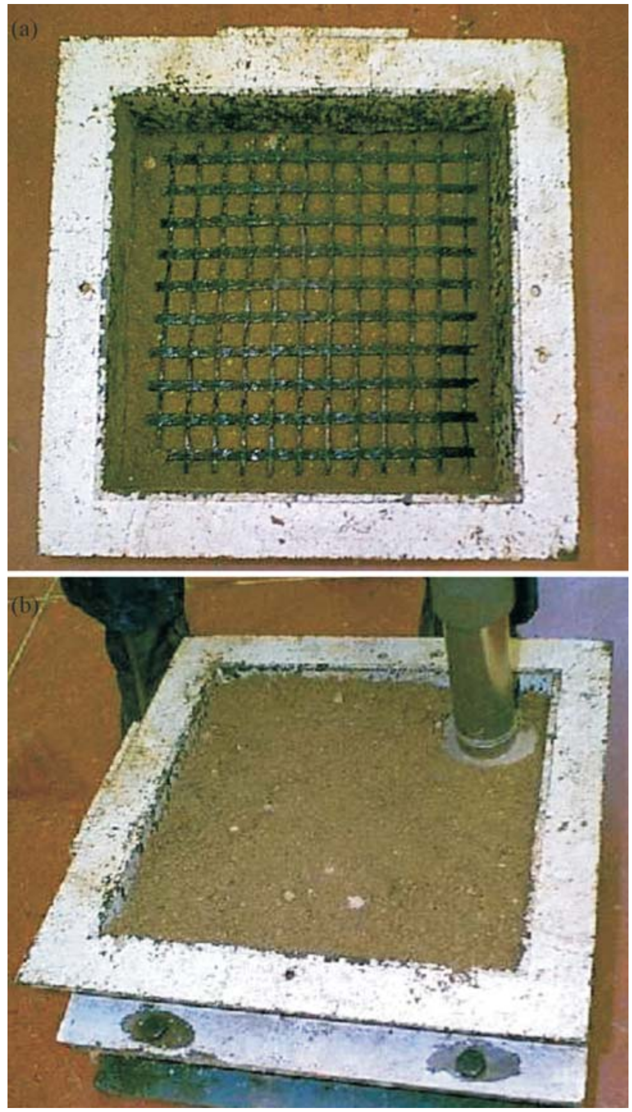


Figure 4 - Experimental simulation of mechanical damage on geogrid. (a) positioning the geogrid; (b) compacting with a manual hammer.

The mechanical damage was evaluated by a reduction factor (f_d), defined as:

$$f_d = \frac{\text{intact strength}}{\text{exhumed strength}} \qquad (1)$$

Table 3 - Geotechnical characteristics of soils.

Soil type	Physical characteristics			Strength parameters	
	G_s	LL (%)	PL (%)	c' (kPa)	ϕ' (°)
Silty clay (at w_{ot})	2.69	29.7	19.0	30	21
Sand (D_r = 80%)	2.71	-	-	16	37

3.2. Unconfined tension on artificially damaged specimens

Unconfined tension tests on artificially damaged specimens were carried out to evaluate the strength loss resulting from intense damage caused by cutting one or more mesh elements. All geogrid specimens were 200 mm wide and 250 mm long, ensuring an effective length of 100 mm between opposing claws. Tensile loading was imposed under a speed of 20 mm/min.

An Instron loading equipment was provided with claws according to the European and Brazilian standards for geotextile's tensile properties by wide-width strip method (ABNT, 1993).

Six intact geogrid specimens were used in these tension tests. Artificial damage was imposed after positioning the specimens in the loading device, without pre-tensioning. Three specimens had their central transverse elements cut (ruptured), as indicated in Fig. 5a. The other 3 specimens were cut in the central longitudinal element (Fig. 5b).

3.3. Pullout testing on artificially damaged specimens

These tests were carried out on 1 m square specimens in a large shearing apparatus. Artificial damage was imposed by cutting one or more mesh elements with a scissor.

The device was initially developed for direct shear tests on soils and rockfill and later modified for pullout testing of geosynthetics (Sayão *et al.*, 2002; Sieira *et al*., 2009).

Initially, the lower half of the box was filled with compacted layers of soil. The damaged geogrid was then positioned and fixed to the claw, before the soil layers were statically compacted in the upper half. The confining pressure was then imposed and the pullout load applied.

During the tests, load and displacement were carefully monitored at the tensional claw, which was positioned at 20 cm distance from the frontal face of the device.

Table 4 presents the pullout testing program. Geogrid specimens with different damage configurations were considered for allowing direct comparison with intact grids.

The experimental program consisted of tests on specimens with 3 or 5 damaged elements, distributed along the central longitudinal element. The damage was imposed by rupturing the longitudinal mesh elements, along the pullout direction (points A, B, C, D and E).

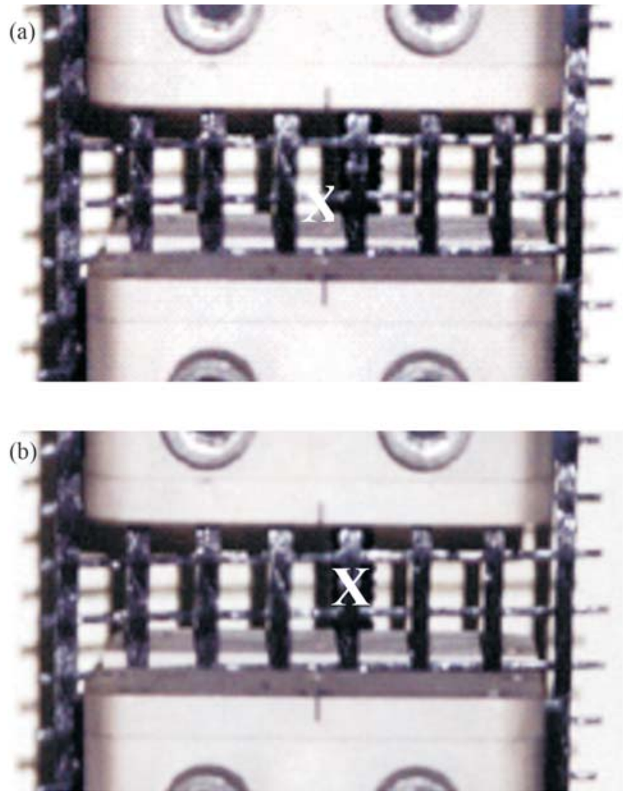
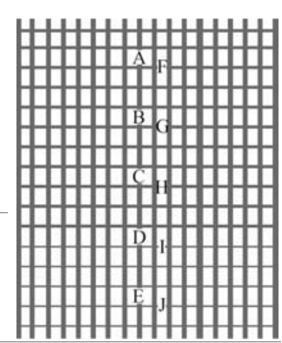


Figure 5 - Position of damaged geogrid elements. (a) transverse element; (b) longitudinal element.

In sandy soil, tests were also carried out on specimens with damages in the transverse direction, distributed along the pullout direction (points F, G, H, I and J). These tests aimed at evaluating the contribution of transverse elements under pullout loading. All tests on damaged specimens were performed under a confining pressure of 25 kPa.

It should be noted that, under a pullout load, longitudinal strips are mainly responsible for mobilizing friction at soil's interface. Transverse elements are responsible for

Table 4 - Pullout tests in artificially damaged specimens.

Soil	N. of damages	Grid element	Damage	Damage position
Sand	0	-	-	
	3	Transverse	G H I	
	5	Transverse	F G H I J	
	3	Longitudinal	B C D	
	5	Longitudinal	A B C D E	
Silty clay	0	-	-	
	3	Longitudinal	B C D	
	5	Longitudinal	A B C D E	

mobilizing passive loads due to geogrid's anchoring within the confining soil. The damage distribution along the longitudinal or transverse strips helped in evaluating the worst damage position along the reinforcement.

The reduction factor for mechanical damage is usually computed from the ratio of intact over damaged strengths, under unconfined tensile conditions. The pullout tests were carried out for finding out the strength loss under confined conditions, which is a common situation in the field.

3.4. Direct shear testing on artificially damaged specimens

The experimental program included direct shear tests on damaged specimens, placed vertically inside the shear box. These tests allowed the investigation of the influence of damage in situations where the failure surface intercepts the reinforcement. In this case, the geogrid becomes tensioned and lends a positive tensile strength to the soil.

These tests were carried out with a shear box of 300 mm x 300 mm, in sandy and silty-clayey soils under a confining level of 100 kPa. The geogrid's position in the shearing box is shown in Fig. 6. The damage was imposed in the longitudinal central strip of the specimen, at the position of the imposed shear plane. The sandy soil was prepared with 10% water content and a relative density of 80%. The silty clay was compacted at optimum water content, reaching a compaction degree of 100%. These conditions were similar to those adopted in pullout and direct shear testing on natural unreinforced soils.

All shearing tests in the 300 mm x 300 mm box followed the ASTM D5321 requirements about the minimum dimension of the box being at least five times larger than the geogrid's openings.

4. Results

Damage effects were evaluated by different tests (unconfined tension, pullout, direct shear) and different types of damage (natural or artificial) imposed to the geogrid specimens. The nomenclature convention adopted for the reduction factors is presented in Table 5. These reduction factors were obtained from the ratio between intact and damaged specimens (Eq. (1)).

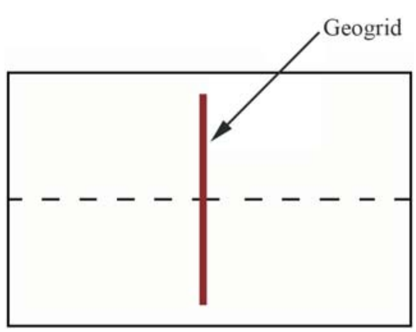


Figure 6 - Position of geogrid inside the direct shear box.

4.1. Unconfined tension on naturally damaged specimens

Table 6 presents the results of reduction factors ($f_d = f_{d1}$) from unconfined tension tests with naturally damaged geogrid specimens. Values of f_{d1} were computed from the ratio between intact and damaged tensile strengths (Eq. (1)). The results indicated a significant strength loss when the compacted cobble was used, with reduction factors from 1.30 (light compaction) to 1.45 (strong compaction).

In sand, strong compaction imposed a reduction factor of only 1.07, while light compaction was insignificant in damaging the geogrid. The compacted silty clay did not suffer any strength loss due to compaction procedures.

Intact geogrid had a tensile strength of 92.4 kN/m, slightly lower than the manufacturer's nominal value of 97 kN/m. This difference may be due to changes in testing procedures, in particular those related to the fixing details of the geogrid (Sieira *et al.*, 2006).

Microscope inspection revealed that, in compacted cobble, the core polyester phylaments were ruptured beyond the PVC coating protection, as indicated in Fig. 7. This is important because the core is responsible for the mechanical characteristics of the geogrids. The function of the coating is to protect the core against damages due to installation and to the use of the reinforced structure. Damage in the core may therefore cause a significant strength loss.

On the other hand, damage on the coating may cause long term problems, as the core is exposed to chemical and biological actions during the operational life of the reinforced mass.

Table 5 - Symbols for reduction factors.

Damage	Test	Reduction factor
Natural	Unconfined tension	f_{d1}
Artificial	Unconfined tension	f_{d2}
	Pullout	f_{d3}
	Direct shear	f_{d4}

Table 6 - Redution factors of geogrids damaged by compaction.

Soil	Compaction	Tensile strength (kN/m)	Factor f_{d1}
Silty clay	Light (2.63 J/cm^2)	92.1	1.00
Sand	Light (2.63 J/cm^2)	92.0	1.00
Cobble	Light (2.63 J/cm^2)	70.1	1.30
Silty clay	Strong (10.52 J/cm^2)	92.0	1.00
Sand	Strong (10.52 J/cm^2)	86.5	1.07
Cobble	Strong (10.52 J/cm^2)	63.5	1.45

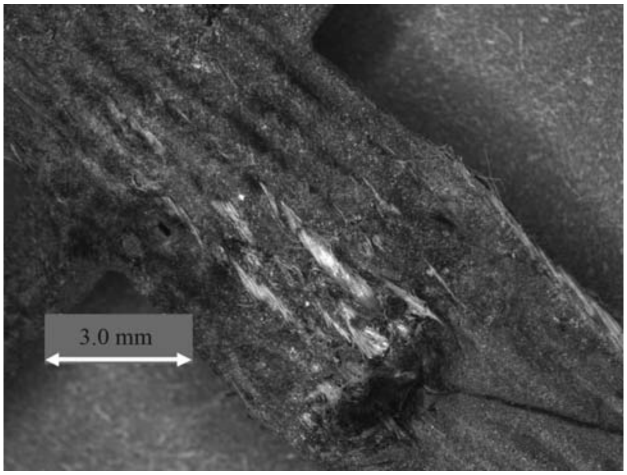


Figure 7 - Microscope inspection of natural damage after laboratory compaction.

4.2. Unconfined tension on artificially damaged specimens

Tests in naturally damaged specimens show that the compaction procedures herein considered did not cause severe damage to the geogrids in fine to medium grained soils (clay or sand). Additional tension tests were then carried out on specimens with intense damage, imposed by rupturing the grid elements with a special scissors.

The reduction factors ($f_d = f_{d2}$) obtained under unconfined conditions are presented in Table 7. The tensile strength of undamaged geogrid is 92.4 kN/m in the same longitudinal direction. Three identical tension tests were performed on specimens with one cut longitudinal element and another three tests were done on specimens with one cut in a transverse element.

Typical results are presented in Fig. 8. As longitudinal elements are responsible for transferring the tensile load along the geogrid, a significant drop in strength is to be expected when one or more of these elements are breached. Rupture of a longitudinal element caused a strength reduction of about 21%, corresponding to a factor f_{d2} = 1.27.

On the other hand, under unconfined tensile loads, transverse elements are mainly responsible for the positioning and configuration of the mesh. Accordingly, the strength reduction was of 9,6%, which corresponds to a factor f_{d2} = 1.11. However, in pullout loading, these elements are responsible for anchoring the grid in the soil mass and the contribution of passive resistance to the overall strength becomes more significant (Jewell *et al.*, 1984).

Figure 9 presents two geogrids after unconfined tension tests. It is noted that failure of the mesh happens at the contact position with the claws. These therefore represent a week point in the testing arrangement and may be responsible for differences in results from tests in different devices.

Table 7 - Unconfined tension tests along longitudinal direction on artificially damaged geogrid.

Ruptured element	Tensile strength (kN/m)	Average	f_{d2}
Longitudinal	74.0	72.8	1.27
	72.0		
	72.5		
Transverse	80.0	83.5	1.11
	84.0		
	86.5		

4.3. Pullout testing on artificially damaged specimens

Reduction factors are usually computed from laboratory tension tests, in which the geogrid is kept unconfined. In the field, however, the geogrid is immersed in the soil mass. Other variables become therefore relevant, such as confining stress, soil type, soil density and grid geometry.

Confined tests reproduce more appropriately the field operational conditions of geogrids within reinforced soil masses. Consequently, reduction factors due to damage shall be more adequately investigated from confined pullout tests.

Table 8 presents the results of pullout tests and corresponding reduction factors ($f_d = f_{d3}$). Factor f_{d3} was computed from the ratio between intact and damaged pullout strengths, in a similar way as previously defined for tension tests.

Pullout results for dense sand (D_R = 80%) are presented in Fig. 10. These results correspond to damages along one longitudinal strip. The pullout strength is seen to drop significantly with increasing number of damaged elements. It is important to note that, in these tests, the geogrid is pulled out from the soil, exposing damage A (Table 4). In this unconfined zone, a gradual increase in the longitudinal

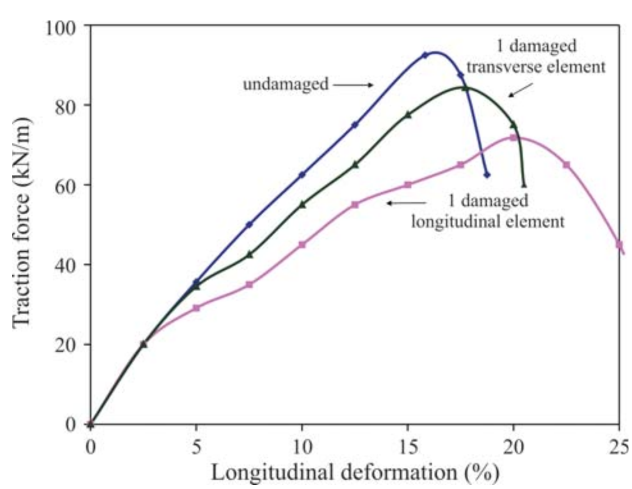


Figure 8 - Tension tests in artificially damaged geogrid specimens.

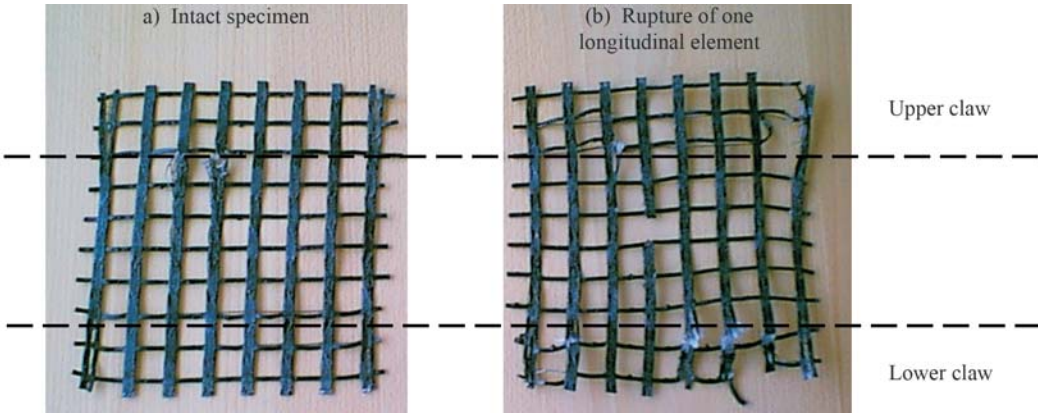


Figure 9 - Geogrid configuration after unconfined tension tests.

Table 8 - Pullout tests on artificially damaged geogrid specimens.

Soil	D_R or GC (%)	N. of damages	Damaged strip	P_{ult} (kN/m)	f_{d3}
Sand	0	0	-	65.1	-
		3	Longitudinal	49.0	1.32
		5	Longitudinal	38.2	1.70
		3	Transverse	55.8	1.16
		5	Transverse	41.9	1.55
Silty clay	100	0	-	51.5	-
		3	Longitudinal	44.1	1.17
		5	longitudinal	42.7	1.20

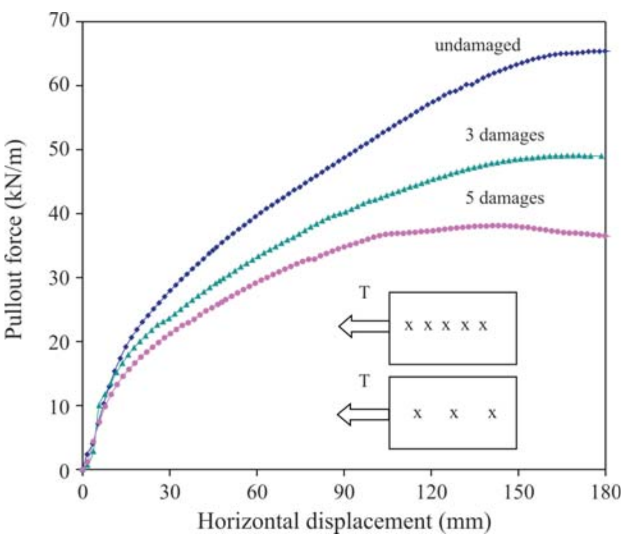


Figure 10 - Pullout tests in sand: damage in geogrid's longitudinal strip.

dimension of the damage was observed, causing a reduction of the geogrid's stiffness.

The longitudinal strip is responsible for transferring the tensional load. Damage reduces the tension stiffness and strength of the geogrid. It should be noted that the pullout displacement is composed by two main parts: deformation and rigid body displacement.

Figure 11 presents the marked variations of strength values and reduction factors with the number of damaged elements for pullout tests in sand. With increasing damage, pullout strength decreases, while factor f_{d3} increases. For five damages in the central strip, f_{d3} reaches 1.70, corresponding to a strength loss of about 42%. These results are somewhat magnified by the grid's exposure in the unconfined region, as previously discussed.

In silty clay soil, similar behavior was noted for damaged geogrids (Fig. 12). Increasing damage caused a decrease in pullout strength, although this decrease was less significant than in sand. With 5 damages, factor f_{d3} was 1.20, instead of the observed value of 1.70 in sand. This was probably related to the lower interface shearing resistance of the geogrid with silty clay, as compared to sand. These results suggest that the effect of damage is higher for coarse grained soils.

Tests results with geogrids damaged in transverse elements are presented in Fig. 13. A significant loss in strength is noted for increasing damage in transverse strips. This is in opposition to the findings drawn from unconfined tension tests, but may be explained by the relative contribution of passive resistance related to transverse elements of the geogrid during pullout loading. Experimental evidence of this passive contribution in overall pullout strength of geogrids has been presented by Palmeira (1987), Palmeira & Milligan (1989) and Sieira (2003).

Pullout tests in artificially damaged specimens had allowed the evaluation of the susceptibility of geogrids to the mechanical damage. A susceptibility index S has been defined by Eq. (2) and can be visualized by the declivity of the curve between the reduction factor and the number of damaged elements.

$$S = \left(\frac{\Delta f_{d3}}{\Delta n}\right) \times 100\% \qquad (2)$$

where Δf_{d3} = variation of reduction factor and Δn = variation of number of damaged elements.

It may be noted that large values of S are related to a higher geogrid's susceptibility to loose strength due to damage.

Figure 14 shows the influence of damage in geogrids inserted in different soils (sand and in silty clay) under pullout loading. An approximately linear drop of the pullout strength in sand may be noted with increasing number of damages in a longitudinal element, resulting in a susceptibility index $S = 8.3\%$. The influence of damage is much less significant in silty clay, for which a decreasing S may be noted for increasing damage. For this clayey soil, when the geogrid goes from 3 to 5 damages the susceptibility index is noted to be $S = 1.8\%$.

This larger pullout reduction for tests of geogrids immersed in sand is related to the higher interlocking of soil grains around the geogrid mesh, as shown in pullout results with intact geogrid (Sieira & Sayão, 2004).

4.4. Direct shear testing on artificially damaged specimens

In the direct shear tests, damage was imposed by cutting one element in the central longitudinal element and the geogrid was placed in a vertical position, as illustrated in Fig. 6.

Figure 15 presents a comparison of test results with intact and damaged geogrid specimens in both sand and

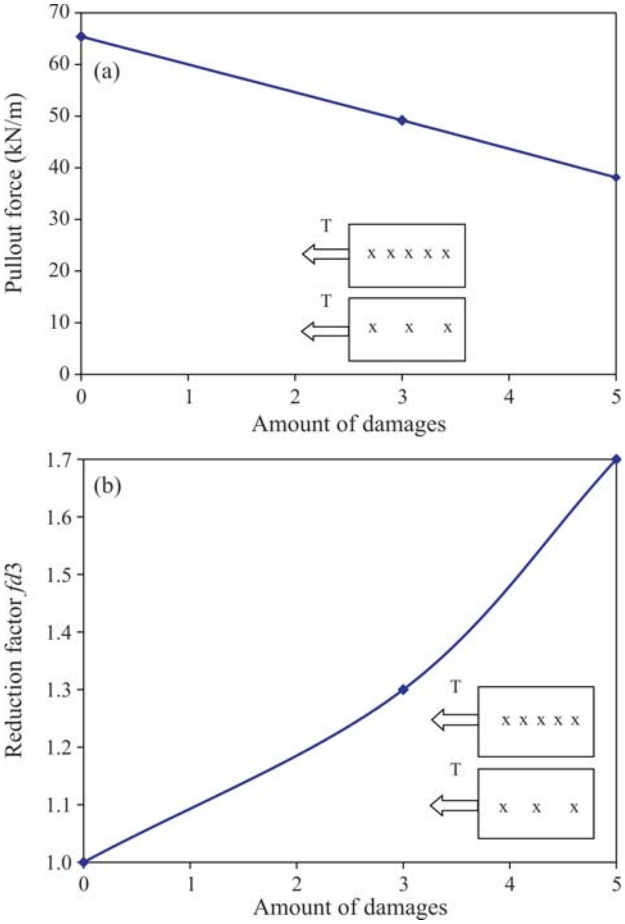


Figure 11 - Pullout tests in sand: influence of artificial damage in longitudinal direction. (a) pullout strength; (b) reduction factor f_{d3}.

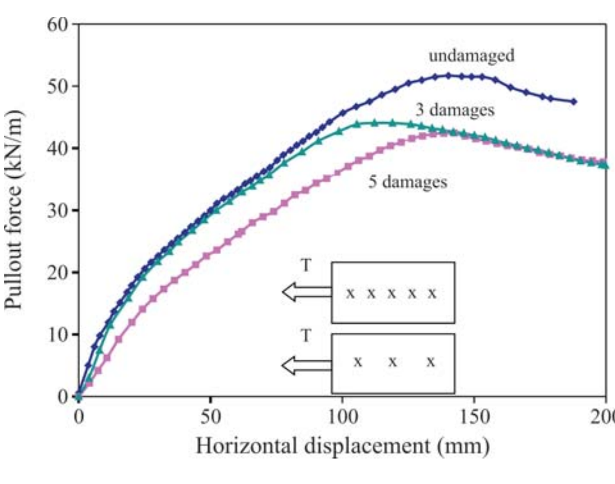


Figure 12 - Pullout tests in silty clay: damage in geogrid's longitudinal strip.

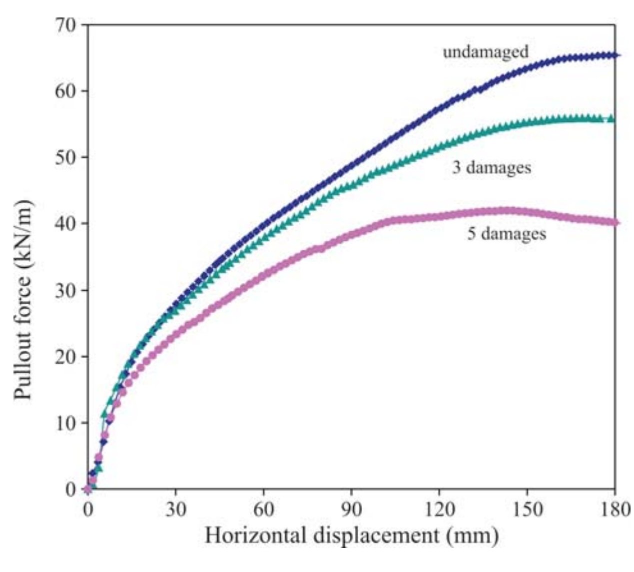


Figure 13 - Pullout tests in sand: damage in geogrid's transverse strip.

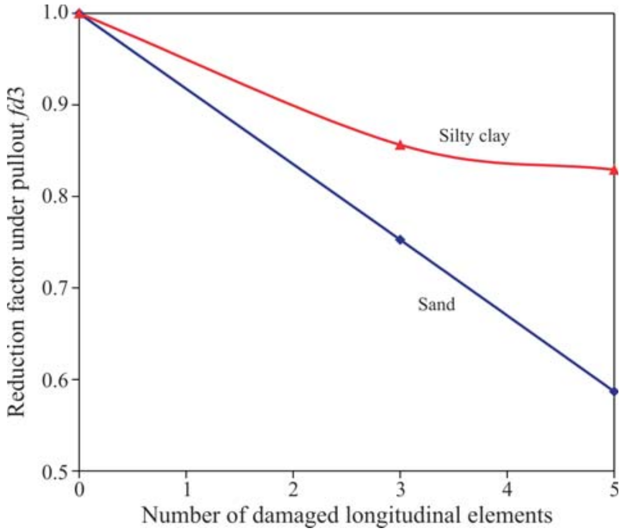


Figure 14 - Susceptibility index for damage of geogrid immersed in sand and in silty clay.

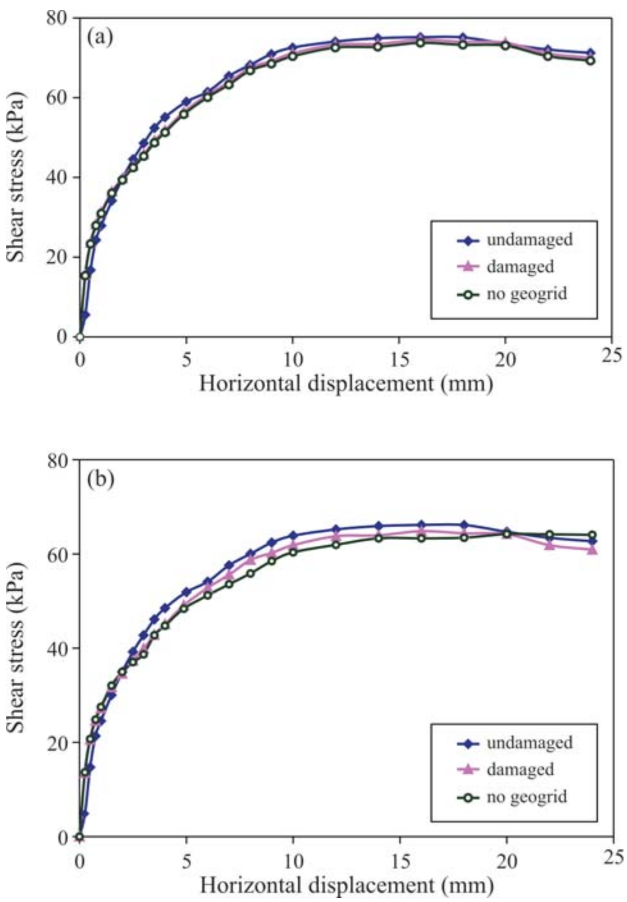


Figure 15 - Direct shear results in artificially damaged geogrid.

silty clay. For performance comparison, direct shear results of soil specimens with no reinforcement are also shown. A confining stress of 100 kPa was applied in all tests.

The loss in strength due to damage is noted to be insignificant. A reduction factor $f_d = f_{d4} = 1.0$ (corresponding to $S = 0$) may be considered representative for both sand and silty clay tests. This observation may be explained with basis on the results previously presented in Fig. 8. Up to a tensile deformation of about 3%, the behavior of intact geogrid is similar to the one with damage in the longitudinal element. In direct shear tests with vertical reinforcement, the geogrid is submitted to traction. Depending on the longitudinal deformation induced by shearing, the mobilized tensile resistance may be not yet influenced by the damage.

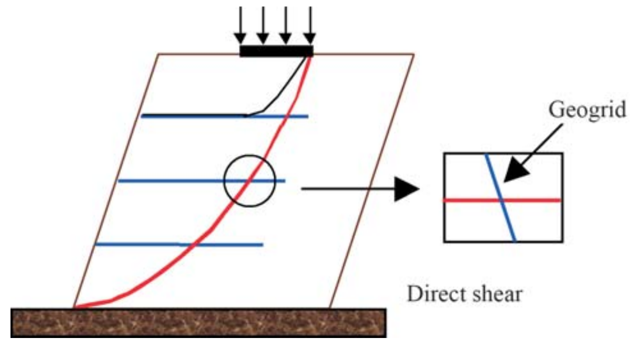


Figure 16 - Field condition simulated by direct shear tests with inclined reinforcement (Palmeira & Milligan, 1989).

It is also noted that geogrids in vertical direction have negligible influence in direct shear results. This explains the insignificant influence of geogrid damage on results of tests, *i.e.*, the presence of the geogrid (with or without damage) has little influence.

These results suggest that, in field situations where the geogrid is nearly perpendicular to the potential failure surface, eventual damage may not compromise the integrity of the reinforced mass. These situations may be found in the upper part of the reinforced fill, as illustrated in Fig. 16.

5. Conclusions

This paper presents an investigation on the influence of mechanical damage on the behavior of geogrids. The experimental program included unconfined tension, pullout and direct shear tests with geogrids in sand and silty clay. Two distinct types of mechanical damage were imposed in the laboratory: natural and artificial damage.

Natural damage was shown to be more relevant when aggressive compaction procedures were imposed to coarse grained soils in contact with the geogrid. In sands, low energy procedures by manual compaction did not result in damaging the geogrid. In silty clay, damage was not significant, even when high energy compaction was applied.

Results of unconfined tension tests in artificially damaged geogrid revealed that rupturing a longitudinal element caused a strength loss of about 22%, corresponding to a reduction factor of 1.27. When a transverse element was ruptured, the strength loss was much less significant.

Under pullout loading, however, transverse elements were shown to contribute significantly to the overall strength, due to its anchoring effect. Therefore, damage in these transverse elements may not be neglected when pull-out conditions prevail in the field.

In direct shear, the results indicated that the relative position of the geogrid relative to the potential failure surface is an important factor. When the geogrid was placed in a nearly perpendicular direction relative to the failure surface, damage in the geogrid was not of concern. This is usually the case of the upper geogrid layers within reinforced fills.

Acknowledgments

The authors are grateful to Maccaferri Brazil for supporting the research on the behavior of geogrids, and to CEDEX (Spain) for the availability of the laboratory facilities for the experimental program. The authors also thank to the Brazilian Research Council (CNPq) for the research grants.

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Geo-Engineering Education and Training. The Past and the Future

Ricardo Oliveira

Abstract. A brief background on education and training in Soil Mechanics, Rock Mechanics and Engineering Geology is presented, highlighting some facts which influenced the development of these sciences. The interplay between them is stressed. The International Societies (ISSMGE, ISRM and IAEG) role in promoting education at different levels is emphasised. The performance of scientists and engineers teams from different backgrounds for the study, design, construction and rehabilitation of major infrastructures and for the solution of many geoenvironmental problems is illustrated. These disciplines are taught today in University undergraduate courses in Civil Engineering, Geological Engineering and Geology and graduate courses are tending to be jointly attended by professionals coming from these three branches. The Bologna Declaration introduced significant modifications in University education and most European countries have already adjusted their education systems to its requirements. Based on some principles of this declaration, perspectives are presented concerning future education and training in geoengineering.
Key words: geo-engineering, teaching, training and professional practice.

1. Introduction and Background

When Civil Engineering courses were established as a result of the separation from military engineering, in the end of the 19th century, students were taught very little on the properties and behaviour of the ground and, in general, related only to foundations in soft ground (soils). Problems associated with tunnel, canal and railway construction and embankments were solved most times without scientific inputs and mainly based on engineering experience.

In that very beginning, Soil Mechanics did not exist as a science yet and its establishment was mainly due to the work developed by Karl Terzaghi. He recognized in the 20's the need for the establishment of principles and theories which could explain the behaviour of the soft ground. His book "Erdbaumechanik auf Bodenphysikalischer Grundlage" (Terzaghi, 1925) is clearly a landmark of what encompasses today the geoengineering activity.

His views at the time can be considered today as prophetic, since they addressed the subjects of the mechanical and the hydraulic behaviour of the ground (soils and also rocks) based on a correct description of the geological conditions and definition of the soil intrinsic properties.

In his paper "Effect of Minor Geologic Details on the Safety of Dams" (Terzaghi, 1929), published almost 80 years ago, he wrote "to avoid the shortcomings associated with present practice requires first of all expert translation of the findings of the geologist into physical and mechanical terms. Next it requires the evaluation of the most unfavourable mechanical possibilities which would be expected under the existing geologic conditions; and finally to assume for the design of the structure the most unfavourable possibilities. These mental operations represent by far the most important, most difficult and most neglected tasks in the field of dam foundations".

Later, many other eminent authors also recognized the need for a proper contribution of Geology in the understanding of the properties and behaviour of soils and rock masses but, as Manuel Rocha stated in 1952 (Rocha, 1952) "Given the complexity of geologic formations it is, in general, indispensable that geologists collaborate in the site investigations of soils, their main role being the definition of the soil structure (attitude, thickness and consistency of the layers, ground water, discontinuities, etc). Only having that information available, it is possible to establish a program for the quantitative determination of the properties of the soils aiming at significant results to be used, having in mind the need to reduce costs and delays with the site investigations. The most convenient education of such geologists is not a classic naturalistic formation but it must be an education based on physics and chemistry".

However, at that time, the Geology courses were essentially devoted to the naturalistic aspects of the Geology preparing scientists for research activities in palaeontology, mineralogy, petrography, geomorphology and geological mapping of outcrops and some excavations. Very little information was given in mathematics, physics and chemistry and no engineering background was supplied to the Geology students.

As the technical requirements increased and as the interaction of the structures with the ground were more relevant, the need for geological information grew and education on basic geology for civil engineers started as well as

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Submitted on September 11, 2008; Final Acceptance on January 26, 2009; Discussion open until August 31, 2009.
Previously published in the Proceedings of the Conference on Education and Training in Geo-Engineering Sciences, Constantza, Romania, 2008. Republished in Soils and Rocks with the authorization of Taylor & Francis.

the cooperation of geologists supplying geologic data relevant for the assessment of the ground behaviour of large projects.

One can say that the first half of the 20th century is the period in which the need for geological input in the large projects has been recognized as indispensable and that professionals with the skill to bridge geology and engineering were lacking.

Civil Engineers had then one or two semester courses in Geology (Geology for Engineers), in most cases of very little use, the main subjects taught being mineralogy, petrography, palaeontology, geomorphology, hydrogeology, etc. The lack of engineering flavour of the geology teaching staff was responsible for the divorce between the subjects taught and their contribution to the solution of engineering problems.

By the fifties, the concept of Applied Geology developed and, then, an effort was made to understand from the geological side what was really relevant to the engineering projects and problems as geological information and how to use site investigations properly.

This recognition had two consequences. Geology for Engineers, a course for Civil Engineering graduates, adjusted its content in order to call the attention of students to the role of geological properties and parameters (lithology, structure, hydrogeology, geomorphology, seismicity, etc) in the definition of the geological models of the ground and of their importance in the stability analysis of civil engineering structures.

Geology students were, at the same time, instructed in the interaction between civil engineering activity and the ground and some courses (first free courses) were given on applications of geological knowledge to the solution of engineering problems and to the design of engineering structures.

Text books by Kryrine and Judd (1957), Goguel (1959), Desio (1959) and Legget (1962) clearly mark that period and helped very much engineers and geologists to learn to work together and to create an atmosphere of reciprocal respect.

These books have been used all over the world as support of many courses (mainly graduate short and long courses) and for self instructions of professionals from both backgrounds.

During the 60's, 30 years after Soil Mechanics was scientifically established, Engineering Geology emerged as a new discipline in the field of Geotechnics and the same happened with Rock Mechanics. By the end of that decade it became accepted that Geotechnics, as a branch of Engineering, embraced Soil Mechanics, Rock Mechanics and Engineering Geology (Fig. 1).

The main factors which have contributed for the "separation" of these disciplines from Soil Mechanics were, on the one hand, the increasing interference of the civil engineering structures with the ground (large excavations, longer and wider tunnels, high embankments on soft ground, large dams, etc.) and, on the other hand, some tragic accidents and natural disasters from which resulted large losses of lives and property.

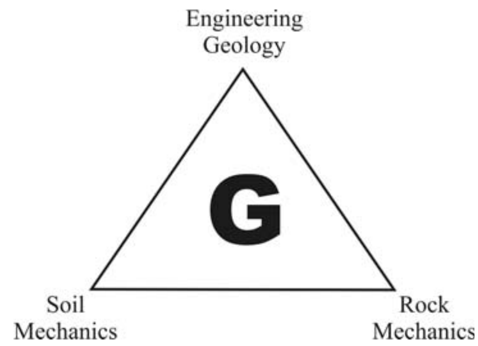

Figure 1 - Geotechnics and embraced sciences.

The stability analysis of rock masses could not be assessed with the same approaches and analysis as for soil masses since their geotechnical behaviour was very much dependent on the discontinuities of the ground and much less on the properties of the rock material.

All those issues proved to be indispensable a more accurate assessment of the ground properties at surface and in depth, this requiring the development of more sophisticated site investigation techniques (geophysical and mechanical) and testing and the knowledge to conduct the interpretation of the results on the basis of the geological model and of the engineering requirements.

As a result of the worldwide recognition of the importance of these two new geotechnical disciplines, international learning societies have been established still in the 60's.

The first ISRM Congress was organized in Lisbon (Portugal) in 1966 and the first IAEG Congress was organized in Paris (France) in 1970 by the National Groups of the respective International Societies.

A Permanent Coordinating Secretariat for the activities of the three Societies has been established in Brussels in 1972 and since then efforts have been made by many people to strengthen the relationship of the Societies ("sister Societies") and of their members, all together promoting the essential and indispensable role of Geotechnics in the sustainable development of the world.

The next step was the introduction of Rock Mechanics and Engineering Geology regular courses in the Civil Engineering curricula and to introduce courses on principles of Soil Mechanics, Rock Mechanics and Engineering Geology in the Geology curricula.

This trend was followed in the most developped countries, namely in Europe and America and it was a most relevant contribution to the advance of Geotechnical Engineering.

Nevertheless, education and training in geoengineering sciences at undergraduate level proved to be not enough to supply geologists and engineers with the knowledge and experience required by the most sophisticated engineering problems and projects.

Academia, understanding the needs of Industry in this and other Engineering branches, adjusted their course structures and degrees and organized to offer graduate courses in different areas.

Graduate courses in Soil Mechanics, in Rock Mechanics and in Engineering Geology were successfully offered to engineers and geologists wishing to specialize in these fields.

The experience of the New University of Lisbon is an interesting example of such offer and it has been reported several times in the past. A paper describing this experience was submitted to the XI European Conference on Soil Mechanics and Foundation Engineering under the topic "The Interplay between Geotechnical Engineering and Engineering Geology" organized by the Danish Geotechnical Society in Copenhagen in May 1995, as a contribution for the discussion in Workshop 1: "Education Issues with Attention to the Professional and Commercial Implications". The paper entitled "Geotechnical education at graduate level. 18 years of experience at the New University of Lisbon (UNL), Portugal" describes the structure of the MSc Courses offered from 1975/76, one in Soil Mechanics and the other in Engineering Geology. Basically the contents of both courses overlap by 50% and, of those, about 20% correspond to Rock Mechanics subjects (Oliveira, 1995).

In the 80's many Universities offering Civil Engineering, Mining Engineering and Geology courses introduced a new undergraduate course in Geologic Engineering following the concept originated in USA with the purpose of reaching a compromise between an engineering education and the geologic content, increasing the number of engineering subjects (mechanics of materials, hydraulics, computer science, etc), reducing the number of the more naturalistic geologic topics and introducing more applied geology.

Furthermore, being an engineering course, the Geologic Engineers were registered as Professional Engineers whenever the course was accredited.

2. The Growing Importance of the Environment

Since long, geoscientists and engineers had to deal with environmental problems, namely natural hazards, like landslides and river and coastal erosion. Books by Varnes (1958), Zaruba & Mencl (1969) and others were devoted to landslides and their control.

In general, men were suffering the consequences of natural disasters, their impacts being loss of lives and property.

As a result of the activity of men, on the one hand, and of the growing and concentration of population in cities (some megacities), on the other hand, the environmental issue turned into one of the most serious problems of the world (Oliveira, 2000).

From the geoengineering point of view, the impacts caused by the construction of large structures and the unappropriate land use required detailed studies of the geologic formations and engineering solutions for their mitigation.

Problems like ground and groundwater pollution, waste disposal, extraction of natural resources, re-use of by-products and others call for a strong support from the geotechnical side and require a continuous interplay between the geoengineering sciences.

The importance of these problems to the control and preservation of the environment was responsible for the development of a new subject, the Environmental Geotechnics.

The first International Conferences have been organized in 1994 in Edmonton (Canada) and in 1996 in Osaka (Japan). This was an initiative of the ISSMGE as a result of the work of its TC 5 on Environmental Geotechnics. In the third Congress organized in Lisbon (Portugal) in 1998 a workshop on "Education in Environmental Geotechnics" was part of the program.

In 1997 the Greek National Group of IAEG organized an "International Symposium on Engineering Geology and the Environment" Theme 9 being devoted to "Environmental courses in geological and geotechnical education". Two of the special lectures of the Symposium were also devoted to education: "Teaching environmental subjects in engineering geological education" (Oliveira, 1997) and "Environmental geology courses within university education" (Rosenbaum, 1997).

All the mentioned subjects related to the environmental protection and to the mitigation and remediation of the effects of engineering construction on the environment call for a close cooperation between Soil Mechanics, Rock Mechanics and Engineering Geology as well as of many other disciplines, and they have been addressed since long by technical commissions established by the three sister societies.

3. Definitions and Boundaries

As said before, Soil Mechanics has been established as a science in the early 30's having its first International Congress in Harvard in 1936, when the International Society of Soil Mechanics and Foundation Engineering was originated. In 1997 the name has been changed to International Society of Soil Mechanics and Geotechnical Engineering to reflect more accurately the activities of the Society.

In the last version of the statutes (Osaka 2006), the aim of the ISSMGE is the promotion of international cooperation amongst engineers and scientists for the advancement of knowledge in the field of geotechnics and its engineering and environmental applications.

The statutes of the ISRM in their 1999 version show some more detail in the definition of the Rock Mechanics field of activity as a science: "Rock Mechanics includes all the studies relative to the physical and mechanical behaviour of rocks and rock masses and the application of this knowledge for the better understanding of geological processes and in the fields of Engineering".

As for IAEG, the last version of the statutes (1992) includes the definition of Engineering Geology as "a science devoted to the investigation, study and solution of engineering and environmental problems which may arise as the result of the interaction between geology and the works and activities of man as well as to the prediction of and the development of measures for prevention or remediation of geologic hazards. Engineering Geology embraces: the definition of the geomorphology, structure, stratigraphy, lithology and groundwater conditions of geological formations; the characterisation of the mineralogical, physicogeomechanical, chemical and hydraulic properties of all earth materials involved in construction, resource recovery and environment change; the assessment of the mechanical and hydrologic behaviour of soil and rock masses; the prediction of changes to the above properties with time; the determination of the parameters to be considered in the stability analysis of engineering works and earth masses; and the improvement and maintenance of the environmental condition and of the properties of the terrain".

Although the detail of the definition of the aim of the three disciplines is quite different, it is clear that there is a significant (and healthy) overlapping between the activities of each of them.

This said, one can state that it is not possible to draw sharp boundaries between Soil Mechanics and Rock Mechanics, between Soil Mechanics and Engineering Geology and between Rock Mechanics and Engineering Geology.

In other words, a specialist in Soil Mechanics has to be instructed to some extent also in Rock Mechanics and in Engineering Geology, another in Rock Mechanics has to be instructed to some extent also in Soil Mechanics and Engineering Geology and another in Engineering Geology has to be instructed to some extent in Soil Mechanics and in Rock Mechanics.

The experience of conducting research in geoengineering subjects and of coordinating large engineering projects proves that in most cases the engineering teams include professionals from different backgrounds and that many activities and decisions have to be jointly discussed and agreed.

However, it seems beneficial to the required interplay to identify the relevant activities which should preferably fall under the responsibility of each group. In a paper presented to a Rock Mechanics symposium more then twenty years ago (Oliveira, 1986) the author presented his views as concerns the "border zone" between Engineering Geology and Rock Mechanics and introduced a methodology for the study of rock masses related to large engineering structures (dams, tunnels, slopes, etc.) which is still followed, in general, today.

It seems clear that Engineering Geology should be responsible for the geologic reconnaissance of the ground and for the definition of the site investigations program. This implies the knowledge of the geophysical and mechanical methods which have to be used in each case, including the relevant in situ tests, namely those performed inside boreholes, as well as how to collect representative samples for laboratory tests. It is the case of permeability tests, dilatometer and pressiometer tests, seismic tests, integral sampling, logging, etc.

The joint interpretation of all the data from these activities, provided they are spread throughout the rock mass and their number allows some statistical analysis, will conduct to the *geotechnical zoning* of the rock mass, each zone being defined on the basis of the geologic conditions and on the values obtained for the relevant parameters. This would be, in most cases, the geotechnical information required for the basic design.

For the final design, some more detailed information may be required, based on a small number of more time consuming and expensive tests, for the detailed stability analysis of the engineering works. These would be best of the responsibility of a specialist in Rock Mechanics since they are very much related to the numerical models which will be used in the analysis, this being clearly a Rock Mechanics task.

A similar approach could be followed for the tentative definition of the boundaries between Soil Mechanics and Engineering Geology.

Soil Mechanics and Rock Mechanics have in common the responsibility of conducting the stability analysis of the ground (and ground / structure) through appropriate numerical models and the design of the solutions which best fit the ground properties and the stability of engineering works.

As said before, the reason for the development of Rock Mechanics as an independent science relies on the fact that the behaviour of soil masses depends in general on the soil properties (physical, mechanical and hydraulic) and the behaviour of rock masses depends much more on the structure of the geologic formations and on the properties of the discontinuities (geometric, mechanical and hydraulic).

There is a wide shadow zone occupied by the weak (soft) rocks / indurate soils, whose behaviour does not fall necessarily in either pattern and they require new laws to explain their rheologic performance. This has been clearly acknowledged about 30 years ago and weak rocks have been subject to specific studies since then and the results reported in many international conferences.

Being a subject of common geoengineering interest a Joint Technical Committee has been recently established, JTC 7 (Soft Rocks and Indurated Soils) which continued the work of other committees set up by each of the Societies years ago.

4. Education and Training

4.1. The role of the international societies

The three Societies have always been concerned with the development of their sciences, being aware that this is very much dependent on education (teaching at different levels) and training.

In general, teaching is best obtained in universities (short specialized courses also in research institutes), and training is best obtained by practice.

This explains the fact that the three Societies have established from their very beginning commissions and working groups to deal with this question and have introduced this subject as themes and workshops of Congresses, Symposia and other Conferences.

A brief reference is made in this lecture only to the Commissions on Education of each Society.

The IAEG appointed a Commission on "Teaching and Training in Engineering Geology" at the time of its 1st Congress in Paris in 1970. This Commission submitted its final report for publication in the Bulletin of the IAEG, after approval by the Council in 1978 (Dearman & Oliveira, 1978).

The report contained suggestions both for:

a) a 4 year undergraduate course on engineering geology.

b) a 1 year graduate course (MSc type) on engineering Geology.

Those suggestions were based on the concept expressed in the report that "the education of engineering geologists has to take into account the need for a good geological background and, at the same time, a knowledge of disciplines dealing with ground properties and an understanding of the behaviour of engineering structures; besides, it is of paramount importance that engineering geologists should have contact with actual engineering works as an essential part of their training".

The syllabus for the undergraduate course and for the graduate course were presented. At that time no concern about the environment was expressed in the proposal, since no geoenvironmental course or subject was mentioned in the report.

The training, preferably in industry, would result from a probation period of one to three years during which the engineering geologist should acquire experience on many facets of the profession.

After publication of this report the Commission was dorment for about 20 years and a new Commission on Teaching and Training in Engineering Geology was established in 1998. It was agreed that the new Commission should include Environmental issues within its remit and should update the report published by the previous Commission in 1978. Furthermore it should prepare a compilation of case histories illustrating savings obtained by using engineering geologists.

The Commission did not continue the work and no document was further produced.

The ISRM established a Commission in Teaching of Rock Mechanics in 1978 which published a Report in the International Journal of Rock Mechanics and Mining Science (ISRM, 1983). The Report contains a statement on the status of, and requirement for, rock mechanics education throughout the world made on the basis of data collected by means of a questionnaire circulated to Universities and other institutions in 1978 and 1979. On the basis of the data analysed, the Commission reached a number of conclusions and made recommendations for future action.

Apart from this, the Report contains a list of text books used in Rock Mechanics courses by the institutions which replied to the questionnaire. The most used are the well known books by (Stagg & Zienkiewicz, 1968), (Coates, 1970), (Jager & Cook, 1976), (Hoek & Bray, 1977), (Obert & Duvall, 1978) and (Goodman, 1980). No mention is made to the first Rock Mechanics text book by Talobre, published in French in 1956 (Talobre, 1956). After some years, the ISRM appointed a new Commission on Education in 1988 which was active until 1999, having presented then a Report in Paris. This report includes: a list of universities and colleges involved in teaching and research in Rock Mechanics, containing 540 entries from 83 countries; a Geotechnical Curriculum Guide; Bibliography of books, journals and videotapes; Educational videotape collection related to several engineering subjects; Educational software collection comprising a set of computer programs. Furthermore, the Commission reported on several initiatives to be continued like the Conference Travel Aid, the 1st ISRM Lecture Tour in China, Education in Numerical Methods for Geo-Engineering and Student Mobility.

For some years all that information was made available to those interested at the home page of the ISRM Commission on Education, namely the Geotechnical Curriculum Guide.

The ISSMFE created a Task Force on Education in Geotechnics in 1990 in co-operative effort with the sister Societies. These efforts continued with the set up of the Technical Committee (TC) 31 in 1994 on Education in Geotechnical Engineering. Since then the topic of education has been discussed in Several ISSMGE Conferences (some already mentioned in this paper). During the XIII ECSMGE, held in Prague, in 2003, a workshop was organized by TC 31 where several issues related to the Bologna Declaration and the changes in academic curriculae were discussed. Apparently this was the last activity of TC 31.

It is clear from the above considerations that Education has always been a major issue for the three sister Societies.

As a result of the increasing cooperation between them a joint Technical Committee (JTC 3) on Education and Training has been established in 2005, to function in accordance with guidelines defined by the Presidents of ISSMGE, ISRM and IAEG.

This Committee met twice in 2006 (Nottingham and Singapore) ant twice in 2007 (Lisbon and Madrid) and it is scheduled to meet in this Conference, in Constantza.

4.2. Training and professional practice

As said before, Education and Training have always been a matter of concern of the geoengineering Societies.

Mention has also been made that Training is best obtained by professional practice and that this training is best succeeded when there is interplay between Engineering Geology and Geotechnical Engineering. As a consequence of this evidence, a joint Technical Committee on "Professional Practice" (JTC 4) has been established by the three sister societies, following the formation of an European Working Group in 2002 for the definition of professional tasks, responsibilities and co-operation in Ground Engineering. This joint European Working Group presented a first report of its activities during the 1st EUROENGEO held in Liège in 2004 dealing with some points of the Terms of Reference which have been agred by the Working Group (Bock *et al.*, 2004).

At that European Conference other contributions on this subject were presented namely the papers from Katzenbach "Some basic considerations about the necessities and possibilities of cooperation between Civil Engineer and Engineering Geologists", from Norbury "Current issues relating the professional practice of Engineering Geology in Europe", and from de Freitas "The necessity of combining geologists and engineers for field work in the practice of Geotechnics".

The evidence of some professional practice is required in many European countries and countries from other continents as a condition to register as member of Associations of Chartered Engineers or Geologists, trying this way to assure the competence of the professionals. Field work, preferably carried out by joint teams (geotechnical engineers and engineering geologists) is considered as an indispensable activity to really put in evidence their skill as practitioners of geoengineering.

4.3. Perspectives

Along the years changes have been introduced in the syllabus of engineering and geological courses in order to progressively adapt them to the needs of the society. Concerning Geoengineering courses this is true for Civil Engineering, Geological Engineering, Mining Engineering and Geology.

In spite of the desire to have similar curricula and duration for each course in most countries of the world, the situation is still very different from region to region, from country to country and even from university to university in the same country.

The globalization of the economy and the mobility of scientists and engineers really call for an effort in the sense that the higher education and training received in school prepare geoengineering professionals as levelled as possible but as well that the level of such higher education satisfy the increasing demands on quality related to the development of the society.

At this point it is important to introduce the Bologna Declaration on European Higher Education signed by 29 countries in June 1999, aiming at the shaping of a higher education system similar in all those countries.

A paper by Seco e Pinto (2007) addresses several aspects of the Declaration and consequences of its application in Europe.

The new system is already operative in most countries and it has been applied in each country to the majority of public and private higher education institutions (Universities and Politechnical Institutes).

For the purpose of this lecture, the paper covers the following simplified version:

a) University Courses

b) Geoengineering formation in Civil Engineering, Geological Engineering and Geology

c) Full program of 3 cycles, the first degree (BSc) after 6 semesters, the second degree (MSc) after 4 more semesters and a third cycle of two semesters providing a diploma of advanced (specialized) studies and being a part of a PhD program for those so wishing.

The first cycle of university courses, corresponding to 6 semesters and 180 ECTS, provides the student basically with a solid scientific background (propedeutic courses) but it is short in geological and geotechnical information. This is a reason why their registration in professional associations is problematic, this issue being discussed now in the framework of European institutions.

In the second cycle, corresponding to 4 semesters and 120 ECTS, the education is oriented to more applied syllabus. Some universities offer specialized branches in this second cycle.

In the case of Civil Engineering and Geological Engineering courses, branches on Geotechnics are offered in several universities.

In the case of Geology, a branch in Applied Geology is offered in some universities and a branch in Engineering Geology in others.

These branches correspond to the last year of the cycle (9th and 10th semester of the MSc).

More specialized geoengineering education is only possible through studies offered in the third cycle as advanced post-graduate or specialized courses, with the duration of 2 semesters as said before. Courses on Geotechnics for Civil Engineering and on Engineering Geology are examples of advanced education.

These courses should preferably be followed by professionals already with some years of training in industry or research under the format of continuous education.

In this case, Civil Engineers, Geologic Engineers and Geologists may attend either course, their option depending on their MSc background and on the practical experience acquired.

The first cycle of engineering courses (Civil and Geological) must include at least one semester course in Geology for Engineers and another in Soil Mechanics.

Engineering Geology, Rock Mechanics and Applied Geoengineering courses (like Underground Construction, Earth Works, Retaining Structures, Foundations, etc.) have to be addressed in the second cycle.

The first cycle of Geology courses should offer semester introductory courses on Engineering Geology, Soil Mechanics and Rock Mechanics in the third year (5^{th} and 6^{th} semester). Applied geoengineering subjects related to natural resources, natural hazards, groundwater, environment, construction, etc are generally provided only in the last year of the second cycle (9^{th} and 10^{th} semester).

It is not the objective of this lecture to detail more as regards future orientation of the geoengineering education system or curricula.

However, the author would like to emphasise some topics which should be included in courses available in the second cycle and in the third cycle, taking into account their importance for the development of the geoengineering activities as they contribute to highlight the role of Geotechnics in the solution of many engineering and environmental problems.

Considerations about natural resources to be used as construction materials, geologic hazards (landslides, subsidence, erosion, earthquakes, volcanoes), impacts caused by engineering works (dams, railways, highways, canals, tunnels), waste disposal, and pollution of soils, rock masses and groundwater should be introduced to engineering students, since the problems associated with these subjects tend to increase as the societies develop.

Furthermore, risk analysis, safety, monitoring and quality control are some concepts which have to be present in the advanced education of geoengineers and scientists as they contribute to make easier the desirable interplay between professionals from both backgrounds and give more responsibility and credibility to their activity.

It is hoped that the work of the Committee JTC 3 will continue to follow the guidelines which have been established in November 2005 and keep monitoring the results of the new education system through enquiries to universities, to employees and to the students.

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Quality Index in the Environmental Management System in Urban Solid Waste Landfills - IQS

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Abstract. The article consists of the application of a new rating methodology for final disposal of urban solid waste (USW) by evaluating the conformity of geotechnical and environmental aspects during the implementation and operating processes. The IQS was proposed when introducing the concepts of Environmental Management, in accordance with ISO 14001, to the Landfill Quality Index (IQA) (Faria, 2002), developed from the Waste Quality Index (IQA) proposed by CETESB (2005). The study focused on the implementing and operating processes, as well as on the control of impacts on the environment and on pollution prevention. Fifteen sites were assessed to confirm the hypothesis. They were rated as inadequate, controlled, adequate and environmental conditions, in accordance with indices obtained with intervals between zero and ten points. It could, therefore, be concluded that in an inventory of rating USW disposal landfills, the use of ISO 14000 as an analytical tool may be extremely helpful to enhance assessment methods. Moreover, environmental management concepts contribute to reducing environmental pollution and, consequently, the associated environmental impacts.

Key words: urban solid waste, sanitary landfill, environmental management, ISO 14000.

1. Introduction

Among the different existing environmental problems, the USW has become one of today's major challenges. The fast growing population requires the production of goods and services, which, in turn, as they are produced and consumed, generate even more waste, their collection and disposal are inadequate and cause significant impacts on public health and the environment.

Waste disposal in landfills is quite common and is the technique most used due to its practicality and low cost. However, landfills cannot be considered merely a place to store waste. They must also be assessed as geotechnical projects, with the behaviour of the different stages of implantation, operation and degradation.

Adequate disposal of USW should be conveniently designed to include concepts relating to the engineering project, knowledge of geotechnics, field investigation and laboratory studies, also covering environmental, economic, political and social aspects, and requires a team of skilled professionals (Mahler & Lima, 2003).

The choice of the best site for final waste disposal is an even more complex problem, since it involves such factors as environmental, economic, transport logistics, structural safety and political (Mahler & Lima, 2003).

Mahler & Lima (2003) also find that the spaces for implementing landfills are becoming ever fewer, since, within the territorial boundaries of the counties, it is hard to choose suitable sites for disposal, which involves a thorough systematic study of various disciplines, such as Geotechnics, Hydrogeology, Hydrology and Climatology.

So this article proposes a set of systematised parameters as basis for structuring and formulating an index relating USW, environment, health and the human being.

Considering the above factors, this proposal was based on the introduction of management requirements, using the standard ISO14001 (Environmental Management System - Specification and Guidelines for use) as a criterion for adapting to the Landfill Quality Index (IQA) (Faria, 2002), through which the proposal is to assess final disposal and treatment of USW from the environmental management viewpoint.

Based on the hypotheses that, considering IQA rating parameters as shown in Tables 1, 2 and 3, a landfill rated "adequate" (or sanitary) by IQA will not guarantee treatment and disposal of its environmentally safe waste; and that the ISO 14000 will be a valuable tool to be used in the landfill rating inventory to verify environmental conditions, as well as aspects relating to the characteristics of the site, its infrastructure and operations.

For example, a landfill with no control, collection and leachate treatment (most significant negative environmental impact) and without effective monitoring of underground water bodies, is IQA-rated as adequate, with score 8.07.

Several underwater water pollution studies show that every uncontrolled landfill causes damage to the environment. Badly built sanitary or controlled, located or operated landfills can alter the quality of aquifers and air and, consequently, contaminate the soil, plants, animals and humans.

Next, the use of ISO 14000 is discussed as a management tool for operating USW landfills then the system adopted for evaluating landfills (IQS) is presented.

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Submitted on December 20, 2007; Final Acceptance on July 8, 2008; Discussion open until .

First, data collected on field visits from twelve USW disposal sites in the States of Rio de Janeiro, two in São Paulo, and one in Pernambuco, were used to apply this rating.

After applying the IQS, the rates obtained were compared analogically with the IQA (Faria, 2002) and IQR rates (CETESB-SEMA, 2005). With this comparison it was possible to confirm the hypothesis under consideration.

2. The Use of ISO 14000 in Urban Solid Waste Landfills

Standard ISO 14001 provided the necessary tools for developing the methodology, and certification requirements could be properly applied to the activities and processes of a sanitary landfill. The soil and subsoil management concept was also applied, consisting of erosion control, salinisation, desertification, proper handling of solid waste, and restoring degraded areas.

When administrating a solid waste sanitary landfill using an environmental management culture, committed to

Table 1 - Characteristics of site (Faria, 2002).

Characteristics of site		
Capacity to support soil	Adequate	5
	Inadequate	0
Permeability of soil	Low	5
	Medium	2
	High	0
Proximity to housing schemes	Far > 500 m	5
	Near	0
Proximity to water bodies	Far > 200 m	3
	Near	0
Depth of groundwater	Over 3 m	4
	1-3 m	2
	0-1 m	0
Availability of material for cover	Sufficient	4
	Insufficient	2
	None	0
Quality of material for covering	Good	2
	Bad	0
Conditions of road-traffic-access system	Good	3
	Regular	2
	Bad	0
Visual isolation from neighbourhood	Good	4
	Bad	0
Legality of location	Permitted entry	5
	Forbidden entry	0
Sub-total 1	Maximum	40

Table 2 - Implanted infrastructure (Faria, 2002).

Implanted infrastructure		
Fencing in the area	Yes	2
	No	0
Gateway/Cabin	Yes	1
	No	0
Control of receipt of cargo	Yes w/ weighbridge	2
	Yes /no weighbridge	1
	No	0
Access at front of work	Good	2
	Bad	0
Caterpillar tractor or compatible	Permanent	5
	Periodical	2
	Non-existent	0
Other equipment	Yes	1
	No	0
Impermeability of landfill base	Yes/unnecessary	5
	No	0
Drainage of leachate	Sufficient	5
	Insufficient	1
	Non-existent	0
Definitive storm water drainage	Sufficient	4
	Insufficient	2
	Non-existent	0
Temporary storm water drainage	Sufficient	2
	Insufficient	1
	Non-existent	0
	Sufficient	3
Gas drainage	Insufficient	1
	Non-existent	0
Leachate treatment system	Sufficient	5
	Insufficient/non-exist.	0
Monitoring underground water	Sufficient	3
	Insufficient	1
	Non-existent	0
Monitoring of surface water, leachate and gases	Sufficient	3
	Insufficient	1
	Non-existent	0
Monitoring soil and waste embankment	Sufficient	3
	Insufficient	1
	Non-existent	0
Fulfils design stipulations	Yes	2
	Partly	1
	No	0
Sub-total 2	Maximum	48

Table 3 - Working conditions (Faria, 2002).

Operating conditions		
Presence of wind-borne elements	No	1
	Yes	0
Daily waste cover	Yes	4
	No	0
Waste compaction	Adequate	4
	Inadequate	2
	Non-existent	0
Presence of vultures-gulls	No	1
	Yes	0
Presence of large quantity of boulder clay	No	2
	Yes	0
Presence of burnings	No	1
	Yes	0
Presence of waste collectors	No	3
	Yes	0
Livestock(cattle, etc.)	No	3
	Yes/nearby	0
Health service waste dumping	No	3
	Yes	0
Industrial waste dumping	No/adequate	4
	Yes/inadequate	0
Functioning leachate drainage	Good	3
	Regular	2
	Non-existent	0
Functioning definitive storm water drainage	Good	2
	Regular	1
	Non-existent	0
Functioning temporary storm water drainage	Good	2
	Regular	1
	Non-existent	0

Operating conditions		
Functioning gas drainage	Good	2
	Regular	1
	Non-existent	0
Functioning leachate treatment system	Good	5
	Regular	2
	Non-existent	0
Functioning underground water monitoring system	Good	2
	Regular	1
	Non-existent	0
Functioning surface water, waste and gas monitoring system	Good	2
	Regular	1
	Non-existent	0
Functioning embankment stabilising monitoring	Good	2
	Regular	1
	Non-existent	0
Corrective measures	Yes/unnecessary	2
	No	0
General data about landfill	Yes	1
	No/incomplete	0
Maintenance of internal access	Good	2
	Regular	1
	Very bad	0
Landfill shutdown plan	Yes	1
	No	0
Sub-total 3	Maximum	52

prevent and reduce pollution, and employing properly trained skilled manpower, it is essential to adopt an environmental management system that includes an organizational structure, planning activities, responsibilities, practices, procedures, processes and resources to comply with an established environmental policy.

This system can be certified, which shows stakeholders how seriously this administration deals with the environmental question; however, ISO 14001 certification does not necessarily imply good environmental performance of the practices, processes, and compliance with the established environmental policies, as provided by ISO 14031. This standard was not used in this study, since the environmental performance assessment (EPA) is only specific for each landfill, when the objective was to have a general rating for them all.

The purpose of certification is to attest that the management system can produce results but without specifying the velocity at which these results will appear. This mistaken routine may lead waste processing to polluting practices, even though the environmental management system is in operation.

3. Landfill Assessment System

Not only the social but also the environmental, sanitary and public health aspects are to be considered (Mahler & Lima, 2003). The Value Analysis Theory was therefore used (Csillag, 1995) as a multi-criterion comparative analytical tool for the coherent converging of these variables, creating weights for the different aspects addressed, and then the Quality Rating of the Environmental Management System for Urban Solid Waste Landfills (IQS) was established.

Considering the discussion herein, the standard NBR ISO 14001 was divided into ten parameters, as follows:

1. Identification of significant environmental aspects and impacts (A);

2. Objectives, goals and environmental programmes (B);

3. Guarantee of necessary resources - humans, technological and financial (C);

4. Training system - competence, consciousness - and internal and external communication (D);

5. Control of SGA documents, registration (E);

6. Emergency plans and programme (F);

7. Control, monitoring and measuring of operations - relating to significant impacts (G);

8. Meeting legal and other approved requirements (H);

9. Internal audit programme (I); and

10. Critical analyses by the administration - considering internal audits, laws, communication, objectives, goals and environmental programmes - corrective and preventive actions - to mitigate impacts (J).

The IQS comprised four groups, the first three being taken from IQA (Faria, 2002) and the fourth added to the rating: Site Characteristics, Implanted Infrastructure, Working Conditions, and Environmental Management System (Table 4).

The first three groups had no alteration and the Value Analysis was therefore not applied. In the last group, when weighting the new parameters, the functions used were the ten listed above, applying the Functional Assessment Matrix (Table 5). This technique permitted each function to be

Table 4 - Environmental Management Assessment parameters (Loureiro, 2005).

Environmental management		
Identify environmental aspects and impacts	Satisfactory	5
	Insufficient	2
	Non-existent	0
Environmental objectives, goals and programmes	Consistent	3
	Inconsistent	1
	Non-existent	0
Guarantee of req. resources.	Sufficient	2
	Insufficient	0
Training & com. system	Efficient	2
	Inefficient	0
Control of docs. and records	Yes	1
	No	0
Emergency programme and plans	Sufficient	4
	Insufficient	2
	Non-existent	0
Control, monit. & measuring of ops.	Effective	4
	Ineffective	0
Complying with legal & other reqs.	Yes	5
	No	0
Internal audit programme	Satisfactory	2
	Ineffective	1
	Non-existent	0
Critical analyses & cor. & prev. action	Consistent	2
	Inconsistent	0
Sub-total 4	Maximum	30

Table 5 - Functional assessment matrix (Loureiro, 2005).

	B	C	D	E	F	G	H	I	J	Total	%	Pt
A	A1	A3	A3	A3	0	A2	H1	A1	A1	14	17.073	5
	B	B1	B2	B2	F2	G2	H3	I2	B2	7	8.537	3
		C	C2	C2	F1	G1	H2	C1	C1	6	7.317	2
			D	D3	D1	G1	H1	D2	J2	6	7.317	2
				E	F3	G2	H3	I3	E2	2	2.439	1
					F	0	F2	F2	F2	12	14.634	4
						G	G1	G2	G2	11	13.415	4
							H	H2	H2	14	17.073	5
								I	J3	5	6.098	2
									J	5	6.098	2
									Total	82	100.00	30

compared with the others, determining at every moment its importance by weighting between 0 and 3 points.

At the end of the comparison, the weights attributed to each function were added up to determine their percentage in relation to the total weights of all functions. Following the IQA criterion of the maximum five-point scoring, the most relevant functions were given this score and the others with less proportionately.

4. IQS Application oo Landfills

The three quality indices (IQR, IQA and IQS) were applied to the 15 (fifteen) waste dumps visited. Part of the area studied among the 12 (twelve) sites visited in the State of Rio de Janeiro corresponds to the middle stretch of the Paraiba do Sul river basin (Fig. 1), and the rest to the Greater Rio Metropolitan Region.

The objectives for choosing these sites were: to compare the current results with those obtained previously by Faria (2002); to present the situation in other disposal areas, in order to have a broader view of local waste management in the State of Rio de Janeiro, and to compare its reality with that of other States.

With the development of this technique, the scoring of the parameters introduced in the IQS was successfully achieved as shown in Table 6.

Figure 1 - Rio de Janeiro State and Paraiba do Sul river basin (UNDP, 1999, in Faria, 2002).

Table 6 - Final IQS rating (Loureiro, 2005).

Total (1+2+3+4)	170
IQS = Sum of scores / 17	
IQS	Assessment
0-6.00	Inadequate conditions (landfill or dump)
6.01-8.00	Controlled conditions (controlled landfill)
8.01-9.00	Adequate conditions (sanitary landfill)
9.01-10	Environmental conditions (environmental landfill)

It was decided not to explicitly refer to the sites assessed for political-administrative questions and interests. A more detailed description of the characteristics of the case studies can be obtained in Loureiro (2005).

In the State of Rio de Janeiro, not only the population of around 2.5 million but also 700 or so industries, various hydroelectricity plants and irrigated agriculture depend on water from this basin. In the Greater Rio Metropolitan Region, approximately eight million inhabitants are supplied from collecting 44 m^3/s from the Guandu River and 5.5 m^3/s from Lajes reservoir, deriving from two transpositions of the Paraiba do Sul river basin. Approximately 160 m^3/s is taken directly from Paraiba do Sul River using the Santa Cecilia pumping station and 20 m^3/s is used from the Pirai river basin (Faria, 2002).

The assessment reports (IQR, IQA and IQS) were completed based on information collected from visual inspections on the sites, some local government data, solid waste landfill operators at each site, professionals in the solid waste sector, and from consulting other reports and papers relating to these landfills.

After accomplishing all work routines, the findings were reported, converging on a weighted average, and a consensus was obtained where each parameter was graded, defining its level of satisfaction, attendance, conformity, effectiveness and/or efficiency.

The main type of soil found in most sites under study was latosol. This soil has a clay fraction of kaolinitic minerals with a high concentration of iron and aluminium. In natural conditions, it is non-saturated, with a high rate of voids and little field capacity, but when suitably compacted it can reach a high supporting capacity with low permeability. These characteristics make the latosol suitable material for daily covering the landfill and base layer and for impermeability (Faria, 2002).

It should also be explained that all sites were called "landfill", regardless of the rating in the assessments, in order of visits.

5. Analysis of Results

During this study, urban solid waste management models were observed in fifteen sites, whose results are given in the Table 7 below.

As can be seen, only one landfill was rated environmental and another rated adequate, which was to be expected, given the strict IQS assessment compared to IQA and IQR. Another three sites were rated controlled and the remaining ten inadequate.

From the landfill rating criteria adopted by IQA and IQR (Fig. 2), 46% of the landfills assessed were rated inadequate, 27% controlled and the other 27% adequate. Now considering the IQS assessment criteria (Fig. 3), the number of landfills in inadequate conditions increased to 66%, and consequently those in controlled conditions dropped to

20% and adequate to 7%, only 7% remaining rated as environmental.

Accordingly, the proposed methodology showed a 20% increase in the quantity of inadequate landfills (dumps), and a 20% drop in the number of adequate landfills (sanitary), confirming the hypothesis that from the previous methodologies, a landfill in adequate conditions did not necessarily maintain environmentally correct operations.

From examining the following graphs, it could be said that the IQS assessment is most restrictive, since the rating in each case was lower than in the other forms of assessment, except for landfill 07, due to the SGA inserted throughout the waste treatment process and disposal, scoring in environmental parameters not assessed by the other methodologies.

Table 7 - Result of landfill quality index assessments (Loureiro, 2005).

Landfill	IQR	IQA	IQS	Rating
01	6.62	6.86	5.76	Inadequate
02	2.31	2.36	1.94	Inadequate
03	6.15	6.43	5.53	Inadequate
04	9.62	9.50	8.71	Adequate
05	3.54	3.64	3.06	Inadequate
06	7.54	7.00	5.94	Inadequate
07	8.77	9.00	9.18	Environmental
08	8.31	8.07	7.59	Controlled
09	2.77	2.64	2.35	Inadequate
10	1.08	1.07	0.88	Inadequate
11	9.08	8.86	7.88	Controlled
12	4.38	4.07	3.41	Inadequate
13	2.00	1.86	1.53	Inadequate
14	6.92	7.29	6.35	Controlled
15	2.54	2.50	2.12	Inadequate

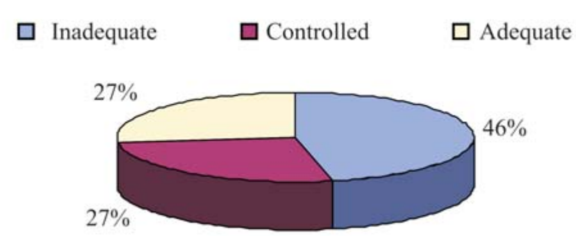

Figure 2 - Rating by IQA and IQR (Loureiro, 2005).

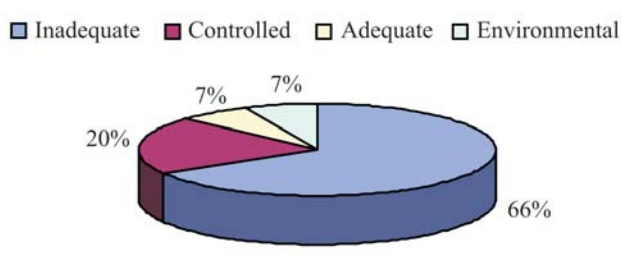

Figure 3 - Rating by IQS (Loureiro, 2005).

6. Conclusions and Final Comments

Following one of the proposals for continuing the line of research on the Waste Treatment Group (GTRES) in the Geotechnics area of COPPE, the objective of this work was to evaluate the degree of conformity of the rating criteria of USW landfills from the environmental management viewpoint, based on ISO 14001 requirements, in terms of implementation, operation and closure of landfills, and interactions with the environment.

Therefore, by applying the IQS assessment methodology in the fifteen case studies, it was possible to confirm the two hypotheses under discussion: that the IQA adequate rating does not guarantee environmentally secure USW treatment and disposal, and that NBR ISO 14001 is a valuable tool that can be used in a landfill rating inventory.

To fully attend IQS environmental parameters when adopting the NBR ISO 14001 requirements, it is fundamental to continue in compliance with the prevailing environmental laws, and to monitor and maintain previous standards of environmental quality, inherent in the aspects and impacts surveyed in the area next to the landfill. This tool and the other standards of the ISO 14000 family may be used to regulate waste disposal throughout Brazil.

The geotechnical parameters (support and permeability of the soil, availability and quality of cover material, compaction, drainage systems, embankment stability, remediation, closure, etc.) were found to be important in several aspects of final waste disposal, since the landfills will inevitably be based on the ground and may be protected by it using an adequate cover. Moreover, it is of the utmost importance to guarantee environmental quality, proper liner compaction, groundwater depth, and permeability of the foundation soil, since this is the main route taken by liquid contaminant, namely, leachate.

Also, like water, soil is becoming ever more important especially since indiscriminate use of spaces and borrow material for partial or final cover will be increasingly scrutinised, considering not only the environmental but also economic, social and financial aspects.

It should be pointed out that Clean Development Mechanism (CDM) concepts can and should be used in the waste sector as an economic-financial support when implementing and operating sanitary landfills, and adopting compost processes, incineration or other procedures such as drying waste, for example, to reduce greenhouse gas emissions to a minimum. Monitoring procedures, collecting and using gases can be done by burning them to generate energy. These aspects shall be included in the assessment system of midsize and large landfills in future procedures, since they are being discussed and further implemented in Brazil.

Lastly, in addition to the need to use skilled personnel, with know-how and multidisciplinary experience, it must be stressed that different landfill assessment systems should be considered and used in accordance with their different sizes. Basic aspects are similar in small, midsize and large landfills, but there are particular features inherent to the size of a landfill, which recommend major differences in assessment systems. Otherwise, distortions may occur since it may be impossible to adopt procedures that are absolutely possible and necessary in some other large landfill.

Acknowledgments

The first author thanks CNPq, CAPES, FAPERJ and DAAD for giving their continuing support to his work. The authors thank the various local governments and their staff for their support during the field visits, and Eng. José Ricardo Ferreira for the technical revision of the text.

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Case History

Soils and Rocks
v. 32, n. 1

Slope Failure Reanalysis of a Dilatant Dense Sand From CPTU and Laboratory Tests

U.F.A. Karim, A. Menkveld

Abstract. Geotechnical properties of dense to very dense sands at Channel Island Harbor in Oxnard, California, are re-evaluated after slope failures to select appropriate strength parameters for remedial and extension works. The revetment is excavated at a slope of 2:1 (horizontal: vertical) in estuarine and dune sands and covered with a geotextile filter and armor rock. Reanalysis of the slope stability explores laboratory and in situ strength conditions of peak, constant volume and residual angles for the sands. A stress-dilatancy theory provides the framework used in re-characterization of the soil and observed slope failure mechanisms. Effective friction angles are recommended for further slope stability analysis at that location.

Key words: slope stability, dense sand, site characterization, shear angle, dilatancy, case history.

1. Introduction

1.1. General

Re-evaluation of slopes and foundations that have collapsed provides insight into working frictional conditions, as long as the effective stress state in the soil over the life of the structure is reasonably well known. In this study, re-characterization of a slope failure in dense to very dense near surface sands for a harbor project at the Channel Island Harbor, Oxnard, California, led to improved assessment of geotechnical properties at the site, particularly effective stress friction angle, ϕ'. Selection of the appropriate friction angle (peak or large strain) is crucial in slope stability calculations, and this case study helped to provide some guidance for evaluating the available long-term strength of the soils.

After a brief description of the project, this paper will discuss results of laboratory and in situ tests analyzed under the critical-state framework adopted for soil re-characterization. Within that framework drained triaxial compression and direct shear testing are the most appropriate. Laboratory index tests of the sand are also used, primarily to compare sands at this site to standard reference sands and for the purpose of developing global correlations. The triaxial tests on reconstituted specimens placed at a range of relative densities of concern for this project are compared to stress-dilatancy relationship from Bolton (1986), to assess the applicability of that relationship as well as compressibility of the sand. Cone penetration test (CPT) data is used to evaluate profiles of relative density and peak effective stress friction angles, based on conventional correlations, for comparison with profiles of peak effective stress friction angles estimated from dilatancy relationships.

The use of peak or constant volume friction angles in re-design studies is further evaluated in limit-equilibrium calculations of slope stability. The friction angles obtained from critical state interpretations of laboratory and in situ test results that reproduce more closely the slope failure are recommended for use in further design.

1.2. Project background

The Channel Islands Harbor (CIH), located in the city of Oxnard, California, USA, was realized when two basins were constructed in the early 1980's by cutting slopes into the existing banks. The basins were excavated in the dry behind a temporary containment dike, and the dike was removed after all grades were established and slopes protection was in place. It is noted that during the grading of the site, sand dunes were flattened, creating an over consolidated profile of the upper sand layers. The slopes were originally designed at an angle of 2:1 (horizontal: vertical) as shown in Fig. 1. Slope protection was constructed by placing armor rock directly on top of a woven geotextile. After the construction of the basins was completed, the uplands soils were graded and improved for development of parking areas, sidewalks, landscaping and building improvements.

The original slopes were designed to a factor of safety of 1.5, which would correspond to using a friction angle of about 40° (Noble, 1996). By the mid-1990's portions of the slopes failed and sediments accumulated at the base. These sediments caused the floating dock sections to ground on the shoals at low tide. Redesign and stabilization for further site extension became necessary as a result of those events.

Selection of appropriate stabilization methods for the current slopes and identifying potential issues for future extensions of the harbor area necessitated geotechnical re-characterization of the old slopes. Evaluations of stratigraphy based on piezocone (CPTU) testing around the perimeter of the harbor identified 3 generalized profiles in the area of concern for stability analyses; (i) alternating sand and

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Submitted on March 6, 2008; Final Acceptance on July 18, 2008; Discussion open until August 31, 2009.

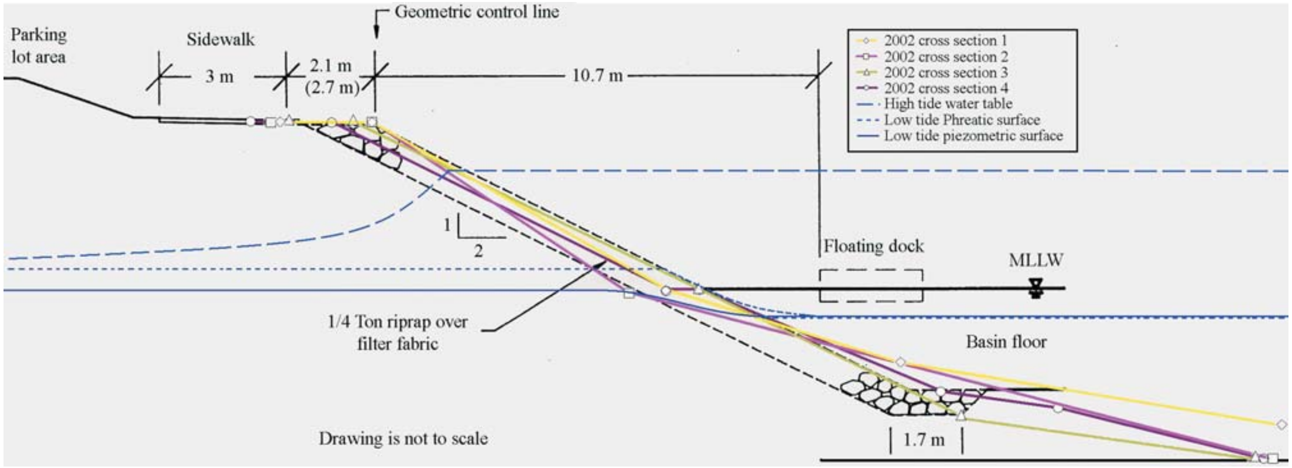

Figure 1 - Original slope design overlain by 2002 topography and range of tidal water conditions.

clay layers of approximately 2 to 3 m thickness; (ii) a continuous dense to very dense sand layer throughout the height of the slope; and (iii) interlayered sand and clay with thickness less than approximately 1 meter. The extents of these three generalized profiles are shown in Fig. 2. Since the continuous dense to very dense sand layers corresponded to areas of greatest slope distress, characterization of the dense to very dense sand and associated slope stability is the focus of this paper.

2. Site Characterization of Sand

Figure 2 - Extents of generalized soil profiles overlain on aerial photograph of site (Fugro, 2002).

2.1. Laboratory testing

The laboratory test program focused on evaluating index and shear properties of the sand for comparison to standard reference sand and interpretations using a critical state framework. Isotropic drained triaxial compression tests on specimens reconstituted at the appropriate relative densities D_r (50-100%) were conducted and the strength dilatancy theory as applied to this sand is assessed. The laboratory index tests are discussed in detail in Menkveld (2002) and include particle size analyses, microscopic evaluation of roundness, specific gravity, minimum index density, and evaluation of the ultimate state friction angle using simplified procedures (Cornforth, 1973; Santamarina & Cho, 2001). Average index properties for the CIH sands are presented in Table 1, as compared to those of standard reference sands. The simplified procedure for evaluating ϕ'_{cv} is shown in Figs. 3a and 3b.

The CIH sand is slightly finer grained than typical reference sands that form the basis of many CPT correlations for sands, but has a similar range of maximum and minimum void ratios. Additionally, the sand contains about 10% fines, which would classify the material as sand with silt, SP-SM under the Unified Soil Classification System (USCS). The higher fines content and smaller D_{50} (equivalent to average diameter for 50% of the sand) would generally lead one to consider the soil as having slightly higher compressibility than the reference sands, but the sub-rounded grains and predominantly sand matrix may lead the soil to behave in a similar manner to the low compressibility Monterey sand under drained penetration. The ultimate state friction angle (ϕ'_{cv}) determined by simplified procedures resulted in a value of approximately 34°, as shown on Fig. 3b. It was observed that some silt particles remained in suspension during the test, which may have resulted in slightly high value of ϕ'_{cv}. This may not be a significant problem in sands with silt, but in silty sands the simplified method is likely not representative of the actual

Table 1 - Grain characteristics of Chanel Islands Harbor sand to reference sands.

Sand[1,2]	D_{50} (mm)	D_{10} (mm)	Fines (%)	C_u	e_{max}	e_{min}	Angularity[3]	Compressibility
CIH_{mean}	0.25	0.07	10	3.4	0.92	0.51	SR	Low
Ticino	0.50	0.41	5	1.6	0.92	0.57	SA to A	Medium
Hokksund	0.39	0.21	5	2.2	0.88	0.54	SA to A	Medium
Monterey 0	0.37	0.25	5	1.6	0.82	0.54	SR	Low

[1]Properties for Ticino, Kokksund and Moterey 0 sand from Kulhawy & Mayne (1990).
[2]Chanel Islands Harbor (CIH) sand properties taken as the mean from laboratory testes. Varience of 5 to 10 percent was observed for D_{50}, e_{max} and e_{min}, and a varience or approximately 20 percent was observed for D_{10}, Fines and C_u.
[3]SR = Subrounded; SA = Subangular; A = Angular.

behavior, and ϕ'_{cv} values on the order of 27 to 30° could be expected (Bolton, 1986).

Bolton recognized that shear strength of cohesionless soils is related to the rate of dilation at failure, which in turn depends on the relative density D_r, the level of mean effective stress p' and soil compressibility. Therefore he developed an empirical method in which he uses Rowe's dilatancy concepts. Bolton showed that the peak secant friction angle ϕ'_{sec} for many sands could be estimated from triaxial tests using the expression:

$$\phi'_{sec} - \phi'_{cv} = 3\,D_r\,(Q - \ln p'_f) \qquad (1a)$$

For plane strain he found that,

$$\phi'_s - \phi'_{cv} = 5\,D_r\,(Q - \ln p'_f) - 1 \qquad (1b)$$

where p'_f (or p'_{ff}, as used in Bolton 1986) is the mean effective stress at failure, and Q is a constant with a value depending on the compressibility and mineralogy of the sand. Bolton suggested a general value of $Q = 10$ for most silica sands.

Drained triaxial compression tests on a dense and medium dense specimen were evaluated using the Bolton (1986) stress dilatancy theory (Eq. (1)). Figure 4 compares laboratory results of CIH sand to trends presented in Bolton (1986). Bolton recommended using a Q value, which is indicative of the soil compressibility, of 10 for quartz and feldspar sands, and lower values for more compressible materials. Triaxial test results for this study agreed well with the Bolton (1986) theory, and seemed to match best with a Q value of 11, indicating a low compressibility soil. It should be noted that a low number of triaxial tests were performed, which limited evaluation of the influence of confining stress, an important aspect of stress dilatancy theory.

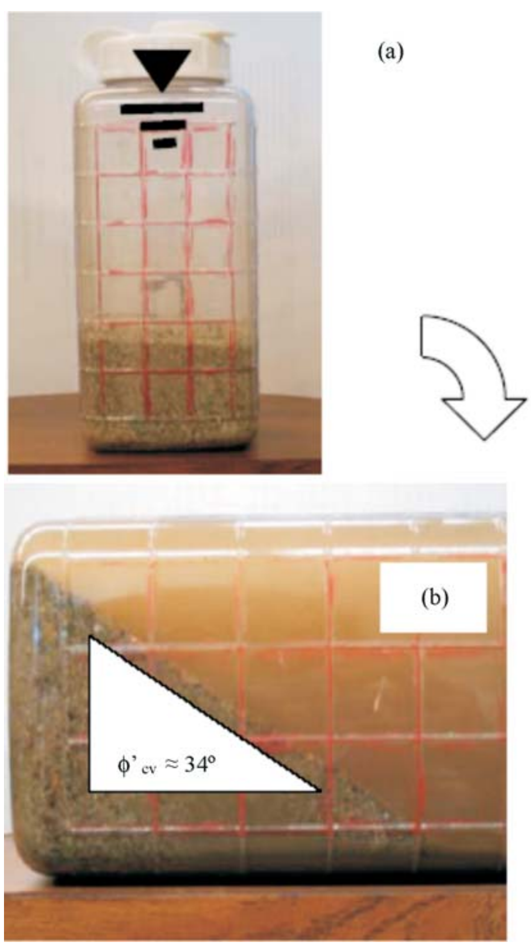

Figure 3 - Method for estimating ϕ'_{cv} based on Cornforth (1973).

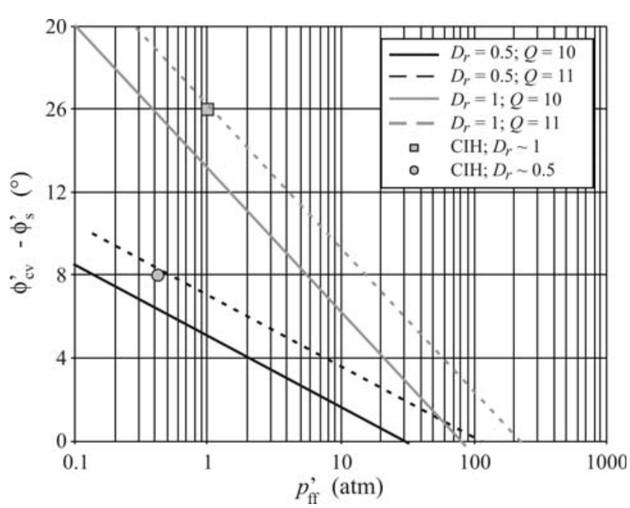

Figure 4 - Evaluation of CIH triaxial data using Bolton's (1986) stress dilatancy theory with $\phi'_{cv} = 34°$ (1 atm = 1 bar = 101.3 kPa).

2.2. *In situ* testing

Sixteen CPTs were performed around the perimeter of the site to generate the zones of generalized soil profiles presented in Fig. 2. As mentioned previously, this study focused on the dense to very dense sands of Typical Profile 2. Also, since the height of the sand dunes were reduced during grading operations, the upper sand profile is over consolidated, which was taken into consideration during analyses. The influence of over consolidation primarily influences cone penetration resistance through an increase in horizontal stress.

Five of the CPTs performed are included under Typical Profile 2, and, due to space constraints, only two of those CPTs will be presented in this section. CPTU-6 and CPTU-11 are located on the west side of the basins, and are separated by approximately 500 m. Analyses related to characterization of these profiles included assessment of relative density, and peak effective stress friction angle. The upper 7 m of the profile is of primary concern for assessment of slope stability issues, and the upper 1 meter is left off plots since it is primarily artificial fill.

Analysis of relative density of the profile was initiated by using correlations presented Baldi et al. (1986), which were primarily based on results of calibration chamber tests in Hokksund and Ticino sands. To account for over consolidation of the deposit, the relative density (D_r) is correlated to penetration resistance normalized by mean effective stress, and over consolidated profiles have an higher cone tip resistance at the same depth than normally consolidated profiles for an equivalent D_r. Profiles of CPT tip resistance (q_c) and estimates of relative density for the dense (D_r = 65%) and very dense (D_r = 100%) states are presented on Fig. 5. Despite being finer grained and having a higher fines content than the reference sands used for the Baldi *et al.* (1986) correlation, the recorded q_c values were 10 to 20 MPa higher than the Baldi *et al.* (1986) correlated value for 100 percent relative density. While at this point it is obvious that the upper sand is very dense, it is desirable to understand the large discrepancy for predicted and observed q_c values, and how this influences the interpretation of sand deeper layers.

Also shown on Fig. 5 are relative density correlations based on equations presented in Mayne & Kulhawy (1995) relating relative density to cone tip resistance normalized by horizontal stress. This correlation encompasses a much larger database of different sand types, with scatter often associated with the compressibility of the sand. The mean value of the correlation plots as the lower bound of the q_c values, with the mean plus 2 standard deviations plotting as the upper bound of the data. From this figure it is inferred that there is significant variability in evaluation of relative density from conventional correlations related to the compressibility of the soil. Additionally, without proper characterization of the soil compressibility, relative density may be significantly overestimated. The variability in correlations seems to be much more significant in dense to very dense soils.

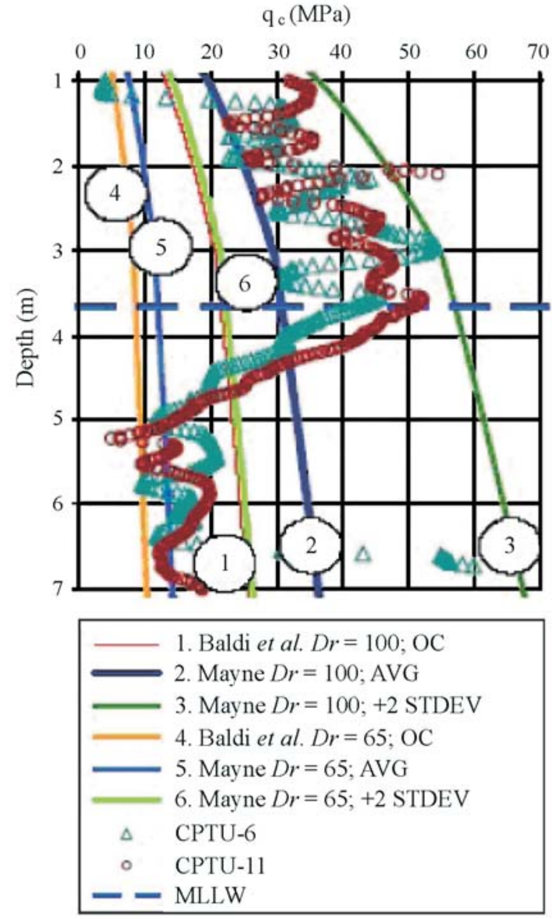

Figure 5 - Range of relative density estimates using typical correlations.

For design purposes, it is desirable to have an estimate of the peak effective stress friction angle. To develop peak strength profiles, correlations between ϕ'_{peak} and q_c as presented in Kulhawy & Mayne (1990) are utilized. Additionally, the influence of soil compressibility on cone tip resistance will be better matched by a change in effective stress friction angle than by a change in relative density (Robertson and Campanella, 1988). So the correlation between ϕ'_{peak} and q_c may be applicable over a larger range of sands than D_r correlations. With a knowledge of ϕ'_{cv} (Fig. 3b) and application in a stress dilatancy framework, profiles of effective stress friction angle can be used to assess relative density of a deposit. The dilatancy component of friction angle for the stress dilatancy theory was calculated by starting at the mean effective stress for a certain depth and moving towards the failure envelope along the 3:1 triaxial stress path. While it is acknowledge that the stress path during cone penetration will be different than triaxial compression, since the q_c-ϕ'_{peak} correlation is based on triaxial test data and the stress dilatancy theory is based on triaxial test data, this is a reasonable stress path to follow for the analyses performed.

Profiles of ϕ'_{peak} are shown for CPTU-6 and CPTU-11 on Fig. 6, along with profiles of ϕ'_{peak} based on stress dilatancy theory for dense and very dense states. The estimates of ϕ'_{peak} from q_c values tend to match well with stress dilatancy theory for a Q value of 11, if the upper soil is classified as very dense and the lower soil is classified as dense. Other than developing a profile of peak friction angle, the combined assessment of relative density and effective stress friction angle analyzed under a stress dilatancy framework, can be used to verify that a constant volume friction angle of 34° is reasonable for this deposit. It should be noted that the mean lower low water table MLLW shown in Figs 5 and 6 is based on high and low tide values (see Fig. 1). This table within the soil slope coincides approximately with the piezometric measurement and shows the relative position from which the cone tests were made at about 1.2 m above the high tide water table.

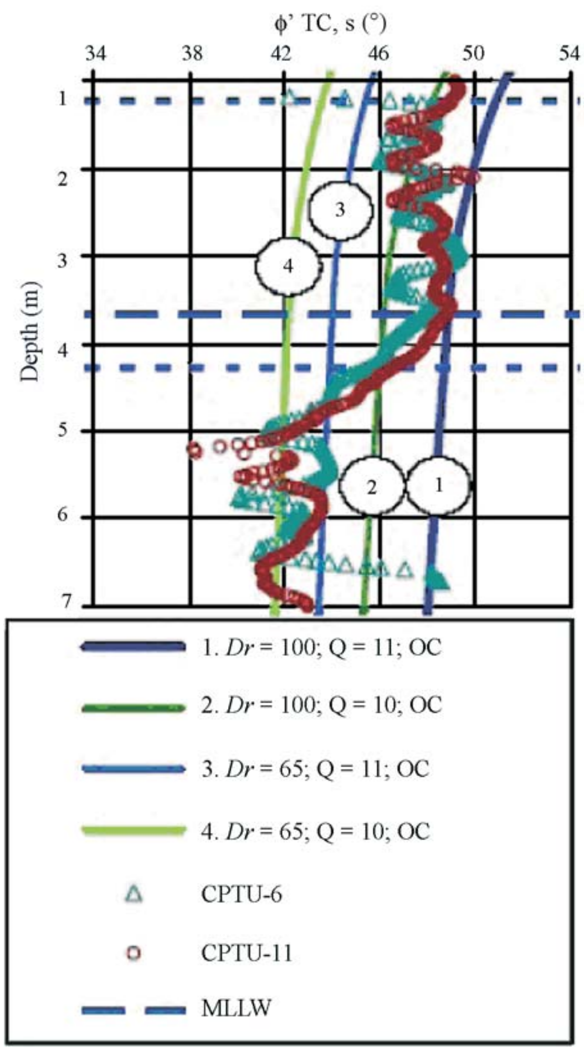

Figure 6 - Range of peak effective stress friction angle as a function of relative density (MLLW: mean lower low water is at approximate depth of low tide piezocone surface as shown in Fig. 1).

3. Slope Stability Analyses

Slope stability analyses are performed using a range of effective stress friction angles developed from the site characterization activities. Slope stability analyses were performed using plain strain and limit equilibrium theory as incorporated into the computer program SLOPE/W[1]. Strength parameters associated with computed critical slip surfaces that match observed failure surfaces are assessed within the previously mentioned stress dilatancy framework to determine the most suitable friction angle under working conditions.

SLOPE/W input parameters include range of friction angles, slope section, water table (high and low tide). The range of friction angles used as input are the peak friction angles obtained from triaxial and direct shear tests, the ultimate friction angles obtained with the Cornforth (1973) method (both the lowest, 31° and the mean value 34° are used). The top sand layer, being denser than the underlying sand, is therefore designated the higher values from the range of friction angles, except when both layers are at the ultimate state. Using secant effective stress friction angle of 50° in the very dense sand and 42° in the dense sand resulted in a factor of safety of approximately 2.0. When the effective stress friction angle was reduced to the ultimate state value of 34° and 31°, factor of safety values of 1.3 and 1.1 were calculated, respectively.

4. Discussion

Friction angles calculated from the dilatancy relationships presented by Bolton (1986) have led to a unified framework that matches laboratory test data, in situ test data, and field performance of a slope failure. As shown in Fig. 6 there is good agreement between the CPT estimates of ϕ'_{peak} and ϕ'_{peak} from stress dilatancy theory. It is therefore inferred from the CPT analyses that the ultimate state friction angle of approximately 34° can be a reasonable value for use in slope stability analyses. This is confirmed by the results of the laboratory tests presented in Figs. 3b and 4.

Due to the cyclic nature of the tidal loading on the slopes, design of remedial measures and future expansion based on slope stability in these sands, use of the ultimate state friction angle, ϕ'_{cv}, equal to 34° for the both the dense and very dense sands is recommended. This is a reasonable choice given the several complicating factors that need to be considered to accurately estimate working parameters for design. Cyclic tidal loading and seepage conditions are among the most important factors, as well as construction methods and materials (*i.e.*, armor rock and filter fabric) used.

1 Geo-Slope International, Calgary, Alberta, Canada, developed the SLOPE/W program.

The safety factor of the re-constructed slopes, with respect to a peak friction angle, may gradually decreased with time due to a number of factors. As noted earlier, failure took place after slope deterioration spanning 10 years. Ultimate state conditions may have been reached after some years because cyclic high and low tides (to factor of safety values ranging between 1.5 and 2.0), and seepage conditions have gradual effects on the initial slope properties. Numerous loading cycles generate enough inelastic strains so that a point on the post-peak part of the stress-displacement curve is approached. Consequently, the sands are eventually controlled by the ultimate state. The original design friction angles of about 40° are reduced due to these effects by 6° leading to reduction in the safety factor from 1.5 to 1.0. It is possible that the designers overlooked this long-term effect and therefore assumed higher friction angles.

5. Conclusions

The primary conclusions of this study are:

• Slope stability failures are possible in dense to very dense sand deposits designed to a factor of safety of 1.5 using peak effective stress parameters, especially if subjected to transient loads, such as tides.

• Relative density correlations that underestimate the compressibility of the soil will overestimate the relative density of the deposit, and possibly lead to unconservative analyses.

• The cone penetration resistance seems to be strongly related to horizontal stress and a dilatancy component, and the normalizing cone tip resistance by mean stress may lead to overestimates of relative density in over consolidated deposits.

• Evaluating relative density based on correlations to ϕ'_{peak} assessed under a stress-dilatancy framework with lab estimates of ϕ'_{cv} seem to be less sensitive to soil compressibility. These relationships can be further calibrated using a limited number of reconstituted triaxial compression tests to further evaluate the soil compressibility.

• Evaluation of CPT tip resistance using empirical correlations and simple laboratory tests can provide insight into the peak and large strain friction angle of the soil.

• Back calculation of slope failure conditions using limit equilibrium analyses provided mobilized friction angles consistent with ϕ'_{cv} values estimated from simplified laboratory index tests and verified using CPT data analyzed under stress-dilatancy theory. The analyses also highlighted that pore pressure conditions and construction materials and methods play an important role in the occurrence of shallow failures.

Acknowledgments

Thanks are extended to Fugro West, Inc., Ventura, California, USA for providing facilities, data, and supervision for this research work, and to the Ventura County Harbour Department for their support during this project. The contributions during the project of J.A. Schneider and P.N. Robins formally with Fugro West, are gratefully acknowledged.

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Notation

Cu = undrained shear strength

D = grain diameter in mm (subscripts 10% passing, 50% passing)

D_r = relative density

e = void ratio (subscript min = minimum and max = maximum)

M = strength parameter

p' = hydraustatic mean effective stress (subscripts f or ff at failure)

q_c = cone tip resistance

Q = value indicative of soil compressibility (stress dilatancy theory, Bolton,1986)

ϕ'_{peak} = maximum effective stress friction angle (from laboratory shear tests)

ϕ'_{cv} = effective stress friction angle at constant volume (laboratory shear test)

ϕ'_{TC} = peak effective stress friction angle (from a CPT cone test)

ϕ'_{sec} = peak secant friction angle

Abbreviations

AVG: Average
CPT: Cone penetration test
CPTU: CPT with pore water pressure measurment
CIH: Channel islands harbour
MLLW: mean lower low water
OC: Over consolidated
STDEV: standard deviation

Obituary - Victor Froilano Bachmann de Mello

Son of a Professor Medical Colonel M.P. and a German Swiss mother Victor de Mello was born in Goa, Portuguese India, in 14 May 1926, attended British boarding school in India, moved to Boston in 1944; as a brilliant student at the MIT he obtained both his BSc and MSc in 1946 and his doctoral degree in 1948. He immigrated to Brazil in 1949 to be a Brazilian, both because of deep-rooted cultural affinities with Goa, and because of the nostalgic challenges of unopened frontiers of tropical civil engineering. It is in Brazil and from Brazil that Victor has grown from his strong roots into a big tree, spreading his teachings to the four winds and the fruit of his works through countless projects built.

His academic skills nourished with Donald Taylor, leading to a marked influence in MIT's shear strength and in the stabilization of clays research projects, the later granting him US Patent 2.651.619.

His enthusiasm in civil engineering involved action and creation on behalf of society, leading him to accept the invitation of Light and Power Company, from Sao Paulo Brazil, to join its department of hydroelectrical power new developments in late 1949. In 1951 he joined Geotecnica, a geotechnical engineering services, design and construction company. Following a return to MIT on 1966/1967 as senior visiting professor, Victor started his career as individual consultant.

His main contributions include embankment and gravity dam engineering, earthmoving, tunnels and underground works, deep urban and port-lock excavations, foundations for high rise buildings, bridges, industries, ports, jetties, breakwaters, highways and railroads. One of his technical passions was probability and statistics applied to engineering design philosophy, together with risk analysis.

As an individual consultant or as a member of international advisory panels he participated on the design and construction of some major engineering projects: Emborcação, Foz do Areia, Guri, Pedra do Cavalo, Tucurui, Yacyreta, and hundreds of other dams in Brazil, in all of Latin America as well as in other countries as Angola, Burkina Faso, China, Irak, Iran, Mozambique, Turkey, Tunisia, etc. – the research and developments proposed by Victor on the behaviour of compacted saprolites and residual soils have influenced dam engineering throughout the world. His activity also included the design and follow up of large open pit mine projects in Brazil, in the Imigrantes highway, Ferrovia do Aço railway, in the geotechnical problems of Confins, Galeão, and Manaus airports, Açominas, Albras, Alumar, Alunorte, Camaçari, Cubatão, and Duque de Caxias refineries and steel and aluminum mills. One of his fascinating contributions was in the Diagnoses of Catastrophic Slope destabilizations in Hong-Kong 1976 -1979.

His professional vision was marked by intense job-generated research/observation and lonely mental experi-

Victor de Mello, a Brazilian geotechnical engineer, died on January 1st, 2009, aged 82.

mentation and debates, with data and interpretation published world-wide. Emphasizing the priority sequence of allegiances as firstly a world citizen, thence a civil engineer for better fulfillment, and finally only subordinately a geotechnical specialist for better engineering, and preaching the preeminence of creativity, and of prescriptions rather than correlations, as dominating geotechnical engineering design. In lecturing on his select case histories he always surprised by stressing from each case the lesson whereby the earnest optimized solution should principally indicate how not to repeat it, if the case chanced to present again.

An enthusiastic and intense perennial challenger and debater, he was often rightly misunderstood as disagreeing with his colleagues, while he was really debating against the topic and his own questionings. His approach has been exposed and expanded in some important papers, among which the Rankine Lecture (1977), Foundations on Clays (1969), The Standard Penetration Test (1971), Thoughts on Soil Engineering Applicable to Residual Soils (1972), Some Lessons from Unsuspected, Real and Fictitious Problems in Earth Dam Eng´g (1975), Philosophy of Statistics in Geotechnique (1975), Behaviour of Foundations and Structures (1977), Behavior of 2 High Rockfill Dams (1984), Foundations of Gravity Dams, Geomechanical Interaction (1984), Destabilization of Rockfill Slopes (1986) Embankment Dams and Dam Foundations (1989), Lessons of Adjustments to Tropical Saprolites and Laterites (1989), Revisiting our Origins (1994), Landslides by Maximized Infiltration: Fundamental Revision of Stability Calculations and Stabilizing Drainages (2003), several being available at his website. Victor was working on a book on his visions on applied soil mechanics, which he left unfinished and will be made available in the near future.

Some of the honors received include being a honorary member of many Societies of Soil Mechanics (Argentina, Japan, Portugal, Southeast Asia, Venezuela), Fellow of the Third World Academy of Science in Trieste Italy, Foreign Associate of the National Academy of Engineering of the USA, President of the International Society of Soil Mechanics and Geotechnical Engineering (1981-1985), Vice-president of the International Society for Rock Mechanics (1970-1974), Founder and President of the Brazilian Society of Soil Mechanics and geotechnical Engineering (1964-1966), recipient of the Terzaghi Award twice in Brazil and of the Manuel Rocha Award in Portugal, Terzaghi Orator ISSMFE (1994), member of the National Academy of Engineering of Brazil and of Argentina.

In an attempt to honoring Victor de Mello and celebrating his contribution to geotechnical engineering the Brazilian and the Portuguese geotechnical societies have created the Victor de Mello Lecture, the first of which was delivered by John Burland in 2008 and is available in many websites.

Victor was also a special human being. His love and strong links to his brothers and sisters started early in their lives, with the six of them being educated at home in Goa, allowed four of them to get higher degrees in the USA and maintained till today, with family gatherings.

Music, literature, dancing, wind surfing, tennis were also among his interests. Victor played the piano, and this helped him to find his way to MIT. His love to music included occidental classics, Portuguese fados, Brazilian popular music, and Indian ragas. Nature and art nurtured him; his everyday drive to his office changed according to which trees flowered in the route pending on the season. His wide cultural background led him to pursue knowledge in a multidisciplinary constellation of authors. And his habit of starting early each day included long, intensive working hours, and also leisure and sports.

Professor de Mello died peacefully of a minor stroke, in his home in São Paulo, Brazil, after a long process of amyotrophic lateral sclerosis (ALS, also called Lou Gehrig's disease). He leaves his wife Maria, his daughter Lucia Beatriz, his son Luiz Guilherme and four grandchildren. A great human being, a true individual and friend, an outstanding practicing engineer, has left our community.

Luiz Guilherme de Mello

Message from ABMS

Professor De Mello will always be remembered for his outstanding contributions to the Geotechnical Engineering knowledge and practice. He produced some remarkable papers and became reference to modern Soil Mechanics, pioneering the Brazilian geotechnical contribution in generating critical evaluation of existing data and performance of real cases, such as the "State of the Art Report on Foundations on Clays" (1969), "The Standard Penetration Test" (1971), the "Rankine Lecture" (1977), "Behaviour of Foundations and Structures" (1977), among others.

His contribution to the understanding of the mechanical behavior on compaction of saprolites and residual soils, emerging from his involvement in design and construction of large dams has influenced contemporary dam engineering.

His contribution for the professional societies can be put into perspective by considering that he was one of the founders of the Brazilian Geotechnical Society (ABMS), President of ABMS (1964-1966), Emeritus Member of ABMS, Honorary Member of SPG (Portugal), Vice-President of ISSMFE (1973-1977), President of ISSMFE (1981-1985).

John Burland presented the 1st Victor de Mello Lecture, during the 4th Luso-Brazilian Conference at Coimbra in 2008. His testimony summarizes our general feelings: "Reflections on Victor de Mello, Friend, Engineer and Philosopher", giving a remarkable testimony of this outstanding engineer and a man ahead of his time.

Jarbas Milititsky
ABMS President

Message from ABGE

Victor de Mello left a remarkable and virtuous influence on Brazilian Geotechnics, with active participation in historical and significant projects for decades, combined with intense academic activities. With his strong personality, he always inspired all that worked close to him, as well as his students and professionals who had him as a reference, encouraging them not to be satisfied with professional bureaucracy and always to dare to seek new and challenging solutions to make projects important to the nation feasible technically and economically.

Prof. de Mello always emphasised the importance of the geological information in his works and the Brazilian Engineering Geology owes him much of its acknowledged status of a fundamental discipline for the conception of engineering projects and for the definition of construction procedures.

ABGE - Brazilian Association for Engineering Geology and the Environment

Message from SPG

It was with great consternation that the Portuguese, Brazilian and Worldwide geotechnical communities took notice of the departure of Prof. Victor de Mello in the first day of the present year (2009).

Victor de Mello was born in Goa (1926), being Portuguese his first nationality. He made his post graduate studies in United States, at MIT, where he obtained his PhD in Civil Engineering, in 1948, in the Geotechnical field. After finishing his studies and graduations, he decided to live and to practice in Brazil, having acquiring the Brazilian nationality and getting married there.

From the beginning his personality distinguished by a profound intelligence, great culture and unsurpassable dynamism, contributing in a single form to the establishment and consolidation of a geotechnical scientific community that from the 50 years is affirming Brazil as one of the reference countries in the international area.

Victor de Mello was a precursor of the Luso-Brazilian geotechnical interchange when he came to professional training at LNEC, in the beginning of his career. His personal and professional characteristics led immediately to the establishment of great friendship ties with numerous Portuguese colleagues of various generations that endured throughout his life.

Distinguished member of the Portuguese Geotechnical Society since 1972, he was the second personality to pronounce the Manuel Rocha Memorial Lecture, in 1985, “Instabilizations of Rockfill Slopes. Conceptual Reappreciations”, published in Geotecnia review n. 47, and he was awarded the Manuel Rocha Research Prize in 1987.

In parallel with some brief reference to his vast and diversified curriculum, as professor, scientist, consultant and designer and also as an international geotechnical community leader, assuming the International Society of Soil Mechanics Vice Presidency between 1973 and 1977, his Presidency between 1981 and 1985, and Vice Presidency of the International Society of Rock Mechanics between 1970 and 1974, the main intention of this note is to stress the great esteem, admiration and respect of the Portuguese geotechnical community for the person and personality of Victor de Mello, expressed by the creation of the Victor de Mello Lecture to be pronounced during the Geotechnical Luso-Brazilian Congresses, that take place every two years.

The fist Victor de Mello Lecture was pronounced by Prof. John Burland, in Coimbra, in April 2008, during the IV Geotechnical Luso-Brazilian Congress.

To his family and in special to his son and our colleague Luís Guilherme de Mello we want to express the condolences of the Portuguese Geotechnical Society.

Laura Caldeira
SPG President

SOILS and ROCKS
An International Journal of Geotechnical and Geoenvironmental Engineering

Publication of
ABMS - Brazilian Association for Soil Mechanics and Geotechnical Engineering
ABGE - Brazilian Association for Engineering Geology and the Environment
SPG - Portuguese Geotechnical Society

Volume 32, N. 1, January-April 2009

Author index

Instructions to Authors

Category of the Papers

Soils and Rocks is the scientific journal edited by the Brazilian Society of Soil Mechanics and Geotechnical Engineering (ABMS) and by the Brazilian Society of Engineering and Environmental Geology (ABGE). The journal is intended to the divulgation of original research works from all geotechnical branches.

The accepted papers are classified either as an Article paper, a Technical Note, a Case Study, or a Discussion according to its content. An article paper is an extensive and conclusive dissertation about a geotechnical topic. A paper is considered as a technical note if it gives a short description of ongoing studies, comprising partial results and/or particular aspects of the investigation. A case study is a report of unusual problems found during the design, construction or the performance of geotechnical projects. A case study is also considered as the report of an unusual solution given to an ordinary problem. The discussions about published papers, case studies and technical notes are made in the Discussions Section.

When submitting a manuscript for review, the authors should indicate the category of the manuscript, and is also understood that they:

a) assume full responsibility for the contents and accuracy of the information in the paper;

b) assure that the paper has not been previously published, and is not being submitted to any other periodical for publication.

Manuscript Instructions

Manuscripts must be written in English. The text is to be typed in a word processor (MS Word or equivalent), using ISO A4 page size, left, right, top, and bottom margins of 25 mm, Times New Roman 12 font, and line spacing of 1.5. All lines and pages should be numbered. The text should be written in the third person.

The fist page of the manuscript is to include the title of the paper in English, followed by the names of the authors with the abbreviation of the most relevant academic title. The affiliation, address and e-mail is to be indicated below each author's name. An abstract of 200 words is to be written in English after the author's names. A list with up to six keywords at the end of the abstract is required.

Although alteration of the sequence and the title of each section may be required, it is suggested that the text contains the following sections: Introduction, Material and Methods, Results, Discussions, Conclusion, Acknowledgements, References and List of Symbols. A brief description of each section is given next.

Introduction: This section should indicate the state of the art of the problem under evaluation, a description of the problem and the methods undertaken. The objective of the work is to be clearly presented at the end of the section.

Materials and Methods: This section should include all information needed to the reproduction of the presented work by other researchers.

Results: In this section the data of the investigation should be presented in a clear and concise way. Figures and tables should not repeat the same information.

Discussion: The analyses of the results should be described in this section. **Conclusions**: The text of this section should be based on the presented data and in the discussions.

Acknowledgenments: If necessary, concise acknowledgements should be written in this section.

References: References to other published sources are to be made in the text by the last name(s) of the author(s), followed by the year of publication, similarly to one of the two possibilities below:

"*while Silva & Pereira (1987) observed that resistance depended on soil density*" or "*It was observed that resistance depended on soil density (Silva & Pereira, 1987)*."

In the case of three or more authors, the reduced format must be used, *e.g.*: Silva *et al.* (1982) or (Silva *et al.*, 1982). Two or more citations belonging to the same author(s) and published in the same year are to be distinguished with small letters, *e.g.*: (Silva, 1975a, b, c.). Standards must be cited in the text by the initials of the entity and the year of publication, *e.g.*: ABNT (1996), ASTM (2003).

Full references shall be listed alphabetically at the end of the text by the first author's last name. Several references belonging to the same author shall be cited chronologically. Some examples are listed next:

Papers: Bishop, A.W. & Blight, G.E. (1963) Some aspects of effective stress in saturated and unsaturated soils. Géotechnique, v. 13:2, p. 177-197.

Books: Lambe, T.W & Whitman, R.V. (1979) Soil Mechanics, SI Version, 2nd ed. John Wiley & Sons, New York, p. 553.

Book with editors: Sharma, H.D.; Dukes, M.T. & Olsen, D.M. (1990) Field measurements of dynamic moduli and Poisson's ratios of refuse and underlying soils at a landfill site. Landva A. & Knowles, G.D. (eds) Geotechnics of Waste Fills - Theory and Practice, American Society for Testing and Materials - STP 1070, Philadelphia, p. 57-70.

Proceedings (printed matter or CD-ROM): Jamiolkowski, M.; Ladd, C.C.; Germaine, J.T & Lancellotta, R. (1985) New developments in field and laboratory testing of soils. Proc. 11th Int. Conf. on Soil Mech. and Found. Engn., ISSMFE, San Francisco, v. 1, pp. 57-153.(specify if CD – ROM)

Thesis and dissertations: Lee, K.L. (1965) Triaxial Compressive Strength of Saturated Sands Under Seismic Loading Conditions. PhD Dissertation, Department of Civil Engineering, University of California, Berkeley, 521 p.

Standards: ASTM (2003) Standard Test Method for Particle Size Analysis of Soils - D 422-63. ASTM International, West Conshohocken, Pennsylvania, USA, 8 p.

Internet references: Soils and Rocks available at http://www.abms.com.br.

On line first publications must also bring the digital object identifier (DOI) at the end.

Figures shall be either computer generated or drawn with India ink on tracing paper. Computer generated figures must be accompanied by the corresponding digital file (.tif, .jpg, .pcx, etc.). All figures (graphs, line drawings, photographs, etc.) shall be numbered consecutively and have a caption consisting of the figure number and a brief title or description of the figure. This number should be used when referring to the figure in text. Photographs should be black and white, sharp, high contrasted and printed on glossy paper.

Tables shall be numbered consecutively in Arabic and have a caption consisting of the table number and a brief title. This number should be used when referring to the table in text. Units should be indicated in the first line of the table, below the title of each column. Abbreviations should be avoided. Column headings should not be abbreviated. When applicable, the units should come right below the corresponding column heading. Any necessary explanation can be placed as footnotes.

Equations shall appear isolated in a single line of the text. Numbers identifying equations must be flush with the right margin. International System (SI) units are to be used. The symbols used in the equations shall be listed in the List of Symbols. It is recommended that the used symbols be in accordance with Lexicon in 8 Languages, ISSMFE (1981) and the ISRM List of Symbols.

The text of the submitted manuscript (including figures, tables and references) intended to be published as an article paper or a case history should not contain more than 30 pages formatted according to the instructions mentioned above. Technical notes and discussions should have no more than 15 and 8 pages, respectively. Longer manuscripts may be exceptionally accepted if the authors provide proper explanation for the need of the required extra space in the cover letter.

Discussion

Discussions must be written in English. The first page of a discussion paper should contain:

• The title of the paper under discussion in the language chosen for publication;

• Name of the author(s) of the discussion, followed by the position, affiliation, address and e-mail. The discusser(s) should refer himself (herself, themselves) as "the discusser(s)" and to the author(s) of the paper as "the author(s)".

Figures, tables and equations should be numbered following the same sequence of the original paper. All instructions previously mentioned for the preparation of article papers, case studies and technical notes also apply to the preparation of discussions.

Editorial Review

Each paper will be evaluated by reviewers selected by the editors according to the subject of the paper. The authors will be informed about the results of the review process. If the paper is accepted, the authors will be required to submit a version of the revised manuscript with the suggested modifications. If the manuscript is refused for publication, the authors will be informed about the reasons for rejection. In any situation comprising modification of the original text, classification of the manuscript in a category different from that proposed by the authors, or refusal for publication, the authors can reply presenting their reasons for disagreeing with the reviewers' comments

Submission

The author(s) must submit for review:

1. A hard copy of the manuscript to Editors - Soils and Rocks, Av. Prof. Almeida Prado, 532 – IPT, Prédio 54 – DEC/ABMS, 05508-901 - São Paulo, SP, Brazil. The first page of the manuscript should contain the identification of the author(s), or

2. The digital file of the manuscript, omitting the authors' name and any information that eventually could identify them, should be sent to **abms@ipt.br**. The following must be written in the subject of the e-mail message: "*Paper submitted to Soils and Rocks"*. The authors' names, academic degrees and affiliations should be mentioned in the e-mail message. The e-mail address from which the digital file of the paper was sent will be the only one used by the editors for communication with the corresponding author.

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Volume 32, N. 1, January-April 2009

Table of Contents