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Table of Contents

Pacheco Silva Lecture

The Pacheco Silva Lecture is delivered each two years by an important geotechnical professional from Brazil or abroad to honour the memory of the distinguished Brazilian geotechnical engineer Francisco Pacheco Silva. Pacheco Silva was a researcher and consultant in geotechnical engineering, as well as one of the founders and past presidents of the Brazilian Association for Soil Mechanics and Geotechnical Engineering - ABMS.

Prof. Paulo Teixeira da Cruz
The 2008 Pacheco Silva Lecturer is Prof. Paulo T. Cruz. Prof. Cruz is Professor of Civil Engineering at the Polytechnic School of the University of São Paulo, São Paulo, Brazil. He holds a BSc. degree in Civil Engineering from the Presbyterian Mackenzie University (Brazil, 1957), MSc. from the Massachusetts Institute of Technology (USA, 1960) and DSc. from the University of São Paulo (Brazil, 1966). Among other subjects, his research interests are focused on the behaviour of compacted soils and dam engineering. For almost 50 years, he has also been an extremely active consultant in geotechnical engineering, particularly on dams, where he designed or acted as consultant in over 100 projects.

Soils and Rocks
v. 31, n. 2

A Partial Review of Barry Cooke and James Sherard 1987 Papers on Concrete Face Rockfill Dams (CFRD)

Paulo T. Cruz

Abstract. Two papers published by Barry Cooke and James Sherard in the Journal of Geotechnical Engineering v. 113:10 (1987) related to Concrete Face Rockfill Dams constitute the State of the Art on assessment, design, construction and performance of CFRD and continue to be a guide for the actual design and construction of such dams. An actual review of some items of those papers is presented, based on new data of the behavior of CFRD built within the 20 years that passed from 1987.
Key words: dams, rockfill, rockfill dams: design, construction, performance.

1. Introduction

The two papers written by Barry Cooke and James Sherard: Concrete Face Rockfill Dams I. Assessment and Concrete Face Rockfill Dam: II Design, published in the Journal of Geotechnical Engineering v. 113:10 (1987) of the American Association of Civil Engineers (Cooke & Sherard, 1987a and b) consolidated the basis for the design and construction of CFRD. A portuguese version of these papers was published by the Brazilian Committee of Dams by Oliveira (1987).

The two papers were based on the design, experience, and performance of the dams built at the time that amounted to 56 (above 50 m in height - Water Power Year Book, 2006). The Foz de Areia Dam (Brazil, 1980) 160 m the highest in the world, is repeatedly mentioned in the papers. Within these 20 years (1987-2007) the number of dams above 50 m built in the world jumped to 233. The actual highest is Shibuya - 232 m (China). However, the rupture of the concrete face of four large dams in 2006 has surprised the world specialists because such ruptures were not foreseen by consultants, designers and constructors. Figure 1 shows a photo of the Campos Novos Dam (202 m - Brazil).one the dams in which ruptures occurred in the concrete face.

These accidents led to a review of Barry Cooke and James Sherard basis for design and construction of CFRD. This review, as discussed below, represents only minor adjustments to the master work of those two great engineers, that had the courage to introduce and "reinforce" this alternative design in relation to the more traditional dams, aiming for economy, safety and speed in construction schedules.

The four dams that had the concrete face ruptured were repaired and are in operation with no risk of failure.

2. Review

The present review is limited to the performance of the rockfill, not only due to the limitation of pages and time for this conference, but mainly due to the limitations of the author's knowledge and experience in other aspects of the dam design.

The changes in the design of the joints, concrete face and reinforcements are referred only briefly. Some of such changes are still under discussion and escape the content of this paper.

2.1. Zoning designation

It is useful to use standard zoning designations and to adopt those common for the ECRD materials; Zone 1 for impervious, Zone 2 for the filter or transition zone directly under the concrete slab, and Zone 3 for the main rockfill. (Cooke & Sherard)

Figure 2 shows a CFRD cross section with the zone designation, and the table contains the materials and construction specifications. It would have been very useful to follow the proposed zoning designation to facilitate exchange of data and easy understanding.

A new zone 2A has been incorporated in new designs, that is, the extruded concrete curb introduced since the Itapebi dam by Resende & Materon (2000). This solution has been used under the concrete face and serves as a support for the slab and a form to the sand layer of zone 2. This detail will come later in this paper.

Figure 1 - Campos Novos Dam.

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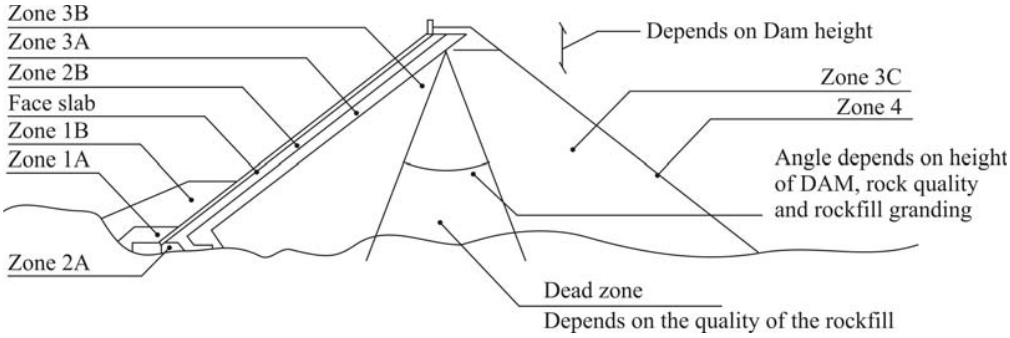


Material	Zone	Classification	Method of placement	Compaction
Imprevious	1A	Silty- Clay with rock fragments ≤ 19 mm	Compacted	0.25 m layer sheeptoot roller
Randon	1B	Silty- Clay with rock fragments ≤ 19 mm	Compacted	0.60 m layer 6 passes of construction equip.
Filter	2A	Sand and gravel $\phi \le 5$ cm	Compacted	0.30 m layer 4 passes vibratory roller 10 t
Transition	2B	Gravel - $\phi \le 100$ cm	Compacted	0.30 m layer 4 passes vibratory roller 10 t
Rockfill transition	3A	Massive basalt with ϕ_{max} 20 cm	Compacted	0.60 m layer 4 passer vibratory roller 10 t
Rockfill	3B	Massive basalt with maximum of 25% of breccias	Compacted	0.8 m layer 4 passes vibratory roller 10 t
Rockfill dead zone	3B 3C	Massive basalt with maximum of 40% of breccias	Compacted	1.2 m layer 4 passes vibratory roller 10 t
Rockfill	3C	Massive basalt with maximum of 25% to 40% of breccias	Compacted	2.0 m layer, 4 passer vibratory rolle 10 t
Downstream	4	Large blocks of massive basalt or basaltic breccias	Placed	

Figure 2 - Concrete face Rockfill Dam typical cross. Section and specification.

Looking into many projects of CFRD this author has seen that the zones suggested by Cooke & Sherard follow the proposed sequence, but the zone designation unfortunately are named with different numbers and notations.

2.1.1. The rockfill behaves as a whole

> *The rockfill embankment is in three zones of increasing layer thickness to give a desirable transition of compressibility and permeability from upstream to downstream. Lowest compressibility is desirable in the portion of the upstream shell which transmits water load to the foundation. Because most of the water load passes into the foundation through the upstream shell it is desirable that the compressibility of zone 3B be made as low as practical to minimize slab settlement. The downstream zone 3C takes negligible water load, and its compressibility has little influence on the settlement of the face slab.* (Cooke & Sherard)

Considering the materials and compaction specifications in the table of Fig. 2 it is clear that the design of this hypothetical dam, follows Cooke's and Sherard's recommendations as stated above. However, when one looks more carefully in the behavior of CFRD one can see that the rock fill behaves more as a whole than as independent zones 3B and 3C. As an example, Fig. 3 shows the end of construction settlements measured in the Itapebi dam. There is a shift of the settlements to zone 3C. During impounding the water load will reach zone 3C.

A simple analysis of the stresses that develop in a CFRD is shown in Fig. 4 at the end of construction and after the reservoir filling for points within the embankment. It is clear that in the upstream shell there is a change in the direction of the stresses from the end of construction to the full water load. Near the face the shear stress decreases and then increases again. In the downstream shell the change in direction of the stresses is practically negligible and the stresses increase with the water load.

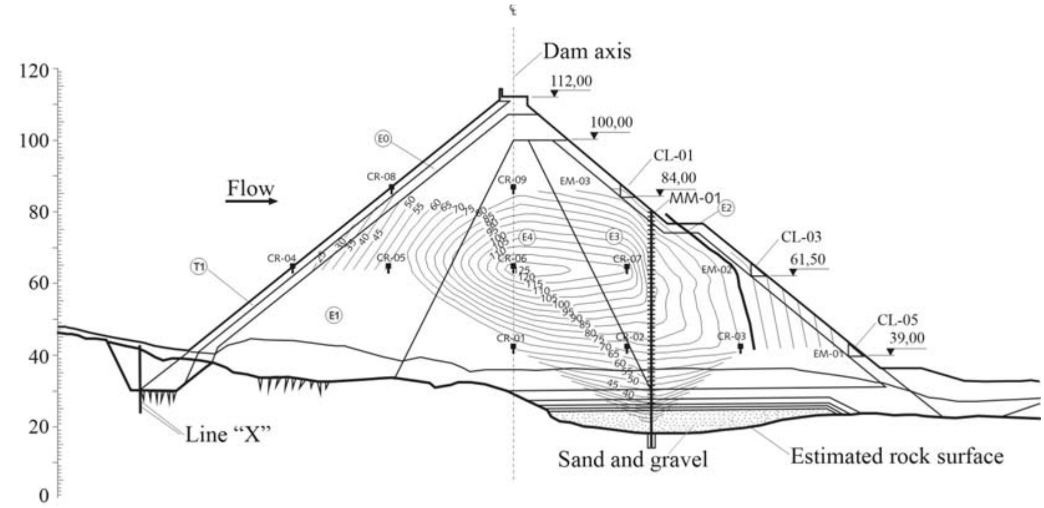


Figure 3 - End of construction settlements measured in the Itapebi Dam (Albertoni *et al.*, 2001).

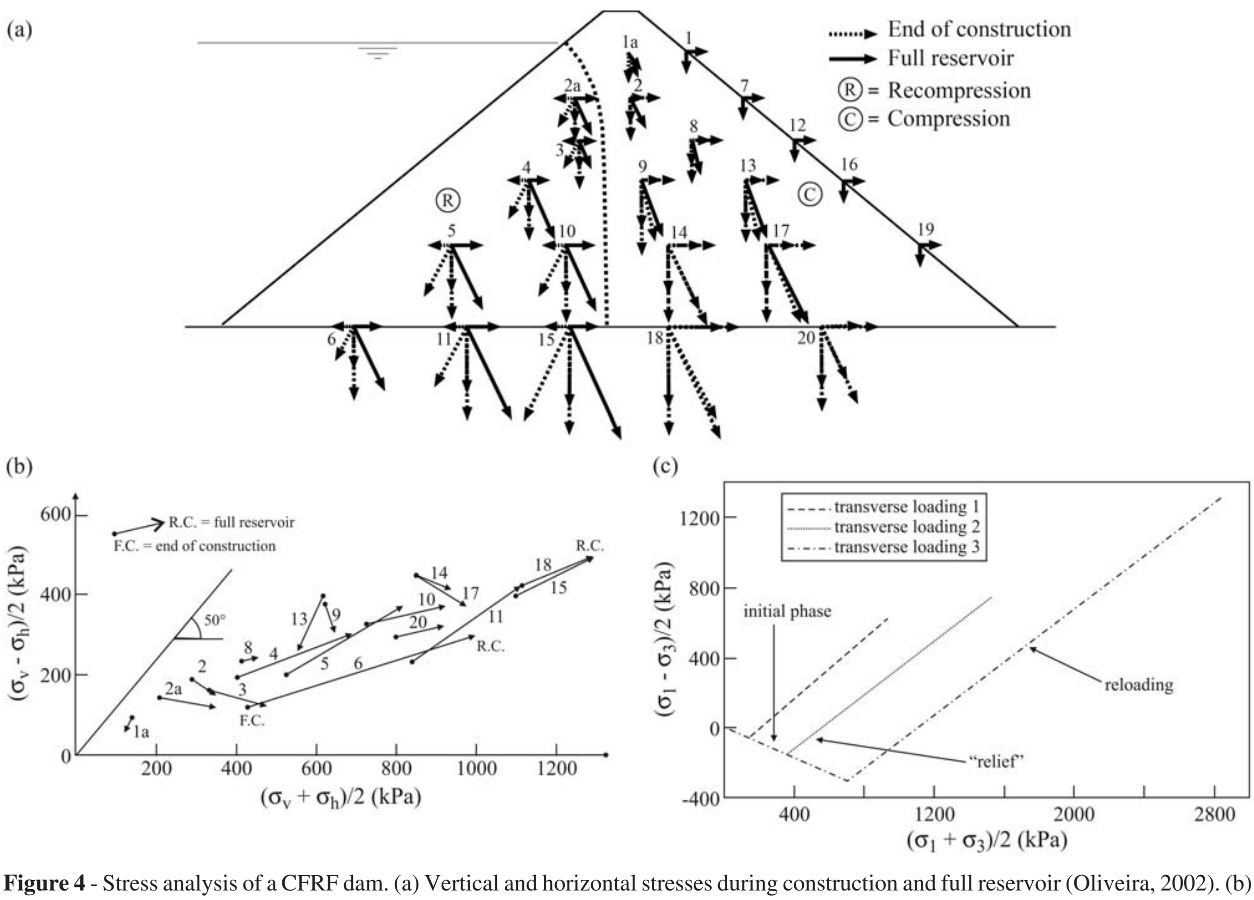


Figure 4 - Stress analysis of a CFRF dam. (a) Vertical and horizontal stresses during construction and full reservoir (Oliveira, 2002). (b) Horizontal and vertical stress paths at end of construction and full reservoir (Oliveira, 2002). (c) Stress paths for laboratory tests (Basso, 2007).

It is possible to separate zones of reloading in the upstream shell and zones of continuous loading downstream. The compression modulus will be different in the zones, as well as the displacements, as shown in the example in Fig. 5.

When the upstream zone is made of gravel and the downstream of rockfill, the lower compressibility of the gravel can lead to bending in the upper third of the face slab as happened in Aguamilpa Dam (Fig. 6). For higher dams, the specifications for compaction in the downstream shell

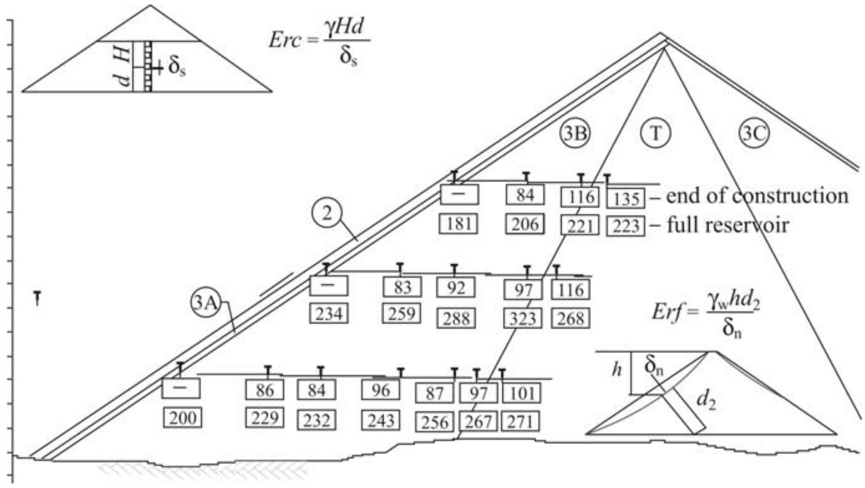
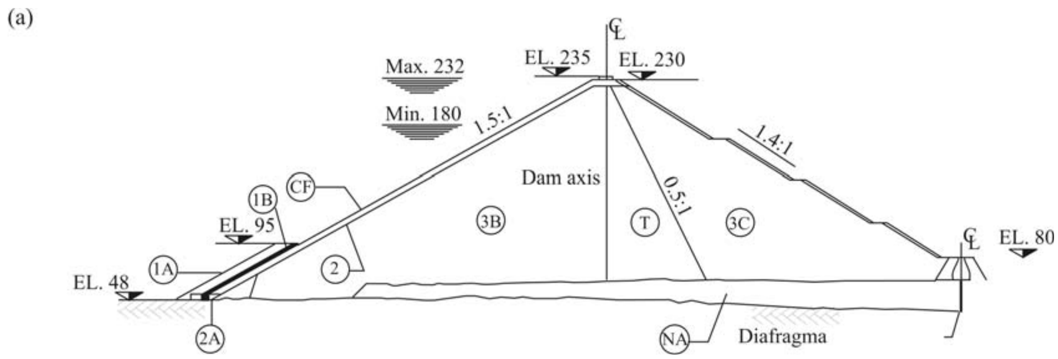


Figure 5 - Deformability modulus at end of construction and full reservoir - El Cajon Dam (Sandoval *et al.*, 2007).



Specification	Zone	Compaction procedure	Compaction specification
Random	1A	Not compacted, just placed 80 cm	
Fine silty sand, ϕ 0.2 cm	1B	Not compacted, just placed 30 cm	
Alluvial gravel and silty sand mixture, ϕ 3.8 cm	2F	Compacted layer 30 cm	Layers 4 passes of 100 kN SDVR
Crushed alluvial gravel sand mixture, ϕ 7.6 cm	2	Compacted layer 30 cm	Layers: 4 passes of 100 kN SDVR Face: 6 passes of 40 kN SDVR or 130 kN PC
Dredged alluvial, ϕ ≤ 4.0 cm	3B	Compacted layer 60 cm	4 passes of 100 kN SDVR
Rockfill 3C with reduced ϕ max 5.0 cm	T	Compacted layer 60 cm	4 passes of 100 kN SDVR
Rockfill (ignimbrite), ϕ 100 cm	3C	Compacted layer 120 cm	4 passes of 100 kN SDVR
Concrete face	CF		
Natural alluvium	NA		

Figure 6a - Aguamilpa Dam.

(zone 3C) have been revised to make then less compressible.

2.1.2. The leakage through the rockfill

The increasing permeability from zone 2 progressively through zones 3A, 3B, and 3C (Fig. 2) is desirable during construction, in the event of a flood before the concrete face is placed. After the concrete face is placed there is no credible face problem that could cause more leakage than the rockfill could handle without damage. (Cooke & Sherard)

In another part of the first paper, Cooke & Sherard discusses the advantages of a placement that will lead to a stratified rockfill.

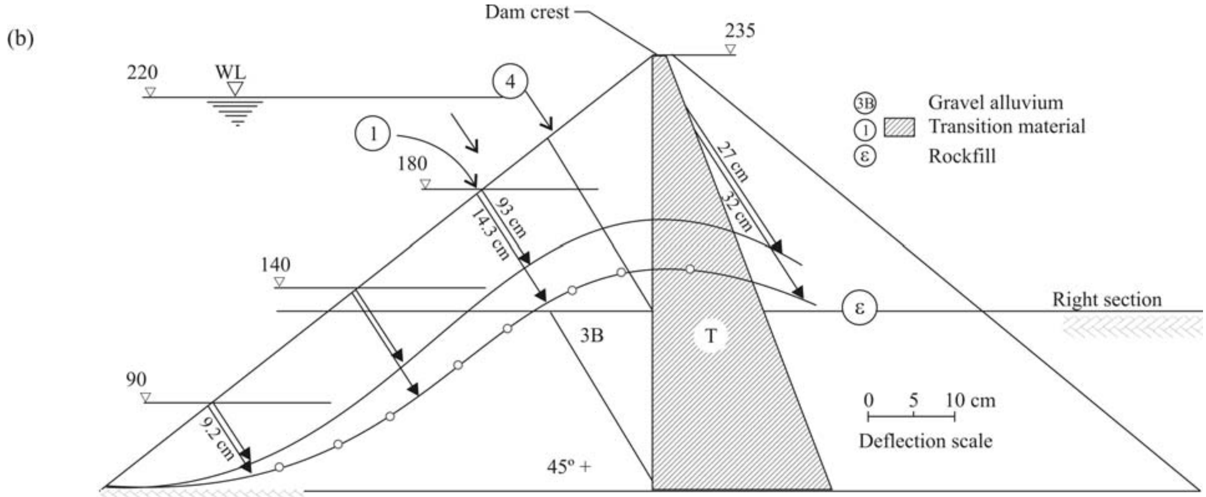


Figure 6b - Face displacements of Aguamilpa Dam (Mori, 1999).

There are no technical disadvantages to the preferred method of placement in segregated layers. In addition to lower cost, there are several advantages. The stratification assures that any flow through the rockfill embankment will travel much more easily in the horizontal direction than vertically. (Cooke & Sherard)

The leakage that passes through fissures, cracks and even fractures at the face of CFRD are controlled by the sand layer, and are considerably less than the flow that will start a progressive removal of the downstream rock blocks. Leakages as high as 1000 to 3000 l/s have been measured in the CFRD Barra Grande, Campos Novos (2007) without any signs of trouble in the downstream rock slope, confirming Cooke & Sherard position. However, if a flood reaches the dam face, before the placement of the concrete slab, the anisotropy of the permeability will not be favorable, because it raises the phreatic line and concentrates the flow in the bottom of the segregated layers as demonstrated by Pinto (1999), in a laboratory experiment with sand and gravel. Under a theoretical point of view Cruz (2005) demonstrated that the higher the point in which the phreatic line reach the downstream shell, the worse is the condition of instability.

2.2. Construction

Regarding construction stages, Cooke & Sherard are quite liberal, as can be seen as follows.

There has been some thinking that the rockfill dam should be completed to full height before starting the placement of the concrete face. The construction and performance experience at the Areia, Salvajina, and Khao Laem Dams conclusively shows that the face slabs can be placed in any sequence convenient to the contractor to obtain maximum schedule and cost benefits.

At the Areia Dam - the highest CFRD to date (160 m), and with the highest settlement - the concrete slab was placed on the lower 80 m of the dam height before the rest of the embankment was completed. The top of the first-stage face slab in the center of the valley moved downstream normal to the slope about 0.6 m while the rest of the embankment was being completed, causing no problem. (Cooke & Sherard)

The movements or displacements of a rockfill occur in two steps or times: a fast initial movement that follows the load increment, ie, the raising of the rock fill or the water in the reservoir. And a second step due to the progressive accommodation of the rock blocks and its progressive breakage due to the reorientation of the stresses. In narrow valleys, according to Cooke & Sherard, *An arching effect develops during the construction of the rockfill that progressively relaxes with time*. This second step is called creep. Usually it may represent a small fraction of the first step, but it is continuous and its effect are felt mainly in the upper half or third of the dam face deflection.

In Campos Novos dam (203 m - Brazil) the face slab was built in 3 phases. The first between March 2003 and August 2003, the second during September 2004 and March 2005 and the final stretch (mostly above the max. water level) at the end of construction. Figures 7 and 8 shows that the vertical displacements measured by settlements cells, due to the construction loads, were still under progress when the first stage of the slab was built. Figure 8 shows the corresponding horizontal displacements. Figure 9 show the face slab movements of the Xingó CFRD during 5.5 years of monitoring.

2.3. Construction materials

Apart from the somewhat rigid requirements for the sandy layer (zone 2) and the transition material (zone 3A) Cooke & Sherard are quite liberal in accepting rocks for the rockfill with relatively low resistance and with fines.

The most important properties of the CFRD embankment are low compressibility and high shear strength.

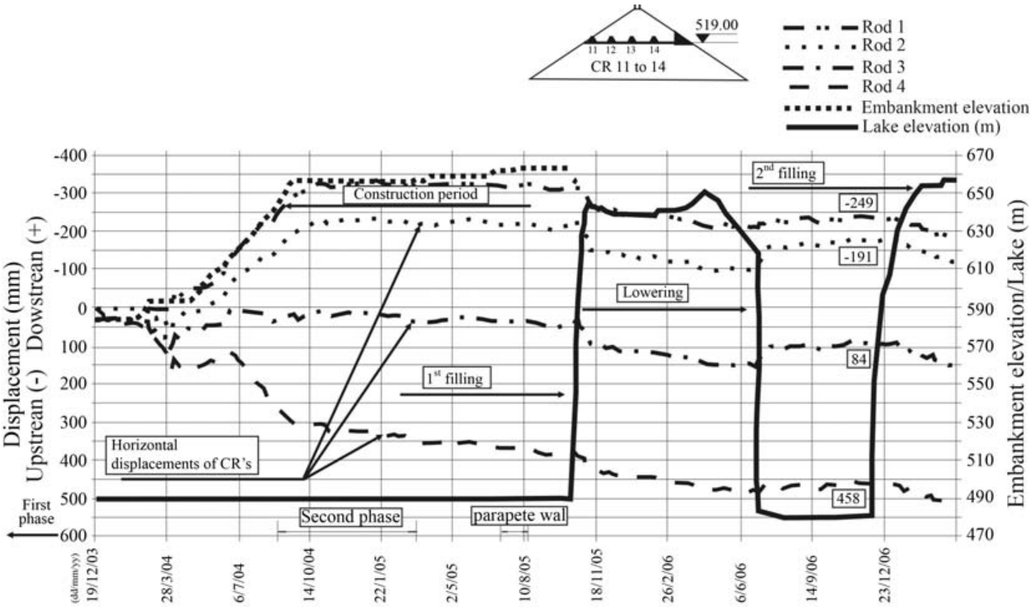


Figure 7 - Horizontal displacements of 1 to 4 CR 15 to 18 (Cruz, 2007).

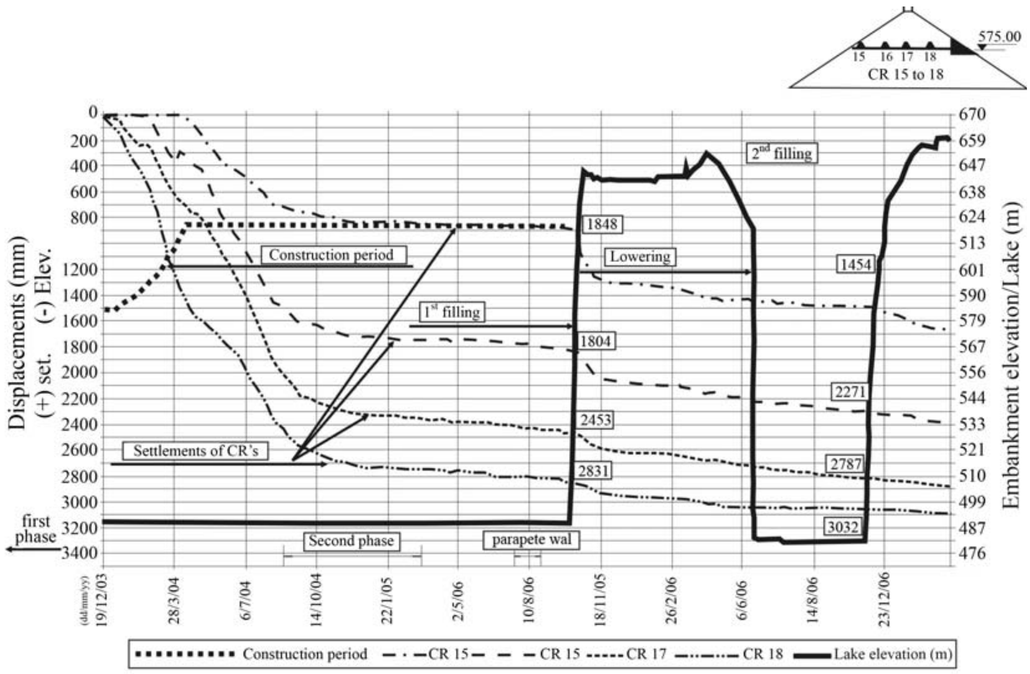


Figure 8 - Vertical displacements of 15 to 18 - stake 13+10,00 - El 575 (Cruz, 2007).

Usually rockfill is highly pervious, but less pervious rockfill can be used in CFRDs by providing special interior drainage zones. As a general rule any quarried hard rock with an average particle size distribution having 20% or less finer than the n. 4 sieve, and 10% or less finer then the n. 200 sieve, will have the needed rockfill high shear strength and low compressibility. These limits may be a better means of defining rockfill by sizes than the common specification requiring a maximum percentage of particles smaller than 2.5 cm.

When a rock fill contains a fine content exceeding these limits, commonly the final evaluation of suitability can be made on the basis of the trafficability of the rockfill surface when the material is thoroughly wetted.

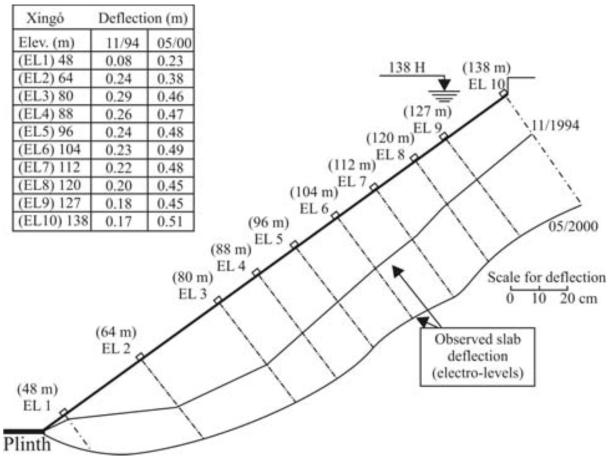


Figure 9 - Face slab movements of the Xingó Dam (Penman & Rocha, 2000).

> *A stable construction surface under travel of heavy trucks demonstrates that the wheel loads are being carried by a rockfill skeleton. An unstable construction surface, with springing, rutting, and difficult truck travel, shows that the volume of soil-like fines is sufficient to make the rockfill relatively impervious. Where the surface is unstable, the fines dominate the behavior and the resulting embankment may not have the properties desired for a pervious rockfill zone.* (Cooke & Sherard)

Restrictions regarding the materials are related to problems in compaction and low permeability, when a drainage filter is provided to drain the leakage. In fact, an analysis of any table containing data of the rockfill materials used in CFRD shows that practically every rock has been used in CFRD rockfill with restrictions only to the presence of fines that would control the behavior of the rockfill. A possible restriction to be observed refers to the use of two very different materials regarding compressibility because, as mentioned before, the dam behaves as a whole. Any more compressible zone will concentrate the displacements and affect the overall distribution of the stresses within the rockfill.

A simple exercise can be done. Let's consider a CFRD 120 m high, with outer slopes of 1,3 (h):1,0 (v), as shown in Fig. 10. The displacement at any point in the slab can be computed by the expression $\Delta = \sigma l/E$, being σ the water pressure, l the distance from the slab to the foundation and E the modulus of compressibility. If E is taken as constant the displacements are those shown by the line 1st solution. If one considers that in the first half the E is the E_R for reloading (as discussed) and in the second half E is E_C (loading) and that $E_R \cong 2E_C$ (as observed) the displacements of the slab are those in the 2nd solution. This simple procedure leads to slab movements very similar to those measured in CFRD (see Fig. 9). If E_C is taken as E_R divided by 3 the displacements correspond to the 3rd solution (arrows in Fig. 10).

2.4. Zone 2

> *The early and primary purpose of the thin zone of finer rock directly under the slab was to provide uniform and firm support for the concrete slab. Crusher-run minus -1,5 - 7.5 cm rockfill has been used.*
> *Recently there has been a trend toward making zone 2 grading have a sufficient quantity of sand-sized particles and fines to improve workability, reduce excess concrete and have reliably low permeability, and have an approximate filter grading.*
> *For face compaction, the roller is first pulled up the slope without vibration for several passes, and then given four upward passes with vibration. A new and promising development is the use of a backhoe-mounted plate vibrator. On several new dams, the plate vibrator is being specified for zone 2 compaction adjacent to the toe slab, and optional for face compaction.*
> *At rainy sites, it is desirable to place the erosion protection as soon as possible after embankment placement. Surface protections comprised of about 50-76 mm of shotcrete or a sprayed sand asphalt skin have been equally satisfactory for general erosion protec-*

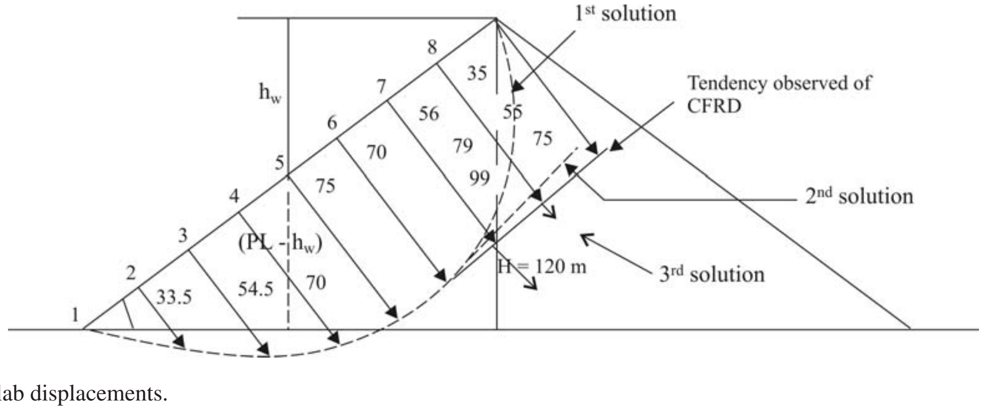


Figure 10 - Slab displacements.

tion, but cannot be relied upon alone to resist large concentrations of runoff from the abutment. (Cooke & Sherard)

Cooke & Sherard recommendations regarding zone 2 have more details than the 3 paragraphs above and should be implemented.

An alternative for the support of the concrete slab was introduced by CNO - Contractor Norberto Odebrecht S.A. for the Itapebi and Itá CFRDs (Resende & Materón, 2000). An extruded concrete wall built as support for the concrete face replaces all the troublesome operations of compaction along the slope of zone 2 and eliminates the asphalt emulsion spray. It serves also as a support to the compaction of the sand layer and provides protection of the sand against rain erosion. Figures 11 and 12 show the curb machine and curb details.

2.5. Joints and reinforcing

The perimeter joint always opens and offsets moderately when the reservoir is filled, and is a potential source of leakage if not well designed, inspected, and constructed. For dams of low to moderate height (less than about 75 m), the joint movement has commonly been only a few millimeters, and joints with current water stop details have usually remained watertight. For some of the higher dams, the joint openings and displacements have been several centimeters. At the 160 m high Areia Dam, the opening in one area was 2.5 cm and the offset 5 cm. No joint leakage occurred, but is probable that the central bulb waters top was ruptured.

Today longitudinal reinforcing is continued through the joints without waters tops. This is considered good practice, is more economical, and has been adapted for more recent dams. With the longitudinal steel passing through the construction joint, there is no need to use water stops.

The face slab has usually been placed in 12 - 18 m wide strips, 15 m being must commonly used. The dimensions should be left to the contractor. (Cooke & Sherard)

In the practice today the horizontal joints are mainly construction joints. The vertical joints in the tension zone near the perimeter joint and near the abutments have special design details (Perez *et al.*, 2007) but in the compression zone the joints usually are built side by side, with a water stop in the bottom. Regarding the perimeter joint ant the vertical joints in the tension zone the concern has been with leakage and only few changes have been introduced since Cooke & Sherard papers of 1987.

The larger displacements that have damage the CFRD with dumped rockfill would not occur in the compacted rockfill. The compressive stresses that develop in the compression joints or within the slaps, due to the movements of the rockfill, would be taken by the compressive strength of the concrete slab. These assumptions were made in the design of CFRD until the recent accidents in four high CFRD. To deal with this problem in high dams in narrow valleys, some of the vertical joints in the compression zones have been left "open", *i.e.* with some space between them to allow movements and avoid the concentration of stresses (Fig. 13). Similar solutions were incorporated in the Shibuya Dam and in the repairs of other dams.

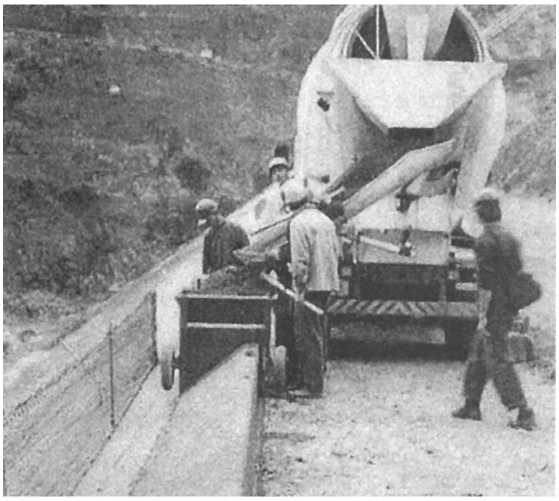


Figure 12 - Curb machine.

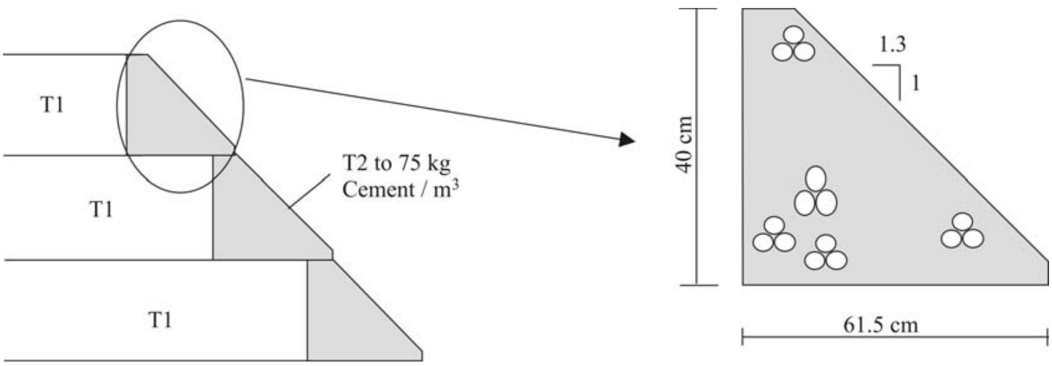


Figure 11 - Curb detail.

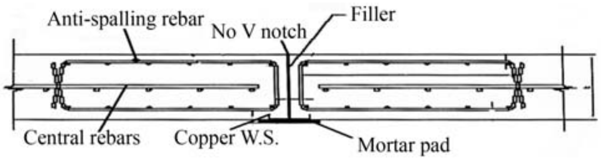


Figure 13 - Karanjulca face slab joint at central compression zone (Perez *et al.*, 2007).

2.6. Instrumentation

The empirical design that prevails in CFRD for the last 50 years has suffer improvements and modifications, because these CFRD have been well instrumented and through the observed data one can analyze the behavior of these CFRD. The views and considerations of Cooke & Sherard of 1987, are actual as can be seen in the following two paragraphs:

> *Instrumentation on CFRD dams has been important in gaining knowledge that has led to design, improvements in joint and face-slab design, evaluation of rock and rockfill, and zoning in the rockfill. The results have given confidence in proceeding with higher dams. Instrumentation is not a requirement for safety monitoring. However, a minimum amount is used.*
>
> *Areia Dam would not be a CFRD without the pioneering engineering of the Hydro-Electric Commission of Tasmania, and the publication of instrumentation results. The owners of the Areia (160 m) and Salvajina (148 m) Dams have extended the practice. The design of the CFRD is essentially empirical. It is based on experience and judgment. Instrumentation results are a major factor in the words "empirical", "experience", and "judgment". Instrumentation data on existing and new dams are important to continuing progress.* (Cooke & Sherard)

But the recent accidents in Mohale, Barra Grande, Campos Novos and Tianshenqiao 1, have shown that the intricate mechanism of stresses transmitted from the rockfill to zone 2, the extruded concrete and the concrete slab are not yet well understood, and that the instrumentation existing in those dams were not able to foreseen what happened.

The movements of the rockfill towards the valley that result in the intricate mechanism mentioned above are not measured. One measures settlements and displacements towards upstream or downstream, but the complete movement is needed. The bench marks placed on the crest and the downstream slope of the dam provide this information, but only in the external contour of the dam. A new instrument is necessary to provide the complete displacements of the dam body. Mathematical models have been developed, but one can not compare predictions with real displacements. Attempts to measure the stresses in the slab with strain gages were done in the Mohale Dam, but in this author's opinion view one still do not have a complete picture of the stresses in the concrete slab.

3. Conclusions

In the 20 years that passed since 1987, more then 177 CFRD above 50 m have been built around the world, most of which have had Dr. Barry Cooke as a consultant. The two of 1987 discussed above papers have guided the design and construction of must of these dams. The purpose of this paper is solely to bring some aspects of CFRD behavior into a discussion that has already been happening in the last 20 years.

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Articles

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Laboratory Behaviour of Rio de Janeiro Soft Clays. Part 1: Index and Compression Properties

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Willy Alvarenga Lacerda, Maria Esther Soares Marques

Abstract. This paper describes the Index and compression properties of Rio de Janeiro sedimentary clays properties and behaviour of ten deposits of soft to medium clays, situated in industrialized and residential areas of Rio de Janeiro City and Rio de Janeiro State. The data reported in this paper is a summary of the geotechnical properties obtained from laboratory and in situ tests carried out under well-defined contour and drainage conditions. The deposits are described in terms of its index properties, stress history, compressibility, consolidation and strength properties.

Key words: index properties, compression, stress history, sedimentary clays, laboratory tests.

1. Introduction

The geology of the Rio de Janeiro State consists primarily of granites and gneissic rocks formed in the Precambrian Age. The sedimentary depositions found in the coastal plains consist mostly of alluvia and marine depositions of the Quaternary Age. Kaolinite is the main clay mineral present in the soft clay deposits, but illite and/or smectite can also be found in a smaller proportion (Antunes, 1978). The presence of organic matter in this reduced environment gives the dark grey color observed in Rio de Janeiro soft clays.

Pacheco Silva (1953) carried out a pioneer and important study on the geotechnical properties of Rio de Janeiro clays. However, the most comprehensive study was carried out on Sarapuí soft clay by researchers of the Federal University of Rio de Janeiro (UFRJ) and Catholic University (PUC-RIO), in the seventies and eighties. Lacerda *et al.* (1977) and Werneck *et al.* (1977) presented the first results of these studies and Almeida & Marques (2002) made a comprehensive update.

A number of other soft clays deposits have been studied in the last 30 years in the City of Rio de Janeiro and vicinity, in association with some engineering works. Some examples of these works are the construction of highways around Guanabara Bay, the construction of the Rio de Janeiro subway, besides other projects such as dams, industrial fills, landfills and others.

In some of these deposits it was possible to perform research projects, by means of master or doctoral studies. Botafogo and Uruguaiana (subway: Vilela, 1976, Lins & Lacerda, 1980), Barra da Tijuca (industrial fill), Caju (sanitary fill: Cunha & Lacerda, 1991) and St. Cruz (industrial fill) sites are located in Rio de Janeiro City. Fluminense Plains, Sarapuí (experimental fills, Ortigão *et al.*, 1983), North Coast, Itaipu (experimental excavation: Sandroni *et al.*, 1984) and Juturnaíba dam (Coutinho & Lacerda, 1994) sites are located in Rio de Janeiro State.

This paper summarizes the geotechnical data of these clay deposits (Futai, 1999), which include: index properties, compressibility and stress history, and undrained and drained strengths. The role of the clay structure on both compressibility and undrained strength is also illustrated. A companion paper (Futai *et al.*, 2008) discusses strength and yield behaviour of the Rio de Janeiro soft clays.

2. Soil Characterization

Schematic geotechnical profiles of the Rio de Janeiro clay deposits studied here are shown in Fig. 1. The thickness of these clay deposits varies from 6 m to 15 m and in all but three sites the clay layer reaches ground level. Also a sand layer underlies all clay layers, a common feature of coastal plains. The main geotechnical parameters of these sites, with emphasis on index and compression properties, are summarized in Table 1, except for Fluminense plains. Depending on the available data, values are presented as average values with standard deviation, absolute values or range of values.

For most sites the average plasticity index I_P is in the range 63-81%. Lower values are observed for Botafogo clay (I_P = 11%) and Uruguaiana clay (I_P = 40%). Greater values are observed for Barra da Tijuca clay (I_P = 120-250%).

Rio de Janeiro clays may be grouped in categories. Botafogo and Uruguaiana clays, located in densely populated areas, with a thick fill layer on the top have Atterberg limits lower than the other sites, as shown in Fig. 2. Their clay content and organic matter are also lower. Due to this

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Figure 1 - Geotechnical profiles of Rio de Janeiro clays.

thick fill layer, in situ unit weight γ for Botafogo and Uruguaiana sites are in the range 16-17 kN/m^3 while for the remaining sites they are in the range 12-14.8 kN/m^3. Also, the vane strength, without correction, measured at Botafogo and Uruguaiana sites are in the range 70-110 kPa, thus much greater than the other sites, in the range 6-30 kPa. Therefore, Uruguaiana and Botafogo sites consist of medium to stiff clays, and the remaining sites consist of very soft to soft clays.

Barra da Tijuca, Itaipu and Juturnaíba sites have higher values of I_p, water content and organic matter content (>20%). These clays are also quite compressible. Their friction angles, ϕ', are also higher due to the presence of fibres in the organic matter.

The remaining clays (Caju, Fluminense Plains, North Coast, Sarapuí and St. Cruz) present organic matter contents below 10% that are enough to influence the behaviour of these clays. For these sites, the water content is in the range 80-150%, and close to the liquid limit, as shown in Fig. 2.

The relationship between water content and organic matter is shown in Fig. 3 for four clays, Itaipú, Juturnaíba, Uruguaiana and Sarapuí. It is well known (*e.g.*, Schofield & Wroth, 1968) that the water content is directly related to compressibility and undrained strength. Therefore, it can be concluded that organic content influences directly compressibility and strength.

Table 1 - Geotechnical properties of Rio de Janeiro clays.

Parameter / clay	Caju (b)	Sarapuí (c)	Santa Cruz (IZ) (d)	Santa Cruz (SZ) (e)	Nothern coast of Guanabara (f)	Itaipú (g)	Juturnaiba (h)	Uruguaiana (i)	Botafogo (j)	Barra da Tijuca (k)
References	Lira (1988); Cunha & Lacerda (1991)	Lacerda *et al.* (1977); Ortigão (1980); Almeida & Marques (2002)	Aragão (1975)	Aragão (1975)	Aragão (1975)	Carvalho (1980); Sandroni *et al.* (1984)	Coutinho & Lacerda (1994)	Vilela (1976)	Lins & Lacerda (1980)	Almeida *et al.* (2000)
Clay layer thickness (m)	12	12	15	10	8.5	10	7	9	6	12
w (%)	88	143 ± 21.7	112	130	113	240 ± 110	154 ± 95.6	54.8 ± 15.9	35	100-500
w_L (%)	107.5	120.3 ± 18.0	59.6	125.4	122	175.4 ± 82.6	132.5 ± 43.8	71.3 ± 30.0	38	70-450
I_P (%)	67.5	73.08 ± 16.1	32	89	81	74.5 ± 30.1	63.59 ± 22.1	40.5 ± 22.03	11	120-250
% clay	-	70	-	54	35	-	60.7 ± 12.74	39.4 ± 10.11	28	28-80
γ (kN/m^3)	14.81	13.1 ± 0.49	13.24	13.44	13.24	12 ± 1.85	12,5 ± 1.87	16.1 ± 1.39	17.04	12.5
S_t	3	2.59 ± 0.69	3.39	2-6	-	4 - 6	5-10	3.00	-	5.0
% organic matter	-	4.13 - 5.54	-	-	-	32.63 ± 20.46	19 ± 10.63	2.56 ± 1,04	-	-
$CR = C_c/(1 + e_o)$	0.27	0.41 ± 0.07	0.32	-	0.26 ± 0.15	0.41 ± 0.12	0.31 ± 0,12	0.31 ± 0.15	0.16	0.52
C_r/C_c	0.21	0.15 ± 0.02	0.10	-	0.16 ± 0.04	-	0.07 ± 0.06	-	0.19	0.10
c_v (m^2/s) x 10^{-8}		9	0.2-18.2	-	0.4	5	1-10		30	2-80
e_o	2.38	3.71 ± 0.57	3.09	3.37	2.91	6.72 ± 3.1	3.74 ± 1.89	1.42 ± 0.36	1.1	-

Mitchell (1993) has shown the influence of the organic content in increasing the Atterberg limits and decreasing both the dry density and the undrained strength. For Sarapuí clay, Coutinho & Lacerda (1994) have clearly shown the influence of the organic content in increasing compression index C_c, compression ratio $CR = C_c / (1 + e_o)$, and secondary compression index $C_{\alpha\varepsilon} = C_{\alpha\varepsilon} / (1 + e_o)$.

The relationship between the I_P and the liquid limit w_L has been traditionally used to classify fine-grained soils. The usual functional relationship between I_P and w_L is:

$$I_P = A\,(w_L - B) \tag{1}$$

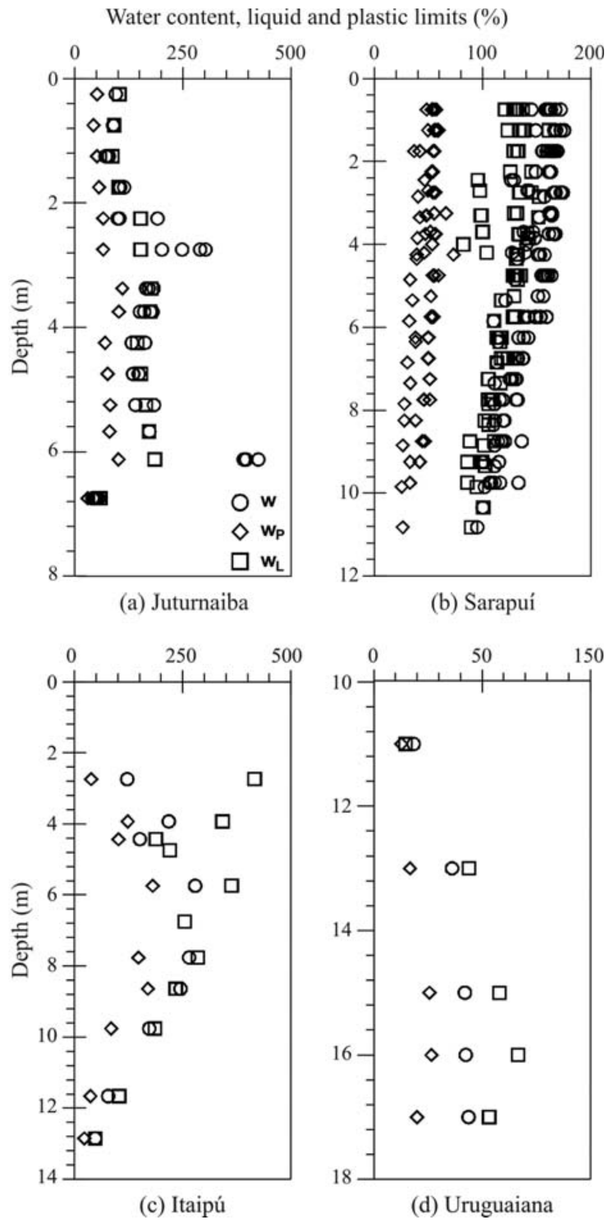

Figure 2 - Water content, liquid and plasticity limits profiles of Rio de Janeiro clays.

The Casagrande A line for soil classification gives $A = 0.73$ and $B = 20$, as shown in Fig. 4. For a large set of data of 520 soils and for I_P and w_L varying between 10-90% and 25-120%, respectively, Nagaraj and Jayadeva (1983) obtained $A = 0.74$ and $B = 8$ for organic soils, yielding a line slightly above the Casagrande line, which still agrees fairly well with data of Rio de Janeiro clays for soils with lower water content.

Four regions in the Plasticity chart were proposed by Futai (1999) and are represented in Fig. 4 and described in Table 2. Three of them are rather well fitted by line A, while region IV is well outside the range of Eq. (1), for Itaipú clay

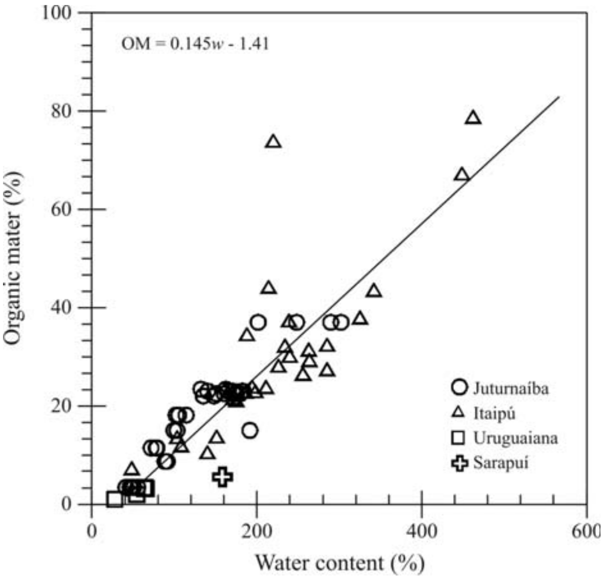

Figure 3 - Organic matter content and water content relationship for Rio de Janeiro clays.

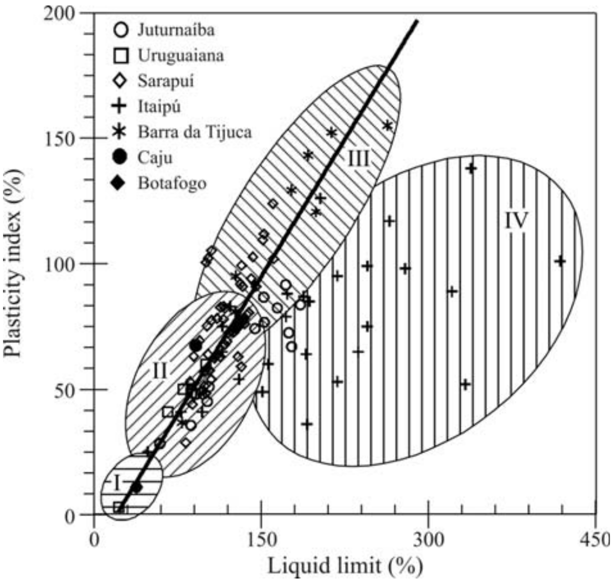

Figure 4 - Plasticity chart for Rio de Janeiro clays (Futai, 1999).

Table 2 - Classification of Rio de Janeiro clays (Futai, 1999).

Classification	Consistency	Plasticity	Compressibility	Strength	e_o	I_P	w_L (%)	$C_c/(1+e_o)$	S_u (kPa)	ϕ' (°)	Rio de Janeiro clays
Region I	Stiff inorganic clays	Low plasticity	Slightly compressible	High undrained strength	< 2	< 10	< 40	0.15-0.35	>50	28-40	Botafogo, Uruguaiana
Region II	Slightly soft organic clays	Medium plasticity	Compressible	Low undrained strength	2-4	10-120	30-200	0.25-0.35	6-15	25-35	Cajú, Barra da Tijuca, Sarapuí
Region III	Medium soft organic clays	High plasticity	Very compressible	Low undrained strength	4-6	>80	>100	0.40-0.60	6-25	30-40	Juturnaíba, Sarapuí
Region IV	Very soft organic clays - peat	Low plasticity	Very compressible	Low undrained strength	>3.5	>130	>150	0.25-0.35	10-25	< 65	Itaipú

and also for Juturnaíba clay, due to their high value of organic matter. This classification is also a function of compressibility and the range of the strength parameters, as shown in Table 2.

3. Compressibility and Stress History

The variation of the overconsolidation ratio OCR with depth, for eight clay deposits, is shown in Fig. 5. The OCR profile for all clays is within a narrow range, which suggests that the stress histories of the clay deposits in the Rio de Janeiro region are similar. The clay deposits sometimes present a dissecated crust with higher OCR value, which thickness can be as high as 4 m.

As seen in Table 1, the compression ratio *CR* of most clays is in the range 0.26-0.32. Outside this range are Barra da Tijuca, Itaipú and Sarapuí clays, with *CR* varying between 0.41 and 0.52, and Botafogo clay, with *CR* = 0.16. Data of compression ratio *CR* were obtained from conventional oedometer tests as well as radial oedometer tests.

Values of compression index C_c and the swelling index C_s of five clays are shown in Fig. 6. C_c and recompression index C_r values with depth present the same trend as the water content and plasticity profiles, as shown in Fig. 2.

Values of C_c of seven clays have been plotted against the water content, as shown in Fig. 7. The correlation found is $C_c = 0.013w$. Similar linear correlations have been obtained for other clays, $C_c = 0.01w$ for Chicago and Alberta normally consolidated clays (Koppula, 1981) of low sensitivity ($S_t < 1.5$); and $C_c = 0.0115w$ for a number of organic silts and clays (Bowles, 1979). Data of compressibility index of eleven reconstituted clays with liquid limit in the range 36-160% (Nagaraj & Miura, 2001) provided $C_c = 0.0103w$. The clay sensitivity is defined by the ratio between intact and remolded undrained strengths, thus it reflects the effects of soil structure cementation, thixotropy and aging, amongst other factors. It has been suggested

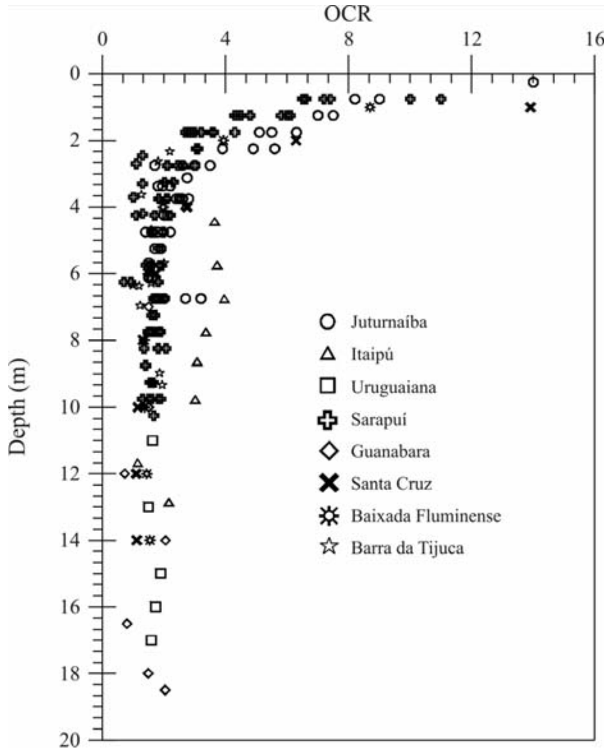

Figure 5 - OCR profiles of Rio de Janeiro clays.

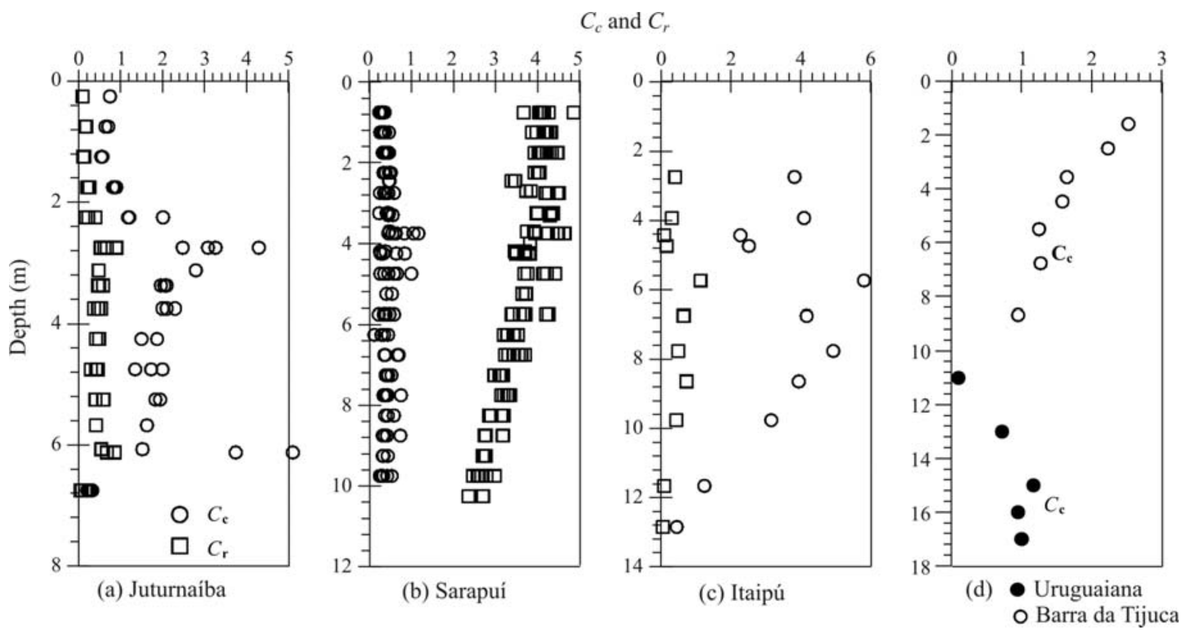

Figure 6 - C_c and C_r profiles of Rio de Janeiro clays.

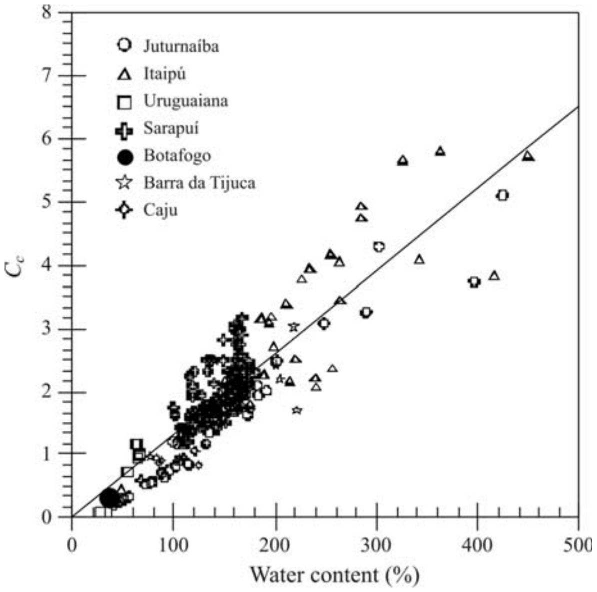

Figure 7 - Average relationship between C_c and water content of Rio de Janeiro clays.

(Leroueil *et al.*, 1983) that the compression index C_c increases not just with water content, but also with clay sensitivity S_t. This behaviour was also observed for Rio de Janeiro clays (Futai, 1999).

4. Summary and Conclusions

A summary of the geotechnical data of Rio de Janeiro clays was presented describing index properties, compressibility, stress history and undrained and drained strengths.

The majority of these clays presented high values of water and organic matter contents. The friction angle is also higher due to presence of fibres in the organic matter. The compression ratio *CR* is in the range of 0.26-0.32, however Barra da Tijuca, Itaipú and Sarapuí clays are more compressible, with *CR* varying between 0.41 and 0.52. The OCR profiles of these clays are very similar, thus indicating similar deposit formation.

The relationship between C_c and water content is $C_c = 0.013w$, well within the range of normally consolidated clays.

Acknowledgments

The geotechnical properties of Rio de Janeiro clays, summarised herewith, result from a large number of studies, particularly master and PhD dissertations. The authors are very much indebted to all involved in these studies, which made the preparation of this paper possible.

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Laboratory Behaviour of Rio de Janeiro Soft Clays. Part 2: Strength and Yield

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Abstract. This paper summarizes the yield and strength behaviour of clays studied in the State of Rio de Janeiro in the last forty years, with emphasis on the role of the clay structure on strength and yield. The undrained strength normalised by the overconsolidation stress is analyzed against the plasticity index I_P. Values of the clay friction angle ϕ' are also analyzed against the plasticity index I_P. Both data are compared with the available literature data, but the Rio de Janeiro clays have higher I_P and also show more data scatter. As far as yield is concerned, it is shown that the use of the SHANSEP method results in the destruction of the clay structure with the loss of its anisotropic characteristics. An evaluation of the level of structure destruction is presented.

Key words: properties of sedimentary clays, laboratory tests, Rio de Janeiro soft clays.

1. Introduction

The sedimentary deposits found in the coastal plains of the Rio de Janeiro State consist mostly of alluvia and marine deposits of the Quaternary Age. A number of soft clays deposits have been studied in the last 30 years in the City of Rio de Janeiro (Botafogo and Uruguaiana, Barra da Tijuca, Caju and St. Cruz) and vicinity (Fluminense Plains, Sarapuí, Itaipu and Juturnaíba dam), in association with some engineering works. In some of these deposits it was possible to perform research projects, by means of master or doctoral studies which generated a good quality data bank. Using this data bank, Almeida *et al.* (2008) summarized index and compression properties of well studied Rio de Janeiro sedimentary clay deposits in the last four decades. This paper complements this study and presents strength and yield behaviour of these clays. Emphasis is given to the role of the clay structure on both strength and yield. Table 1 summarizes the main geotechnical properties of the clays analyzed in the present paper, with emphasis on strength properties. Additional information on these clays may be obtained on Almeida *et al.* (2008).

2. Strength

2.1. The sample quality

It is initially useful to analyse the relationship between the undrained strength and the specimen diameter. Figure 1 shows the variation of undrained strength S_u of Sarapuí clay measured (Ortigão, 1980) by unconsolidated undrained triaxial UU tests normalised by S_{uref}, which is the undrained strength of UU tests carried out on the reference 127 mm diameter samples. The average values of S_u/S_{uref}, increase with a D_1/D_2 ratio expressed by:

$$\frac{\dfrac{D_{sh}}{D_{ts}}}{\dfrac{D_{shref}}{D_{sh}}} = \frac{D_1}{D_2} \tag{1}$$

where D_{sh} = shelby diameter; D_{shref} = reference shelby diameter (in this case, 127 mm) and D_{ts} = test specimen diameter.

The quality of test data depends on sampler diameter relationship (relation between the diameter of the shelby sampler used and the diameter of the reference shelby sampler) and also on the specimen diameter relationship (relation between the diameter of the sampler and the diameter

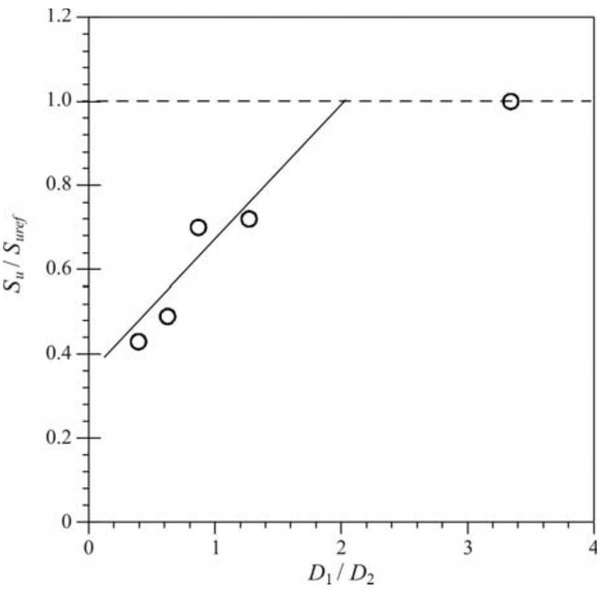

Figure 1 - Variation of S_u/S_{ust} with D_1/D_2 (Futai, 1999).

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Table 1 - Geotechnical properties of Rio de Janeiro clays.

Parameter/clay	Caju (b)	Sarapuí (c)	Itaipú (g)	Juturnaíba (h)	Uruguaiana (i)	Botafogo (j)	Barra da Tijuca (k)
References	Lira (1988); Cunha & Lacerda (1991)	Lacerda *et al.* (1977); Ortigão (1980); Almeida & Marques (2002)	Carvalho (1980); Sandroni *et al.* (1984)	Coutinho & Lacerda (1987)	Vilela (1976)	Lins & Lacerda (1980)	Almeida *et al.* (2000)
w (%)	88	143 ± 21.7	240 ± 110	154 ± 95.6	54.8 ± 15.9	35	100-500
I_P (%)	67.5	73.08 ± 16.1	74.5 ± 30.1	63.59 ± 22.1	40.5 ± 22.03	11	120-250
g (kN/m^3)	14.81	13.1 ± 0,49	12 ± 1,85	12.5 ± 1.87	16.1 ± 1.39	17.04	12.5
S_t	3	2,59 ± 0,69	4-6	5-10	3.00	-	5.0
% organic matter	-	4.13 - 5.54	32.63 ± 20.46	19 ± 10.63	2.56 ± 1.04	-	-
S_u (UU) (kPa)	6-12	8.64 ± 3.26	7.5 ± 3.53	18.7 ± 5.43	70.9 ± 25.1	20-90	15.5 ± 6
S_u vane (kPa)	-	8-20		6-30	-	70-110	6-30
S_u/σ'_{vm} (UU)		0.35	0.49	0.34	0.33	0.30	0.42
ϕ' (degrees)	27	25-30	21-65	25-65	34 ± 5	28	40
E_u/σ'_{v0} (UU)	171.5 ± 119	101 ± 154	59.7 ± 34.5	21 ± 0.16	40.4 ± 34.9	234 ± 123(CIU)	-
E_u/S_u	403.5 ± 47	129.7 ± 69.1	33.7 ± 12.1	101.7 ± 87.9	69.6 ± 29	292 ± 117(CIU)	-
e_o	2.38	3.71 ± 0.57	6.72 ± 3.1	3.74 ± 1.89	1.42 ± 0.36	1.1	-

of the test specimen). For higher D_{sh}/D_{ts} ratio, the specimen will be far enough from the sample edges, thus clay structure disturbance due to sample insertion is lower. Even though there is few data, the analysis of Sarapuí clay data suggests that D_1/D_2 values were equal or higher than 2. The test specimen diameter, therefore, should be less than:

$$D_{ts} = \frac{D_{sh}^2}{254} \qquad (2)$$

Substituting in Eq. (2) the more commonly used 4" shelby tube D_{sh} = 100 mm, it is concluded that a test specimen with a 38 mm diameter should be adopted. As this is the minimum test specimen adopted in triaxial tests, it is also concluded that the sampler must have diameters equal or higher than 100 mm, as recommended in the Brazilian Code of Practice for soil sampling (NBR 9820, 1997). This finding is valid for the standard specimen preparation procedure of sample extrusion from the shelby used for the present data analysis (Ortigão, 1980).

More recently a new procedure of sample preparation has been proposed (Ladd & De Groot, 2003) which is in increasing acceptance (Sandroni, 2006) and does not involve sample extrusion from the shelby. Therefore the above finding may not be valid if this procedure is used and further studies are necessary. The Soil Mechanics Laboratories of USP (LMS-EPUSP) and COPPE-UFRJ have used the Ladd and De Groot (2003) proposed procedure.

This finding is restricted to homogeneous non-fissured clays. For fissured clays, the sample should be large enough to include a representative number of fissures. Thus, a small sample is usually not representative.

2.2. The influence of the stress level on the natural clay structure

The influence of high stresses on the structure and anisotropy of clays will be further illustrated at this point. For this purpose, data of CIU triaxial tests, more easily available than CK_oU triaxial tests, will be used together with the clay structure disturbance index (Futai, 1999), ID, defined by:

$$ID = \frac{\sigma'_c}{\sigma'_{cm}} \qquad (3)$$

in which σ'_c is the confining stress applied to the triaxial specimen and σ'_{cm} is the isotropic overconsolidation pressure.

Figure 2 presents the variation of the normalised deviator stress $q/M\sigma'_c$ with ID for four clays, where q and M are respectively the deviator stress and the slope of the critical state line as used by Atkinson & Bransby (1978). Generally the structure disturbance index is defined as the relationship between the maximum confining stress and the isotropic overonsolidation pressure. However in this analysis, due to the lack of data of isotropic overconsolidation pressure, the overconsolidation pressures obtained after oedometer tests were used. Since the relationship between isotropic and oedometric overconsolidation pressure for natural clays varies in the range of 0.44 to 0.66 (Díaz-Rodriguez *et al.*, 1992), depending on the friction angle, the ratio between the ID presented in Fig. 2 and ID of Eq. (3) lies also in this range.

For higher ID, thus lower $q_f/M\sigma'_c$ ratios, structure disturbance is caused by isotropic consolidation with a loss of strength, associated to the loss of anisotropy. For ID lower than 1, the soil is at an overconsolidated state, where structure is very important.

Figure 2a presents the variation in $q/M\sigma'_c$ for Juturnaíba clay. The value of $q/M\sigma'_c$ decreases rapidly for ID smaller than 1, and then it gets about constant. Figure 2b shows a similar behaviour for other Rio de Janeiro clays.

The conclusion that may be taken from Fig. 2 is that when these clays are consolidated to about three times the overconsolidation pressure, there is a tendency for $q_f/M\sigma'_c$ to become constant. After this stress level is achieved, the clay structure is destroyed.

The interpretation and meaning of ID is illustrated in Fig. 3, in which the deviator stress normalised by the confining stress is plotted against ID. A low value of ID means that the soil is at an overconsolidated state, while a very high ID value is associated to a normally consolidated state of the soil. The strengths at points A and B represent the natural condition and remoulded strengths, respectively. Soil sensitivity can be related to the ID value. The strength of a sensitive soil drops from A to B, with small variations of axial deformation, indicating that soils with lower ID value presents higher sensitivity.

2.3. Undrained strength

The normalised undrained strength (S_u/σ'_{vm}, laboratory data, where σ'_{vm} is the overconsolidation pressure)

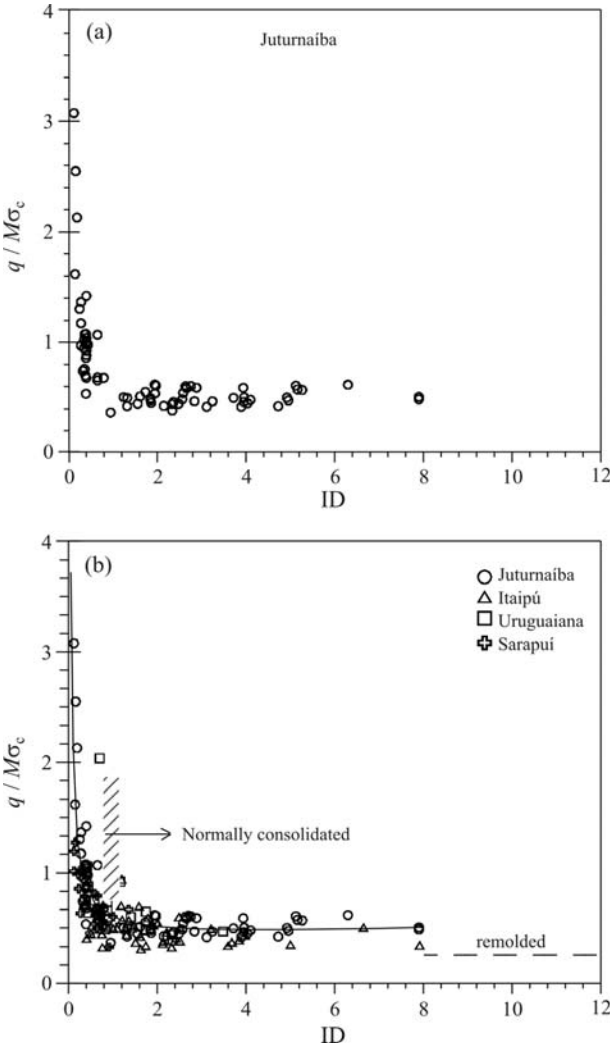

Figure 2 - Influence of structure and anisotropy on undrained behaviour of Rio de Janeiro clays (Futai, 1999).

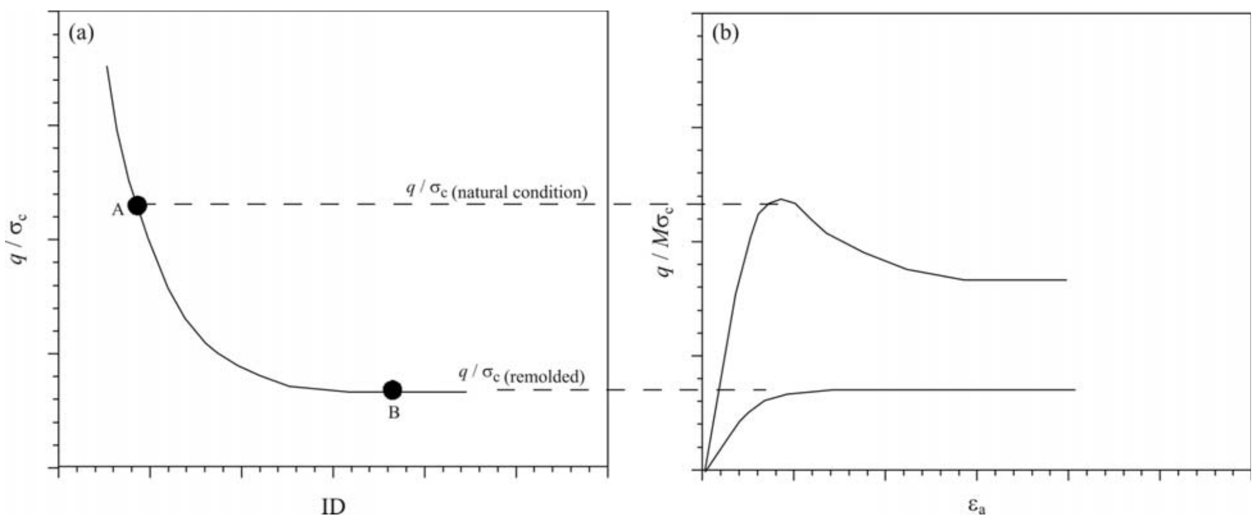

Figure 3 - Interpretation of ID parameter (Futai, 1999).

variation with plasticity index (I_P) for Rio de Janeiro clays is shown in Fig. 4, together with data from Eastern Canadian clays (Leroueil *et al.*, 1983; Marques *et al.*, 2004). The range of I_P for the Rio de Janeiro clay is higher than that of Canadian clays and because of scatter of the data a good fit is not easily obtained. Some of the of S_u/σ'_{vm} values for Rio de Janeiro clays are higher than the proposed relationship, which can be explained by sampling quality, since lower S_u/σ'_{vm} values reflect a higher quality of sampling with the increase of σ'_{vm}. In any case, it may be observed a trend of the increase of S_u/σ'_{vm} with I_P.

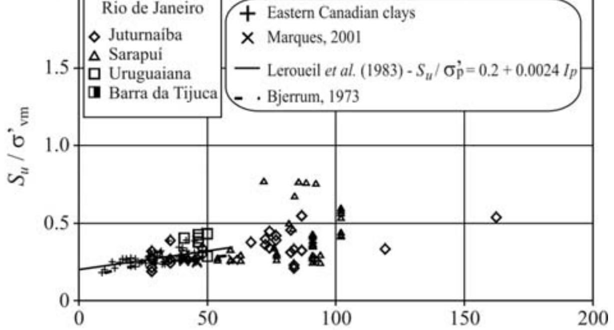

Figure 4 - Variation of S_u/σ'_{vm} with I_P.

2.4. Drained strength

Kenney (1959) obtained a relationship between clay friction angle (ϕ') and I_P and concluded that ϕ' decreases with I_P. Rio de Janeiro clays present this behavior just for I_P less than 40%. However, for values of plasticity index greater than 40% a significant scatter can be observed, as shown in Fig. 5, since ϕ' can be higher due to influence of fibres in the organic matter. Pinto (1992) also presented similar results for other Brazilian clays. Coutinho & Lacerda (1987) and Mitchell and Coutinho (1991) showed that the friction angle increases with organic matter content.

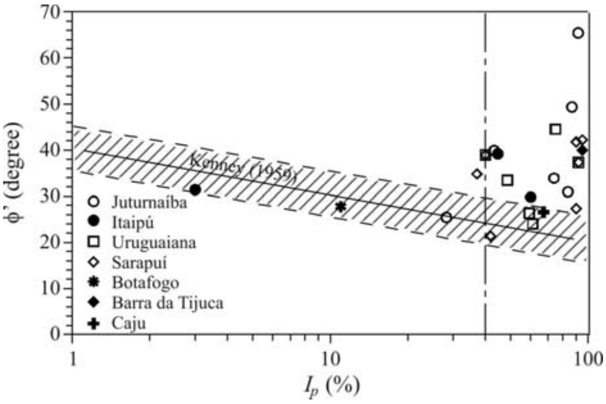

Figure 5 - Relationship between friction angle and I_P of Rio de Janeiro clays.

3. Yield

Yield curves obtained from CIU triaxial tests performed on Sarapuí (Ortigão, 1980) and Botafogo (Lins & Lacerda, 1980) clays are shown in Fig. 6. In these tests the natural clay structure was destroyed because of the application of stresses up to eight times the overconsolidation stresses, in a procedure similar to the SHANSEP (Ladd & Foot, 1974) method. In this figure q and p' are respectively the deviator stress and the average effective stress as used in the Critical State Theory. These variables are normalized against p'_e the Hvorslev equivalent pressure (Atkinson & Bransby, 1978) and M, the slope of the critical state line. As a result of high isotropic consolidation stresses have been applied, which destroyed the natural clay structure, the yield envelope is centred in the hydrostatic axis. This point is further discussed below.

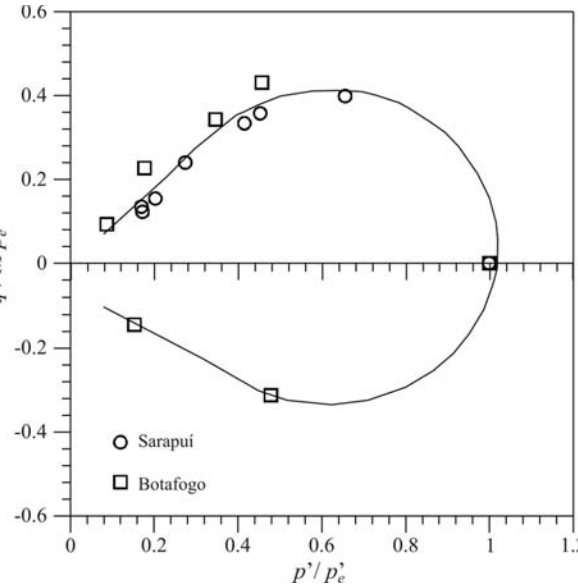

Figure 6 - Normalised yield curves of Sarapuí and Botafogo clays from CIU tests (Futai, 1999).

Yield curves of natural clays are usually normalised by the overconsolidation stress σ'_{vm} (*e.g.*, Díaz-Rodriguez *et al.*, 1992). For Rio de Janeiro clays, however, better representations of the yield envelopes were obtained by normalising data by the product $M\sigma'_{vm}$ (Graham *et al.*, 1988), where M is the inclination of the critical state line in the Cambridge q:p' plot, given by (Atkinson & Bransby, 1978):

$$M = \frac{6\sin\phi'}{3-\sin\phi'} \tag{4}$$

Figure 7 illustrates how M may be a useful normalising parameter of clay behaviour. The yield curves obtained using data from triaxial tests on overconsolidated specimens are shown in Fig. 8. Despite the lack of data for extension triaxial conditions, it is observed that all clays analysed

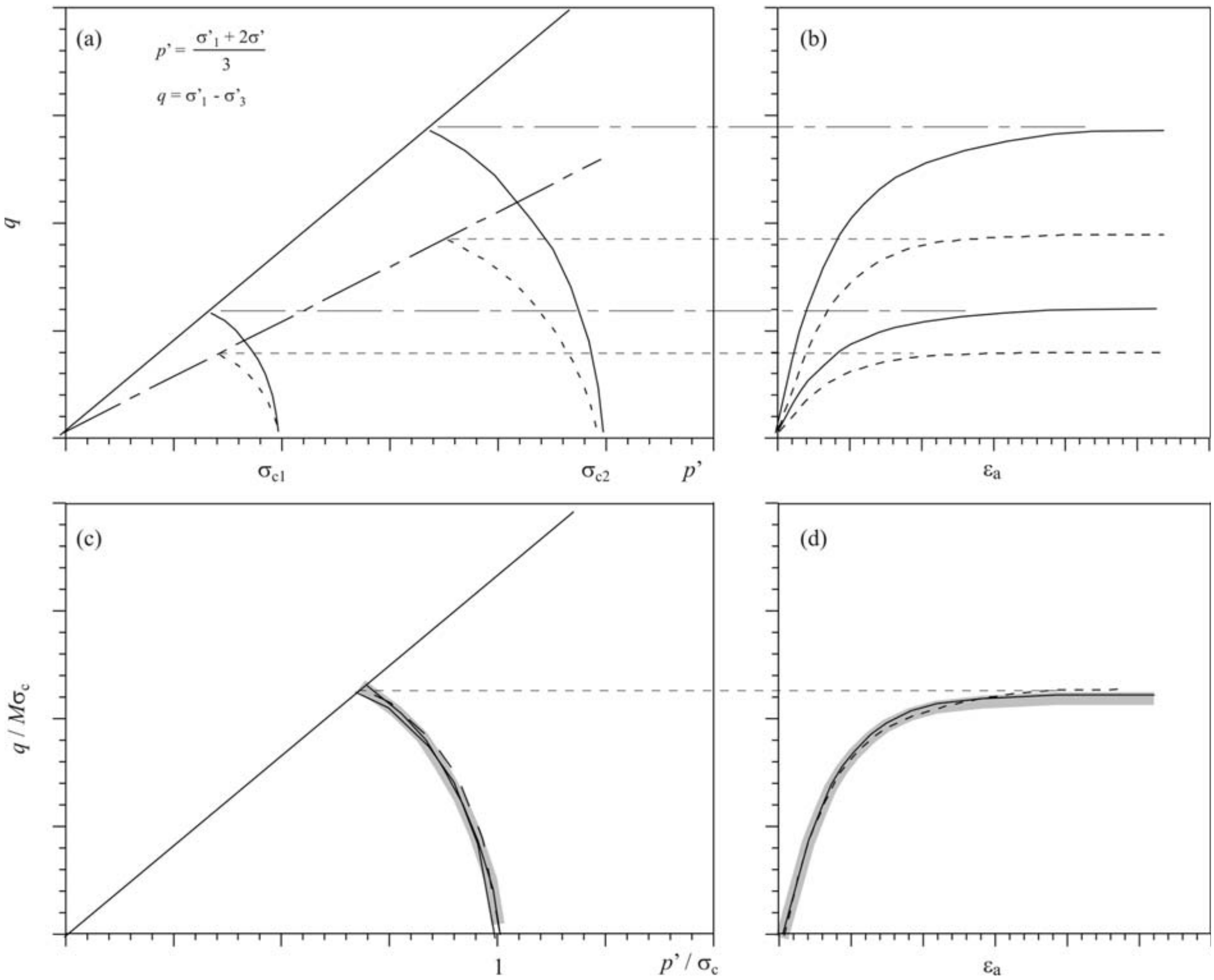

Figure 7 - Normalisation of clay behaviour.

show yield envelopes asymmetrical with respect to the hydrostatic line (p'/σ'_{vm} axis). These yield envelopes are in accordance with previous findings (*e.g.*, Díaz-Rodriguez *et al.*, 1992) and reflect the influence of the structure and anisotropy of natural clays.

Figure 9(a) shows all yield curves normalised with respect to overconsolidation pressure, in a single plot (Futai, 1999). However, the scatter shown in this figure is reduced when normalisation is made with respect to the average effective yield stress p'_m, as shown in Fig. 9b (Graham *et al.*, 1988), p'_m being defined by:

$$p'_m = \frac{\sigma'_{vm}}{3}(1+2K_0) \tag{5}$$

where

$$K_0 = 1 - \sin\phi' \tag{6}$$

Graham *et al.* (1988) proposed that this kind of normalization decreases the effects of mineralogy on the yield curve. Figure 9(b) shows that for the clays presented in this study, there are two well defined trends: the lower normalized yield curve represents soft clays and the other yield curve represents medium clays and organic clays, with high concentration of fibres and organic matter. The mineralogical composition of these clays is very similar; however, the organic matter has an important influence on the behaviour and may be responsible for the differences on the normalised yield curves. The data scatter in Fig. 9 is similar to that found by Graham *et al.* (1988) for clays less plastic than Brazilian clays.

The yield envelopes for Sarapuí clay presented in Figs. 8 and 9 are put together in Fig. 10 and the differences observed result from the high stresses used in the SHANSEP approach, which destroy the structure and anisotropy of natural clays. This behaviour is presented schematically in Fig. 11, where the stress increase expands and changes the shape of the yield envelope, which then becomes symmetrical with respect to the hydrostatic axis.

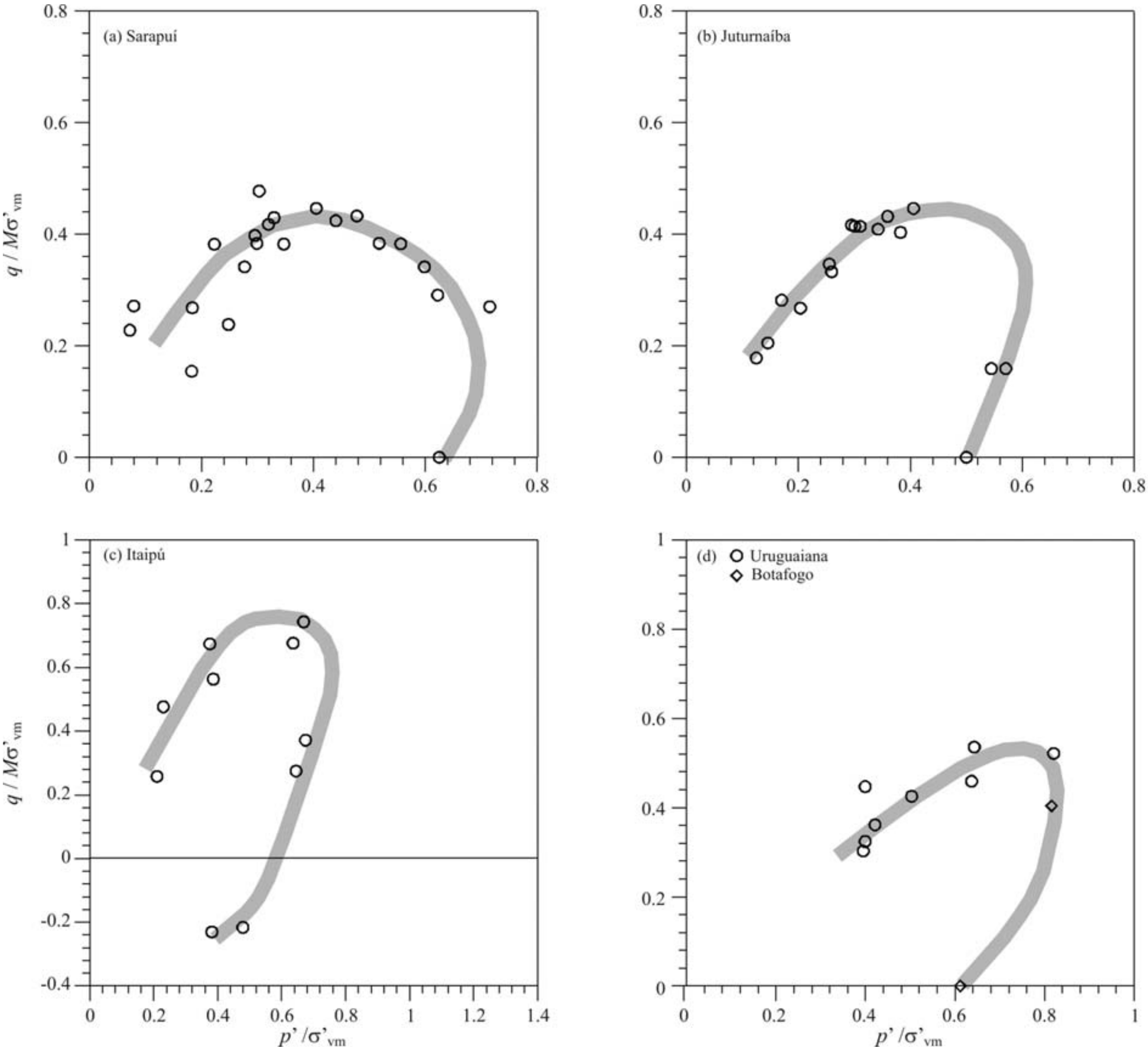

Figure 8 - Limit state curves of five Rio de Janeiro soft clays (Futai, 1999).

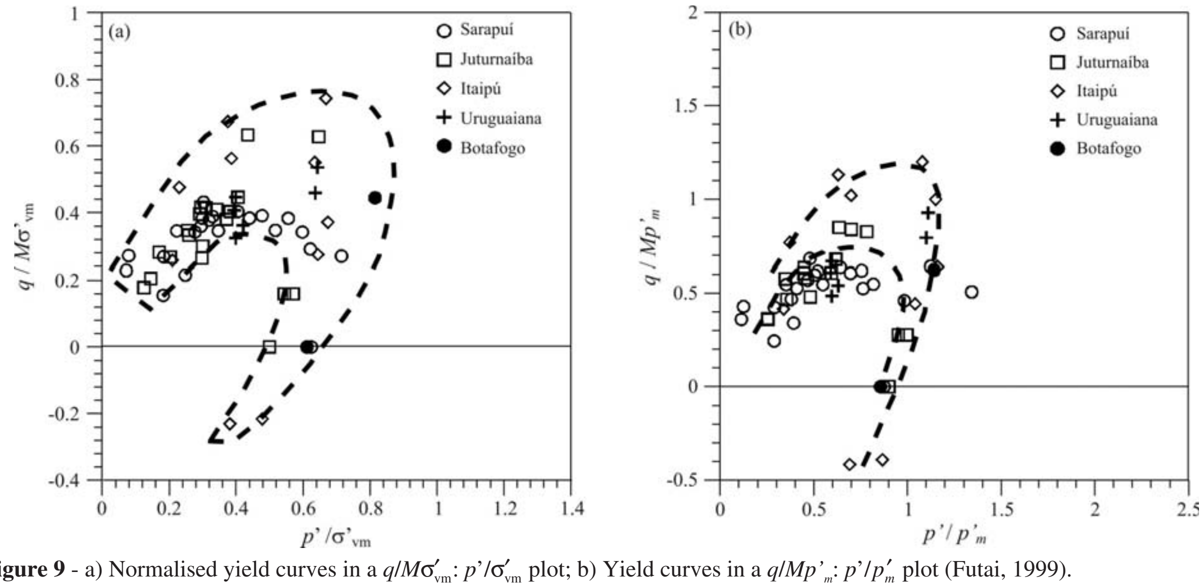

Figure 9 - a) Normalised yield curves in a $q/M\sigma'_{vm}$: p'/σ'_{vm} plot; b) Yield curves in a q/Mp'_m: p'/p'_m plot (Futai, 1999).

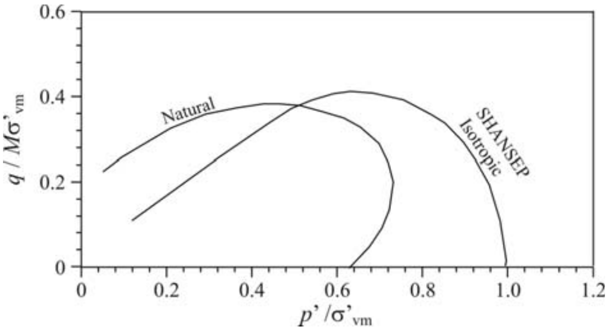

Figure 10 - Yield curves for Sarapuí clay (Futai, 1999).

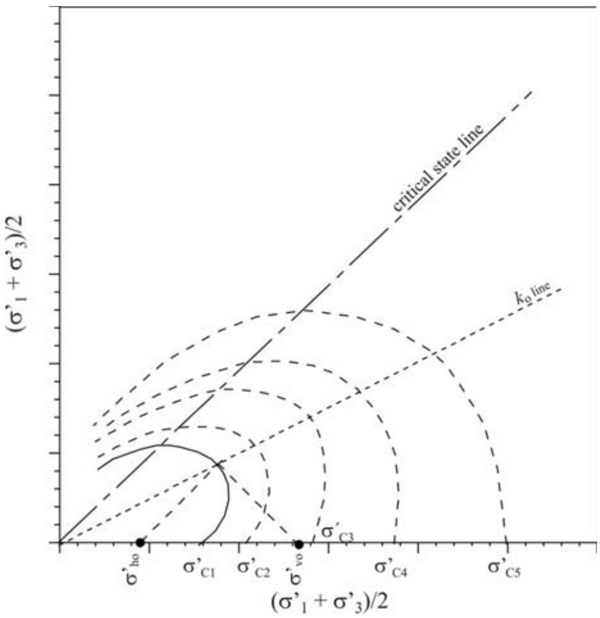

Figure 11 - Variations on the shape of yield curves with isotropic loading (Futai, 1999).

4. Conclusions

This paper summarized a study on the strength and yield behaviour of Rio de Janeiro sedimentary clays. Emphasis was given to the role of the clay structure on both strength and yield.

Regarding strength the following main conclusions were reached:

1. The specimen disturbance increases with the increase of the specimen diameter for the conventional technique of sample extrusion; further studies should be carried out to investigate the new technique for specimen preparation proposed by Ladd & De Groot (2003);

2. The normalised undrained strength S_u/σ'_{vm} plotted against the plasticity index showed that for the high I_P range of Rio de Janeiro clays S_u/σ'_{vm} values are higher than those of Canadian and Scandinavian clays.

3. It is shown that, for I_P values higher than 40%, the friction angle ϕ' increases with I_P due to the presence of the organic matter, which can contain fibers.

Regarding yield, the following main conclusions were reached:

1. It is observed that when high stresses are used, such as in the SHANSEP approach, the structure and the anisotropy of the natural clays are destroyed, which will influence its behaviour.

2. At lower stress levels the yield curves obtained were asymmetrical with respect to the hydrostatic axis p' due to the influence of anisotropy and structure of these clays.

3. The use of the ratio between maximum consolidation stress used in the test and clay overconsolidation pressure made possible to assess the influence of the degree of structuring caused by the consolidation process in the laboratory.

Acknowledgments

The geotechnical properties of Rio de Janeiro clays, summarised in this paper, result from a large number of studies, particularly master and PhD dissertations. The authors are very much indebted to all involved in these studies, which made the preparation of this paper possible. The authors are grateful to Esther Marques for reviewing the manuscript and for the suggestions provided. The authors are also grateful for the financial support from CNPq and Pronex/MCT.

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Soil Landslide Risk Charts of the Urban Area of Ponte Nova-MG

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Abstract. This paper addresses the construction of the soil landslide risk charts of the urban area of the city of Ponte Nova, Minas Gerais state, Brazil. A systematic survey of office and field data was considered to elaborate thematic risk charts (lithostructural, geological-geotechnical, slope and soil use and occupation), which when crossed permitted the construction of current and potential soil landslide risk charts. This study endeavored to supply a diagnosis of the risk situations in the urban area of Ponte Nova, to help in the adoption of future short, mid and long term political and structural measures to reduce, correct and prevent catastrophic events in areas associated to urban occupation.
Key words: risk chart, landslides, Ponte Nova, thematic risk charts.

1. Introduction

The phenomenon of hillside and slope instabilities frequently causes accidents whose damage can be greatly increased when they occur in urbanized areas. Modifications in topography, hydrological and hydrogeological conditions and geomorphological processes, especially those that result from urbanization of new areas, are often the main causes of these phenomena.

Regarding the result of anthropogenic action in urban areas, generally with a high occupation density of the flat areas or those with little slope, the occurrence of landslides is due to the fact that urbanization expands to the steep hillsides that are geotechnically less favorable for use. This is a common problem in the underdeveloped and developing countries, where disorderly growth and occupation of geotechnically unsuitable areas, especially by the low income population, has created an increasing number of problems of hillside instabilities.

Considering the importance of cartography of landslide risks for planning of the use and occupation of open areas, the present paper is concerned with the elaboration of soil landslide risk charts for the city of Ponte Nova based on a study by Natali (1999).

2. Information on the Study Area

2.1. Geographic location

The study area is the urban portion of Ponte Nova, located in Zona da Mata, southeast of Minas Gerais State, covering approximately 9 km^2. The area lies between the UTM 7740/7743 and 716/721.5 coordinates and the 20°25'00" latitude south and 42°54'40" longitude west geographic coordinates.

Ponte Nova is bounded by the municipalities of Rio Doce, Santa Cruz do Escalvado, Urucânia, Guaraciaba, Teixeiras, Amparo do Serra, Jequeri, Barra Longa, Acaiaca and Oratórios and has, according to the IBGE (2000), a resident population of 48.997 inhabitants in the urban area.

2.2. Physiographic aspects

The Cwa and Aw climatic types occur in the Zona da Mata region (microregion of the Ponte Nova Forest) according to the Köppen climate classification. The Cwa type, humid with hot summers, is predominant in the higher zones and is characterized by a short dry season, mean annual temperature between 19.5 °C and 21.8 °C and annual rainfall between 1,100 mm and 1,400 mm. The Aw climate,

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tropical wet and dry, dominates the lower parts of the area, with mean annual temperature between 21.4 °C and 24.7 °C, well-defined seasons and rainfall concentrated from October to March.

The regional hydrographic network belongs to the Doce River Basin, where the main rivers are the Piranga and the Casca, that drain, respectively, the East and West portions of the region from south to north, forming a drainage network characterized by lithostructural control.

The municipality of Ponte Nova is cut mainly by the Piranga River that rises in the Serra da Mantiqueira Mountains and is called the Doce River after the confluence with the Carmo River, close to the city of Rio Doce. The Vau-Açu stream and the Manso, Pedreira and Passa Cinco creeks are the main affluents of the Piranga River within the research area.

The vegetation of the region is characterized by fragments of Atlantic Rain Forest, more usually found on hill tops. The current floral aspects reflect anthropogenic action on the natural environment, and one can refer to a landscape combined of pastures and scrub, fragments of native forest but pastures predominate significantly.

The study region is characterized geomorphologically by low round-topped hills shaped like half oranges, a typical structure of granite-gneiss landscapes, gentle hillsides and open valleys. The morphological features of the area shows a strong influence of the geological variables and one can refer to the lithology, stratification and faults.

The urban area studied of Ponte Nova lies within the limits of a semi open valley, concentrated in the lower parts, on the banks of the Piranga River, and spreads progressively to the hillsides. The region is divided into four dominions, characterized as follows: (i) orthogneiss of the Mantiqueira complex; (ii) paragneiss of the São Sebastião do Soberbo metamorphic suite; (iii) metavolcanic and sedimentary sequence of the Dom Silvério group; (iv) high degree rocks from the Juiz de Fora complex.

3. Methodology

3.1. General considerations

In the present study, the whole urban sector of Ponte Nova was defined as an area of interest for the elaboration of the landslide risk chart. A data set was used regarding: (i) 18 topographic bases; (ii) 17 pairs of aerial photographs on the scale of 1:8,000; (iii) orthophotographs on the 1:10,000 scale as a base for the geological field mapping; and (iv) the IDRISI software version 2 for WINDOWS to generate maps in the "raster" format.

3.2. Preliminary survey of the area and past landslides description

The urbanized neighborhoods and those undergoing urbanization in Ponte Nova were surveyed using the orthophotograph on the 1:10,000 scale, mainly those located in the hillside sectors. The set of critical points was located on the base map (orthophotograph) for the geological-geotechnical assessment in the investigation phase. The types of characteristics landslides of the area including mainly soil/saprolite and block falls were identified using this preliminary field survey, the data collected at the inventory stage and with the help of aerial photographic analysis performed at the Minas Gerais Electrical Company (CEMIG) Office in Ponte Nova.

3.3. Field investigation

In this phase, most of the resources necessary to elaborate the chart of landslide risks for Ponte Nova were assembled and the conditioners and influencing attributes on the landslide processes were defined and the systematic field surveys carried out.

3.3.1. Conditioner and attribute definition

To draw up the chart of landslide risks, the following conditioners/attributes were investigated, considering the type of landslides defined in the study area, as follows:

• Geological-geotechnical compartments: the lithology, main geological structures (stratification, fault, fracture), other features of interest, such as landslide scars, rock outcrops and existing rock outcrops and their influences on the landslide processes in the study area;

• Slope: the critical slope intervals were considered for triggering landslide processes, that were analyzed by the chart of slope risks using the SIG/IDRISI software;

• Soil use and occupation: the form of use and occupation of the urban soil was taken into account, the population flow of each neighborhood located on the hillsides, construction types, cut slopes inclinations for house and road constructions, basic infrastructure (rainwater drainage system and street surfacing) and their effects on landslides and correlated processes;

• Vegetation: the existence of vegetation foci was verified on the urbanized hillsides and their relationships with the mass gravitational movement processes;

• Rainfall: rainfall data from January 1960 to February 1998 were analyzed to obtain correlations with the past landslides.

3.3.2. Systematic field surveys

The systematic field surveys included two main and parallel activities, namely: (i) lithostructural mapping; and (ii) landslide recording and the description of natural and cut slopes. The studies were carried out in a period of approximately 60 days, with logistical support from the Ponte Nova City hall.

The lithostructure was mapped using as base map two orthophotographs on the 1:10,000 scale from 1987, supplied by CEMIG. The mapping study involved cartography and description of the lithology and geological structures that presented direct relationships with the landslides and correlated processes, such as stratification, faults and fractures. The landslides were recorded and the natural and cut slope inclinations were described parallel to the development of the lithostructural mapping.

3.4. Photographic interpretation and cartography

This stage was developed by interpreting aerial photographs and orthophotographs that enabled observation of standard features characteristic of landslides and the local geology. Pairs of aerial photographs on the 1:8,000 scale and two orthophotographs on the 1:10,000 scale covering all the urban area of Ponte Nova were used throughout the study.

The cartography studies were carried out shortly after finishing the field surveys, using photographic interpretation. Pre-existing maps and data were used in this task, along with data collected in the field phase and aerial photographic interpretation. The required thematic charts were drawn up for the elaboration of the final chart of landslide risks. The main maps elaborated were:

- Lithostructural, developed from systematic field surveys and data collected during photo interpretation. They were on the 1:10.00 scale, based on two orthophotographs on the 1:10,000 scale dated 1987 that was supplied by CEMIG. This map contained the lithology and main structural features of the area including their atitudes;
- Geological-geotechnical, prepared on the 1:10,000 scale and containing the main parameters of the study area, such as: lithology, structural features, rock outcrops, landslide scars, slopes and points of water springs. This information was collected in field surveys, photographic interpretations on the 1:8,000 scale and in orthophotographs on the 1:10,000 scale;
- Slope chart, elaborated for the Ponte Nova urban area based on the classification system used by IPT (1991). This chart was developed from a planialtimetric base using the IDRISI software version 2;
- Soil use and occupation chart, elaborated on 2,000 scale from the planialtimetric base, orthophotographs on the 1:10,000 scale and aerial photographs on 1:8,000 scale.

3.5. Use of digital cartography techniques and SIG

At this stage, the charts and thematic maps generated in the study were transformed to the digital format, using the AutoCAD R14 program for data application of the SIG techniques by the IDRISI software version 2 for Windows.

4. Results

4.1. Systematic field surveys

Tables 1 and 2 show the results of the field surveys, complemented by the interpretation of the aerial photographs and orthophotograph. Table 1 shows the position of the visited cut slopes, the cut slopes that presented instabilities (superficial or deep slides and/or slide scars), and theirs respective slope inclinations. Table 2 summarizes the situation of urban occupation, vegetation and drainage in the neighborhoods.

Regarding urban occupation, in Table 2 the term orderly was used to designate the neighborhoods that presented basic infrastructure, such as paved streets and electrical cable network, rainwater discharge network, sewage network, and the term disorderly was used to categorize the

Table 1 - Inclinations of the visited cut slopes and of the unstable slopes.

Neighborhood	Inclination of the visited cut slopes				Inclination of the unstable slopes			
	$15° < \phi \leq 45°$	$45° < \phi \leq 60°$	$60° < \phi \leq 75°$	$75° < \phi \leq 90°$	$15° < \phi \leq 45°$	$45° < \phi \leq 60°$	$60° < \phi \leq 75°$	$75° < \phi \leq 90°$
Tijuco				1				1
Guarapiranga	1	1	1	4		1	1	3
N.S. Auxiliadora				1				1
Vale Verde	1				1			
Santo Antônio	1			3	1			2
Triângulo				3				3
Triângulo Novo	1			2				1
CDI				2				
Vila Alvarenga			1	2				
S.C. de Jesus			1	8			1	3
São Geraldo				1				1
Esplanada			1	2			1	

Table 1 (cont.)

Neighborhood	Inclination of the visited cut slopes				Inclination of the unstable slopes			
	$15° < \phi \leq 45°$	$45° < \phi \leq 60°$	$60° < \phi \leq 75°$	$75° < \phi \leq 90°$	$15° < \phi \leq 45°$	$45° < \phi \leq 60°$	$60° < \phi \leq 75°$	$75° < \phi \leq 90°$
Vila Pachequinho				1				
Copacabana				2				1
Centro			1	4			1	3
10 de Maio				1				1
Gavetão			2	3				3
Vila Oliveira				4				2
Cidade Nova				1				
Fátima				3				1
São Pedro				5				
Novo Horizonte				1				
Nova Almeida				1				
Palmeiras			1	1				1
Raza			1	2				1
Lot. Novo 1				1				
Lot. Novo 2				1				1
Paraíso			1					
Antônio Girundi				1				

Table 2 - Urban occupation, vegetation and drainage of Ponte Nova neighborhoods.

Neighborhood	Urban occupation	Vegetation		Drainage		
		Tree	Low	Natural	Superficial	Rain water collector system
Tijuco	Sparse (orderly)	Sparse	Sparse	Efficient	Deficient	Deficient
Guarapiranga	Dense (orderly)	Sparse	Sparse	Deficient	Deficient	Efficient
Auxiliadora	Medium (orderly)	Sparse	Medium	Deficient	Deficient	Efficient
Vale Verde	Médium (orderly)	Sparse	Sparse	Efficient	Deficient	Efficient
Santo Antônio	Dense (orderly to disorderly)	Sparse	Sparse	Deficient	Deficient	Efficient
Triângulo	Dense (orderly to disorderly)	Sparse	Sparse	Efficient	Deficient	Deficient
Triângulo Novo	Dense (orderly to disorderly)	Sparse	Sparse	Efficient	Deficient	Deficient
CDI	Sparse (orderly)	Sparse	Sparse	Efficient	Deficient	Deficient
Vila Alvarenga	Dense (desorderly)	Medium	Medium	Efficient	Deficient	Deficient
S. Coração de Jesus	Dense (orderly to disorderly)	Sparse	Sparse	Efficient	Deficient	Deficient
São Geraldo	Médium (orderly to desorderly)	Sparse	Sparse	Deficient	Deficient	Deficient
Esplanada	Dense (orderly)	Sparse	Sparse	Deficient	Deficient	Deficient
Vila Pachequinho	Medium (desorderly)	Medium	Medium	Efficient	Deficient	Deficient
Copacabana	Médium (orderly)	Sparse	Medium	Efficient	Deficient	Deficient
Centro	Dense (orderly to desorderly)	Sparse	Inexistent	Efficient	Deficient	Efficient

Table 2 (cont.)

Neighborhood	Urban occupation	Vegetation		Drainage		
		Tree	Low	Natural	Superficial	Rain water collector system
10 de Maio	Dense (orderly to desorderly)	Sparse	Sparse	Deficient	Deficient	Efficient
Vila Oliveira	Medium (orderly to desorderly)	Sparse	Sparse	Efficient	Deficient	Deficient
Cidade Nova	Sparse (orderly to desorderly)	Sparse	Sparse	Efficient	Deficient	Deficient
Fátima	Dense (orderly to desorderly)	Sparse	Sparse	Efficient	Deficient	Deficient
São Pedro	Dense (orderly to desorderly)	Sparse	Sparse	Deficient	Deficient	Deficient
Novo Horizonte	Sparse (orderly to desorderly)	Sparse	Sparse	Efficient	Deficient	Deficient
Nova Almeida	Medium (orderly)	Sparse	Sparse	Deficient	Deficient	Deficient
Palmeiras	Dense (orderly)	Sparse	Inexistent	Deficient	Deficient	Deficient
Raza	Médium (orderly)	Medium	Médium	Efficient	Deficient	Deficient
Lot. Novo 1	Inexistent	Sparse	Médium	Deficient	Deficient	Deficient
Lot. Novo 2	Inexistent	Sparse	Medium	Efficient	Inexistent	Inexistent
Paraíso	Sparse (orderly)	Sparse	Sparse	Efficient	Deficient	Deficient
Antônio Girundi	Sparse	Sparse	Medium	Deficient	Inexistent	Deficient

neighborhoods that presented some deficiency in this sense. The classification of orderly and disorderly was also used to characterize the neighborhoods that showed simultaneous levels of efficiency and deficiency in their basic infrastructure.

It is pointed out that usually the neighborhoods with orderly to disorderly occupation were those that presented good initial planning, but as they grew and became populated, they extrapolated the limits of this planning, making them deficient in some aspects of the basic infrastructure. It can also be stated that most of these neighborhoods presented high population density, occupied most of the hillsides in Ponte Nova or began to develop in a part of the town and grew to the hillside, such as the neighborhoods of Santo Antônio, Triângulo, Triângulo Novo, Sagrado Coração de Jesus (Pacheco), São Geraldo, Centro, 10 de Maio, Vila Oliveira, Cidade Nova, Bairro de Fátima, São Pedro and Novo Horizonte.

The tree and scrub vegetation was sparse in almost all of the occupied area and was almost always represented by fruit, flowers and low growing plants typical of back yards, such as grasses. There was denser vegetation on the hillside tops, where remains of native vegetation combined with pasture were observed, but outside the occupied areas.

The urban perimeter of Ponte Nova was in a precarious situation regarding drainage structures, as understood from the data presented in Table 2. Throughout the study area there was a great deficiency in the drainage systems, whether natural, surface or rainwater. Consequently, the areas where there were no efficient drainage systems, a fact almost always caused by disorderly urban growth, was where the greatest risks of soil mass landslides were found. In these areas, after executing cuts and fills for civil engineering construction, the removed material was usually thrown down the hillside, without technical criteria, adding to the inconvenience of waste disposal and rubble resulting from improper occupation. In the rainy period, especially, these loose materials slid under the action of their own weight or by the reduction of their shear strength and destroy engineering works further down the hillsides.

4.2. Rainfall data

Rainfall data of the area under study collected at the Piranga rainfall station (code 02043010) and supplied by CEMIG are presented in Figs. 1, 2 and 3, encompassing the period 01/1960 to 12/1998.

Figure 1 shows that the period of greatest rainfall in the region studied was from October to March, and January was the month with the highest monthly average rainfall (55,99 mm), followed by December (49.6 mm) and in third place November (49.19 mm). The lowest rainfall measurements were recorded in June (7.52 mm), July (7.69 mm) and August (10.2 mm). It is known that water is usually considered the main generating agent of mass gravitational movements and that rain duration and frequency are the most important factors in triggering landslide processes. Corroborating this point of view, it was observed that the periods when the highest rainfall was recorded, that is, from October to March, as shown in Fig. 1, the highest indices of accidents related to landslides occurred.

Figure 2 depicts the maximum monthly rainfall from October 1996 to March 1997, which is the wet season in the

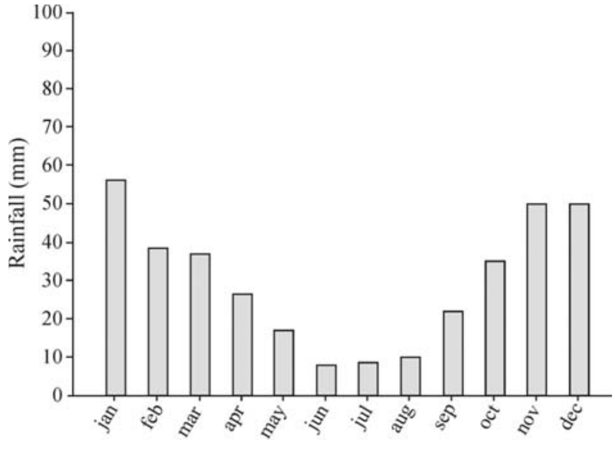

Figure 1 - Means of the maximum monthly rainfall in Ponte Nova from January 1960 to February 1998.

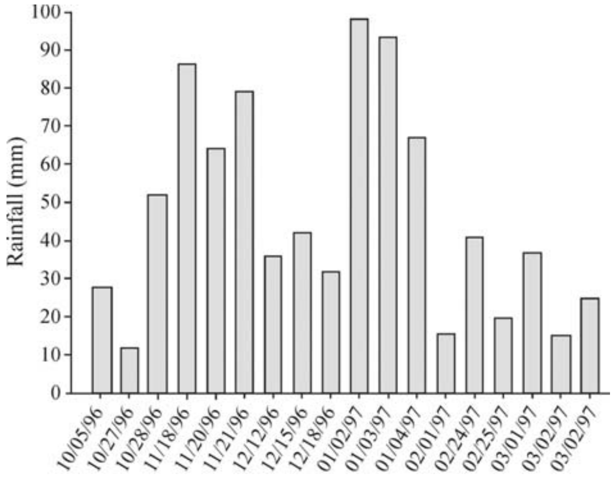

Figure 3 - Maximum rainfall of the three wettest days of each month in Ponte Nova from October 1996 to March 1997.

region. One of the highest monthly rainfalls of the last 20 years was observed in this period, reaching 98.3 mm rain in January.

It can be concluded from the analysis of the rainfall data presented in Figs. 2 and 3 that in the period from October 1996 to March 1997, January was the month that presented the highest rainfall rates and the values observed were among the greatest in recent times. It was at the start of this month (January 1997), between the days 01, 02 and 03 that there was one of the greatest floods of the Piranga River in the period analyzed. In this month there were also several landslides with housing destruction in Ponte Nova that were recorded by the Ponte Nova Civil Defense.

Figure 3 shows the maximum rainfall for the three wettest days of each month, also for the period from October 1996 to March 1997. This figure shows that on the days that it rained most, 02/01/97 with a rainfall of 98.3 mm; the next day, 3/01/97, with 91.3 mm and the days 18/1196 and 21/1/96, with daily rainfall rates of 86 mm and 79 mm, respectively.

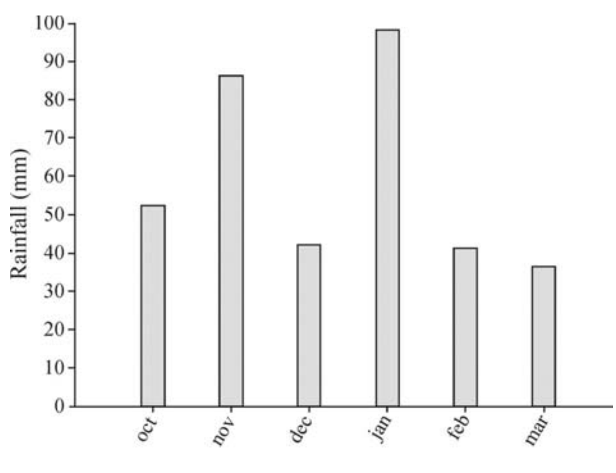

Figure 2 - Maximum daily rainfall of each month in Ponte Nova from October 1996 to March 1997.

4.3. Photographic interpretation and cartography

The photographic interpretation studies, based on the use of 17 pairs of aerial photographs on the 1:8.000 scale and two orthophotographs on the 1:10,000 scale, covered the urban area of the municipality of Ponte Nova. The photographic interpretation served as support for the systematic field survey, when the geological, lithostructural, soil use and occupation maps were made. The cartography studies resulted in the following maps and risk charts:

- Lithostructural map: this map, Fig. 4, contains the lithology and the main structural features and their attitudes in the study area. The whole area of residual altered gneiss soil was mapped and at some distinct points, gneiss outcrops were observed. An outcrop of basic rock (amphibolite) was also mapped and some recent formations, mainly in islands formed in the Piranga river bed. The gneiss stratification was outstanding among the structural features observed in the area, with angles between 5° and 30° but no preferential orientation in function of the intense deformation of the gneiss in this region;
- Geological-geotechnical map: this map, Fig. 5, shows the lithology, rock outcrops, landslide scars, water spring points, and identified erosion foci. These parameters were determined by field surveys and photographic interpretation studies. The landslide scars are highlighted in this map that occurred in the mapped area, mainly on the disorderly occupied hillsides. It is also of interest to observe the points of water springs and erosion foci;
- Slope chart: this chart is shown in Fig. 6, and the values of the percentages used are shown in Table 3, according to the classification criteria adopted by IPT (1991). It is emphasized that in function

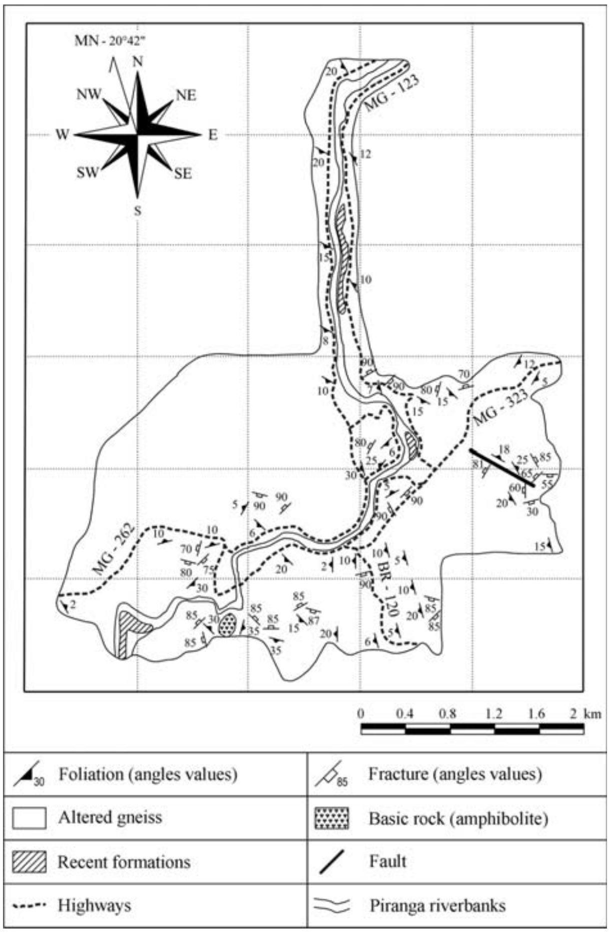

Figure 4 - Lithostructural map of the urban area of Ponte Nova.

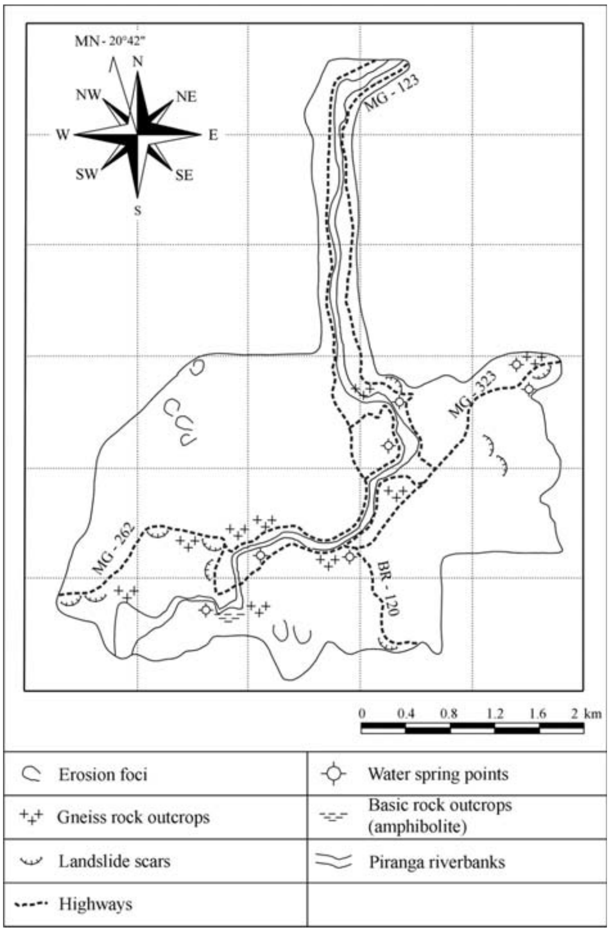

Figure 5 - Geological-geotechnical map of the urban area of Ponte Nova.

Table 3 - Slope intervals adopted, in percentage (IPT, 1991).

Interval	Significance of the limits
0 to 15%	15%: maximum slope tolerated for vehicle circulation
15 to 30%	30%: maximum slope permitted by law for hillside occupation without special planning
30 to 50%	50%: technically recommended slope limit for hillside occupation

of the characteristics of the terrain to be occupied, the slope contours may suffer alterations, decreasing the limit value by 50% for smaller contours according to the geotechnical study of the area. Technically, it is emphasized that areas with more than 50% slope can also be occupied, providing there are geotechnical solutions available;

- Soil use and occupation chart: this chart, Fig. 7, shows the types of characteristic occupation of the area studied. Four types of occupation were mapped, considering mainly the population flow of each neighborhood analyzed as follows: dense occupation, medium occupation, sparse occupation and nonexistent occupation. Of these four, the areas of dense occupation make up most of the city of Ponte Nova and presented the greatest concentration of landslides. The nonexistent occupation was characterized by contours within the urban area that have not yet been used by the population. In these locations, the vegetation was in the form of pastures, landslide occurrence was not observed and a direct relationship was inferred between anthropological action and susceptibility to landslides.

4.4. Current and potential soil landslide charts of the urban area of Ponte Nova

By crossing the thematic maps developed in the present study, following the suggestion by Cerri (1990), two soil landslide risk charts were obtained for the urban area of Ponte Nova, considering the current and the potential risks. Figures 8 and 9 show both the maps, elaborated on the

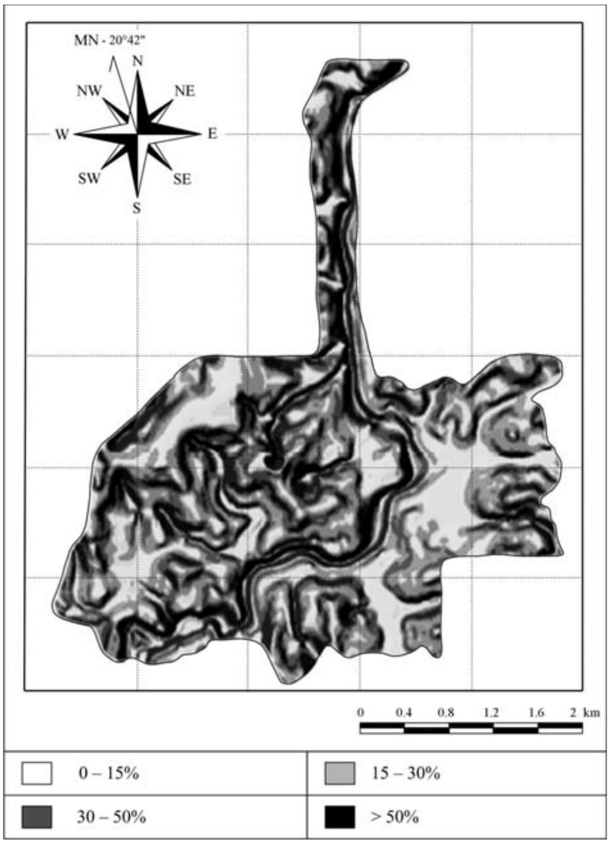

Figure 6 - Slope map of the urban area of Ponte Nova.

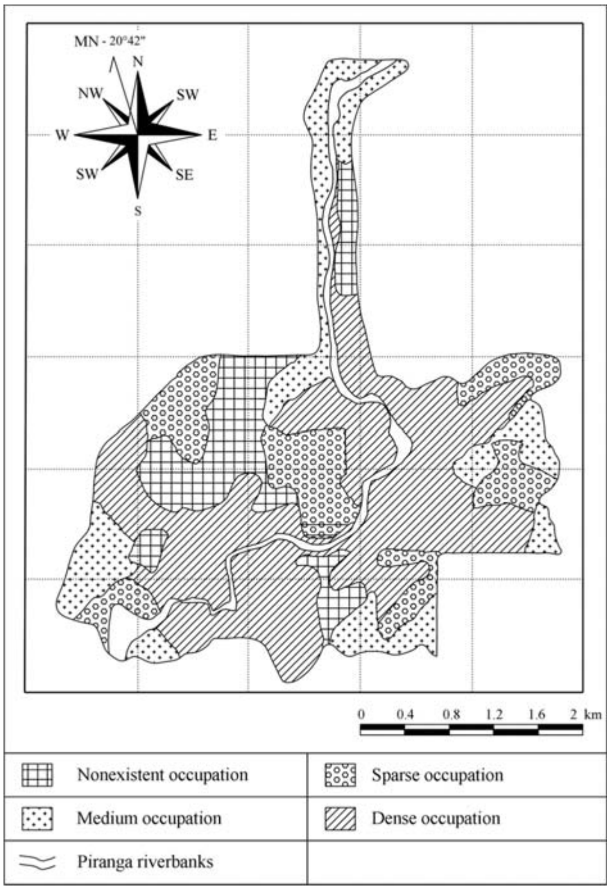

Figure 7 - Soil use and occupation chart of the urban area of Ponte Nova.

1:10,000 scale that have keys showing risk areas, their characteristics and recommendations for use and occupation, subdivided for the current risk and the potential risk situations. The use of the term *landslide risk* refers to the areas that may suffer movement and/or mass soil detachment.

It is emphasized that when elaborating the charts of landslide risks for the urban area of Ponte Nova a risk analysis was adopted where the classes and degrees of risk resulted from the qualitative assessment of the risk situations identified, conjugating the areas susceptible to landslides and possible social and economic damage. Several parameters were considered for the identification of the risk situations, notably vegetation, monthly and daily rainfall rates for the rainy season, anthropogenic action and especially slope inclination.

Four degrees of risk were adopted in the landslide risk classification, namely inexistent, low, medium and high. These degrees of risks were used to determine the zones with real or potential risk (susceptibility). It is pointed out that the use of this subdivision of risks aimed to establish, within the study region, areas that should receive immediate action (current risk) or planning (potential risk) from public authorities. Thus the different degrees of risk have the function of helping public administration in Ponte Nova in prioritizing these structural and nonstructural actions to be taken in the future.

It is pointed out, that in the study area the landslides occurred most frequently in young residual pink colored gneiss soil and less frequently in the soil with an overlay of red sand-clay colluvium. It was observed these landslides followed predominantly a circular rupture patter and were mostly induced by anthropogenic intervention combined with periods of heavy rainfall. It was detected that all the landslide scars analyzed and mapped were related to the occurrence of cut slopes without technical criteria, with unsuitable height and geometry and no superficial drainage structures. Furthermore, based on the analysis of the aerial photographs and field visits, landslides were not identified in the natural hillsides within the slope contours reported in the slope map presented in Fig. 6.

5. Conclusions

This is study endeavored to present a basic diagnosis of the risk situations in the urban area of the city of Ponte Nova, Ponte Nova, Minas Gerais state, Brazil, by establishing a landslide risk chart to (i) supply technical elements to help in aspects pertaining to urban planning of the city and

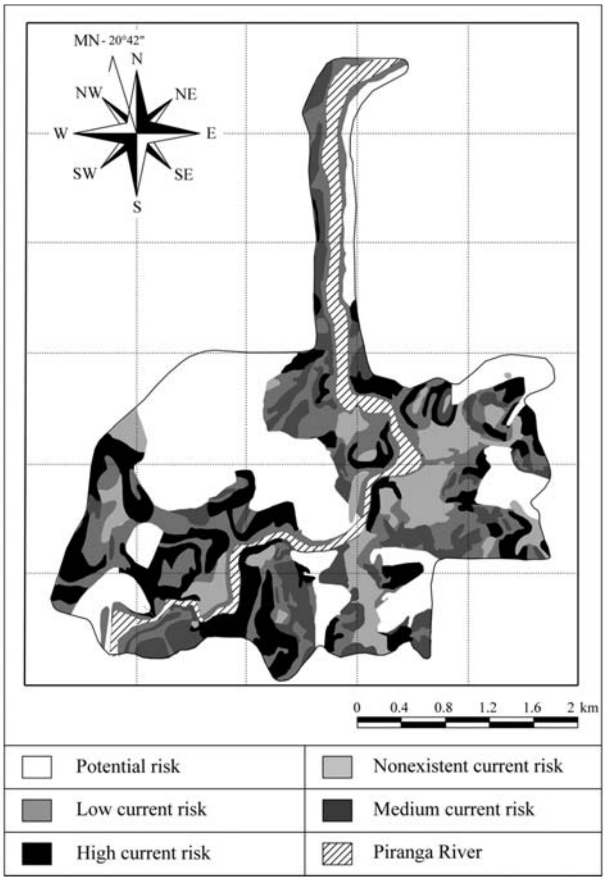

Figure 8 - Current soil landslide risk chart of the urban area of Ponte Nova.

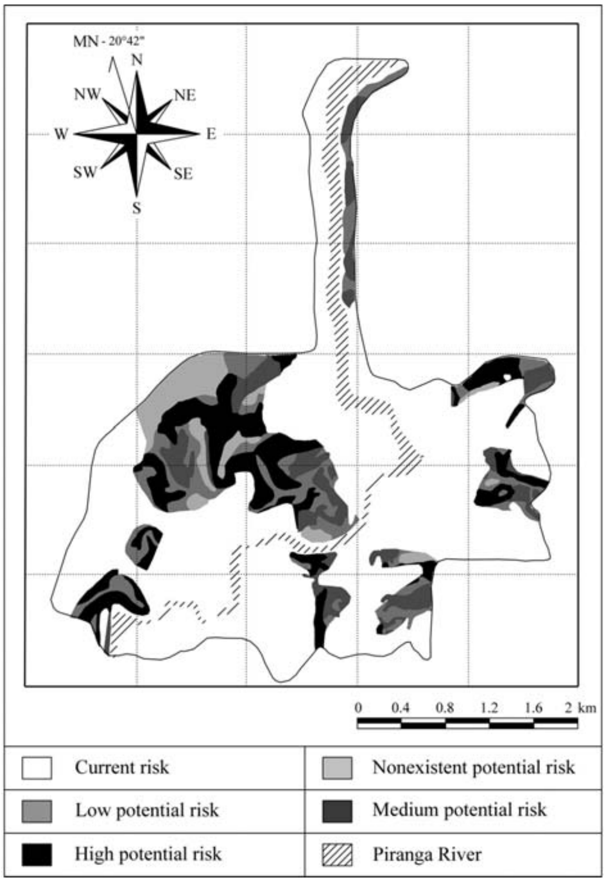

Figure 9 - Potential soil landslide risk chart of the urban area of Ponte Nova.

soil use and occupation; and (ii) supply elements for the adoption of corrective measures and especially, medium and long term preventative measures by the municipal authorities.

Specifically regarding the chart of landslide risks, the key for Figs. 8 and 9 shows in detail the areas with greatest landslides problems (high-risk areas) that need special care and priority of execution of corrective or preventative measures.

Acknowledgments

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Mineralogical Characteristics of “Entablature” and “Colonnade” Basalt Occurrences in the Northern Portion of the Paraná Basin, Brazil

Ronaldo Lima Gomes, Jose Eduardo Rodrigues

Abstarct. This work presents the results of mineralogical characterization carried out on samples obtained from massive (“colonnade”) and columnar jointed basalt (“entablature”) located in the northern portion of the Paraná basin, more precisely, in lava-front associated outcrops of some quarries of São Paulo State, Brazil. Besides some traditional material test methods, alternative tests such as the methylene blue adsorption, abrasion pH and conductivity were carried out. The joint analysis of this information assisted in material qualification, providing a comparative analysis between the “Colonnade” and “Entablature” relative to its use as aggregate for civil construction purposes.

Key words: aggregates, mineralogical characterization, basalts, construction materials.

1. Introduction

The concept of “entablature-colonnade” was first applied in basaltic rocks of the Parana basin by Souza Jr (1992). According to this author, with the exception of the outlying portions of amygdaloidal nature, certain lava flows present structural compartmentations that can be divided in two distinct jointing systems known as “colonnade” and “entablature”.

In general, the core of basaltic flows normally corresponds to a coarser texture rock with more widely spaced fractures due to lower cooling speed of the lava at the interior of the flow. On the other hand, the upper and inferior peripheral zones present faster cooling and are consequently characterize by a more expressive fracturing and fine vitreous texture of the constituent minerals. In general however, the entablature located in the interior of the flow is found to present a characteristics degree of fracturing and mineral texture which are compatible with fast cooling lava which is contrary to expected. The first scientific works establishing the basic characteristics and the relating nomenclature of these joints were published by Tomkeieff (1940) and Spry (1962).

The genesis model of “entablature-colonnade” concept conceived by Long & Wood (1986) considers the flows were covered by surface water soon after overflow. This water must have percolated through the already consolidated surface crust of the flow, penetrating right into its interior and thus causing rapid cooling and consequently a high degree of cracking of the entablature compartment. Both the “entablature” (columnar jointed basalt) and the “colonnade” (massive basalt) compartments present mineralogical paragenesis typical of basic rocks constituted of plagioclase feldspar, pyroxenes (augite), magnetite and variable amounts of vitreous substance. The amount of vitreous substance and the mineral texture are the main differences between the two compartments. In general, the “entablature” presents 60% vitreous substance whereas for the “colonnade”, this amount is about 20%. Another differentiating mineralogical characteristic between the two compartments is the morphology of magnetite crystals. The “entablature” presents a dendritic morphology for this mineral while the “colonnade” presents well-formed crystals due to a more gradual cooling.

In several cases, the degradation of rocks used as construction materials, especially basaltic rocks, has been linked to clay content (Scott (1955), Day (1962), Collet *et al.* (1962), Smith (1970), Schneider *et al.* (1968), Struillou (1969), West *et al.* (1970), Farjallat (1972), Hayase *et al.*(1972), De Alba & Sesana (1978), Rimoldi (1982), Frazão & Caruso (1983), Roisenberg *et al.* (1984)). In general, the tendency to degradation results of interaction of internal and external variables. The internal variable is in the rock itself, for example: type and clay content, cation exchange capacity, concentration of ions available on clay with reflections on the cation exchange, particle size and initial content of water. The external variables comprise external conditions to which the rock is exposed and features induced bay mining, transport and processing (Kuhnel, 2003), for example: clay mineral ability to access water, possibility of expansion of the clay mineral and nature concentration of ions dissolved in water that sometimes can reduce the expansion. These variables affect the mechanical behavior and, therefore, their quality as construction materials (Kuhnel, 2003; Korkanç & Tudrul, 2004).

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In this way, the vast majority of scientific publications produced in the last four decades on the degradation of rocky materials of basaltic flow origin (Korkanç & Tudrul, 2004; Moon & Jayawardane, 2004; Kühnel & Katshi, 1997; Scott, 1955; Sveinsdottir, 1999; Van Atta & Ludowise, 1976; Van Rooy, 1991 and Wylde, 1976) do not approach the concept of "entablature-colonnade". This fact raises various questions with respect to these materials as to: whether the great amount of vitreous substance present in the "entablature" is would compromise its use as aggregate for civil construction purposes, what behavior these materials might present under alteration tests and consequently what resulting alterability they might show. However, the ease of identifying compartments in the field makes this concept being used unconsciously by professionals in the field basalt exploitation. Massive basalt is informally known as "massive" whereas the more fractured "entablature" (columnar jointed basalt) is called denominated "granulated" or "granular". On the other hand, vesicular basalt are called "frog eye".

In this context, the objective of the present work is to present a mineralogical characterization of these rocks with the aim of generating information that can be used for the evaluation of the rock quality when used as aggregate for civil construction purposes.

2. Study Method

The method applied was initially aimed at classifying different types of basaltic flows according to the "entablature-colonnade" concept. To do this, a detailed geological-geotechnical mapping of the surface rock and mining front in ten quarries in the interior of the São Paulo State was carried out. The field studies carried out permitted the definition of the principal types of flows and their classification based on the "entablature-colonnade" concept. Details of this classification can be found in Gomes & Rodrigues (1999) and Gomes (2001).

Each sample studied refers to an "entablature" or "colonnade" compartment of flow profiles. The vesicular basalt was used in only one case. With the objective of comparing the technological and alterability characteristics of the basaltic flows of colonnade joints to those of intrusive basic rocks, a sample was collected from a diabase quarry. Table 1 presents the sample designation, the joint type and the origin of the samples.

Table 1 - Location and type of the studied samples.

Sample	Compartment	Localization
SCC	Colonnade	Washington Luís High Way Quarry, São Carlos, São Paulo State
SCE	Entablature	São Carlos City Quarry, São Carlos, São Paulo State
SAN	Colonnade	Santo Antônio Quarry, Araraquara, São Paulo State
ANT	Entablature	Santo Antônio Quarry, Araraquara, São Paulo State
IND	Colonnade	Inderp Quarry, Ribeirão Preto, São Paulo State
SAD	Entablature	Said Quarry, Ribeirão Preto, São Paulo State
VES	Vesicular basalt	Said Quarry, Ribeirão Preto, São Paulo State
PAV	Colonnade/ diabase	Paviobras Quarry, Rio Claro, São Paulo State
BP	Colonnade	Bica de Pedra Quarry, Jaú, São Paulo State
SM	Colonnade	Pedralite Quarry, São Manuel, São Paulo State
SIQ	Entablature	Siqueira Quarry, Assis, São Paulo State
WS	Colonnade	Ws Quarry, Assis, São Paulo State

In general, the samples used in the test were crushed material obtained from the quarry. In the case of the vesicular basalt, blocks of this material were collected and later crushed in the laboratory. For tests in which cylindrical test samples were necessary, such as those used for the uniaxial compression, representative rock blocks were collected from the sampling point.

Firstly, routine material characterization tests such as the determination of physical indexes (ABNT, 1984), uniaxial compressive strength (ISRM, 1978), determination of the crushing strength, Los Angeles abrasion value (ABNT, 1983), and the soundness test by ethylene glycol immersion (ABNT, 1992), were conducted with the aim of identifying and prequalifying the material. Besides these well known technological tests, alternative tests such as Methylene Blue Adsorption test (MBA) (Verhoef & Van de Wall, 1998), pH abrasion (Grant, 1969 and Malomo, 1980) and conductivity abrasion test (Gomes, 2001) were carried out. Parallel to these tests, X-ray difractometry was used mainly for detecting and quantifying deleterious minerals. Techniques employed for detailed observation such as optical and Scanning Electron Microscope (SEM) was used to obtain a detailed insight on the mineralogy, petrography and microstructure of the samples.

Mineralogical and textural characteristics of the rock were determined from studies thin section samples. According to these studies, mineralogical composition, texture, pore structure, opaque and secondary mineral content and rate of weathering of the basalt in different compartments were evaluated using the *K1* and *K2* Petrographical Indexes proposed by Frazão & Paraguassu (1994) and Frascá (1998), and the Rsm Index "Secondary mineral rating" proposed by Cole & Sandy (1980).

3. Mineralogical Characterization

3.1. Petrographical characteristics

From the macroscopic point of view, basalts possess a certain coloration that varies from dark ash to greenish. They also present a fine granulation, are mesocracratic to melanocratic, have a homogeneous aspect, without apparent structures with exception of the vesicular- amygdaloidal basalts. Sample PAV (diabase) has an average granulation, a greenish gray coloration and homogeneous aspect. Under a petrographic microscope, both the basalts as well as the diabase are composed of ripiform crystals of plagioclase (labradorite) among which are prismatic crystals of clinopyroxene (augite) and opaque minerals (titanite and magnetite) thus configuring a subophitic or intergranular textural array (Table 2). In the case of basalts, mainly in the "entablature" (columnar jointed Basalt), concentrations of volcanic glass and clay minerals confer to the rock, locally, an "intersertal" textural aspect. It can also be noted that the samples studied show marked differences relative texture, more specifically, with regards to the size of the plagioclase and clinopyroxene crystals. The samples "entablature" (columnar jointed Basalt) associated are shown to present a finer texture than colonnade (massive basalt) samples which confirms the genesis of the "entablature-colonnade" concept. It is also observed that the values maximum and minimum plagioclase and clinopyroxene crystal sizes are related respectively to samples of diabase and vesicular-amygdaloidal basalt.

With exception of sample *SAN* (massive basalt) which presents a high amount of iron oxides and hydroxides as a result of its more advanced stage of alteration, the amounts of clay minerals, volcanic glass and iron oxides and hydroxides are found to present higher values in the columnar jointed compartment samples than in the colonnade samples.

Except for the vesicular basalt with about 25% of its total volume consisting of vesicles and amygdales, the studied samples were found to present only rare occurrences of these structures. These amygdales generally are always encountered filled by green and brownish clay minerals and often accompanied by chalcedony, quartz, zeolites and carbonate.

Observations of sample PAV under a petrography microscope as well as the naked eye accused the occurrence of small metallic mineral points of colored yellow coloration (reflected light) scattered out within the sample. These minerals were preliminarily identified as possible sulphides. In order to ascertain what type of metallic mineral was present, a punctual chemical analysis was carried out using an electronic microprobe. Analyzes were carried out on three occurrence points of the sample and in all points analyzed, the presence of sulfur and iron elements was accused. However, the analysis did not furnish any quantitative data relative to the percentages of the elements necessary to identify the type of sulphide. Notwithstanding, for the present work, the most probable occurring mineral was supposed to be pyrite.

Table 3 shows the main mineralogic features necessary to determine the *K1* and *K2* indexes proposed respectively by Frazão & Paraguassu (1994) and Frascá (1998). These indexes, correlate sane essential minerals (S1) and the corresponding altered minerals (S2), the smectites-filled amygdales (S3), interlinked smectites (S4), smectite-filled microcracks (S5), other types of microcracks (S6), the amount of volcanic glass (S7), and the amount of secondary minerals including oxides and hydroxides of iron (S8) according to the following expressions:

$$K1 = S1/(S2 + S4 + S5) \quad (1)$$

$$K2 = S1/(S2 + S3 + (S4 * 2) + (S5 * 2) + S6) \quad (2)$$

It is worthwhile pointing out that an increase in the two indices increase translated into a better quality of the rocky material for use for civil construction purposes.

An alternative method of analyzing the petrographic characteristics would be for use of the R_{sm} index - "Secondary mineral rating" considered by Cole & Sandy (1980). This index is based on the mineralogical and textural characteristics of the rock through the following:

$$R_{sm} = (P*M)T \quad (3)$$

where R_{sm} = the rate of secondary mineral; P = percentage of secondary mineral; M = the secondary mineral value and T = the texture value.

The obtained secondary mineral value M was 10. This corresponds to clay minerals of the smectite group. The obtained textures associates to the clay minerals present are given by: (T = 0.3) partial alteration of the phenocrystals and incomplete filling of vesicles; (T = 0.4) completely altered phenocrystals; (T = 0.5) homogeneous distribution in the matrix; (T = 0.6) complete filling of vesicles; (T = 0.7) irregular distribution in the matrix and with filling of isolated microcracks; (T = 1.0) filling of irregular microcracks and (T = 2.0) filling of interconnected microcracks. Table 4 presents the values of R_{sm} index for the sample studied.

The samples SM, SCC, PAV, BP and WS showed the highest rates of *K1* and *K2* indexes and SIQ, SAN and VES the lowest. The samples SCE, IND, ANT and SAD showed intermediate values. The presence of approximately 28% of volcanic glass in the mineralogy of sample ANT was responsible for the reduced petrographic index, *K2* of this sample. Sample SAD was shown to present a low value of the petrographic index *K2* due to the presence of interlinked smectites observed under a petrographic microscope. This could thus adversely influence its quality. However, its very compact structure which confers a low porosity and water absorption to this sample can nullifies the influence of these clay minerals on the results of the alterability tests.

Table 2 - Petrographical characteristic of the studied samples.

Sample	Lithology	Texture	Mineralogy	Hydrothermal alteration	Weathering	Amygdalas	Clay minerals	Microcracks
SCC	Basalt/ Colonnade	Subophitic Granulation: Plagioclase (0.40-0.1 mm) Clinopyroxene (0.6-0.05 mm)	Plagioclase (35%) Clinopyroxene (30%) Opaques (10%) Clay minerals(03%) Volcanic glass (22%)	Weak Some type of argilation, mainly in the clinopyroxenes	Weak or absent No presence of oxides or hydroxides of iron detected	Absent	Presence of a brownish green coloration clay mineral (smectite) associate to the alteration of clinopyroxenes	Absent
SCE	Basalt/ Entablature	Ophitic to subophitic Granulation: Plagioclase (0.15-0.05 mm) Clinopyroxene (0.02-0.04 mm)	Plagioclase (30%) Clinopyroxene (25%) Opaques (10%) Clay minerals(05%) Volcanic glass (27%) Oxides-Hidroxides of iron (03%)	Weak Associated to vitreous clayey/argilaceous mesostasis and the argilation of clinopyroxenes	Weak Presence of oxides and hydroxides of iron filling microcracks of the clinopyroxenes	Absent	Presence of a brownish green coloration clay mineral (smectite) associate to vitreous clayey/argilaceous mesostasis and the alteration of cpx	Presence of microcracks in the pyroxenes
SAN	Basalt/ Colonnade	Ophitic to subophitic Granulation: Plagioclase (0.16-0.24 mm) Clinopyroxene (0.16 mm)	Plagioclase (30%) Clinopyroxene (25%) Opaques (05%) Clay minerals(10%) Volcanic glass (20%) Oxides-Hydroxides of iron(10%)	Average Associated to the vitreous clayey/argilaceous mesostasis and the argilation of clinopyroxenes	Average Presence of oxides-hydroxides of iron surrounding the opaques and filling pyroxenes microcracks	Rare occurrence of amygdalas filled by greenish clay minerals associates to calcedony and carbonate	Presence of a brownish green coloration clay mineral (smectite) associate to vitreous clayey/argilaceous mesostasis	Presence of microcracks in the rock, pyroxenes and Plagioclases
ANT	Basalt/ Entablature	Subophitic Granulation: Plagioclase (0.15-0.05 mm) Clinopyroxene (0.02-0.04 mm)	Plagioclase (30%) Clinopyroxene (25%) Opaques (10%) Clay minerals(05%) Volcanic glass (28%) Oxides-Hydroxides of iron(02%)	Weak Some argilation in clinopyroxenes and Plagioclases	Weak	Absent	Presence of a brownish green coloration clay mineral (smectite) associate to vitreous clayey/ argilaceous mesostasis and the alteration of cpx	Absent
IND	Basalt/ Colonnade	Subophitic Granulation: Plagioclase (0.16-0.3 mm) Clinopyroxene (0.04-0.08 mm) Opaque (0.04-0.08 mm)	Plagioclase (35%) Clinopyroxene (30%) Opaques (10%) Clay minerals(05%) Volcanic glass (20%)	Weak Some argilation in clinopyroxenes, forming zones of alteration in the rock	Weak or absent Presence of oxides and hydroxides of iron not detected	Absent	Greenish clay mineral distributed in zones throughout the rock due to the alteration of clinopyroxenes	Absent
SAD	Basalt/ Entablature	Subophitic Granulation: Plagioclase (0.08-0.24 mm) Clinopyroxene (0.04-0.08 mm) Opaque (0.02-0.04 mm)	Plagioclase (35%) Clinopyroxene (30%) Opaques (05%) Clay minerals (20%) Volcanic glass (10%)	Weak Some argilation, mainly in the clinopyroxenes	Weak or absent No presence of oxides and hydroxides of iron detected	Absent	Presence of brownish green clay mineral (smectite) associate to the alteration of clinopyroxenes	Absent
VES	Basalt/Vesicle-amygdaloidal	Intersertal Granulation: Plagioclase (0.04-0.12 mm) Clinopyroxene (< 0.02 mm) Tonsils/amygdalas (0.8-3.0 mm)	Plagioclase (25%) Clinopyroxene (20%) Opaques (05%) Clay minerals+ Volcanic glass + Oxides-Hydoxides of iron (50%)	Average argilation in clinopyroxenes and Plagioclases	Weak Rare presence of oxides and hydroxides of iron surrounding the opaques	Presence of amygdalas representing about 25% of the volume of the rock filled with greenish clay minerals, calcedony and carbonates	Presence of brownish green clay mineral filling the interlinked amygdalas and interstices (> 0.5 mm)	Presence of intragranular (clinopyroxene) and intergranular (Rock)microcracks

Table 2 (cont.)

Sample	Lithology	Texture	Mineralogy	Hydrothermal alteration	Weathering	Amygdalas	Clay minerals	Microcracks
PAV	Diabase	Intergranular to subophitic Granulation: Plagioclase (0.3-0.8 mm) Clinopyroxene (0.2-0.3 mm)	Plagioclase (45%) Clinopyroxene (40%) Opaques (10%) Sulphides (05%)	Weak Some argilation in clinopyroxenes and Plagioclases	Weak or absent No presence of oxides and hydroxides of iron detected	Absent	Presence of brownish green clay mineral (smectite) associated to the alteration of clinopyroxenes	Microcracks present in rock, pyroxenes and Plagioclases
BP	Basalt/ Colonnade	Subophitic Granulation: Plagioclase (0.10-0.4 mm) Clinopyroxene (0.05-0.6 mm)	Plagioclase (35%) Clinopyroxene (30%) Opaques (10%) Clay minerals(05%) Volcanic glass (20%)	Weak Some argilation, mainly in the clinopyroxenes	Weak or absent	Absent	Presence of brownish green clay mineral (smectite) occupying intergranular spaces	Absent.
SM	Basalt/ Colonnade	Subophitic Granulation: Plagioclase (0.35-0.1 mm) Clinopyroxene (0.1-0.05 mm)	Plagioclase (35%) Clinopyroxene (30%) Opaques (15%) Clay minerals(05%) Volcanic glass (15%)	Weak Presence of argilation in the clinopyroxene-opaque contacts	Weak Presence of oxides-hydroxides of iron surrounding the opaques and filling microcracks of the pyroxenes	Absent	Presence of chestnut color clay mineral in the contacts between cpx and opaque and filling some cracks in the Plagioclase.	Rare in Plagioclases and clinopyroxenes, and absent in rock
SIQ	Basalt/ Entablature	intergranular Granulation: Plagioclase (0.1-0,15 mm) Clinopyroxene (0,04 mm)	Plagioclase (30%) Clinopyroxene (30%) Opaques (05%) Clay minerals(20%) Volcanic glass (15%)	Average Associated to vitreous argillaceous mesostasis and the argilation of clinopyroxenes	Weak. Presence of oxides-hydroxides of iron surrounding the opaques and filling microcracks of the pyroxenes	Absent	Presence of brownish green clay mineral (smectite) associate to vitreous clayey/argilaceous mesostasis and the alteration of Plagioclases and clinopyroxenes	Microcracks present in rocks, pyroxenes and Plagioclases
WS	Basalt/ Colonnade	Subophitic Granulation: Plagioclase (0,4-0,2 mm) Clinopyroxene (0,15-0,06 mm)	Plagioclase (40%) Clinopyroxene (35%) Opaques (10%) Clay minerals(05%) Volcanic glass (10%)	Weak Presence of argilation in the clinopyroxene-opaque contacts	Weak Presence of oxides-hydroxides of iron surrounding the opaques and filling microcracks of the pyroxenes	Absent	Presence of chestnut color clay mineral in the contacts between cpx and opaque and filling some cracks in the Plagioclase	Rare in Plagioclases and clinopyroxenes, and absent in rock

Table 3 - Petrographical Characteristic of the samples (modal analyzes (%)).

	SCC	*SCE*	*SAN*	*ANT*	*IND*	*SAD*	*VES*	*PAV*	*BP*	*SM*	*SIQ*	*WS*
S1	73	62	50	60	72	68	40	90	72	77	55	80
S2	2	3	10	5	3	2	10	5	3	3	10	5
S3	0	0	1	0	0	0	20	0	0	0	0	0
S4	1	3	2	4	5	8	0	0	1	0	5	0
S5	0	0	1	0	0	0	0	0	0	0	3	0
S6	0	0	2	0	0	0	0	0	0	0	2	0
S7	22	27	20	28	19	10	25	0	20	15	15	10
S8	2	5	10	2	1	2	5	5	3	5	10	5
K1	24.3	10.3	3.8	6.7	9.0	6.8	4.0	18.0	18.0	25.7	3.1	16.0
K2	18.2	6.9	2.6	4.6	5.5	3.8	1.3	18.0	14	26.0	2.0	16.0

Note: S1 = plagioclase + clinopiroxene + sane opaque; S2 = plagioclase + modified clinopiroxene ; S3 = amygdala with smectites; S4 = interlinked smectites ; S5 = smectite filled microcracks ; S6 = other microcracks; S7 = volcanic glass; S8 = secondary mineral + oxides and hydroxides of iron; *K1* = petrographic Index (Frazão & Paraguassu, 1994); *K2* = Frascá, 1998).

Table 4 - Values of Rsm for the sample studied.

S	M	T	(%)	R_{sm}	S	M	T	(%)	R_{sm}	S	M	T	(%)	R_{sm}	S	M	T	(%)	R_{sm}
SCC	10	0.3	2	23	VES	10	0.3	10	175	ANT	10	0.3	5	53	SM	10	0.3	3	34
		0.6	0				0.6	20				0.6	0				0.6	0	
		0.7	1				0.7	0				0.7	4				0.7	0	
		1.0	0				1.0	0				1.0	0				1.0	0	
		0.5	2				0.5	5				0.5	2				0.5	5	
SCE	10	0.3	3	55	PAV	10	0.3	5	40	IND	10	0.3	3	49	SIQ	10	0.3	10	145
		0.6	0				0.6	0				0.6	0				0.6	0	
		0.7	3				0.7	0				0.7	5				0.7	5	
		1.0	0				1.0	0				1.0	0				1.0	3	
		0.5	5				0.5	5				0.5	1				0.5	10	
SAN	10	0.3	10	110	BP	10	0.3	3	31	SAD	10	0.3	2	72	WS	10	0.3	5	40
		0.6	1				0.6	0				0.6	0				0.6	0	
		0.7	2				0.7	1				0.7	8				0.7	0	
		1.0	1				1.0	0				1.0	0				1.0	0	
		0.5	10				0.5	3				0.5	2				0.5	5	

S: sample; M: mineral; T: texture.

3.2. X-ray diffractometry

X-ray diffractometry accused the presence of plagioclase, pyroxene and metallic minerals of the magnetite/hematite type and titanite in all the samples. Clay minerals of the smectite group were identified in various amounts in samples *IND, VES, BP, SM and WS*. In the other samples, the diffractogram peaks, representative of these clay minerals, were not significant enough to be identified. On comparing the diffractogram of the columnar jointed basalt and massive basalt samples, it is easy to note that the difference between the plagioclase peak intensity and pyroxene peak of intensity is higher for the massive basalt samples. This comparison was made discounting the influence of the test "background" values. This difference in diffractograms is better observed on comparing the diffractograms of samples *SCC* with those of samples *SCE*, of samples *IND* with those of samples *SAD* and of samples *SAN* with those of samples *ANT*. One observes that the diffractograms of the columnar jointed basalt ("entablature") samples (*SAD, ANT and SCE*) present peak plagioclase values, approximately as high as the peak pyroxene value. For the massive basalt (“colonnade”) samples (*IND, WS, BP, SM, SCC* and *SAN*), this relation varies approximately between two the five times. From the obtained diffractogram results, the SAI index (“Smectite Alteration index”) considered by Houston & Smith (1997) (Table 5) was determined for each sample. This index is based on the correlation between the Smectite peak of intensity (cps) - 15 Å and the Plagioclase peak intensity (3,2 Å) after deducing the test “background” level.

High values of this index, that varies of 0 the 1, indicate a marked presence of clay minerals of the smectite group was detected from the diffractogram.

3.3. Physical indexes

The average apparent dry specific mass was observed to vary between 2.948 (g/cm^3) corresponding to sample *BP* and 2.243 (g/cm^3), corresponding value sample *VES*. However, the saturated apparent specific mass was found to vary between 2.958 (g/cm^3) and 2.415 (g/cm^3), corresponding respectively to samples *SIQ* and *WS,* and *VES*. The apparent porosity varied between 17.27% and 0.60% values associated respectively to samples *VES* and *IND* while the water absorption was shown to present values between 7.78% and 0.20% also corresponding to samples *VES* and *IND* (Table 6).

3.4. Strength indexes

The direct compression strength was found to vary between 60 MPa and 290 MPa, corresponding to samples *VES* and *SAD* respectively. In many of the studied samples, the Los Angeles abrasion strength values were found to be inferior to the 31,6% loss. Samples VES and IND showed extreme values of the modified crushing strength from average, respectively 20.73% and 11.85% loss (Table 6).

3.5. Ethylene glycol immersion

This test was carried out to rapidly simulate the reaction between water and expansive clay minerals in the sample. Ethylene glycol is one of the materials that reacts with swelling clays of the smectite group to form an organo-clay complex having a larger basal spacing than that of the clay mineral it self.

The samples were immersed in an ethylene glycol solution for a period of 21 days and the amount of particles affected evaluated after every 3 days.

Table 6 and Fig. 1 presents results of the ethylene glycol immersion test for all samples with exception of samples *SCC, SCE, PAV, IND and SAD* which were not affected by this test.

It is noted that at the ninth cycle, samples *VES*, *SAN* and *SIQ* presented the first trace of spalling, corresponding to respectively, 30%, 10% and 10% of fragments affected. Then after in the tenth second cycle, samples *WS* (16%) and *ANT* (8%) also manifested fragmentation with the percentage of fragments affected shown in parentheses for each case while during the fifth cycle, sample *SM* presented 8% of fragments affected. Finally, in the eighteenth cycle of the

Table 5 - Values for determining the SAI index.

Sample	Plagioclase		Smectite		SAI
	“Background” (cps)	Peak (cps)	“Background” (cps)	Peak (cps)	
SCC	241	941	202	212	0.014
SCE	286	632	238	247	0.026
SAN	241	1297	201	217	0.015
ANT	296	627	195	228	0.100
IND	284	656	199	221	0.059
SAD	294	560	181	204	0.086
VES	350	654	293	488	0.641
PAV	291	645	206	210	0.011
BP	243	842	240	282	0.070
SM	244	861	200	230	0.049
SIQ	295	1016	208	220	0.017
WS	267	770	201	238	0.074

Table 6 - Results of technological and alterability tests for the samples studied.

Sample/test	SCC	SCE	SAN	ANT	IND	SAD	VES	PAV	BP	SM	SIQ
Dry unit weight (kg/m^3)	2.912	2.893	2.910	2.889	2.923	2.916	2.243	2.862	2.948	2.921	2.945
Water absorption (%)	0.27	0.30	0.29	0. 54	0.20	0.29	7.78	0.27	0.33	0.47	0.39
Porosity (%)	0.77	0.87	0.84	1.57	0.60	0.83	17.27	0.77	0.97	1.38	1.15
Ethylene glycol index (%)	0	0	59	8	0	0	86	0	8	8	22
MBA (g/100 g)	0.22	0.52	0.60	0.60	0.22	0.22	2.41	0.22	0.45	0.37	0.68
CEC (meq/100 g)	0.70	1.62	1.88	1.88	0.70	0.70	7.56	0.70	1.41	1.16	2.12
Uniaxial compressive strength (Mpa)	265	242	160	230	175	290	60	191	180	124	86
Crushing test (%)	12.20	14.10	13.48	14.15	11.85	14.31	20.73	14.00	13.34	13.62	11.98
Los Angeles abrasion (%)	13.09	14.91	16.65	14.84	13.87	13.32	31.58	16.32	15.88	13.26	16.33

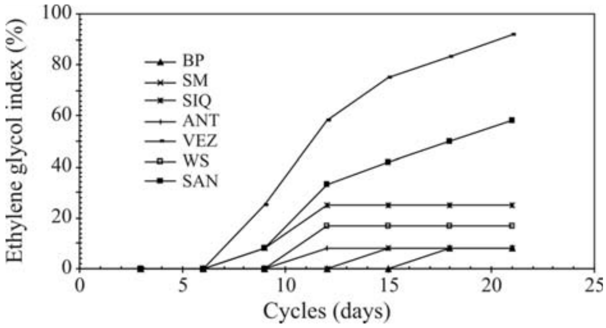

Figure 1 - Evaluation of the immersion in Ethylene glycol.

test, sample *BP* also showed about 8% of its fragments affected by ethylene glycol immersion (Fig. 1).

A close look at Fig. 1 shows a constant behavior of samples *SM*, *WS*, *ANT* and *BP* vis-à-vis the percentage of fragments affected by the immersion test. However, this was not the case for samples *VES* and *SAN*. Here, a gradual increase in the amount of fragments affected with time until the end the end of the test was noticed while sample *SIQ* showed a rather increasing disaggregation up to the 12^{th} cycle and later maintained this behavior constant.

From the above-exposed, the studied samples can thus be ranked in decreasing order disaggregation of fragments affected as a function of the time (cycles) necessary for the first disintegration as follows: *VES* > *SAN* > *SIQ* > *WS* > *ANT* > *SM* > *BP*. It is worth emphasizing here that samples *SCC, SCE, PAV, IND and SAD* were not affected by this test.

3.6. Methylene blue adsorption test

Verhoef & Van de Wall (1998) define the index MBA (Methylene Blue Adsorption) as a mass of methylene blue necessary to recover, with a monomolecular layer, the particles contained in 100 g of sprayed fragments of aggregates. Depending on the amount and type of clay mineral present, the adsorption of methylene blue will be variable.

The concentration of the solution test was 3 g of methylene blue per liter of distilled water as suggested by Verhoef & Van de Wall (1998). The test is based on determining the peak of maximum adsorption of methylene blue dye to 2 g rock material sprayed, by the addition in quantities of 1 to 5 cm^3 the solution of methylene blue, added to the slurry (2 g of material + 30 mL distilled water).

The maximum adsorption occurs when there is saturation of dye in aqueous solution, indicated by the emergence of an "aureole" bluish around the core of the drop of the suspension when put on filter paper. From the volume of the solution of methylene blue consumed in the saturation of slurry (rocky material sprayed + water) it is estimated the values of CEC and MBA.

The methylene blue test results expressed as the amount of methylene blue adsorbed (MBA), the cation exchange capacity (CEC) are presented in Table 6. Observation shows that the maximum value of MBA is 2.41 (g/100 g) corresponding to sample *VES* while the least value is 0.22 (g/100 g) for samples *PAV, IND, SAD* and *SCC*.

Considering the cation exchange capacity, the observed maximum value was 7.56 (meq/100 g) for sample *VES* while the minimum value was 0.70 (meq/100 g) corresponding respectively to samples *PAV, IND, SAD* and *SCC*.

3.7. Abrasion pH

According Malomo (1980), during weathering, the feldspars are undergoing hydration and as a consequence the release of ions, in general, alkali and alkaline earth elements. These ions combine with the free OH^- ions in the solution. Thus, the greater the amount of ions of alkali and alkaline earth elements released lower the concentration of OH^- free in solution and thus will lower the pH value. The pH of abrasion is known as the pH of the solution comprises the feldspar sprayed more distilled water, in different concentrations.

The sample consisted of pulverized material (# 0063 mm) resulting from the crushing testing. The test was based on the measurement of pH of sample mixed in distilled water, as follows: a) addition of 0.75 g of sample in 48 mL of distilled water; b) shake the solution for 3 minutes; c) after resting for 1 min, achieve a measure of pH; d) adding more 0.75 g of the sample and repeating the above procedure; e) adding more 0.5 g of the sample and repeating the above procedure and f) adding more 1.0 g of the sample and repeating the above procedure.

The measured pH values at concentrations of 1/64, 1/32, 1/24 and 1/16, for all samples, are given in Table 7. A concentration of 1/16, that is 0.063 (g/mL) was adopted as the representative of the abrasion pH for the studied samples.

From Fig. 2, three different pH zones can easily be noticed for all concentrations. The first zone characterized for pH values above of 9.34 is representative of sample *VES* while the second zone associated to pH values varying between 8.04 and 9.09 is representative of samples *SCC, SCE, SAN, ANT, IND, SAD, BP, SM, WS and SIQ*. The third zone on the other hand presents pH values between 6.26 and 6.28 and is associated to results of sample *PAV*. This permits us ascertain the applicability of pH abrasion test as a viable identification tool for samples containing a marked presence of clay minerals (Zone 1) represented in the present case for vesicular-amygdaloidal basalt samples.

Grant (1969) related the abrasion pH to the presence of clay minerals in samples which adsorb part of primary ions in solution and consequently increasing OH^- ion concentration and the pH. Clay minerals, mainly of the 2:1 type, present distributed negative charge on their surface. This charge also adsorbs free hydrogen ions in solution thus causing an increase in OH^- concentration and consequently the pH.

The low pH values sample *PAV* represented Zone 3 are indicative of the presence metallic mineral rock in the rock, in particular, the sulphides. Pyrites (FeS_2) and the pyrrhotite ($Fe_{1-x}S$) are the most abundant sulphides of iron in rocks and a study of their oxidation is of prime importance in the evaluation of the durability and quality of aggregates for civil construction purposes. In general, the oxidation of sulphides is known to produce sulfates, oxides and free ions of H^+. As this reaction progresses, the concentration of H^+ free ions increases and consequently the acidity of the solution.

3.8. Abrasion conductivity

Following the same procedure adopted for determination of abrasion pH, the conductivities measured at concentrations of 1/64, 1/32, 1/24 and 1/16 are presented in Table 7. The representative abrasion conductivity of the sample adopted at a concentration of 1/16 was (0,063 g/mL).

In accordance with Fig. 3, three groups of conductivities can be identified. Group 1, represents high conductivity values representative of sample *PAV* followed by Group 2, representative of the values of sample *VES* while Group 3 is made of conductivity values representative in the remaining samples which are found to show low conductivity.

The increase in conductivity is due mainly by the increase in ions concentration and type of ions in the solution. In the case of sample *PAV*, representative of high conductivities of Group 1, the prominent cause of increased conductivity is the increase in H^+ concentration resulting from the oxidation of sulphides present in the sample which in

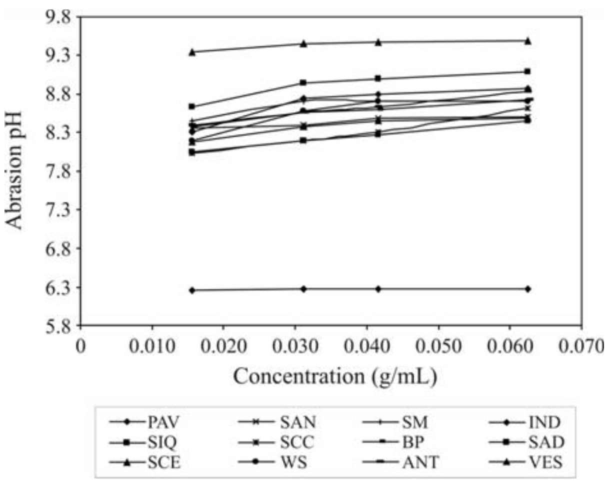

Figure 2 - Values of abrasion pH of the samples studied.

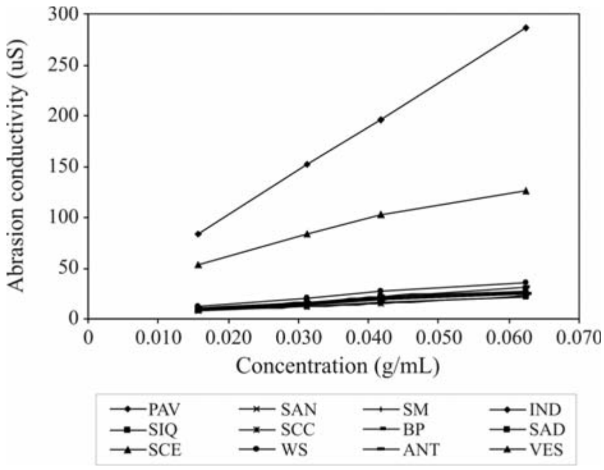

Figure 3 - Values of abrasion Conductivity of the samples studied.

Table 7 - Values of abrasion pH and Conductivity of the studied samples.

Concentration / sample	0.016 (g/mL)		0.031 (g/mL)		0.042 (g/mL)		0.063 (g/mL)	
	pH	Conductivity (uS)	pH	Conductivity (uS)	pH	Conductivity (uS)	pH	Conductivity (uS)
SCC	8.03	8.04	8.19	12.00	8.31	15.20	8.61	23.00
SCE	8.18	9.80	8.37	15.70	8.44	20.30	8.48	26.50
SAN	8.35	9.90	8.41	15.00	8.48	19.00	8.51	24.00
ANT	8.39	11.00	8.56	17.00	8.64	22.50	8.83	31.00
IND	8.31	9.20	8.74	14.30	8.80	19.00	8.87	24.70
SAD	8.63	10.00	8.94	16.30	9.01	21.50	9.09	28.00
VES	9.34	54.00	9.45	83.00	9.47	102.50	9.49	126.00
PAV	6.26	84.00	6.27	152.00	6.27	196.00	6.28	203.04
BP	8.37	9.00	8.56	14.30	8.61	18.50	8.72	25.00
SM	8.44	10.76	8.70	17.00	8.70	22.00	8.70	27.00
SIQ	8.04	8.52	8.19	13.00	8.26	17.00	8.45	22.00
WS	8.19	12.00	8.57	20.00	8.71	28.00	8.71	36.00

turn is responsible for the fall in pH of the solution. Concerning the Group 2 sample (*VES*), the observed relatively higher conductivities than for samples of Group 3 are probably associated to the increase alkaline and alkaline earth elements ions in the solution and, also due to high concentration of OH^- which results, as mentioned before, in high pH values. The Group 3 sample conductivity is probably influenced only by the amount of alkaline and alkaline earth metal ions in solution.

4. Conclusions

The application of the Long & Wood (1986) model for the classification of the studied basaltic flows was found to be very satisfactory given its capability of reproducing the origin of the "entablature-colonnade" compartmentation.

From the geological engineering point of view, the mineralogical, textural and structural differences between the "entablature" (Columnar jointed basalt) and "colonnade" (massive basalt) compartments reflect the different geotechnical and technological behaviors. The "entablature" being highly fractured and containing high amounts of vitreous substances in its mineral composition is found to presents some characteristics which may not favor its use for civil construction purposes.

The fact that the technological characterization tests were carried out almost simultaneously provided excellent information for determining numerous quality indices such as: the petrographic *K1* and *K2* indexes, petrographic index R_{sm} (secondary mineral rating), SAI Index (smectite alteration index).

Despite the very close similarities between the petrographic characteristics of the studied samples, the petrographic indices *K1*, *K2* and R_{sm} clearly indicated the low quality samples *SIQ*, *SAN* and *VES*.

The x-ray diffractometry identified different amounts of clay minerals of the smectite group in samples *IND, VES, BP, SM* and *WS*. In the other samples, the diffractograma peaks, representative of these clay minerals, did not permit their identification.

With exception of sample *VES* corresponding to vesicular basalt, the physical indices, direct compression strength, Los Angeles abrasion strength and the crushing strength, when analyzed separately, were found to furnish insufficient information to differentiate the samples according to quality.

The test results obtained from the immersion of rock fragments in ethylene glycol were found to be effective in differentiating the samples according to their likelihood of disintegration.

The alternative tests the methylene blue adsorption test, abrasion pH and conductivity were all found to be useful. The methylene blue test was applicable in determining the presence and amount of clay minerals while the abrasion pH and conductivity tests were used in verifying the presence of metallic mineral of the sulphide type. In general, these tests are quite outstanding due to the ease and rapidity of obtaining test results.

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Sergey Andreevich Yufin

Geotechnical engineer and professor (1945-2008)

Sergey Andreevich Yufin was a professor from Moscow State University of Civil Engineering, an influential personality in international geotechnical engineering and tunneling practice and teaching for almost 40 years, died 2008, aged 62. He is survived by his wife and son.

In 1968 he graduated from Moscow State University of Civil Engineering, Faculty of Hydraulic and Geotechnical Engineering. The University activity began in 1967 as Assistant at the same University and since that time Sergey Yufin permanently worked at the Faculty of Hydraulic and Geotechnical Engineering. In 1974 Sergey Yufin defended his thesis, in which the finite element method for underground construction analysis was shown for the first time in Russia. In 1990 Sergey Yufin defended his doctoral thesis. The 90-s was the most difficult years for the University because of the situation in Russia Economy and it was during these years that Sergey Yufin showed himself as one of the Leaders of the Faculty, a brilliant teacher and scientist.

Sergey Yufin has been invited to the prestigious universities and research centers of Austria, Germany, Norway, USA, Switzerland and Portugal where he worked as a Professor or leading scientist. In Russia he was in charge of research works on grants of CRDF and Swiss National Scientific Fund. He participated in projects and researches of objects in geotechnical and water construction both in the former USSR and in England, India, and USA. Sergey Yufin was a specialist in the field of underground construction, underground space usage, and computer modeling of geotechnical goals.

Sergey Yufin was also Director of ACUUS, IACMAG, member of ARMA, ITA, ISRM; member of the Research Council on development of underground space of Russian Tunnel Association; member of editorial staff of several foreign magazines on geotechnical and underground construction and Head of the *Center of Underground and Special Engineering* of the Moscow State University of Civil Engineering. He was member or Chairman of organizing committees of international conferences and congresses on geomechanics or computer modeling, and author of 150 scientific publications, several monographs, a textbook, a number of teacher editions. Many works of Sergey Yufin are published abroad. He had governmental awards.

My first professional contact with Sergey Yufin was during ISRM International Symposium EUROCK93, held in Lisbon, 1993, where we had an important contribution as a General Reporter. Since there a very intensive professional relationship was established between Sergey Yufin and Portuguese institutions like SPG, University of Porto and LNEC. His last contribution was during the 11th ISRM Congress held in Lisbon, last summer in 2007.

The first Sergey Yufin's cooperation with ABMS & SPG jointed sponsored events was with the 3rd edition of the Workshop on *Applications of Computational Mechanics in Geotechnical Engineering* held in Porto, 1998, followed by the 4th edition in Ouro Preto, 2003, and finally by the 5th and last in Guimarães, 2007. These editions started as a result of a joint research projects involving organizations from Brazil and Portugal and can be considered, in my personal opinion, maker of the recent series of Brazilian-Portuguese Geotechnical Congresses. In 2007 the Journal Soils and Rocks acquired the status of an international journal and for the Editorial Board Sergey Yufin was chosen.

This note is a simple mention to the memory of a friend that leaves us.

L. Ribeiro e Sousa

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

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Volume 31, N. 2, May-August 2008

Table of Contents