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A SLOPE INSTABILITY CASE HISTORY INVOLVING SWELLING CLAY IN SOUTHERN BRAZIL

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ABSTRACT

This paper describes a slope instability case history involving swelling clay. A 9-m high slope was cut in 3 sedimentary layers during the construction of a single-carriage road. A complex slope failure mechanism was identified during site investigation, consisting of: (a) a progressive surficial degradation, particularly of the lower swelling clay layer, (b) a deep-scated slope failure, and (c) the toppling failure of the upper stronger layers. Site investigation included SPT testing, undisturbed sampling and *in situ* suction measurement. Laboratory testing consisted of: (a) soil characterization by X-ray diffraction analysis, particle-size analysis and Atterberg limits tests; (b) evaluation of effective shear strength parameters using direct shear tests and ring shear tests; and (c) determination of soil-water characteristic curves. Slope stability analyses were carried out, followed by comparison with observed field performance.

KEYWORDS

slope stability, swelling clay, site investigation, laboratory testing, case history

INTRODUCTION

In 1993, during the construction of RS-736, a single-carriage road in the center of Rio Grande do Sul state, 2 cuts were made through 3 sedimentary layers near the town of São Jerônimo. The first cut was 340-m long and the height was about 9 m. The second cut was about 180-m long and 8-m high. During construction, the cutting operation was difficult, suggesting high *in situ* shear strength. As a result, relatively steep slopes (about 40°) were built. Some time after road opening both cuts showed increasing signs of slope instability. The second cut failed completely about one year after construction. Figure 1 outlines the cross-section of both slopes. The change of geometry of the second slope - which was monitored over a 4-year period - is also shown in Fig. 1.

According to both the geological interpretation and the geotechnical site investigation, 3 different slope failure mechanisms seemed to be superimposed on both cuts. The first failure mechanism was the progressive degradation of the slope, particularly of the lower swelling clay layer. The degradation process started with the breakage of the exposed soil in small lumps (about 10 mm to 20 mm long) followed by sliding of shallow wedges (roughly 0.5 m thick). The main field evidence of this degradation was the frequent

accumulation of rain-washed debris near the slope base. Blocked surface drains was the main consequence of debris accumulation. Once each wedge had moved, undisturbed soil was exposed, and the degradation process was renewed. As a consequence, the slope geometry near the base slowly changed until a second, more significant failure mechanism was triggered. This consisted of a larger, deep-scated failure with low shear strength mobilized along the slip surface. The third failure mechanism was the subsequent toppling failure of the upper stronger layers. The whole process was retrogressive, becoming active mainly after heavy rainfall and also after debris accumulated on the road were cleared (see Fig. 1).

The São Jerônimo slope failures were investigated in two stages. The first stage was carried out soon after the main failure happened; both site investigation and laboratory testing were then concentrated on the first cut. Results were presented by Leipnitz (1995). The second stage was carried out recently; it consisted of further *in situ* tests and laboratory tests, concentrating mainly on the second cut. *In situ* suction measurements, multiple reversal direct shear tests, and ring shear tests were performed during this stage. Corresponding results are presented in this paper.

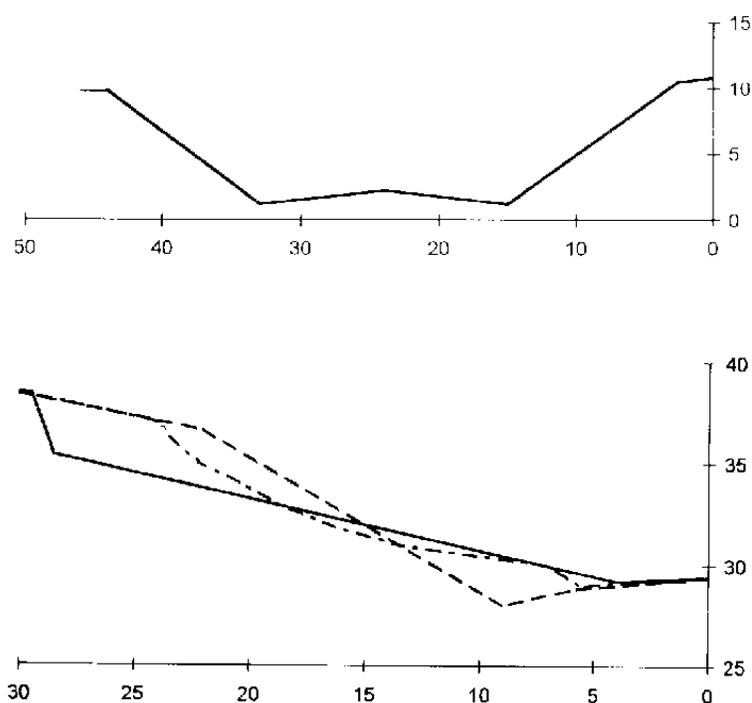


Fig. 1 São Jerônimo slope failure: cross-sections of both cuts.

GEOLOGICAL INTERPRETATION

The local geology is part of the Paraná Basin which covers most of Southern Brazil. The characteristic feature of this basin is a 400-m to 1.200-m thick sequence of volcanic flows, mainly basalt and rhyolite. The Southern boundary of this sequence is situated about 20 miles north of São Jerônimo. A package of sedimentary rocks is situated below the volcanic rocks. This package consists of many layers of weak sandstones, siltstones, claystones, and shales. Some of these rocks contain swelling clay minerals. At the São Jerônimo site, three main soil layers have been identified. The lower layer is a light grey stiff clay with some amount of pink clay. A clear field swelling behaviour is shown by this layer: X-ray diffraction analysis revealed the presence of smectite. The middle layer is a reddish-grey hard silty clay with pockets of a light grey clay; it is composed chiefly by kaolinite, although the light grey pockets showed the presence of some smectite. The top layer is a reddish-brown sandy silt; it is comparatively thin at the failed slope site. This layer is not considered to be part of the main sedimentary strata: it is instead a recent out-wash deposition.

SITE INVESTIGATION

Site investigation consisted mainly of SPT testing (on both cuts), in situ suction measurements (on the second cut), and basic soil tests (on both cuts). For the first cut, SPT values varied between 5 and 7 in the top layer, between 10 and 24

in the middle layer, and between 10 and 15 in the lower layer. For the second cut, $13 < N(\text{SPT}) < 24$ was measured in the middle layer, falling to $N(\text{SPT}) \approx 12$ in the lower layer.

Representative samples were collected from these soils, and basic soil tests were performed in the laboratory (moisture content, Atterberg limits, soil unit weight). Table 1(a) outlines main soil data for the first cut. It should be noticed that layers 1 and 2 are unsaturated. On the other hand, the degree of saturation of layer 3 is near 100%. This is consistent with field observations. Table 1(b) shows corresponding data for the second cut.

Table 1. Basic soil data

(a) first cut

soil	ω_L (%)	ω_P (%)	ω (%)	e	S_r (%)	% <2 μm
layer 1	39	21	15.2	0.76	53	45
layer 2	45	22	17.2	0.64	74	28
layer 3	71	26	39.4	1.00	98	59

(b) second cut

soil	ω_L (%)	ω_P (%)	ω (%)	e	S_r (%)	% <2 μm
layer 2	36	26	16	0.74	62	38
layer 3	83	33	24	0.89	64	71

It is clear that layer 3 deserves closer attention because it shows high plasticity and a large clay content. Notice that layer 3 is similar for both cuts, but a difference in composition is observed for the case of layer 2.

Except for layer 3, both cuts are largely unsaturated. In order to evaluate the magnitude of suction, and its possible influence on slope behaviour, in situ suction measurements were carried out on the second cut. The Imperial College suction probe (Ridley and Burland, 1993; Ridley and Burland, 1995) was used for this purpose. This instrument has the remarkable capability of measuring very high suctions (at least 800 kPa) without cavitation. Measurement time is also relatively short (about $\frac{1}{2}$ hour for reaching equilibrium). For the measurements carried out, 2 boreholes were excavated about 2 meters away from the crest of the second failed cut, using a 70-mm diameter 'dutch' hand auger. When the depth of 2 m below the slope crest was reached (i.e. in the middle of layer 2) each borehole was cleaned out using a bucket auger and lined using a PVC tube with a 50-mm internal diameter. A special 35-mm

diameter cutting tool (described by Ridley and Burland, 1995) was afterwards used to excavate a small extension to the bottom of the initial borehole. The suction probe was inserted in the PVC tube and then carefully lowered into contact with the soil. In order to achieve good contact between the probe ceramic disc and the soil, this disc was previously coated with a kaolin paste. This procedure caused a small amount of disturbance to the local suction, and this took about ½ hour to approach the in situ value (Fig. 2). Equilibrium suctions were low, with values between 10 kPa and 14 kPa. These measurements were made after the 1996 end-of-winter heavy rainfalls. At that time, the failure process was particularly active in the second cut.

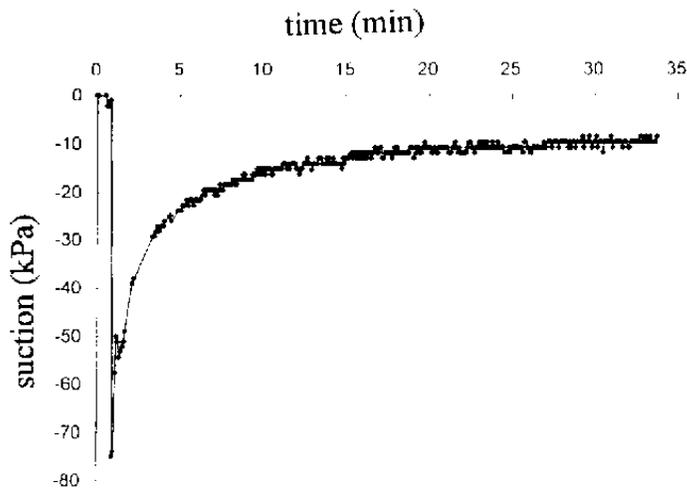


Fig. 2 Layer 2: measurement of suction.

LABORATORY SHEAR STRENGTH TESTING

Two series of laboratory shear strength tests were carried out using samples taken from the São Jerônimo site. The first series, consisting mainly of direct shear tests, was performed some time after failure, using undisturbed samples taken from the first cut. All specimens were tested under a soaked condition, with the rate of 0.3 mm/h (drained). Normal stresses applied in these direct shear tests varied between 31 kPa and 205 kPa. Table 2(a) shows corresponding peak and critical state effective shear strength parameters. It should be noticed that only one testing stage was used, without reversal. Therefore, no attempt was made of determining residual shear strength in these direct shear tests.

The second series of laboratory tests was carried out using undisturbed samples taken from the second cut (for layers 2 and 3). This series included the evaluation of residual shear strength for both layers. Direct shear tests were performed according to the multiple reversal technique suggested by Skempton (1964). At least 3 shearing reversals were applied to each specimen, as shown by Fig. 3. It can be noticed that layer 3 has a very clear post-peak decrease in shear strength, up to a total displacement of about 25 mm. For layer 2, there

is a smaller post-peak decrease in shear strength, stabilizing after a displacement of about 10 mm.

Table 2. Shear strength parameters.

(a) first cut.

Soil	γ (kN/m ³)	c'_{peak} (kPa)	ϕ'_{peak} (°)	c'_{cv} (kPa)	ϕ'_{cv} (°)
layer 1	17.4	3.0	34	-	-
layer 2	19.7	34.0	38	0.0	37.5
layer 3	18.4	32.5	15	0.0	27.5

(b) second cut.

Soil	γ (kN/m ³)	c'_{peak} (kPa)	ϕ'_{peak} (°)	c'_{cv} (kPa)	ϕ'_{cv} (°)
layer 2	19.4	24.3	40.0	0	35.5
layer 3	17.7	18.9	29.4	0	17.2

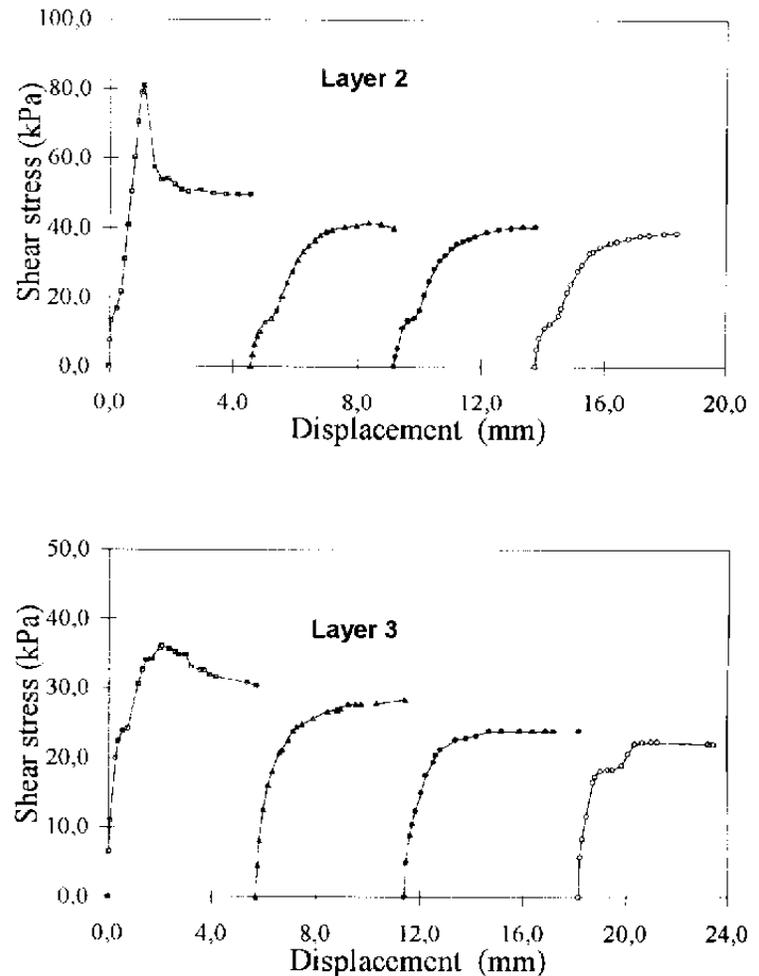


Fig. 3 Shear stress x displacement curves obtained with multiple reversal direct shear tests (for $\sigma'_n = 50$ kPa).

Further tests were carried out using Bromhead's ring shear apparatus (Bromhead, 1979). Results of these tests are shown in Fig. 4. For layer 2, little loss of strength was observed even after a total displacement of 250 mm. On the other hand, layer 3 showed a significant reduction of shear strength for large displacements, characterizing the mobilization of residual shear strength. The residual angle of internal friction of this layer was about 14° (as evaluated by direct shear tests) or 11° (as evaluated by ring shear tests).

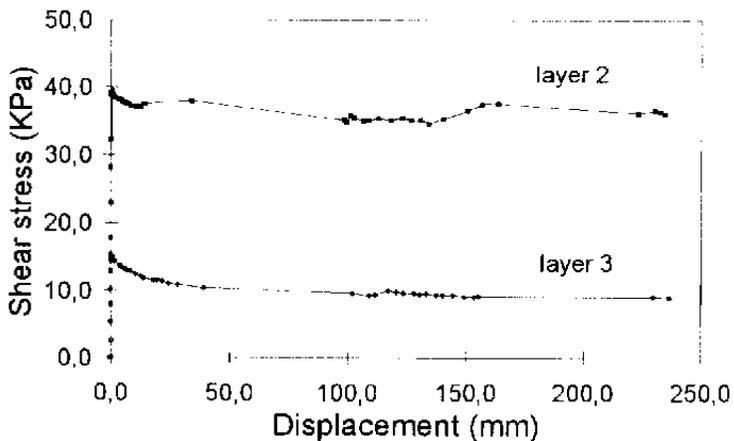


Fig. 4 Shear stress \times displacement curves obtained with ring shear tests (for $\sigma'_{vn} = 50$ kPa).

SLOPE STABILITY ANALYSIS

Slope stability analyses were carried out using Bishop's routine method with c' and ϕ' given in Table 2. These analyses tried to reproduce the first-time deep-seated failures. The analyses assumed the end-of-construction slope geometry for both cuts. The value of suction present at the layer 2 was considered to be zero, an assumption which seems to be consistent with the critical condition of heavy rains (bearing in mind the low suctions measured in situ). For the second cut, the water level was assumed to be at the contact between the middle and lower layers, for consistency with results of laboratory tests and field observations. When peak shear strength parameters were used, the value of factor of safety (F) computed for both cuts was about 2.0. Therefore the observed failures could not be explained by the analyses considering peak shear strength parameters.

On the other hand, when shear strength parameters corresponding to the critical state condition were used, F was 1.03 for the first cut, and 1.02 for the second. Such good agreement with observed first time failures indicates that some shear strength degradation has actually occurred. The soil probably lost part of its shear strength due to deformations associated to cycles of wetting/drying near the surface, and swelling upon unloading. This is consistent with the swelling behavior shown by the smectite-rich layer

3. After failure, the mobilized shear strength tends to drop even further, as shown by the large-displacement direct shear and ring shear data. The result in the field is a very soft and unstable debris mass overlying layer 3, for the second cut. It is likely that residual shear strength has been mobilized at the contact between these debris and layer 3. This brings maintenance difficulties: any attempt to clear away debris accumulated over the road surface triggers further soil movements.

CONCLUSIONS

The investigation of a failed slope cut through 3 sedimentary layers was described in this paper. Effective shear strength parameters were determined from direct shear tests and ring shear tests. The smectite-rich layer 3 shows a drop in shear strength for large displacements, as evaluated by multiple reversals direct shear tests and ring shear tests. A smaller post-peak shear strength reduction was shown by layer 2. Although the soil in the top layers was mainly unsaturated, in situ suction measurements showed that the value of suction was very low after end-of-winter heavy rainfalls. Slope stability analyses were therefore carried out assuming zero suction. When first-time failures were analysed using peak shear strength parameters, the values of factor of safety were about 2. Factors of safety near to 1 were obtained for first-time deep-seated failures only when critical state shear strength parameters were assumed. This implies that some shear strength degradation has occurred before failure took place; field observation is consistent with this assumption.

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