Geological-Geotechnical Characterization of Slopes Belonging to the Serra do Mar Paranaense, Brazil

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Abstract. An extensive campaign of field and laboratory tests were performed on samples of residual soils and colluviums present in the morphoscultural sub-unit comprising Serra do Mar Paranaense located in Southern Brazil. The geotechnical investigations included the physical characterization, *in situ* hydraulic conductivity and the mechanical behavior of the soils, by means of conventional direct shear tests, smooth interface direct shear tests, and CIU triaxial tests. The results showed that both the superficial colluvium and residual soils found along this stretch have similar granulometry, generally classified as silty sand soils, with *in situ* hydraulic conductivity of around 10^{-4} cm/s. Grain size curves show less dispersion in the case of residual soils compared with colluvial soils. The residual and colluvial soils had average peak and residual friction angles of 32° and 26°, respectively, with variations and differences attributed to the complex variety of the lithotype present in the region. Regarding cohesive intercept, a greater disparity was found in the results; however, these results corresponded to the literature. These results are relevant because they provide a framework to evaluate the stability of road slopes, together with other pertinent information, such as slope declivities and layers, water table, suction parameters and rain scenarios, both in specific cases within or close to the region, and in areas of similar geological material.

Keywords: colluvial soils, residual soils, Serra do Mar, shear strength, smooth interface, soil strength parameters.

1. Introduction

Serra do Mar is a mountain range encompassing approximately 1,500 km of the East/South coast of Brazil, going from the state of Rio de Janeiro to the North of the state of Santa Catarina (Cerri *et al.*, 2018; Vieira *et al.*, 2018). The range is classified into three large geomorphological compartments: Plains Compartment, Mangrove Compartment and Mountain Ranges and Hills Compartment, characterized by a diversity of lithological types, including granites, schists, gneisses and migmatites (Massad, 2010).

According to Listo & Vieira (2015) and Vieira *et al.* (2018), the Mountain Ranges and Hills compartment of the Serra do Mar is one of the main geomorphologic compartments frequently affected by mass movements of the shallow landslide type. Among the events that have occurred over time, the following have been significant: Caraguatatuba in 1967, Cubatão in 1985, Ilhota, Gaspar and Luís Alves (state of Santa Catarina) in 2008, Angra dos Reis in 2010 and in the mountain regions of Rio de Janeiro and Paraná in 2011. Listo & Vieira (2015) state that Serra do Mar is one of the most important reliefs of Brazil, both in geomorphologic terms when described as a function of its genesis and evolution, and by its strategic importance, in connecting the largest import and export harbors of the South and Southeast regions, as the port of Santos, the busiest in South America (Vieira *et al.*, 2018). As such, it has a dense network of communication and important service routes supporting economic development, for example: roads, railways, water pipelines, gas pipelines, transmission lines, urban installations and energy industries (Cerri *et al.*, 2018). Thus, a varied research has been and is being performed in this region, from which several questions have arisen, for example, if the results from the models will be more efficient with the use of geotechnical values collected *in situ* (Listo & Vieira, 2015).

The occurrence of different soil types throughout the study area and the need to obtain a better understanding of the geological-geotechnical behavior of the region, reflect the importance of the experimental study of the predominant materials. Based in this context and in the absence of geotechnical information about those materials, this paper presents the geological-geotechnical characterization of a stretch of Serra do Mar, based on results obtained from samples from the superficial colluvial and residual soils of migmatite and granite found in the region. Characterization

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tests for determination of physical parameters of soils is the primary and fundamental stage for several analyses, such as determining the prediction of both mechanical and hydraulic behavior in engineering, mining and environmental processes, as well as susceptibility analyses for the determination of areas at risk of mass movements.

Some studies have described the geological and geotechnical characteristics of the soils found along the Serra do Mar in order to obtain parameters as internal friction angle, cohesive intercept, granulometric distribution, hydraulic conductivity and structural features (Mendes *et al.*, 2006; Furlan *et al.*, 2011; Mendes *et al.*, 2015; Sestrem *et al.*, 2015; Advincula, 2016; González *et al.*, 2017; González, 2017; Cerri *et al.*, 2018; Vieira *et al.*, 2018; Trevizolli, 2018) looking to increase the knowledge about the region.

Other studies and researches focused on contributing to the understanding of the surface and subsurface dynamics of water at the Serra do Mar (Mendes *et al.*, 2006; Soares *et al.*, 2012; Mendes *et al.*, 2015; Oliveira *et al.*, 2016; Trevizolli, 2018, Picanço *et al.*, 2019).

The sub superficial layer of the natural slopes from Serra do Mar is unsaturated. After the dry season, infiltration capacity is high and rainfall events can cause significant effects on the distribution of soil matric suction (Bicalho *et al.*, 2015). During the rainy season, the infiltration of rainfall into the ground develops positive pore pressures by raising the water table and reducing suction levels. Consequently, the soil shear strength decreases, caused by the increase of natural moisture content and unit weight in an global stability analysis for shallow slides, like thin layer sliding (Ortigão & Sayão, 2004), being one of the causes that explain the occurrence of slope instability at Serra do Mar (Victorino, 2015; Sestrem *et al.*, 2015; González *et al.*, 2017).

The series of tests presented in this paper is part of a research with the main objective of compiling geotechnical data, in order to allow a later evaluation of the stability of road slopes along the stretch of BR-376/PR in the Serra do Mar Paranaense, through the identification of the common type of mass movement and the elaboration of susceptibility and safety factor (FS) maps.

2. Geological Background

Lithotypes present in the area of interest include characteristic rocks of the Atlantic Orogenic Belt (Gneiss-Migmatitic Complex), areas of colluvium and talus deposits (MINEROPAR, 2005). Among the lithotypes of the Gneiss-Migmatitic Complex, defined by Siga Jr. *et al.* (1995) as the Atuba Complex, are associations of stromatic migmatite with biotite-hornblende-gneiss paleosome, mica-quartz-schist, ultrabasite, metabasite and amphibolite; ophthalmic migmatites with biotite-gneiss paleosome, biotite-hornblende-gneiss and hornblende-gneiss with local quartzites; biotite-gneisses; ocelar gneisses, interdigitated with stromatic migmatites with the occurrence of banded and leucocratic gneisses and feldspathic schist; undifferentiated migmatites with amphibolites and quartz-feldspathic veins associated with migmatites "*dent de cheval*", local pegmatite and aplo-granites; norites, enderbites, charno-enderbites, gneisses, meta-quartz-diorites, metadiorites, metagabros, including serpentinites and steatites; foliated granite suite, undifferentiated metasomatic granites or anatexia.

MINEROPAR (2002) defines the Complex as a set of stromatic migmatites, granite gneisses, granite gneisses and pebbles, meta-ultrabasic rocks, metabasites, amphibolites and quartzites. There are frequent intercalations of amphibolite bodies, sometimes with garnetiferous or magnesium schists, from centimetric lenses to metric bodies. Features related to the second phase of migmatization are common, with pink mobilizates (K-feldspate), either consistent or not with the gneiss banding. The association of norites, enderbites, charno-enderbites, and others corresponds more properly to a granulite complex. Two calcium-alkaline trends were identified inside this set: one tonalite (norito enderbitic) and the other norite-jotunite-opdalite-charnockitic. In metamorphic terms, a recrystallization event in the order of 800 °C within the granite facies was identified in this sector of the Complex. The granite foliate, anatexytic and metasomatic is inserted in the Gneiss-Migmatitic Complex due to the close relation with the embedded migmatites, in contrast to the granitoid rocks of the Granitic-Gneiss Complex, which is considered intrusive.

In the area of interest, there are also some granitic bodies, defined in the literature as belonging to the Alcali-Granites Suite of Upper Proterozoic - Paleozoic age, with different dimensions, ranging from small stocks to batholiths. It is locally named "Granito de Morro Redondo", a regional toponym These massifs are characterized by alkaline nature, equigranular texture and isotropy, in contrast to the pronounced foliation of gneisses and imbedded migmatites, where contact is normally made through fault zones (MINEROPAR, 2002).

Regarding the colluvium areas of the quaternary age, Angulo (2004) described them as sediments associated with the Serra do Mar slopes, in which no evidence of transport by low viscosity flows was observed. It is described by the author as predominantly fine sediments, with variable proportions of sand and pebbles, usually without structures. Pebbles may be dispersed in the matrix or concentrated in levels or lines (stonelines), with the frequent occurrence of more than one superimposed colluvium with different texture or color characteristics. According to Angulo (2004), colluviums seem to have been originated by slow mass movement processes, involving weathering processes, however, the lines of pebbles and buried soils attest to the complexity of their evolution.

In the description of these sediments and deposits of quaternary age, Angulo (2004) refers to the deposits of talus as sediment accumulations that frequently occur in the

foothills of the steep slopes and whose deposition surfaces create ramps of strong inclination. These ramps are predominantly characterized by debris fall process with no evidence of fluvial processes. In some cases, the ramps described had ravines, with parallel and non-radial patterns, such as in fans. The distribution of the lithotypes is presented in Fig. 1.

The relief, with strongly corrugated morphology in this environment, favors the occurrence of residual soils in the upper third portion of the slopes, conditioned by the incipient pedogenesis development associated to the action of the surface runoff. Thus, these soils have characteristics linked with the original material, represented by variable granite lithotypes, migmatites and gneisses (Furlan *et al.*, 2011; Cerri *et al.*, 2018). The colluvial soils, on the other hand, may be found in the lower two-thirds of the slopes, where they gain depositions from upwind and upstream erosive processes.

3. Material and Methods

The area of study in this paper includes the segment of BR-376/PR within Serra do Mar (Fig. 2). The area extends approximately 32 km and begins in the city of São José dos Pinhais (a city belonging to the metropolitan region of Curitiba, State of Paraná), and continues until just before the border of the State of Santa Catarina, between kilometers 649 and 681 of BR-376/PR. The area includes relief units from the First Plateau Paranaense and the morphosculptural sub-unit of the Serra do Mar Paranaense. The study of hydrographic basins supplies relevant information to the research, for further analysis of rainfall distribution at slope stability (Soares *et al.*, 2012; Vilanova, 2015; Gonzalez, 2017; Cerri *et al.*, 2018; Vieira *et al.*, 2018).

3.1. Geotechnical investigation program

The strategy for choosing the points of geotechnical investigation was initially defined by the identification of the different geologic units, described in the geologic map (Fig. 1) and in field visits. Existing reports and geotechnical projects also helped in the identification and previous localization of the depots of colluvial soils and landfill areas. The location of the investigation points (Fig. 3) was limited to the area next to the borders of road BR-376/PR, between km 660 and 680, due to the difficulty of access caused by the dense vegetation and rough topography, characteristics of Serra do Mar.

The elevation range was defined by eight classes, with intervals of 200 m between each one, beginning with < 200 m, 200 m-400 m, 400 m-600 m, 600 m-800 m, 800 m-1000 m, 1000 m-1200 m, 1200 m-1400 m and > 1400 m, with a distribution of 1.6%, 8.7%, 10.7%, 18.4%, 47.4%, 7.4%, 4.7% and 1.1% for each class, respectively. On the basis of EMBRAPA (2006) classification, and con-

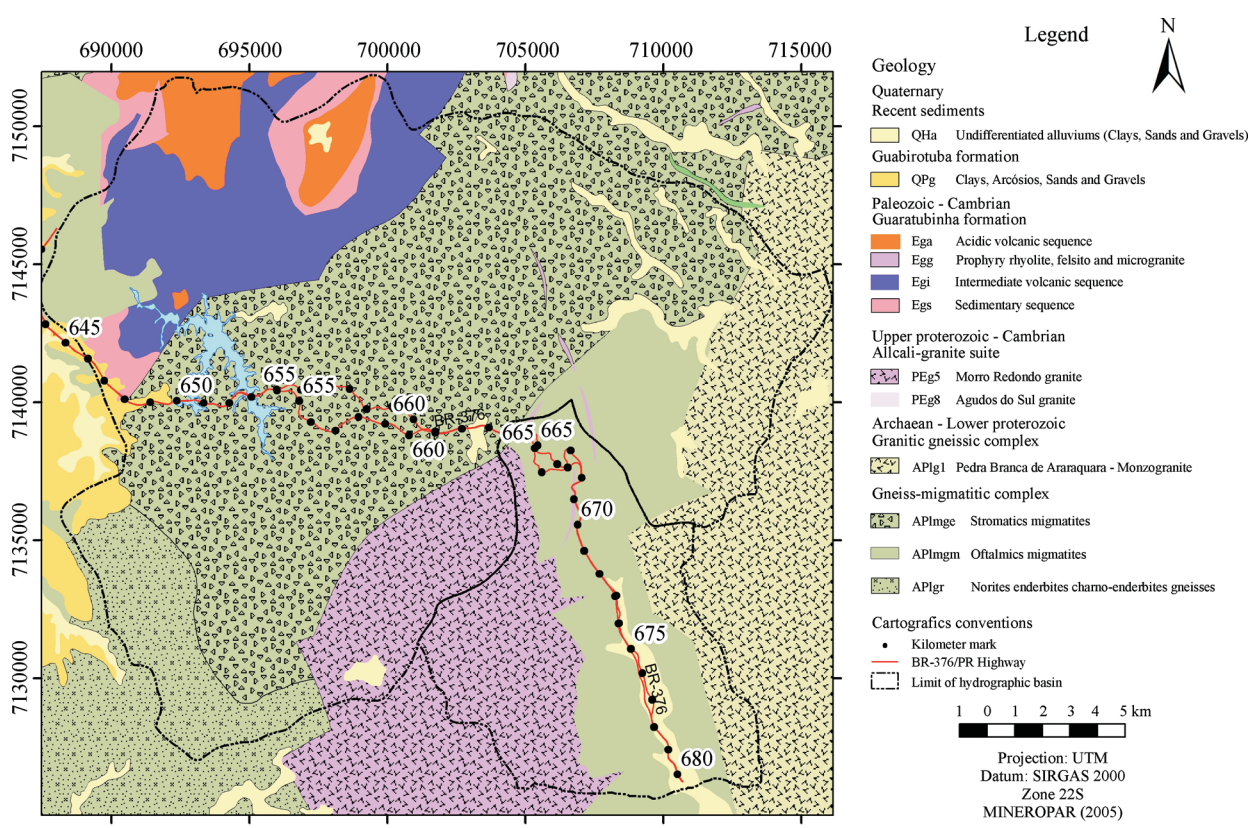

Figure 1 - Geologic map with the main lithotypes (Modified from MINEROPAR, 2005).

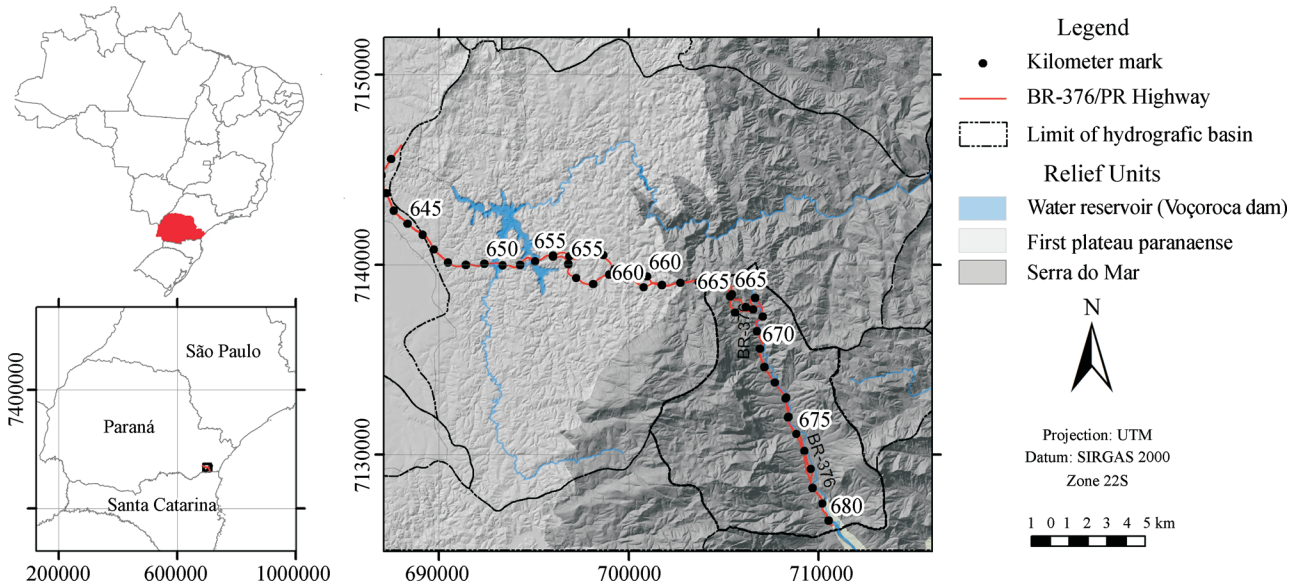

Figure 2 - Location of the study area in Serra do Mar, State of Paraná, Brazil.

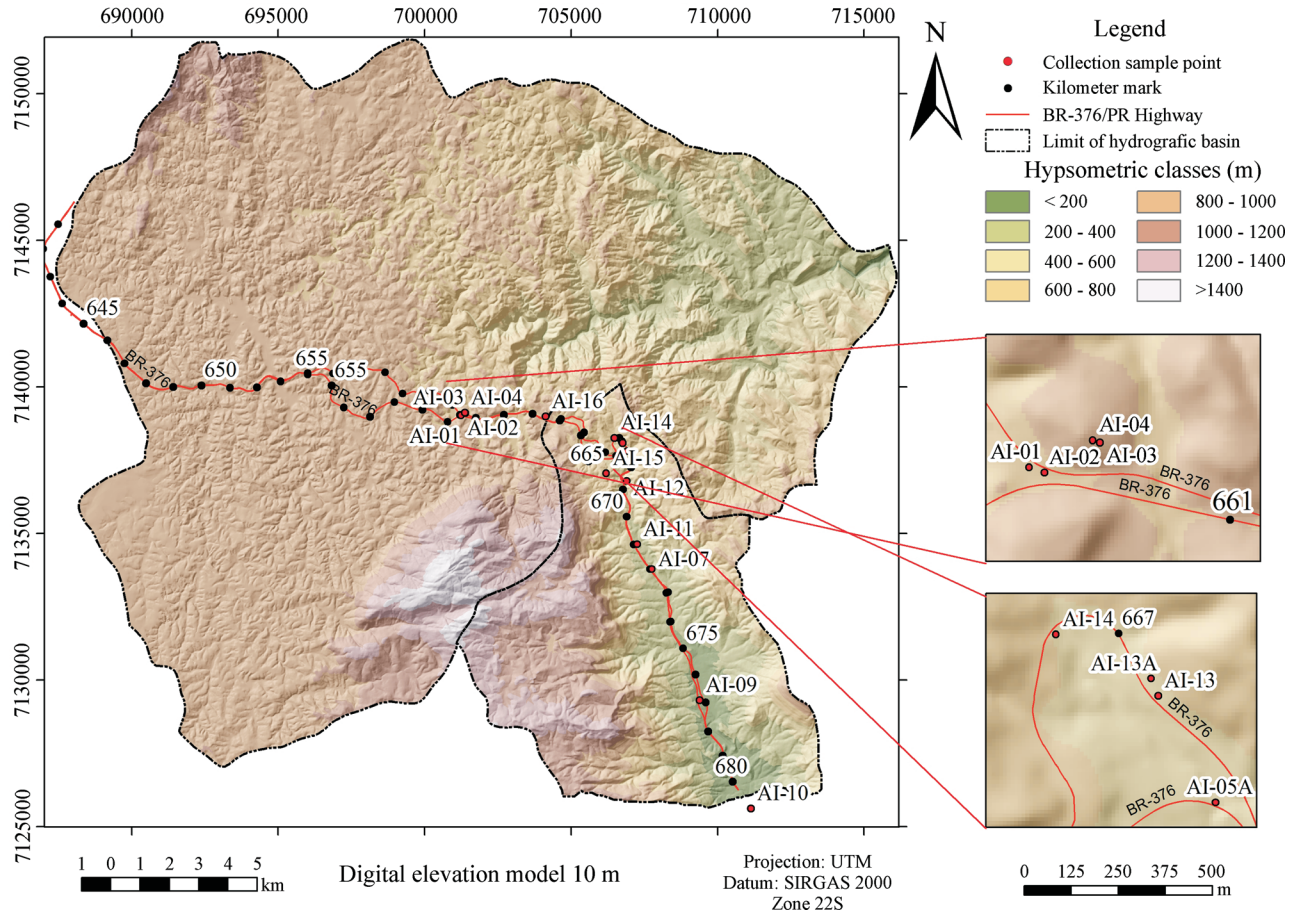

Figure 3 - Location of undisturbed samples.

sidering the slopes declivity, the area presented 45.1% of the area as Strongly Wavy, 22.1% as Wavy, 19.4% as Hilly, 7.5% as Softly Wavy, 3.6% as Steep and 2.4% as Plane.

The geotechnical investigation was distributed along the large area of study in order to increase its representativeness (Fig. 3). The soil has a process of natural forma-

tion, making its properties strongly dependent on the geological processes acting in their genesis and, therefore, the parameters defined for each type of soil carry some intrinsic variability due to the heterogeneity of the material. The undisturbed samples were obtained by excavation, avoiding roots and organic material. No signals of wall instability of the excavations were noticed at any point. Then, blocks received paraffin in all their surroundings and were involved with fabric, receiving paraffin once again. Such procedures follow standard NBR 9604 (ABNT, 1986), which tries to preserve the natural moisture conditions and to ensure the physical integrity of the sample. Before being moved from the wells, the blocks were packed in wood boxes and the voids filled with sawdust. The collection procedure involved also the identification of the caps of the boxes. Besides this procedure, from the bottom of each well approximately 3 kg of deformed material were taken, in order to proceed with geotechnical characterization tests.

3.2. Field and laboratory tests

Information of location and type of material sampled along the studied stretch are found in Table 1. The elevation and UTM coordinates were obtained via navigation GPS with precision indicated by the manufacturer of approximately 5 m. In addition, for relative location of the material sampled, data of the approximate km of the track were inserted. With regard to the location of the road where the sampling was made, it was classified as N for the track in direction North and S for the track in direction South, being LS for left side and RS for right side of the road, as well as the UTM coordinates (Zone 22S), along with the respective values of East (E) and South (S) and the depth of the disturbed and undisturbed samples to be used in the tests. The depths of sampling were defined based on field visits, in which land scars of ancients mass movements and the common type of appearance were identified, and were classified as translational and subsurface (González, 2017).

The 11 (eleven) samples were submitted to geotechnical characterization (moisture content, granulometric analysis, Atterberg Limits and particle density) and 5 of them were selected to carry out conventional direct shearing tests (with measurement of mechanical strength for specific and residual deformations), and another 4 samples to carry out consolidated undrained triaxial compression tests (CIU).

In places where samples were retrieved, permeability *in situ* tests were also carried out using a mini disk infiltrometer type. The *modus operandi* of the test consists basically in verifying the volume of water that infiltrates the soil in a given interval of time along with a given suction. For this, the upper and lower chambers of the test setup are filled with water, the upper chamber controlling the applied suction and the lower being used as a graduated reservoir to determine the water infiltrating the soil (Victorino, 2015).

Fatehnia *et al.* (2014) describe that the area measured by this type of method is small because of limitations of the disk size of the infiltrometer and small depth of the test. However, it is mainly used for determining the hydraulic properties of the superficial layer of the soil. Moreover, contrary to other devices that only measure the flow under submerged or saturated conditions (such as, for example, the double-ring permeameter), the tension disk infiltrometer is able to measure the non-saturated hydraulic conductivity of the soil. In order to measure the hydraulic conductivity of the soil, a negative potential (suction) must be made over its surface. In the current study, the depth of measurement was defined in 2.0 m (at the depth of the undisturbed samples) due to the low thickness of the potential failure plane.

The soil mechanical strength parameters were determined via direct shear and triaxial compression tests car-

Table 1 - Information about points of geotechnical investigation.

Sample ID	km	Road	Side	Origin of soil	Elevation (m)	UTM Coordinates		Depth (m)
						E (m)	S (m)	
AI-01	659	S	LS	Residual	804	0701199	7139033	1.5
AI-02	659	S	RS	Colluvium	809	0701199	7139033	1.0
AI-03	659	N	RS	Residual	800	0701387	7139099	2.3
AI-04	659	N	RS	Residual	818	0701368	7139105	1.0
AI-07	672	S	RS	Residual	311	0707750	7133769	1.2
AI-09	676	S	LS	Colluvium	178	0709382	7129296	0.9
AI-13	668	N	RS	Colluvium	582	0706740	7138096	1.5
AI-13A	668	N	RS	Residual	601	0706721	7138133	1.0
AI-14	667	N	RS	Residual	618	0706474	7138248	1.3
AI-15	666	N	RS	Residual	652	0706169	7137050	1.0
AI-16	664	N	RS	Residual	829	0704124	7138979	0.8

ried out at the Laboratory of Soils of the Institute of Technology for Development (LACTEC). The geotechnical characterization was performed at the Laboratory of Materials and Structures (LAME) of the Federal University of Paraná (UFPR).

Direct shear tests were conducted according to the British Standards BS 1377 - part 7 (BSI, 1990a). Three shearing apparatus were used: two of model L02900 from Wille Geotechnik and one model Shear Trac II made by Geocomp. The test specimens were trimmed in square rings of 100 mm side and approximately 20 mm height. After trimming, the specimen was slowly extruded into the shear box, a setting load was applied, and the soil was submerged in water (under inundation). A consolidation pressure was then applied (30, 60 and 90 kPa) during a minimum period of 24 h. Settlement readings were taken as a function of time to allow appropriate calculation of consolidation coefficients (c_v = 2.2 × 10^{-5} to 6.2 × 10^{-4} cm^2/s for 30 kPa; 8.7 × 10^{-5} to 2.3 × 10^{-3} cm^2/s for 60 kPa and 2.5 × 10^{-4} to 3.9 × 10^{-3} cm^2/s for 90 kPa) and to ensure that the sample has reached equilibrium prior to the start of shearing. The shearing rate was defined from parameters of consolidation as proposed by Gibson and Henkel (1954, cited by Head, 1981), allowing to perform the test in drained conditions. The shearing rate adopted for all tests was 0.07 mm/min.

Skempton (1964), cited by Kanji (1998) evaluated the use of parameters of peak or residual strength for determining the safety factor of slopes. Kanji (1998) considers that this choice should be based in the level of stress and deformation of the slope, also considering geotechnical and geologic aspects, such as the presence of joints and fissures, degree of weathering and development of progressive failure. The author concluded that the presence of fissures and joints may lead to a progressive failure until the material reaches residual strength, suggesting, thus, the adoption of residual strength values in these cases.

Kanji (1998) proposed a simple test for obtaining residual strength parameters whose procedure consisted in molding clay soils until their liquid limit and shearing them over a polished (smooth) surface. The main difference between this procedure and the conventional one is that the lower half of the shear box is filled with polished rock, making the soil sample slide over it. The author suggests that using this technique the particles of soil become oriented in the interface. Advincula (2016) uses the test specimen trimmed from the undisturbed block; however, in order to create the polished interface, a thin steel wire was used for cutting the sample directly in the polished surface and to evaluate the residual shearing strength.

Based on Kanji (1998) and Advincula (2016), for the residual strength test of this research, the soil was molded directly in the undisturbed block and sheared over a polish surface that consisted in a granite block with dimensions of 100 mm × 100 mm × 20 mm. This interface was placed in the lower part of the shear box, in order to assess the residual shear strength of the sample. This test integrates both methods previously described, taking the best procedure from each one.

Regarding triaxial tests, they were executed using a Wille Geotechnik shearing apparatus, model UL60, according to British Standards BS 1377 - part 8 (BSI, 1990b). Triaxial shear tests were performed using cylindrical soil specimens of 50 mm in diameter by 100 mm in height. The complete saturation of triaxial specimens was achieved by employing two methods: percolation of water and application of back pressure. Back pressure in the order of 350 kPa was enough for *B* pore pressure parameter to reach the minimum of 0.95 (BSI, 1990b). For the shearing stage the effective compression pressures were between 15 kPa and 90 kPa with an axial displacement rate of 6.3 × 10^{-2} mm/min.

The definition of the shear strength parameters of the saturated soil depended upon the failure criteria used to determinate it. Among the most commonly used methods, are applied: peak diverting tension (maximum deviation tension), constant inclination, maximum ratio of the principal stresses (σ'_1/σ'_3), stress path and specific deformation (residual strength).

Both direct shear tests and triaxial tests were performed with saturated samples considering that this is the worst scenario, and better control conditions are possible during the tests (Advincula, 2016; González, 2017; Trevizolli, 2018).

4. Results and discussion

4.1. Geological aspects

Through information obtained from field work compared with information available in the literature about the area of interest, it was verified that:

Along the studied stretch of the road, residual soils were found with more frequency and greater thickness from half way up the slope to the top of the corrugated reliefs. Two main soil types have been identified that are related to this rock matrix: residual soils of migmatites/gneisses and residual soils from granite.

The residual soils of migmatites/gneisses are the most abundant and are characterized by the predominance of diverse colors (yellows, reds and whites). They are predominantly silty, compact and have low permeability and plasticity. This type of soil is frequently in the presence of relict structures from the parent material, characterized by a whitish color, which comes from concentrations of felsic materials, such as feldspar and quartz. More reddish and yellowish coloration levels are due to greater concentrations of mafic minerals in the matrix rock, such as micas of biotite type, and amphiboles.

Although the geologic map of the region indicates a distance between the presence of granites and the area of study, outcrops of this lithotype were observed during site visits, on slopes adjacent to BR-376/PR highway, which

were then mapped in addition to weathered residual soils. The residual soils of granite were characterized by lighter colors evidence of clay of kaolin type, resulting from the alteration of feldspars. The presence of colluvial soil was marked by dark grey color, lack of structure and thickness of approximately 50 cm. A layer of organic soil was also found, about 20 cm thick, and dark in color.

The residual soils from granite had lack of relict structures in the form of levels of different colorations, white to reddish. The relict structure observed in some cases is characterized by a differentiated granulometric aspect, given by the weathering alteration of feldspars of clear colors and of mafic minerals of reddish colors, oriented according to an incipient foliation that may be both of metamorphic and magmatic origin, having mineral granulometry from medium to coarse (ALS, 2014).

Most colluvial soils are characterized by the reddish, brownish or yellowish color, like bricks, of silt-sandy matrix and without evidence of relict structures from the matrix rock. They appear over residual soils, in some cases being very clear the transition between both, especially by a sudden change of coloration. They are located mainly in the more flattened tops or in the lower portions of the strands and have very irregular thickness (ALS, 2014).

The definition of colluvium soils is given as the material composed of blocks and/or grains of any dimension, transported by gravity and accumulated in the base or at a small distance of steeper slopes or rocky cliffs (Lacerda & Sandroni, 1985). This type of soil is often found in areas situated in the lower third of slopes and mountains, where the relief is strongly corrugated and makes no reference to the origin of the soil. Depending of transport factors, it may involve more than one type of material.

Contacts between colluviums and residual soils of migmatite are more accentuated in outcrops. Between horizons A and B (colluvium soils), the sudden change of coloration marks the transition between them. The transition from colluvium for residual soil (horizons C) is more subtle, marked by the change from a material without structure, or massive, to a material with relict structure, inherited from the matrix rock. Besides, the disperse whitish colors indicates the presence of weathered feldspars, in the case of residual soil, feature not verified in colluvium soil.

In more restricted places of Serra do Mar Paranaense, inside the interest area of this study, there are alluviums associated with the main channels in the section of the medium rivers course such as the "São João", among others. Those alluviums cover an extensive region of Guaratuba, in the portion of the coastal plain, in the riverbeds of "São João, Cubatão, Cartãozinho and Canavieiras" rivers. They are constituted by sediments of river deposition, predominating sand and riverbeds of gravel, which may be associated with depots of meadow and slope. The meadow depots appear in restricted areas along some draining, characterized by unconsolidated sediments, of small thickness, constituted by silts and clays, partly turfy and with sand of different granulometry. Inside those depots may also appear gravel riverbeds, of Holocene age, where predominate quartz and quartzite pebbles, well selected and rounded, indicating effective transport (ALS, 2014).

4.2. Characterization tests

Characterization tests encompassed obtaining the soil natural moisture content, Atterberg Limits and the particle density of soil samples collected in field, representing the surface material composing the hillside. Also, *in situ* hydraulic conductivity tests were performed at the bottom of the well of the undisturbed soil samples collected in the field. Results from characterization tests are shown in Table 2. Except for samples AI-13 and AI-16, classified respectively as clayey sand and silty gravel soil, the other samples were classified as silty sand, according to USCS (ASTM, 2017).

According to the plasticity chart relating liquid limit (LL) *vs.* plasticity index (PI) (Fig. 4), most of the samples have behavior belonging to silty soils (below line A) and all samples had liquid limit (LL) under 50% (left of line B), being, therefore, classified as soils with low compressibility and low to average plasticity. Line A distinguishes clay soils (placed above) from silty soils (placed below). Line B, describing the degree of compressibility of soils, separates the materials with liquid limit lower and greater than 50%.

Only sample (AL-07) of residual soil from migmatite exhibited behavior belonging to clay soils (over line A), al-

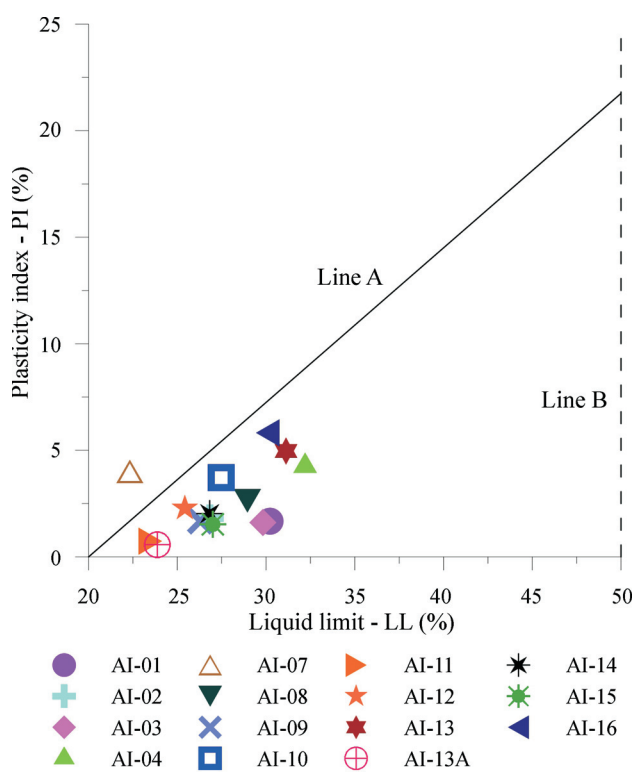

Figure 4 - Plasticity chart.

though the amount of clay present in the granulometric distribution was 3.8%. Granulometrics classification, Atterberg Limits and the particle density results are coincident between them, which indicates accuracy in the procedures. When compared by means of granulometric and the plasticity chart classification as silty sand soils, the behavior corresponds to silt of low compressibility and low plasticity.

Results of granulometric analysis are presented in Table 3, which displays the percentage of material retained in each granulometric range.

The granulometric analysis of residual soil from migmatite showed that the soil lies in a well-defined range with

Table 2 - Values obtained by soil characterization tests.

Sample ID	Natural moisture (%)	Liquid limit (%)	Plastic limit (%)	Particle density (g/cm^3)	USCS classification
AI-01	25.6	30.2	28.5	2.773	Silty sand
AI-02	19.3	26.8	24.6	2.645	Silty sand
AI-03	23.5	29.8	28.2	2.694	Silty sand
AI-04	26.6	32.2	27.8	2.804	Silty sand
AI-07	19.2	22.3	18.3	2.680	Silty sand
AI-09	27.4	26.3	22.9	2.645	Silty sand
AI-10	22.8	27.5	20.5	2.637	Silty sand
AI-11	18.9	23.5	22.7	2.626	Silty sand
AI-13	30.6	31.1	26.1	2.693	Clayey sand
AI-13A	20.5	23.9	21.1	2.699	Silty sand
AI-14	22.9	26.8	24.8	2.677	Silty sand
AI-15	25.2	27.0	25.5	2.666	Silty sand
AI-16	23.1	30.1	24.3	2.634	Silty gravel

Table 3 - Granulometric analysis of soil samples.

Soil	Sample ID	Soil classification (USCS)	Gravel (2.0 mm < % < 60 mm)	Sand (0.06 mm < % < 2.0 mm)	Silt (2 μm < % < 0.06 mm)	Clay (% < 2 μm)
Residual from migmatite	AI-01	Silty sand	0.0	32.1	63.3	4.6
	AI-07	Silty sand	2.2	38.4	55.7	3.8
	AI-13A	Silty sand	4.6	34.9	56.4	4.1
	Average		1.6	32.7	61.1	4.6
	Standard deviation		1.9	4.7	6.3	1.6
Colluvium	AI-02	Silty sand	0.4	37.6	55.3	6.7
	AI-09	Silty sand	4.4	23.8	60.0	11.8
	AI-13	Clayey sand	0.1	20.4	48.8	30.7
	Average		0.3	29.0	52.1	18.7
	Standard deviation		0.2	12.2	4.6	17.0
Residual from granite	AI-03	Silty sand	1.2	32.3	59.2	7.3
	AI-04	Silty sand	0.1	25.6	71.1	3.2
	AI-14	Silty sand	0.1	35.1	54.3	10.5
	AI-15	Silty sand	0.2	38.1	56.5	5.2
	AI-16	Silty gravel	32.8	25.3	27.0	15.0
	Average		9.4	30.6	49.5	10.6
	Standard deviation		15.7	7.1	15.2	4.1

predominance of silty material (Fig. 5). The average amount of silt in samples is equal to 61.1%, with a standard deviation of 6.3%. The fraction of sand in samples was 32.7% on average, with standard deviation of 4.7%. The fractions of gravel and clay represented a small portion of the soils with averages of 1.6% and 4.6% respectively, and standard deviations of 1.9% and 1.6% respectively. The particle density was approximately 2.717 g/cm^3 and standard deviation of 0.049 g/cm^3. The natural moisture content of these samples was 21.8% with standard deviation of 3.4%.

Samples AI-14 and AI-15 related with the residual soils from granite showed very close granulometric distribution and consistence index (Fig. 6). In contrast, sample AI-16 had a predominance of gravel (32.8%) and high liquid limit, probably associated with the greater concentration of clay (15.0%). For all samples (residual and colluvial soils) the liquid limit (LL) had average of 27.3%, with 3.5% standard deviation and the plastic limit (PL) had average of 26.1% with 1.8% standard deviation. The particle density was approximately 2.695 g/cm^3 and standard deviation of 0.065 g/cm^3, with a moisture content in natural condition of 24.2% and standard deviation of 1.6%.

The granulometric distribution of colluviums showed greater dispersion when compared with residual soils (Fig. 7). While in the sample of colluvium soil AI-02 predominated the occurrence of silty sand material, sample AI-13 had a significant percentage of clay of approximately 30.7%. A small amount of gravel was found in the two samples. The greater liquid limit (LL) corresponding to samples of colluvium material was found in sample AI-13 (31.1%), which may be explained by the clay portion present in the material. The average value for plastic limit (PL) was 24.5%, for particle density was 2.661 g/cm^3 (with standard deviation of 0.028 g/cm^3), and for natural moisture content was 25.8% (with standard deviation of 5.8%).

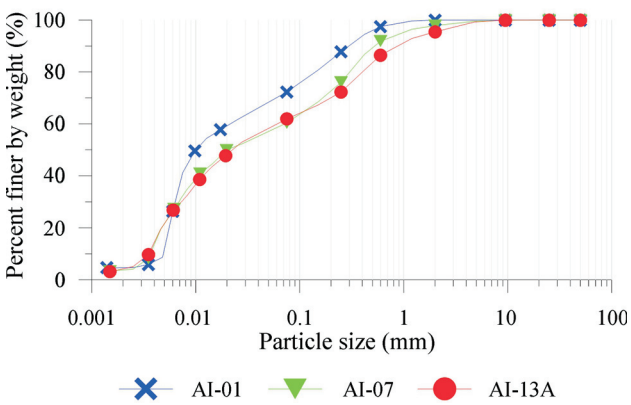

Figure 5 - Granulometric curves related to migmatite residual soil.

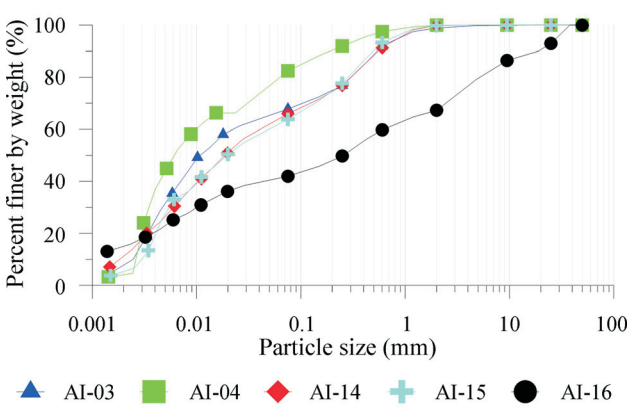

Figure 6 - Granulometric curves related to granite residual soil.

4.3. *In situ* permeability tests

By means of *in situ* permeability test of superficial soil, made at the bottom of the sampling wells of the undisturbed samples, average values of hydraulic conductivity around 10^{-4} cm/s were obtained, with minimum value of 3.1×10^{-5} cm/s and maximum value of 1.0×10^{-3} cm/s, getting as result coefficients defining a low degree of permeability, according to Terzaghi & Peck (1967). Such average of hydraulic conductivity is found in the interval defined by Casagrande & Fadum (1940), corresponding to very fine sand and silt. According to Lambe & Whitman (1969), those values correspond to sandy clay. For Pinto (2006), this value of hydraulic conductivity is considered characteristic of clay and fine sand. Das (2007) defines these values of hydraulic conductivity as belonging to clay or silt. Therefore, the value of the hydraulic conductivity obtained is found adequate for soils of the studied site, classified as silty sand, according to the USCS granulometric analysis.

Vieira *et al.* (2018) used in their investigation values of 10^{-4} and 10^{-3} cm/s obtained with Guelph Permeameter in three land scars in an experimental basin located in the Serra do Mar (Copebrás basin, São Paulo State), either in colluvial soil or in migmatite saprolite's of the Embu Complex and Costeiro Complex. According to the authors, two types of regolith above an intensely fractured bedrock were observed: colluvial soil with depths about 1.0 m, formed by pedogenesis over transported material, with a sandy-clayey texture matrix and partially weathered bedrock; and sapro-

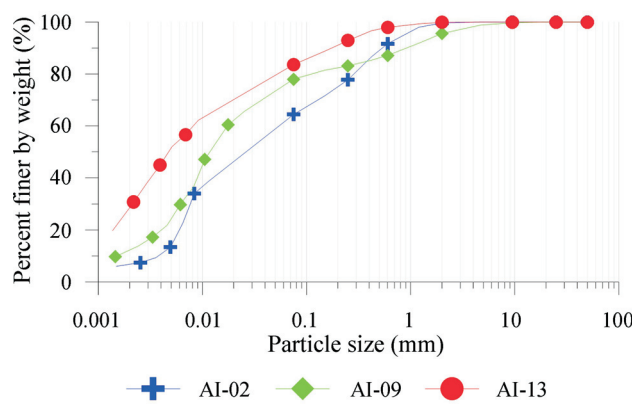

Figure 7 - Granulometric curves related to coluvium soil.

lite (about 3.0 to 4.0 m depth), more sandy than the overlying horizon, with evidence of structures inherited from bedrock.

Trevizolli (2018) studied the *in situ* permeability in a slope located at Serra do Mar, km116 in Barra do Turvo - State of São Paulo, using Guelph Permeameter. The author obtained a hydraulic conductivity of 10^{-5} cm/s for the superficial soil of the slope. The soil of the slope investigated was classified as colluvium and residual soil from Migmatite rock, with granulometric analysis resulting in clayey sand and clayey sand with gravel, respectively.

Therefore, comparing the results found here with previous research (Vieira *et al.*, 2018; Trevizolli, 2018), the soil particle size resulting from the weathering of migmatite is partially similar to the other regions at Serra do Mar with similar lithotypes, generating hydraulic conductivity between 10^{-5} and 10^{-3} cm/s for this soil.

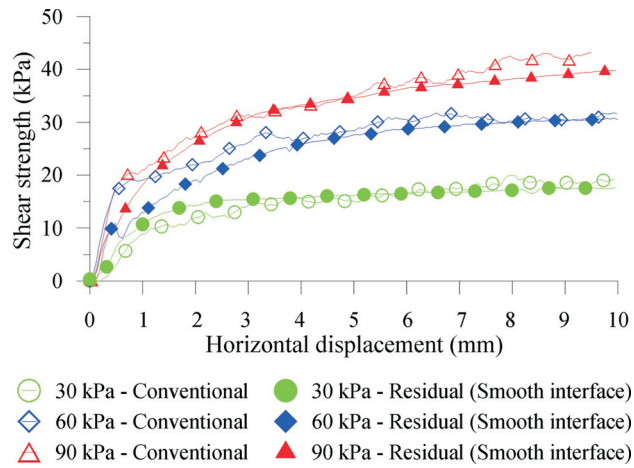

Figure 9 - Shear strength *vs.* horizontal displacement for migmatite residual soil (AI-01).

4.4. Mechanical behavior tests

Conventional and smooth interface direct shear tests were performed for obtaining parameters of peak and residual strength, in both cases under inundation, in addition to triaxial CIU test (Fig. 8).

Based on the shearing stress-horizontal displacement response shown in Fig. 9 from conventional and smooth interface direct shear tests, the failure mode of the test specimens happened in a ductile way, in other words, with no peak (Fig. 9). This characteristic is typical of the behavior of sandy soils with void ratio greater than critical, in other words, loose sands.

Table 4 and Table 5 show a summary of shear strength results: friction angle (ϕ') and cohesive intercept (c'), normal stress, degree of saturation initial (i) and final (f) from conventional direct shear tests and smooth interface method.

The determination of the parameters of maximum shear strength of the soils considered a horizontal displacement of 8 mm, which was established considering the shear stress-displacement response. For this displacement value, the shear stress is nearly constant. For residual strength test (smooth interface) larger values of horizontal displacement were required (11 mm) in order to reach constant values of shear stress (Tchalenko, 1970; Suzuki, 2004; Advincula, 2016; González, 2017; Trevizolli, 2018).

For colluvium soils, the cohesive intercept was greater than that observed for the residual soil from migmatite, and similar to that of the residual soil from granite.

Skempton (1985, cited by Kanji 1998) interpreted the residual strength of soils as a function of their fraction of clay (grains smaller than 2 μm). The author concludes in its work that, for soils with percentage of clay over 50%, the angles of residual friction found are lower and have great differences between peak and residual values. On the other hand, when the percentage of clay is smaller than 25%, residual values much closer to the peak strength values are found. This trend was verified in the present research.

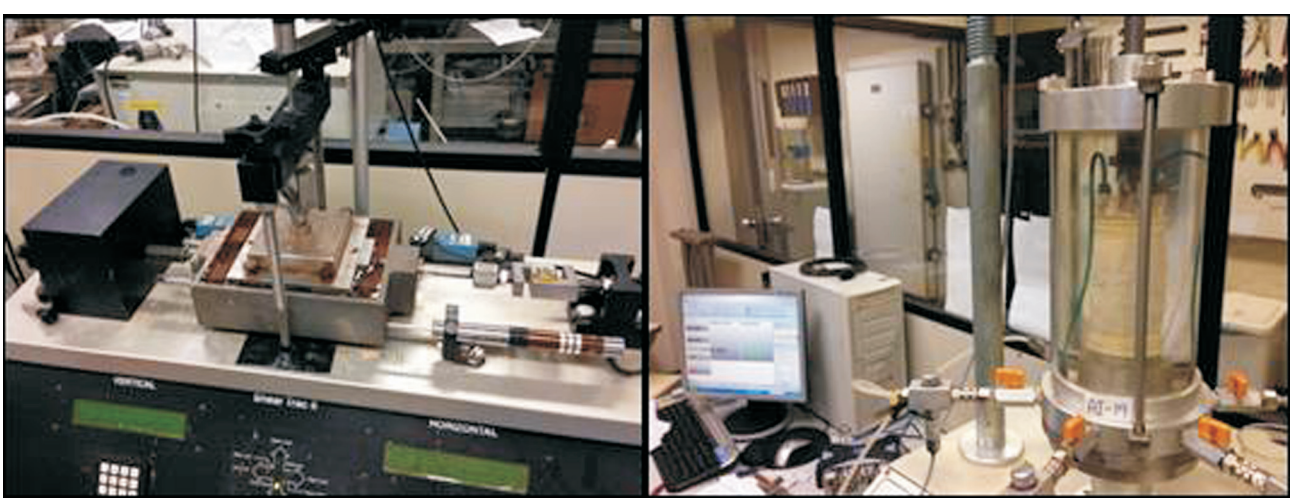

Figure 8 - Conventional direct shear (left) and triaxial CIU (right).

Table 4 - Parameters of soil strength from conventional direct shear tests.

Material	Sample ID	Normal stress (kPa)	Degree of saturation (i) (%)	Conventional direct shear strength test			Degree of saturation (f) (%)
				Displac. (mm)	c' (kPa)	ϕ'(°)	
Residual soil from migmatite	AI-01	30; 60; 90	60	8.0	6.5	27.9	91
	AI-13A	30; 60; 90	90	8.0	0.0	38.3	100
Colluvium	AI-02	30; 60; 90	70	8.0	8.2	26.0	93
	AI-09	30; 60; 90	80	8.0	14.8	37.9	100
	AI-13	30; 60; 90	90	8.0	11.7	31.8	100
Residual soil from granite	AI-14	30; 60; 90	60	8.0	14.2	28.1	87

Table 5 - Parameters of soil strength from direct shear with smooth interface method.

Material	Sample ID	Normal stress (kPa)	Degree of saturation (i) (%)	Direct shear strength method of smooth interface			Degree of saturation (f) (%)
				Displac. (mm)	c' (kPa)	ϕ'(°)	
Residual soil from migmatite	AI-01	30; 60; 90	60	11.0	0.0	25.6	92
Colluvium	AI-02	30; 60; 90	70	11.0	0.0	26.8	93
	AI-09	30; 60; 90	80	11.0	11.8	18.6	100
	AI-13	30; 60; 90	90	11.0	0.0	24.4	100

Therefore, sample AI-13 that has 30.7% of clay had difference of 7.4° between the friction angle obtained in the conventional shearing test and the smooth interface test, while sample AI-01, with 4.6% of clay fraction, had a difference of 2.3° between those two tests.

Sample AI-09, pertaining to colluvium soil, had a difference of 19.3 degrees between the conventional direct shearing test (37.9°) and the residual strength by smooth interface (18.6°), showing inconsistent result and too low for residual friction angle.

In sample AI-13A, the presence of relict structures was identified. This characteristic has a tendency of conditioning the failure surface and having strength parameters under or over the ones defined in the conventional direct shear test, depending on the position of structures and weakness plans about the shear surfaces in the shear box.

By comparing the present results with other researches, the behavior failure mode in a ductile way (no peak) was also observed in the study of Advincula (2016) on the determination of peak and residual strength of colluvium tropical Brazilian soils in the State of Rio de Janeiro, for normal stresses from 25 kPa to 200 kPa. Although those are colluviums soils, Advincula (2016) studied a sample of residual soil from migmatite, which exhibited peak in failure at the convencional direct shear test. For colluvium soils, the values of both residual and peak friction angles obtained by the cited author were between 22.5° and 37.4°, with values of cohesive intercept between 0.0 and 9.5 kPa for residual strength and 0.8 and 19.8 kPa for peak strength. In the sample of residual soil from migmatite, the peak friction angle was 36.4° with a cohesive intercept of 36.5 kPa, while the corresponding residual values were 15.3° and 7.0 kPa, respectively. The author attributes the decrease of strength from peak to residual values, to the presence of mica in the mineral composition (Rigo *et al.*, 2006; Advincula, 2016).

Trevizolli (2018) performed conventional and smooth interface direct shear tests (at normal stresses from 50 kPa to 200 kPa) in colluvium and residual soils (under inundation) from Migmatite rock at Serra do Mar slope in Barra do Turvo - State of São Paulo. The author found failure mode in a ductile way (no peak). The values of both residual and peak friction angle obtained by the cited author were between 17.8° and 30.2°, with values of cohesive intercept between 4.7 and 16.8 kPa.

Suzuki (2004) performed conventional direct shear tests (at normal stresses from 25 kPa to 400 kPa) in colluvium and residual soils (under inundation) from Serra do Mar at Morretes - State of Paraná. The author also found failure mode in a ductile way (no peak) for most samples tested. The resistance parameters varied between 2.0 and 21.0 kPa for cohesive intercept and 28.3° to 34.6° for friction angle.

By means of triaxial tests, it was verified that the behavior of soil not always had a defined peak and, in some tests, the material was identified with a strain-hardening behavior, without a well-defined failure (Fig. 10). Thus, for processing results according to the material behavior, dif-

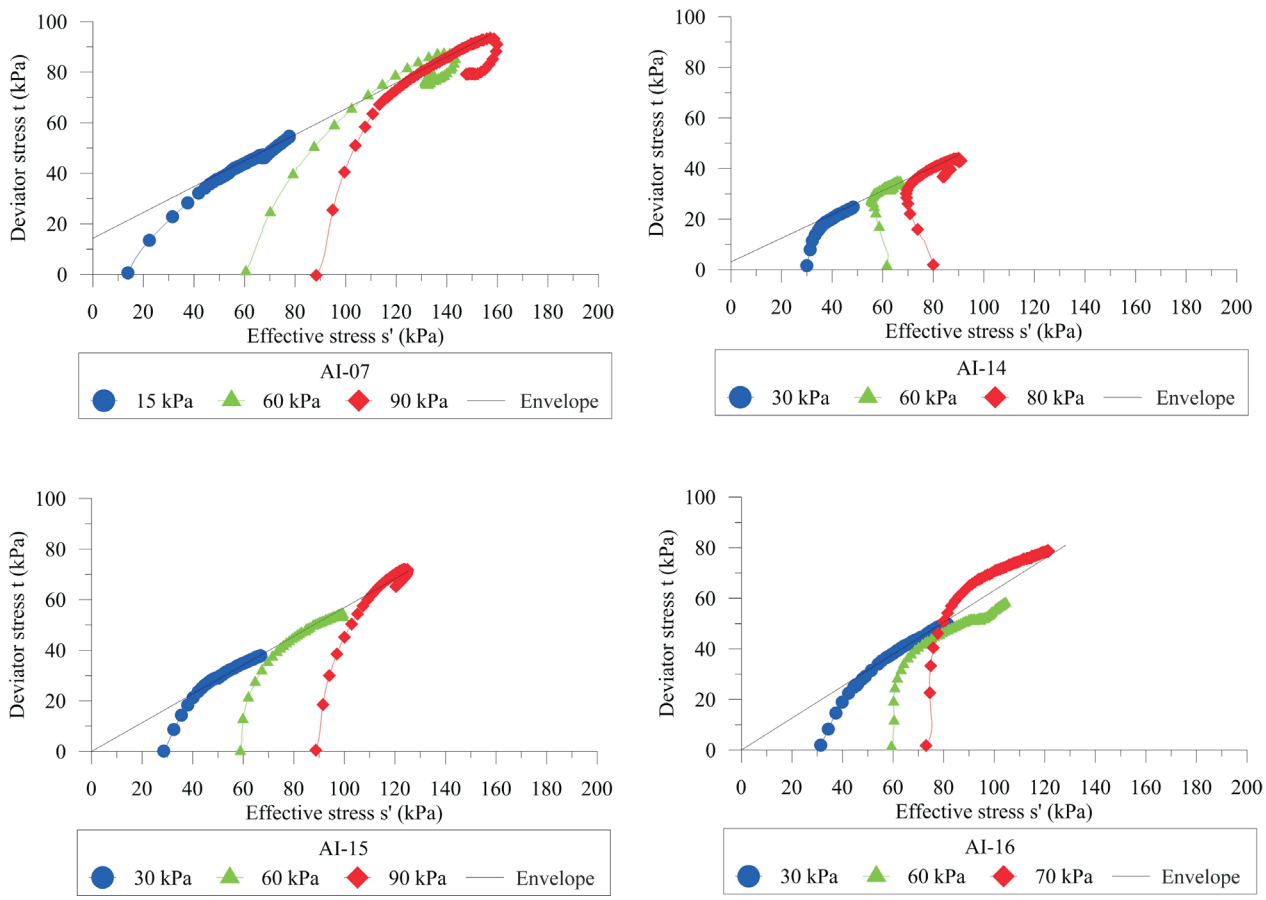

Figure 10 - Stress path curves obtained through Triaxial CIU test in samples (a) AI-07; (b) AI-14; (c) AI-15 e (d) AI-16.

ferent failure criteria were used. Triaxial tests presented a strain-hardening behavior, so the criteria of maximum rate of the principal stresses (σ'_1/σ'_3) and stress paths were applied.

The results obtained with CIU triaxial tests are presented in Table 6, as well as the values of degree of saturation initial (i) and final (f), the value of *B* pore pressure parameter and the failure criteria applied.

Contrary to that presented in the conventional direct shearing test, the residual soil from migmatite subjected to triaxial test showed higher values of cohesive intercept, while the samples of residual soils from granite showed lower values. The friction angle showed no significant variation in the studied soils.

Generally, when comparing the results obtained from conventional direct shear tests with results from CIU triaxial tests, the values of internal friction angle were concordant, for all samples tested here (residual and colluvium soil). The minimum value of the friction angle determined from the conventional direct shear test was around 26.0°

Table 6 - Parameters of soil strength obtained with triaxial tests.

Material	Sample	Degree of saturation (i) (%)	*B* pore pressure parameter	Degree of saturation (f) (%)	Failure criteria	c' (kPa)	ϕ' (°)
Residual soil from migmatite	Al-07	65	1.00	100	σ'_1/σ'_3	12.7	33.3
					Stress path	14.3	32.2
	AI-14	63	0.97	99	σ'_1/σ'_3	1.6	28.9
					Stress path	3.1	27.8
Residual soil from granite	AI-15	58	0.96	99	σ'_1/σ'_3	0.5	35.1
					Stress path	0.0	34.8
	AI-16	81	0.98	99	Stress path	0.0	43.2

and the maximum value around 38.3°, with average of approximately 31.7°. In the CIU triaxial test, the minimum value found was 27.8° and the maximum value of 43.2°, with average of approximately 33.6°. Consequently, the average peak friction angle of 32° was adopted as being a conservative value for a regional approach.

Results obtained in tests of residual shear of smooth interface (for residual soil from migmatite and colluvium) presented minimum values, around 18.6°, showing inconsistent result and too low for residual friction angle, and maximum around 26.8°. Thus, the average residual friction angle of approximately 26° was assumed.

Results regarding the cohesive intercept showed more disparity (for all samples: residual and colluvium soil), with values being obtained by means of direct shear tests (ranging from 0.0 kPa to 14.8 kPa, with average equal to 9.2 kPa) similar in magnitude of minimum and maximum to the ones obtained by means of CIU triaxial tests (ranging from 0.0 kPa to 14.3 kPa, however with average equal to approximately 4.6 kPa).

In the detailed analysis of sample AI-14 (Residual soil from Granite), in which direct shear and CIU triaxial test were carried out, a small difference in the friction angle was observed. In direct shear test, the friction angle was 28.1° and from CIU triaxial test was 28.7°. The cohesive intercept was significantly disparate, being 14.2 kPa for conventional direct shear and 1.6 kPa for CIU triaxial test.

Values of cohesive intercept obtained in CIU triaxial tests for granite residual soils were closer to the results obtained in direct shear tests of smooth interface (0 kPa). The triaxial test carried out with migmatite residual soils showed greater values of cohesive intercept (average of 13.5 kPa), being different from the results obtained with samples of granite residual soils, which showed values between 0.0 and 3.1 kPa, with average of 1.0 kPa.

Once again, comparing the present results with other references about CIU triaxial tests in Brazilian tropical soils, Advincula (2016) obtained values of friction angle in colluvium soils between 29.5° and 31.1° and cohesive intercept between 6.4 kPa and 11.8 kPa.

With respect to residual soils, Carvalho (2012) performed triaxial CIU tests in migmatite residual soil in the state of Rio de Janeiro, obtaining as results values of friction angle of 21.5° and cohesive intercept of 105.8 kPa. It must be noted that as the pressures applied by Carvalho (2012) are between 25 and 500 kPa, the values of friction angle and cohesive intercept analogous to tests described in this paper could not be correlated. However, analyzing only effective stresses of 25, 75 and 150 kPa from the work of Carvalho (2012), the value of cohesive intercept and friction angle were 74.2 kPa and 27.7°, respectively, being more coherent with the results of the present research, however with a greater cohesive intercept.

In order to have knowledge about the results of shearing strength in residual terms, Bressani *et al.* (2001) presented a review of several references for different types of Brazilian tropical soils, which made possible to observe that the average angle of residual friction is 25° (excluding results from residual basalt soil, grey and red clay listed) and cohesive intercept has a lower value, around 0 kPa.

Tonus (2009), based on data presented by Dell'Avanzi *et al.* (2007) from triaxial CIU and CID tests, direct shear and ring shear tests, presented a summary of the parameters of peak strength found for different lithostratigraphic units, whose granulometric descriptions are similar to the ones observed in the current research. From statistical analysis, the author obtained friction angles between 29° and 36° (average of 32°) and cohesive intercept between 13 and 29 kPa (average of 21 kPa) for residual soils. With respect to colluvium soils, the friction angle ranged between 25° and 31° (average of 28°) and the cohesion between 6 and 21 kPa (average of 14 kPa). Those results have high coefficient of variation, with values around 20% for the friction angle and of 95% for cohesive intercept.

Regarding the high degree of variation for parameters of peak friction angle and cohesive intercept, a similar trend was observed in the study presented by Mezzomo & Bertuol (2013). By means of retro-analysis made in soils (residual and colluvium) of several slopes along the Serra do Mar, those researchers obtained an average friction angle of 25° and average cohesive intercept of 10 kPa, with coefficient of variation of 12% and 72%, respectively.

Generally, when compared with data found in literature, the results obtained with both direct shear tests and CIU triaxial tests carried out in this research are coherent and similar considering the variability of the material found in the region (Table 7).

The comparison of the shear strength parameters obtained by two different methods (CIU triaxial tests and conventional direct shear tests) is very interesting because of the differences between the apparatus applied and the principles. In CIU triaxial tests the soil fails along planes of weakness, while in direct shear tests the soil rupture occurs in a pre-established plane. (Lambe & Whitman, 1969; Pinto, 2006; Das, 2007).

At Serra do Mar, where the occurrence of mass movements with shallow-type landslide is common, as typical of the geological-geotechnical context of the region, conventional direct shear tests to obtain shear strength parameters are simple, with low cost and fast execution time comparing with CIU triaxial tests. In addition, they are useful as well as representative of the field conditions: plane slip surface at low confining stresses.

Furthermore, in these slopes progressive failure frequently occurs and, therefore, residual shear parameters are relevant as entry parameters in landslide susceptibility analysis. One more time, direct shear tests with smooth in-

Table 7 - Summary of some results from shear strength parameters of Serra do Mar soils.

Authors	Source of results	Type of soil	Friction angle (ϕ')	Cohesive intercept (c')
Present research	Direct shear tests	Residual and Colluvium	18.6°-38.3°	0.0-14.8 kPa
	CIU triaxial tests	Residual and Colluvium	27.8°-43.2°	0.0-14.3 kPa
Bressani *et al.* (2001)	Shearing strength in residual terms	Brazilian tropical soils	25°	0.0 kPa
Suzuki (2004)	Direct shear tests	Residual and Colluvium	28.3°-33.0°	2.0-21.0 kPa
Tonus (2009)	CIU and CID triaxial tests, Direct shear and ring shear tests	Residual	29°-36°	13.0-29.0 kPa
		Colluvium	25°-31°	6.0-21.0 kPa
Carvalho (2012)	CIU triaxial tests for high confining stress	Residual	21.5°	105.8 kPa
Carvalho (2012)	CIU triaxial tests for low confining stress	Residual	27.7°	74.2 kPa
Mezzomo & Bertuol (2013)	Retro-analysis	Residual and Colluvium	25.0°	10.0 kPa
Advincula (2016)	Direct shear tests	Residual	15.3°-36.4°	7.0-36.5 kPa
Advincula (2016)	Direct shear tests	Colluvium	22.5°-37.4°	0.0-19.8 kPa
Advincula (2016)	CIU triaxial tests	Colluvium	29.5°-31.1°	6.4-11.8 kPa
Trevizolli (2018)	Direct shear tests	Residual and Colluvium	17.8°-30.2°	4.7-16.8 kPa

terface can be applied to obtain those parameters, as was done in this paper.

5. Conclusions

This paper examined geological-geotechnical characteristics of slopes belonging to a segment of BR- 376/PR, within Serra do Mar in Southern Brazil.

The classification of colluvium soils and superficial granite and migmatite residual soils were, in part, similar with regard to both characteristics and granulometry: predominantly silty sand, with mean value of particle density of 2.691 g/cm^3, plasticity index on average equal to 3.2% and natural moisture of 23%. The *in situ* hydraulic conductivity obtained was around 10^{-4} cm/s in the superficial layer of the soil, characterized as the potential failure surface.

Different from what can be found in the literature about residual soils present in the region of study, namely that migmatite residual soils and granite residual soils are clayey soils, this research observed great amounts of silt and sand and low amount of clay (in average, for all samples, 53% of silt, 8% of clay and 32% of sand). This difference among the described information may be the consequence of the variation of the composition that the lithotypes of this complex have and because these residual soils are younger. Granulometric curves had a similar trend among residual soils, while colluvial soils had a greater dispersion in the granulometric distribution, confirming the heterogeneity of the material when transported and the difference of material weathered *in situ*. Colluvium and residual areas were hard to limit, this being one of the main limitations of the research.

This finding is observed when comparing results from the characterization of those materials, made by means of granulometric analysis, Atterberg Limits and particle density, as well as from parameters of mechanical strength, obtained by means of conventional direct shear test and smooth interface tests, and of CIU triaxial tests, which were analogous.

Concerning parameters of mechanical strength, the residual and colluvial soils found in the studied area (a portion of Serra do Mar) showed average peak and residual friction angle of 32° and 26°, respectively. For cohesive intercept a greater disparity of results was obtained, but also according to literature.

Therefore, the soils found along the studied area, even if different with respect to the geological material defining their genesis, have similar physical characteristics as well as mechanical behavior. Such results, together with other relevant parameters (*e.g.*, slope declivities and layers, groundwater level, rainfall records, suction records and soil retention curve), may be used as input data for future studies, such as in the evaluation of the stability of road slopes, either in individual cases or in regional scale, in nearby areas or having a similar context of geologic material.

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