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ORIGINAL PAPER

Effects of Cyclic Variations of Pore Pressure on the Behaviour of a Gneiss Residual Soil

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Abstract Many slope failures in residual soils have been attributed to increased soil pore pressure. During the rainy season, water infiltration occurs, which can lead the slope to rupture by increasing the pore pressure. However, some landslides have been observed with pore pressures below those required for failure. Hypotheses for such ruptures include the occurrence of creep or fatigue due to cyclic variations in the piezometric level on the slope. This paper presents a study on the effect of pore pressure on the behaviour of the residual soil of the Soberbo hillside in Rio de Janeiro, Brazil, subjected to monotonic and cyclic pore pressure. Triaxial tests that simulate the increasing piezometric level (constant shear drained—CSD tests), drained creep (DC-CSD tests) and cyclic pore pressure variation (CPP tests) in slopes were carried out. The CSD test results showed that the failure due to increasing pore pressure is essentially dilatant and confirmed the peak strength obtained from conventional drained triaxial tests. Sudden failure was observed in all CSD tests at the end of the pore pressure increase phase. The results show that the failure of the slope may occur due to drained creep and the loss of soil shear strength under cycling pore pressure.

Keywords Pore pressure · Constant shear drained tests · Tropical soils · Creep · Rain-induced failure

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1 Introduction

Residual soils are commonly found in tropical regions since high temperatures and abundant rainfall create favourable conditions for chemical reactions that decompose rock, transforming it into soil. The combination of deep highly weathered soil, steep slopes and intense rainfall make tropical regions more prone to mass movements. Landslides are considered one of the most important environmental problems in hilly tropical areas, and most landslides are rain-induced.

During rainy periods, water infiltration may provoke a significant increase in soil pore water pressure.

Brand (1981) suggested that this phenomenon could be represented by triaxial tests and proposed that the field stress path of a soil element in the failure slope could be represented by an increase in pore pressure, keeping the total stress constant−constant shear drained test (CSD tests). This stress path may generically represent a slope stress path even if the initial condition corresponds to an unsaturated condition.

The results of triaxial tests with failure induced by increasing the pore pressure have been reported by different researchers. Brenner et al. (1985), Santos et al. (1996), Zhu and Anderson (1998), Tsukamoto (2002), Ng and Petley (2009); Junaideen et al. (2010) performed tests in residual soils. Test results in colluvial soils have been reported by Anderson and Riemer (1995), Anderson and Sitar (1995), Dai et al. (1999a and b). The slope behaviour by CSD tests in loess was studied by Xu et al. (2011, 2012) and Zhou et al. 2014 and in pyroclastic soils by Olivares and Damiano 2007. CSD tests have been used in sands to understand their behaviour related to drained instability (Anderson and Riemer 1995; Chu et al. 2003, 2012,2015; Lourenço et al. 2011; Dong et al. 2016).

The behaviour of residual soils in tests with constant deviator stress and an increase in pore pressure under drained conditions is relatively well understood. As the pore pressure increases, the mean stress p' decreases and the soil undergoes axial and volumetric strain. Initially, both deformations are small and increase when the stress state approaches the strength envelope. In most cases, the volumetric behaviour is dilative even when in conventional triaxial tests the behaviour is contractive. Contractive behaviour in the CSD stress path has been observed in very loose specimens.

However, the slope behaviour in residual soils subjected to creep and cyclic pore pressure variations has not been adequately studied. Lacerda (1989) reported that some landslides in residual soil slopes could not be explained by a simple increase in piezometric level, i.e., some failures occurred with pore pressure lower than that necessary for the stress path to reach the strength envelope. Lacerda (1989) hypothesized that this failure mechanism may be attributed to soil fatigue caused by cyclic variations in the soil pore water pressure. Demers et al. (1999) explained the failure in clay slopes in the Maskinongé area, Quebec, by using the hypothesis of loss of strength due to fatigue in the soil. Leroueil (2001) also discussed the effects of creep and fatigue in structured soils and its influence on slope instability triggering.

According to Lacerda (1989), the typical profiles of residual soils exhibit increases in strength and permeability with depth. However, a low-permeability soil layer is also commonly found due to the deposition of fine-grained soil particles or the precipitation of leached ions from the upper soil layers. This low-permeability layer is located at the base of the mature residual soil. Below that layer is the young residual soil, also called saprolitic soil. Figure 1 shows a schematic representation of a well-developed residual soil profile found in tropical areas (Lacerda, 1989). The influence of the variation in pore pressure affects the saprolitic layer that is permanently saturated.

The piezometric level varies throughout the year; during the rainy season, it is high (HPL), and during the dry season, it is low (LPL). Depending on drainage conditions, artesian groundwater may occur. Figure 2 shows the stress path during the seasonal variation of the piezometric level. During the dry season, the pore pressure is represented by Δu_{0}, and during the rainy season, the pore pressure may significantly increase to

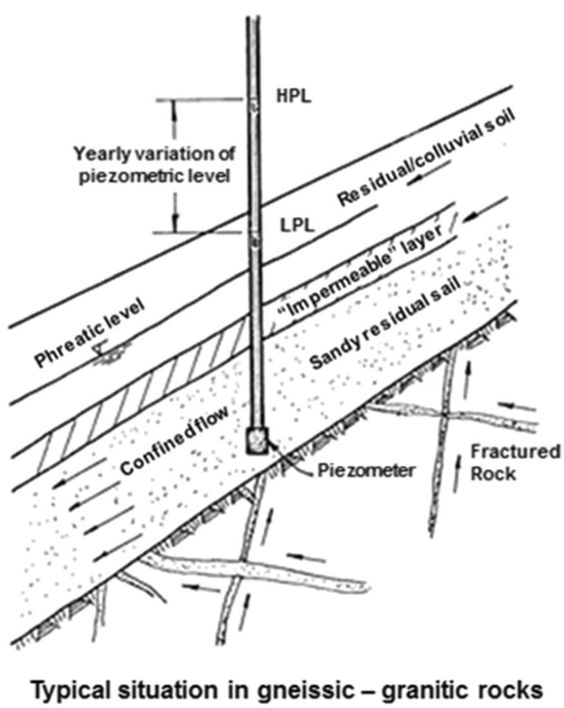

Fig. 1 Seasonal variation in piezometric levels in a typical profile of a well-developed residual soil (Lacerda 1989)

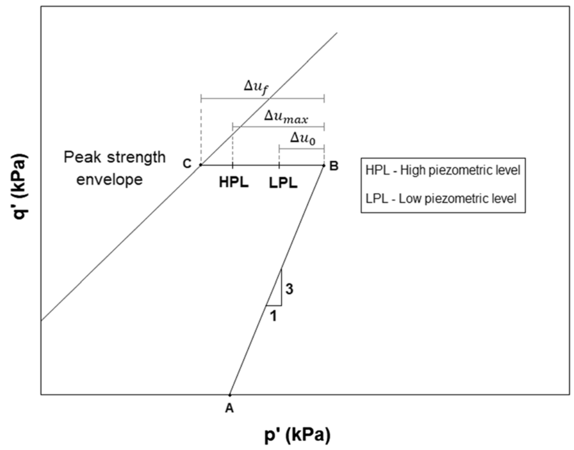

Fig. 2 Stress path in the soil specimen in a triaxial test under cyclic pore water pressure (Lacerda 1989)

Δu_{max}. Eventually, in the very rainy season, the state of stress may reach the strength envelope (Δu_f in Fig. 2), and slope failure would occur. Moreover, the cycling of the pore pressure between Δu_{max} and Δu_0 may lead to the creep failure (or fatigue) of the slope. Fatigue is associated with the variation in the effective stresses and the corresponding degradation of the peak shear strength of the soil to its residual shear strength. Creep is related to the strain of the soil under constant stress state conditions. According to Lacerda (1989), fluctuations in water pressure may decrease soil cohesion.

The aim of this paper is to evaluate the effect of different pore pressure scenarios on the stability of residual soil slopes. Triaxial tests were performed to i) evaluate the mechanism proposed by Brand (1981) in terms of the monotonic increase in soil pore pressure, in which failure is achieved by increasing pore pressure under constant drained shear; ii) evaluate the behaviour of the soil under constant stress state conditions below and near failure in a drained creep; and iii) verify the effect of cycling pore pressure on the strain and degradation of the shear strength of the soil, as proposed by Lacerda (1989).

Triaxial tests that simulate the field stress path were carried out: constant shear drained (CSD) tests, modified CSD tests with a drained creep phase (DC-CSD tests), and tests with cyclic pore pressure (CPP tests), as proposed by Lacerda (1989). The stress paths followed in the tests simulate those shown in Fig. 2. In the CSD tests, the stress path corresponds to the BC segment. In the DC-CSD tests the stress path follows from point B to the HPL point, and then this stress state is maintained over time. In the CPP tests, the stress state varies cyclically along the stress path represented by the LPL-HPL segment.

2 Experimental Programme and Testing Procedures

2.1 Soil Studied

Laboratory tests were performed on undisturbed block samples taken from the Soberbo hillside in Rio de Janeiro, Brazil. Soberbo soil is a residual soil of gneiss. Soberbo soil properties have been reported by Silveira (1993). The plasticity index of this soil varies from 5 to 14%, and the percentage of fines varies from 40 to 50%. According to the Unified Soil Classification System (USCS), this type of soil is classified as silty sand (SM). The average physical characteristics of the residual soils of Soberbo are shown in Table 1.

2.2 Performed Tests

Four types of tests were carried out: (i) conventional consolidated drained triaxial tests (CD); (ii) drained triaxial tests that led to failure by increased pore water pressure under constant shear stress (CSD), as suggested by Brand (1981); (iii) modified CSD, in which samples were subjected to drained creep periods (DC-CSD) before being led to failure by increased pore water pressure under constant shear stress; and (iv) cyclic pore water pressure tests (CPP), as proposed by Lacerda (1989).

The specimen dimensions were 5 cm in diameter and 10 cm in height. Specimens were saturated by back pressure in increments of 50 kPa. The soil

Table 1 Characteristics of residual soils of Soberbo

Property	Value
Water content (%)	31.4
Unit weight (kN/m^3)	17.2
Specific gravity of solids	2.85
Dry unit weight (kN/m^3)	13.1
Degree of saturation (%)	78
Void ratio	1.15

specimen was assumed to be fully saturated for values of Skempton's B parameter greater than 0.95. In the performed tests, an average value equal to 0.98 was obtained. Two to three days and back pressure up to 200 kPa were required for full saturation of the soil specimens.

After saturation, the soil specimens were subjected to isotropic consolidation. During this phase, the volume and height of the soil specimens were simultaneously monitored by electronic and mechanical devices. The t_{100} for isotropic consolidation varied from 2 to 4 min, indicating very fast pore pressure dissipation. Nevertheless, the next phase of testing, which led the soil specimens to failure, was initiated after 24 h of isotropic consolidation.

In all tests, filter paper strips were used to increase the rate of pore pressure dissipation inside the specimens. Corrections were made to account for the resistance of the membrane and filter paper strips in the determination of soil stresses. The CD test samples were led to failure at an axial strain rate of 0.06 mm/min. Axial displacement, volumetric variation and axial load were monitored until the end of the test.

The CSD test shearing was performed in two phases. The confining pressure was initially maintained constant, and the axial stress was raised in steps up to 50% of the deviator stress to failure verified in the CD tests [considering the same confining stress in both tests]. As shown in Fig. 2, the stress path followed in this phase is represented by path AB. The next phase consisted of an increase in pore water pressure in steps until soil specimen failure keeping the total stress constant (path BC). The number of increments and their duration was planned to reach failure in 4 h, approximately the same time to failure as observed in the CD tests. The minimum time of a given increment was 10 min, which was generally sufficient to stabilize the axial and volumetric strains, except for stress levels close to failure.

The creep tests were carried out using the same procedure described for the CSD tests. The first phase consisted of an isotropic consolidation, Point A in Fig. 2. Then, a deviator stress was applied (stress path AB). From point B, the pore pressure was increased to a given point before failure and then maintained under drained conditions at a constant stress state (DC-CSD). During the creep phase, soil volume variation and axial strain were monitored.

During cyclic pore pressure tests (CPP), the increase in pore pressure varied from the minimum (Δu_o) to maximum values (Δu_{max}), as shown in Fig. 2. Parameter U_c, as defined below, was proposed by Lacerda (1989) to normalize the cyclic pore pressure. The minimum and maximum values of pore pressure represent the low and high piezometric levels, respectively, as shown in Figs. 1 and 2. Each step of constant pore pressure was maintained for 10 min. The number of cycles and axial and volumetric deformations were recorded during the test.

$$U_c = \left[(\Delta u_{max} - \Delta u_0)/\left(\Delta u_f - \Delta u_0\right)\right] \times 100(\%)$$

in which Δu_f is the increase in pore pressure related to failure determined by the CSD test.

In tests in which failure was not achieved during the creep period [or in cyclic pore pressure tests] additional steps to increase the pore pressure were performed to lead soil specimens to failure by using the same procedure adopted in the CSD tests.

2.3 Test Programme

Conventional drained triaxial tests (CD) were conducted at confining pressures of 25, 50, 100, 200 and 600 kPa. These conventional tests were performed to obtain the strength envelope and the critical state parameters. Note that the landslides are usually shallow. However, this stress range was defined to provide a broad view of the soil behaviour under different conditions.

The test conditions of the performed CSD, DC-CSD and CPP tests are shown in Tables 2 and 3.

3 Test Results and Analyses

3.1 Conventional Consolidated Drained Tests (CD)

Figure 3 presents all the results for the CD tests. The behaviour obtained in the tests with confining pressures of 25 and 50 kPa was dilative. Tests with confining pressures over 100 kPa presented contractive behaviour. This behaviour is typical for overconsolidated soils. However, there is no geological evidence showing that this soil has been subjected to an overconsolidation process due to stress loading.

Table 2 Performed CSD, UC-CSD and DC- CSD tests

Test identification	σ_c' (kPa)	σ_d (kPa)	$\sigma_d/\sigma_{d(f)}$	Δu(kPa)	$\Delta u/\Delta u_f$
CSD—50-01	50	100.98	0.61		
CSD—100-01	100	147.00	0.58		–
CSD—200-01	200	243.50	0.53		–
CSD—600-01	600	586.30	0.51		–
DC- CSD—50-01	50	101.38	0.61	34	0.89
DC- CSD—100-01	100	152.05	0.60	55	0.85
DC- CSD—100-02	100	152.00	0.60	60	0.92
DC- CSD—100-03	100	147.15	0.58	52.5	0.81

Note: σ_c', effective confining stress; σ_d, deviator stress in tests with monotonic increases in pore pressure; $\sigma_{d(f)}$, deviator stress to failure determined in the CD tests; Δu, increment in pore pressure to the onset of creep phase; Δu_f, increment in pore pressure at failure

Table 3 Performed CPP tests

Test identification	σ_c' (kPa)	Δu_0 (kPa)	$\Delta u_{max.}$ (kPa)	Δu_f (kPa)	U_c (%)
CPP-50–01	50	10	30	40	67
CPP-50–02	50	10	40	40	100
CPP-50–03	50	10	34	38	86
CPP-50–04	50	10	32	36.5	83

Note: Δu_0, Δu_{max}, and Δu_f are defined in Fig. 2

This type of behaviour should be attributed to structural effects inherited from the gneissic rock that produced the residual soil.

Figure 3c illustrates the mobilized shear strength, represented by obliquity (q'/p'), observed during the tests. Tests under low confining stress showed a peak followed by a decrease in obliquity, while the tests under high confining stresses exhibited increased obliquity until the end of the test. In Fig. 3c, it was possible to observe that the performed tests did not reach critical state conditions. In tests under lower confining stress, obliquity was still declining by the end, whereas it was increasing in tests in which high confining stress was used. It was possible to estimate obliquity (q'/p') in the ultimate test condition as being between 1.20 and 1.46, which means a friction angle from 30° to 36°. The average value may be taken as a representative value of the friction angle at a critical state (ϕ_{cv} = 33°).

In Fig. 4, the stress path in the p':q' and v:p' planes is presented. Figure 4a shows that the stress path crosses the critical state line in tests under low confining stress levels. In plane v:p', it was possible to find the critical state line from the final test values (Fig. 4b). Figure 5 shows the critical state line and peak strength envelope obtained in the tests under low confining stress (CD 25, CD 50 and CD 100). The corresponding cohesion and friction angle determined under low stress are 18 kPa and 29.7°, respectively.

3.2 Tests with Failure Induced by Increasing Pore Pressure (CSD)

The results for the CSD tests are shown in Figs. 6 and 7. All tests were performed in two phases. In the first phase of testing, there was an increase in the deviator stress under constant confining stress, while in the second phase, the pore pressure was raised under constant deviator stress. In Fig. 6, the stress paths observed in the phase of pore pressure increase in the CSD tests are presented.

Figure 6a shows the peak strength envelope and the critical state line obtained in the CD tests. Good agreement can be observed among the peak strength envelope and the critical state line and the CSD test results. Tests under high confining stress failed close to the critical state line, and those under low confining stress failed close to the peak strength envelope.

Figure 6b shows the test path and critical state line in the v:p' plane. The results of the CSD tests appear to

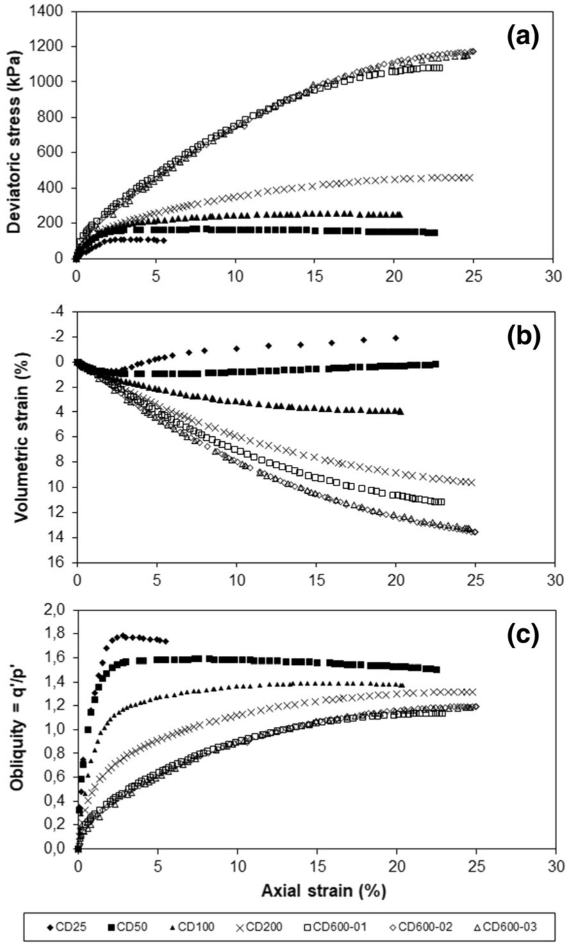

Fig. 3 Results for the CD test

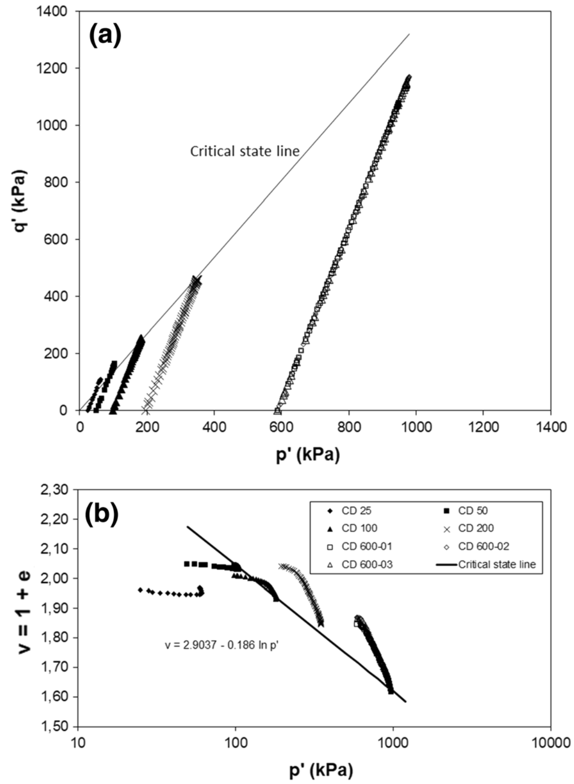

Fig. 4 Results for the CD tests: **a** stress path, peak strength and critical state line in the p′:q′ plane and **b** path and critical state line in the v:p′ plane

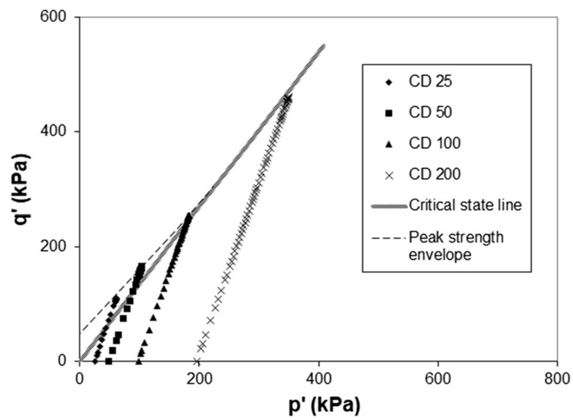

Fig. 5 Stress path, peak shear strength envelope and critical state line (CD tests)

agree with the critical state line determined by the CD tests. During the phase of pore pressure increase, in all performed tests, an increase in volume and axial strain was observed, as shown in Fig. 6b and c, respectively. The axial strain increases slowly until near failure. A sudden rise in the strain rate was observed close to failure. Dilative behaviour was obtained in all CSD tests during the increasing pore pressure phase. Dilation was greater in samples under low confining stress.

3.3 Drained Creep Tests (DC-CSD)

The results for the drained creep tests are shown in Figs. 7 and 8. Figure 7 shows the stress state at the beginning of the creep period and its positions in relation to the soil failure envelope and critical state line. The axial and volumetric deformations that occurred during the creep phase are shown in Fig. 8. As indicated in Fig. 7, among all performed tests with confining stress of 100 kPa, the DC-CSD 100-02 test is the closest to failure with a $\Delta u/\Delta u_f$ ratio of 0.92

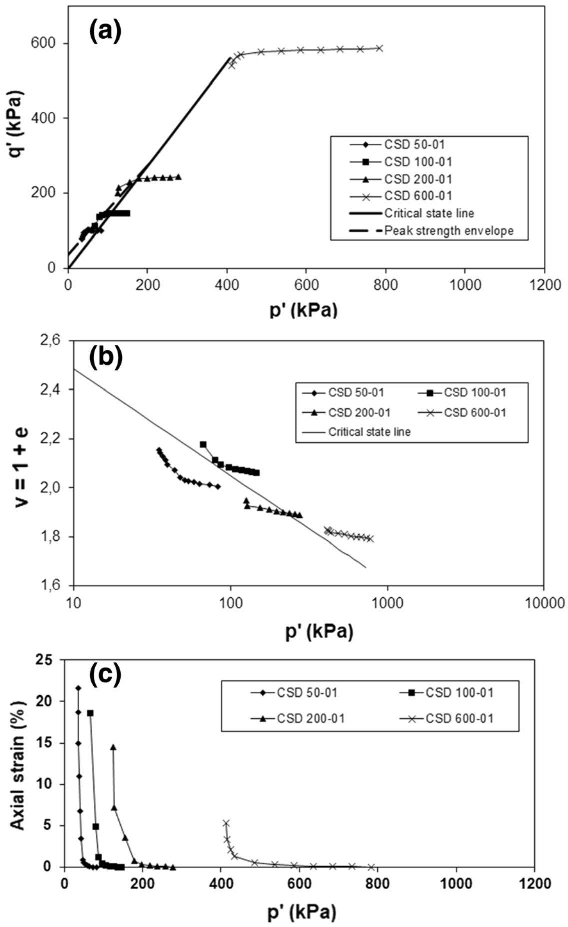

Fig. 6 Results for the CSD tests: **a** stress path, peak strength envelope and critical state line; **b** test path and critical state line in the v:p′ space; and **c** axial strain observed in the CSD tests

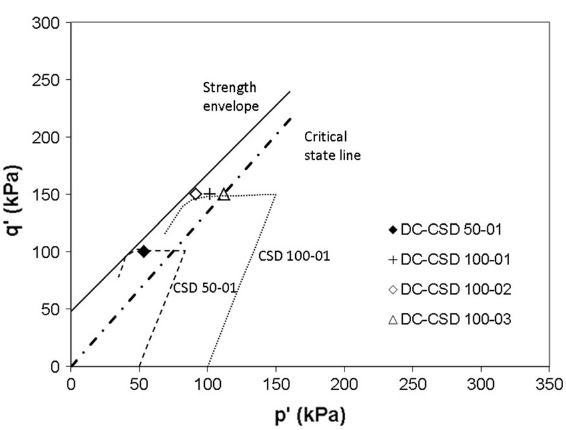

Fig. 7 Stress state in drained creep (DC-CSD)

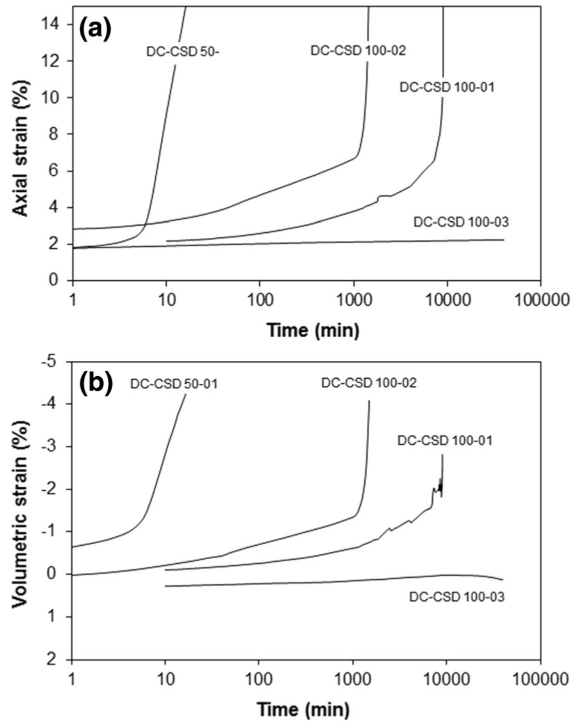

Fig. 8 Results for the DC-CSD tests: **a** axial strain vs. time in the drained creep tests (DC-CSD) and **b** volumetric strain vs. time in the drained creep tests (DC-CSD)

(Table 2), followed by the DC-CSD 100-01 test with $\Delta u/\Delta u_f$ of 0.85 and the DC-CSD 100-03 test with $\Delta u/\Delta u_f$ of 0.81. Note in Fig. 8 that the time to failure of the DC-CSD 100-02 test was lower than the value observed in the DC-CSD 100-01 test, while the DC-CSD 100-03 test did not fail during the period of testing. In Fig. 8, a dilatant behaviour was observed during the drained creep period. These results demonstrate that the tested residual soil may loose strength under drained creep. Critical boundaries to the start of soil strength reduction during a drained creep period may be related to the soil critical state line.

The DC-CSD 50-01 test with confining stress of 50 kPa, even with $\Delta u/\Delta u_f$ of 0.89, exhibited an increase in dilatancy compared to the results of tests with confining stress of 100 kPa and with short time periods to failure under drained creep (Fig. 8). Lower confining stress and a high void ratio would lead the soil specimens to be more susceptible to soil resistance reduction during the creep period of tests. Higher volumetric dilative strain was indeed observed in the CSD tests under low confining stress (CSD 50-01 and CSD 100-01 in Fig. 6b).

The results for the undrained creep tests in the CSD stress path presented by Santos et al. (1996) demonstrated that for the Soberbo residual soil during the undrained creep period, the soil stress path moves away from the soil failure envelope. The results for the drained creep tests shown in Figs. 7 and 8 demonstrate that failure occurs under drained conditions. The test results also show that during a drained creep period, the soil shear strength in addition to the corresponding ultimate condition (critical state condition), may vanish, while the time to failure would decrease accordingly with the increase in the stress level.

3.4 Tests with Cyclic Pore Pressure (CPP)

The stress path of the CPP tests together with the peak strength envelope and the critical state line are shown in Fig. 9. The U_c parameter values for the CCP 50-01 and CPP 50-02 tests were 67% and 100%, respectively. Two other tests with the confining stress of 50 kPa were carried out by using a U_c parameter of 86% (CPP 50-03 test) and 83% (CPP 50-04 test). In all tests, the u_0 values were established to maintain the stress state on the critical state line. Thus, in all the tests, the cyclic pore pressure varied between the critical state line and the peak shear strength envelope of the soil.

In Fig. 10, for tests with an isotropic confining stress equal to 50 kPa, the results of axial and volumetric strain versus the number of cycles are shown. As expected, the CPP 50-02 specimen failed at the first cycle of testing since the higher value of pore pressure was the one needed for failure determined in the CSD test (U_c = 100%). The results show that the number of cycles for failure increases as the U_c parameter decreases (Fig. 11).

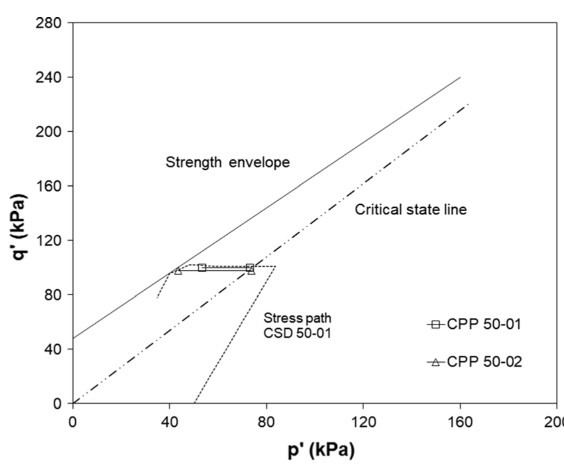

Fig. 9 Pore pressure variation in the CPP tests

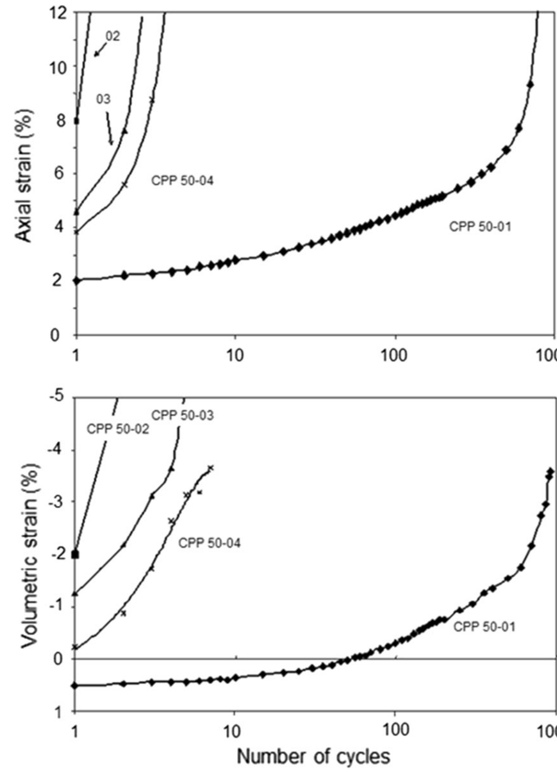

Fig. 10 Axial and volumetric strain vs. number of cycles (CPP tests at a confining stress of 50 kPa)

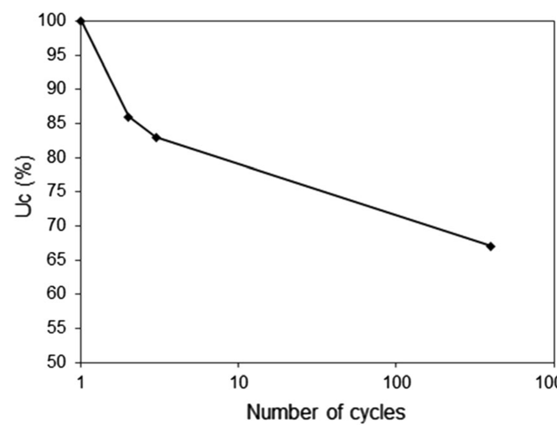

Fig. 11 Number of cycles to failure as a function of the Uc parameter

During each cycle of pore water pressure, specimens are led to an increase in the axial and volumetric strain. All deformations were partially recovered at the end of each cycle. Nevertheless, an increase in the plastic strain was observed after each cycle, and the accumulated strain led the soil specimens to fail. These results are compatible with those observed in the drained creep tests. Both cyclic and creep tests show that the soil shear strength in addition to the critical state line may be lost by the accumulated strain. Similar results were obtained by Harley et al. (2016) in tests with cyclic pore pressure performed on overconsolidated till. They showed that cohesion is lost if the stress state is maintained or cycled beyond the critical state line.

3.5 Results Discussion

The residual soil in the present study shows higher peak shear strength than ultimate strength and dilative behaviour under low confining stress. At high confining stress, the maximum and ultimate deviator stresses in the CD tests were the same, and the soil showed compressive behaviour. Since there is no geological evidence of past overconsolidation in this soil, the observed peak strength may be attributed to the structural effects inherited from the original rock. In fact, Silveira (1993) showed that this residual soil retains the same microstructure as the original rock with strong interlocking between quartz and feldspar grains and mica booklets.

The CSD tests confirmed the peak shear strength of the soil, as determined by the CD tests. The increasing in pore pressure led to a sudden failure in all the tests. Independent of the stress level, the constant shear drained triaxial tests showed dilative behaviour. Nevertheless, tests with low confining stress exhibited greater dilation than those with high confining stress.

The dilative behaviour detected in the CSD triaxial tests was also observed in undisturbed (Anderson and Riemer 1995; and Zhu and Anderson 1998) and compacted residual and colluvial soils (Dai et al. 1999a, b; Tsukamoto 2002 and Junaideen et al. 2010) as well as dense sands (e.g., Chu et al., 2015). Contractive behaviour in the CSD stress path was observed in very loose reconstituted samples (Anderson and Riemer 1995; Tsukamoto 2002, Ng et al. 2004; Junaideen et al. 2010; Chu et al. 2012) and undisturbed loess sediments (Xu et al. 2011, 2012).

Creep tests (DS-CSD) showed that the volume increases and specimens may fail during the creep phase after rising pore pressure with opened drainage. In these tests, time to failure decreased as the state of stress approached the failure envelope (peak stress condition). The lower limit from which failure occurs seems to be related to the critical state line. This means that drained creep may lower the shear strength of the soil beyond the critical state line. Moreover, the failure mechanism is related to soil dilation. Drained creep tests performed at the same state stress level show that the lower the confining stress was, the higher the dilation and the faster the failure of the soil specimens.

Time dependent behaviour in CSD triaxial tests conducted in in undisturbed residual soil was also reported by Ng and Petley (2009). Harley et al. (2016) showed a reduction of the shear strength in the creep test carried out in overconsolidated till. According to Harley et al. (2016), this reduction is related to the stress state conditions between the critical state line and shear strength envelope.

Cyclic pore pressure (CPP) generally exhibited the same pattern as that observed in drained creep. The decrease in shear strength occurs for pore pressure cycling conditions corresponding to the stress state of the soil between the critical state line and peak strength envelope. Due to high dilation at low confining stress, a decrease in confining stress promotes a reduction in the number of cycles to failure. The mechanism of the reduction in soil shear strength due to the cyclic pore pressure tests is related to the accumulated strain that occurs after each cycle. During the cycling pore pressure, stress state conditions near the shear strength envelope imply increased axial and volumetric (dilative) strain, as observed in the drained creep tests. There is a decrease in the strain rate of the soil as the state of stress moves towards to the minimum value of the pore pressure. However, accumulated axial and volumetric strain are maintained. Further cycles increase the accumulated strains, and the process may lead the soil specimens to rupture. The failure is triggered when the accumulation of plastic strain reaches a critical value. The accumulated strain during creep or cyclic tests may reach the value of the strain at failure in conventional triaxial test. At this strain, the strain rate increases until failure.

Figure 12 shows a conceptual model for the rain-induced triggering mechanism in residual soil slopes.

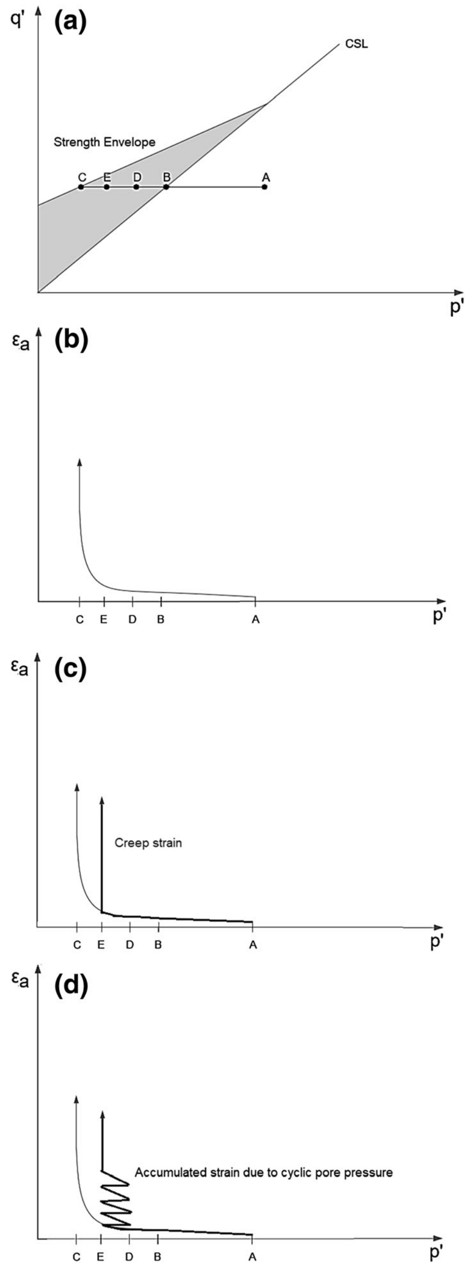

Fig. 12 Conceptual model of the rain-induced slope failure: **a** critical state line, shear strength envelope and possible stress state conditions during increase in the piezometric level; **b** soil strain during monotonic increase in pore pressure; **c** soil strain during the creep phase; and **d** accumulation of soil strain during cyclic variation in the piezometric level

Point A in Fig. 12a represents the initial effective stress state of a soil element inside the slope. Raising the piezometric level moves the stress state horizontally along the ABDEC path. Failure occurs at point C. The shear strains during the rise of the piezometric level are shown in Fig. 12b. According to this model, strains increase slightly with the elevation of the piezometric level, until yield occurs, and then increase rapidly until failure.

The proposed conceptual model considers that if the stress state remains constant for a long period of time within the shaded area in Fig. 12a, between the critical state line and the peak strength envelope, the soil undergoes creep strain which can may cause the slope to fail. Figure 12c illustrates the strains that occur during the creep on the slope. Initially, strain behavior is identical to that shown in raising of the piezometric level up to the stress corresponding to point E. From that point stress remains constant, but strains continue to occur until failure. The failure time is associated with the position of the stress state in the shaded area. The closer to the peak strength envelope, the shorter the failure time. If the stress state is below the critical state line, no creep failure occurs. Loss of strength due to creep only occurs if the stress state is in the shaded area of Fig. 12a.

The model considers that the situation in which there are cyclical variations of piezometric level is similar to the creep. For example, if the stress state in the soil varies cyclically between points D and E, shown in Fig. 12a, strains would occur as depicted in Fig. 12d. The cyclic variation of pore pressure represents the seasonal variations of the piezometric level on slopes. In dry seasons the piezometric level is low, which is represented by point D, but rises during the rainy season. Thus, the stress state moves to point E. Under these conditions, the soil undergoes strains. When the piezometric level drops, strain rate decreases, however, prior strain is maintained. A new cycle of increased pore pressure causes a resumption of strains. The continuity of stress cycles between points D and E results in the accumulation of strains, which may cause slope failure, as shown in Fig. 12d.

4 Summary and Conclusions

The tests performed in the Soberbo residual soil show higher peak shear strength than the critical state strength and dilative behaviour under low confining stress. At high confining stress, the maximum and critical state deviator stresses were the same, and the soil showed compressive behaviour. As there is no geologic evidence of past overconsolidation in this soil, the observed peak strength may be attributed to structural effects inherited from the origin rock. Sudden failure was observed in all constant shear drained tests at the end of the pore pressure increase phase.

Failure was observed during the drained creep tests, with essentially dilatant behaviour. The time for failure in the drained creep tests decreased with the increase in stress level. The lower limit at which failure occurs may be related to the critical state line. Soil shear strength beyond the critical state envelope may disappear during a drained creep. The failure mechanism may also be related to dilatancy. In the drained creep tests at low confining stress, a higher dilatancy is exhibited, and failure occurred faster than in samples submitted to high confining stress.

Cyclic pore pressure tests showed a similar pattern to that observed in the drained creep tests. The cyclic pore pressure tests demonstrated that the soil shear strength between the critical state line and peak strength envelope may disappear if the increase in maximum pore water pressure varied during this interval. At low confining stress, higher dilatancy and fewer cycles were necessary for a reduction in soil shear strength. The lower soil shear strength due to cyclic pore pressure tests was related to strain accumulation. During cycling, high pore pressure approached the state of stress in the strength envelope, which led to an increase in axial and volumetric (dilative) strain, as observed in the drained creep tests. At each new pore pressure cycle, residual deformations accumulated, causing the soil specimen to fail.

The tests carried out show some aspects related to slope instability in residual soil. During the rainy season, a classical failure of a slope may occur due to the increase in the groundwater level that leads to stress state conditions corresponding to the peak shear strength envelope. Moreover, failure promoted by creep or cycling pore pressure may also occur at stress levels below the peak strength envelope. Failures resulting from cyclic pore pressure and creep are interrelated processes. During the rainy seasons, the high pore pressure approximates the soil stress state near failure. During those periods, strain along the failure surface occurs as a drained creep. The soil strains are not recovered, and in further periods, strain accumulation may lead the slope to failure.

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References

Anderson SA, Riemer MF (1995) Collapse of saturated soil due to reduction in confinement. J Geotech Eng. https://doi.org/10.1061/(ASCE)0733-9410(1995)121:2(216)

Anderson SA, Sitar N (1995) Analysis of rain-induced debris flows. J Geotech Eng. https://doi.org/10.1061/(ASCE)0733-9410(1995)121:7(544)

Brand EW (1981) Some thoughts on rainfall induced slope failures. In: Proceedings of 10th international conference on soil mechanics and foundation engineering, Stockholm, Sweden, 15–19 June 1981. A.A. Balkema, Rotterdam, The Netherlands. pp 373–376

Brenner RP, Tam HK, Brand EW (1985) Field stress path simulation of rain-induced failure. In: Proceedings of 11th international conference on soil mechanics and foundation engineering, vol 2. CRC Press, Boca Raton, pp 991–996

Chu J, Leroueil S, Leong WK (2003) Unstable behaviour of sand and its implication for slope instability. Can Geotech J. https://doi.org/10.1139/T03-039

Chu J, Leong WK, Loke WL, Wanatowski D (2012) Instability of Loose Sand under Drained Conditions. J Geotech Geoenviron Eng. https://doi.org/10.1061/(ASCE)GT.1943-5606.0000574

Chu J, Wanatowski D, Leong WK, Loke WL, He J (2015) Instability of dilative sand. Geotech Res. https://doi.org/10.1680/gr.14.00015

Dai F, Lee CF, Wang S (1999a) Analysis of rainstorm-induced slide-debris flows on natural terrain of Lantau Island, Hong Kong. Eng Geol 51:279–290

Dai F, Lee CF, Wang S, Feng Y (1999b) Stress–strain behaviour of a loosely compacted volcanic-derived soil and its significance to rainfall-induced fill slope failures. Eng Geol 53:359–370

Demers D, Leroueil S, d'Astous J (1999) Investigation of a landslide in Maskinongé Québec. Can Geotech J. https://doi.org/10.1139/t99069

Harley R, Sivakumar V, Hughes D, Donohue S (2016) The strength reduction of overconsolidated till under static and dynamic pore water pressure conditions. Geotech Lett. https://doi.org/10.1680/jgele.15.00156

Junaideen SM, Tham LG, Law KT, Dai FC, Lee CF (2010) Behaviour of recompacted residual soils in a constant shear stress path. Can Geotech J. https://doi.org/10.1139/T09-129

Leroueil S (2001) Natural slopes and cuts: movement and failure mechanisms. Geotechnique. https://doi.org/10.1680/geot.2001.51.3.197

Lacerda, WA (1989) Fatigue of residual soils due to cyclic pore pressure variation. In: Proceedings of 12th international conference on soil mechanics and foundation engineering, vol X, pp 3085–3087

Lourenço SDN, Wang G, Chu J (2011) Aspects of sand behaviour by modified constant shear drained tests. Environ Earth Sci. https://doi.org/10.1007/s12665-010-0573-8

Ng KY, Petley DN (2009) The use of pore pressure reinflation testing in landslide management in Hong Kong. Q J Eng Geol Hydrogeol. https://doi.org/10.1144/1470-9236/08-008

Ng CWW, Fung WT, Cheuk CY, Zhang L (2004) Influence of stress ratio and stress path on behavior of loose decomposed granite. J Geotech Geoenviron Eng. https://doi.org/10.1061/(ASCE)1090-0241(2004)130:1(36)

Olivares L, Damiano E (2007) Postfailure mechanics of landslides: laboratory investigation of flowslides in pyroclastic soils. J Geotech Geoenviron Eng. https://doi.org/10.1061/(ASCE)1090-0241(2007)133:1(51)

Santos OF Jr, Lacerda WA, Ehrlich M (1996) Discussion of collapse of saturated soil due to reduction in confinement. J Geotech Eng. https://doi.org/10.1061/(ASCE)0733-9410(1996)122:6(505)

Silveira GC (1993) Geotechnical properties of the residual and colluvial soils from Soberbo Road landslide. M.Sc. Dissertation, Federal University of Rio de Janeiro (in Portuguese)

Tsukamoto Y (2002) Discussion of determination of shear strength of Hawaiian residual soil subjected to rainfall-induced landslides. Geotechnique 52(2):143–144

Xu L, Dai FC, Tham LG, Tu X, Jin Y (2011) Landslides in the transitional slopes between a loess platform and river terrace, Northwest China. Environ Eng Geosci XVII(3):267–279

Xu L, Dai FC, Gong QM, Tham LG, Min H (2012) Irrigation-induced loess flow failure in Heifangtai Platform North-West China. Environ Earth Sci. https://doi.org/10.1007/s12665-011-0950-y

Zhu JH, Anderson SA (1998) Determination of shear strength of Hawaiian residual soil subjected to rainfall-induced landslides. Geotechnique. https://doi.org/10.1680/geot.1998.48.1.73

Zhou YF, Tham LG, Yan WM, Dai FC, Xu L (2014) Laboratory study on soil behavior in loess slope subjected to infiltration. Eng Geol. https://doi.org/10.1016/j.enggeo.2014.09.010