Embankment Failure on Clay near Rio de Janeiro

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Abstract: The construction on an instrumented trial embankment on soft clay is described. Field measurements of pore-pressure and vertical and horizontal displacements were carried out and the results are discussed. Foundation strength was evaluated through vane tests, UU triaxial tests and SHANSEP method. Embankment failure occurred when its height was about 3 m. Stability analysis were carried out to evaluate methods of assessing soft foundation strength.

Introduction

An extensive research program on the behavior of embankments founded on soft soils, sponsored by the Brazilian Highway Research Institute (IPR), has been jointly conducted by COPPE-UFRJ, at the Federal University of Rio de Janeiro. It includes field and laboratory investigations, theoretical analyses, and large-scale field trial embankments on a very soft, dark grey deposit near the city of Rio de Janeiro.

For such experiments, a testing site was selected, located at about 7 km from the city of Rio de Janeiro, as a typical example of a region where roads, industrial plants, and urban developments are under construction.

The first trial consisted of an instrumented embankment built on the clay deposit and taken to failure in December 1977. Some of the results are described in this paper.

Site and Soil

The testing site is situated in a very flat swampy area, locally known as "Fluminense Plains," which covers a surface area of about 150 km^2 around Guanabara Bay (Fig. 1). At the site, the clay deposit is about 11 m thick and overlies sand and gravel layers.

The interest in the geotechnical properties of this clay is not recent. Early studies were carried out by Silva (24) and, more recently, additional investigations have been carried out under the sponsorship of IPR (4,5,11,23,31).

As may be seen from Fig. 2, the liquid limit varies from 120%–160%, near the ground surface, to 90%–100% at the bottom of the deposit; the in situ water content being slightly higher than these values. This is generally an indication of high sensitivity in a clay. However, the sensitivity of this deposit, as determined from field vane tests by Collet (4), is low, ranging from 2–4, with an average of 2.6. The plastic limit de-

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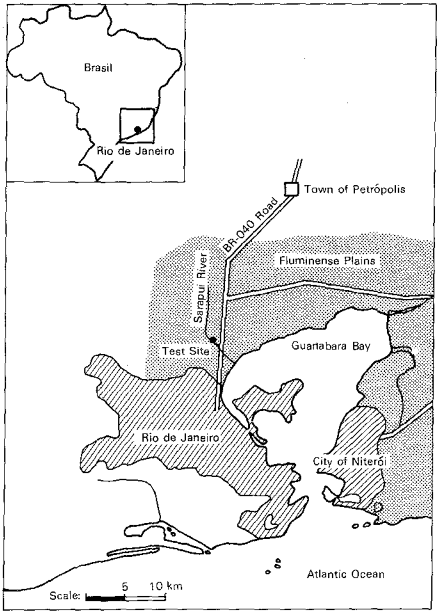

FIG. 1.—Test Site Location

creases from 60%–80% near the top, to 50%–60% at the bottom. Saturated unit weight has a mean value of 13.2 kN/m^3. X-ray diffractometry and differential thermal analysis (DTA) have shown that kaolinite is the predominant mineral, but small amounts of illite and montmorillonite are also present. Organic matter content is about 5% and this is responsible for the dark grey color of the clay.

Stress history was evaluated from 63 high quality odometer tests, carried out in consolidation rings of 80 cm^2 cross-sectional area (110.9 mm diam), on samples obtained with a 125 mm diam piston sampler. In the early phases of these tests, small load increments were applied to the specimens, until a sharp bend was observed on the strain-log, $\bar{\sigma}_v$, curve, allowing a more accurate determination of the overconsolidation or max-

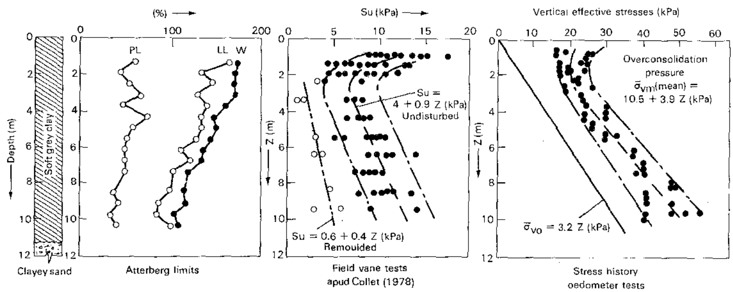

FIG. 2.—Summary of Geotechnical Properties, Rio de Janeiro Soft Gray Clay

imum past pressure, $\bar{\sigma}_{vm}$. The results indicate the presence of an upper clay crust, extending down to a depth of 2.5 m, below which average $\bar{\sigma}_{vm}$ values seem to vary according to

$$(\bar{\sigma}_{vm})_{\text{mean}} = 10.5 + 3.9\,z \quad \text{(1)}$$

in which z = depth in meters; and $\bar{\sigma}_{vm}$ in kPa. As the water table is close to the soil surface, it oscillates with the seasons between +0.2 m and −0.5 m with respect to ground elevation. Observations during the period of the construction of the embankment have shown that the water level was very close to the ground surface and, thus, the effective vertical stress is given by the expression $\bar{\sigma}_{vo} = 3.2\,z$ in which $\bar{\sigma}_{vo}$ is in kPa. From these results, the overconsolidation ratio (OCR) can be evaluated. At the upper clay crust, OCR values seem to be very high, and below the crust it seems to decrease with depth according to

$$\text{OCR} = \frac{3.28}{z} + 1.22 \quad \text{(2)}$$

These results are in accordance with previous data published by Lacerda, et al. (11), and Costa-Filho, et al. (5).

Undrained Shear Strength

Undrained shear strength (Su) was evaluated from field and laboratory tests. Laboratory triaxial UU tests were performed on specimens of different sizes, obtained from samplers of various diameters, and the results are tabulated in Table 1. The SHANSEP approach (12) was employed in order to evaluate Su from undrained triaxial compression tests on clay specimens consolidated both under anisotropic ($\overline{\text{CKoU}}$-C) and isotropic ($\overline{\text{CIU}}$-C) conditions. The results of these tests are presented in Fig. 3, in which Su/$\bar{\sigma}_{vc}$ is the normalized undrained strength, i.e., undrained strength divided by vertical effective consolidation stress. Thus, using stress history data from odometer tests, Su in situ was subsequently deduced by applying the procedure recommended in Ref. 12.

A comparison between mean Su values obtained from different testing methods is presented in Fig. 4, also included is Su obtained through the

TABLE 1.—Summary of UU Triaxial Tests Results

Sample diameter, in millimeters (1)	Specimen diameter, in millimeters (2)	Number of tests (3)	Linear Regression Results: S_u, in kilopascals z, in meters (4)	Coefficient of correlation (5)	Standard error of estimate, in kilopascals (6)
50	50	15	4 + 0.059z	0.06	2
63	50	22	3.5 + 0.24z	0.23	3
63	36	30	4.6 + 0.43z	0.31	3
125[a]	38	21	3 + 1.2z	0.74	2.5
125	100	8	4.1 + 0.52z	0.83	0.9

[a]Tested by Costa-Filho, et al. (5).

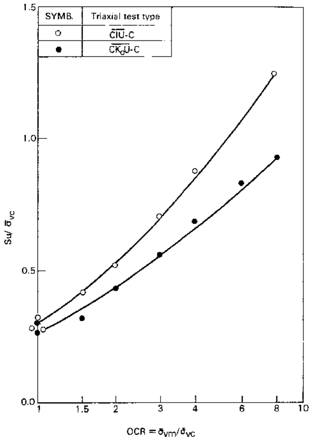

FIG. 3.—Normalized Undrained Strength versus OCR—Triaxial Test Results

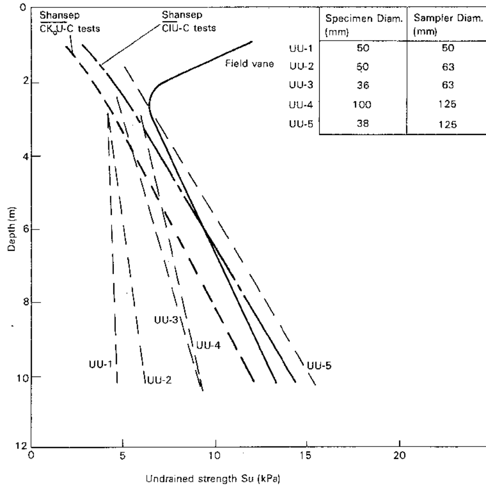

FIG. 4.—Comparison between Mean Values of Undrained Strength from UU Tests, Field Vane, and SHANSEP

relationship Su = 0.22 $\bar{\sigma}_{vm}$ suggested by Mesri (19) and also Trak, et al. (28). This figure shows the difficulty involved in laboratory tests—and this also includes the SHANSEP method—for evaluating the increase in strength at the upper clay crust, which was detected by field vane tests. Samples of the upper clay crust were few and showed a fair amount of vegetable matter (roots, etc.) and voids, which made specimen trimming somewhat difficult. Below the crust, there is a general trend of agreement of mean Su values obtained from field vane tests, SHANSEP, and high-quality UU tests, in which excessive sample disturbance was avoided.

Field vane tests indicate the presence of an upper clay crust, extending to a depth of 2.5 m, along which the Su values seem to decrease with depth (Fig. 2). Below this depth, field vane data indicate increasing Su values.

Embankment Design

Plane strain conditions are usually required in a trial embankment in order to duplicate the most common situation in road embankments on soft soils. A very long trial embankment, however, is not practical, and adequate dimensions should be established for the prototype. Another characteristic of a trial embankment is that the instrumentation is generally located in one main section. The success of the experiment depends on failure taking place at such a section. Considering these points, the trial embankment was constructed, as shown in Figs. 5–6, with the following characteristics: a steeper slope, $1(v){:}2(h)$; a stable slope, $1(v){:}6(h)$; a base length of 80 m; and a base width of 40 m. In order to avoid the possibility of failure occurring outside the instrumented zone, it was decided to confine laterally this main section by means of two triangular

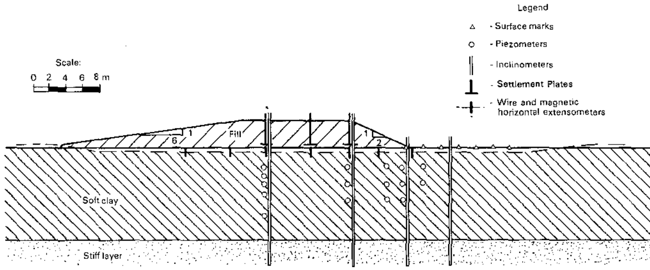

FIG. 5.—Main Section—Instrumentation Scheme

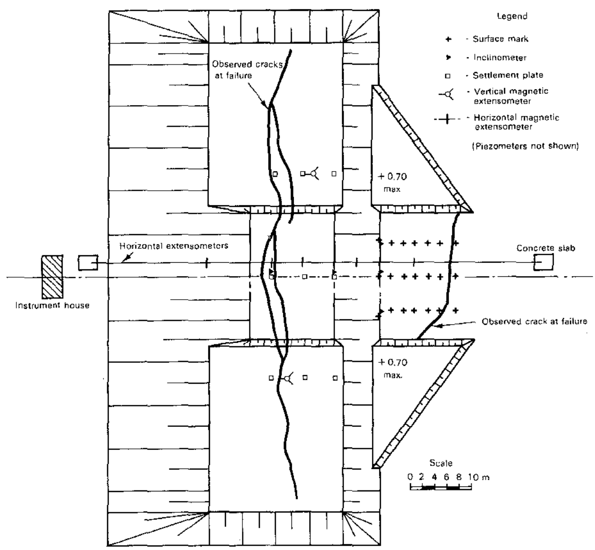

FIG. 6.—Instrumentation Plan

berms of 0.70 m in height (Fig. 6), 20 m apart from each other. In addition, the embankment height at the main section was raised in advance of the berms, always keeping a difference in height of 0.50 m.

An embankment with such geometry certainly departs from the desired plane strain conditions, but this disadvantage was, at the time, preferred to the risk of a failure occurring outside the instrumented zone. The three-dimensional effect caused by the adopted geometry is subsequently analyzed in this text.

INSTRUMENTATION

The layout of the instrumentation of the trial embankment is shown in Figs. 5–6. It included piezometers, magnetic and horizontal wire extensometers, inclinometer casings, settlement plates, and surface marks.

The piezometers were of the hydraulic type (Fig. 7), consisting of a

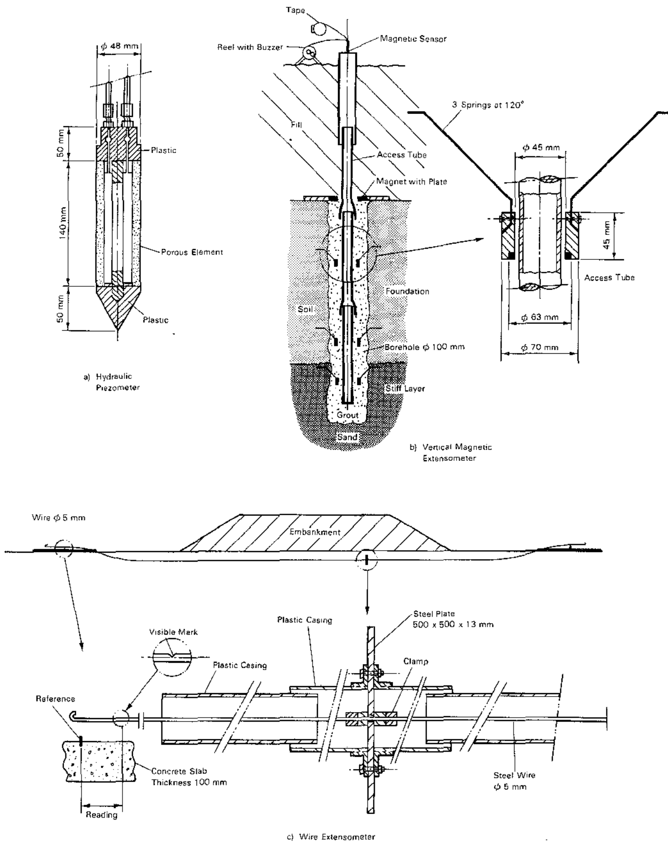

FIG. 7.—Instrument Details

porous high air entry ceramic tip, 48 mm in diam and 140 mm long, connected by twin polyethylene coated nylon leads to mercury manometers, located in an instrument house.

Out of the 21 piezometers of this type, 10 were installed in pairs under the axis of the embankment, and the remaining 11 were located under or near the steeper slope of the embankment. The inclinometer casings (six in all) were surveyed by a SINCO Digitilt Inclinometer.

For monitoring horizontal displacements at the base of the embankment, two independent systems were adopted. The first was a simple mechanical device called a wire extensometer, details of which are shown in Fig. 7. Readings were taken by measuring the distance of a visible mark on the wire to a reference embedded in the concrete slab, with the wire stretched by a 50 N force. The movement of each plate could be monitored from both ends of the wire, providing a check for the displacement measured.

The second system, positioned at the same locations as the wire extensometer targets, consisted of a horizontal magnetic extensometer which employs the same principle as that originally developed by Burland, et al. (2). Aluminium plates 400 mm long and 250 mm wide, containing a magnetic ring, were laid down along the base of the embankment. A 38 mm diam plastic access tube was passed through the magnetic rings and extended to reach both concrete slabs outside the embankment. Through this access tube, an electric sensor attached to a measuring tape was introduced, and readings could be obtained. Ground horizontal displacements in front of the embankment's steeper slope were observed through surface marks located as shown in Fig. 6.

As seen in Figs. 5–6, many instruments were located in clusters, permitting duplication in the observation of the same measurement, thus providing independent checking of the readings and the instrumentation performance.

Embankment settlements were observed by 21 surface marks, 9 settlement plates, and 2 vertical magnetic extensometers as shown in Figs. 6–7. Only three settlement plates were located at the main instrumented section. The others were laterally positioned outside of the failure zone aiming at observing settlements at embankment lateral sections which should remain stable even after failure.

Embankment Construction and Failure

Before construction, the instrumentation was observed during a long period until the initial performance of the instruments was considered to be satisfactory and zero readings were defined. Fill material was a silty-sand residual soil. Initially, a 0.50 m thick layer was placed on the area, except at the main instrumented zone where the initial layer was 1 m thick in order to minimize the possibility of damage to the instrumentation. To avoid destruction, small and light bulldozers were employed for spreading and compaction, and trucks were not allowed to cross the instrumented zone.

The embankment was then lifted in layers of 30–40 cm thick. After placement of each layer a set of readings was made. Fill unit weight was determined by in situ tests and averaged 18 kN/m^3, with a standard deviation of 0.35 kN/m^3.

The first indication that failure was being approached occurred when the embankment height was 2.5 m. A crack, about 1 cm wide, opened along the embankment crest. Until then the instruments had not shown any signs of imminent failure. On the following day, the embankment height was lifted to 2.8 m. As a consequence, severe cracking on the embankment crest occurred and sudden increases were observed in the readings of some instruments. Despite these failure signs, a clear slip had not yet occurred and bulging of the clay surface in front of the steeper slope was not clearly evident. Such characteristics, if observed in an ordinary engineering structure, would certainly cause construction stoppage due to risk of a failure. However, considering the research nature of this trial embankment, it was decided to continue fill placement until a clear slipping occurred. The embankment height was raised to 3.1 m. Again, a marked change was observed in most instrument readings, followed by an increase in embankment cracking, the width of which reached values of about 10 cm. In addition, the edges of the cracks now showed a difference in level of about 10 cm, clearly indicating the embankment failure. Bulging of the clay surface in front of the steeper slope was then evident, but there was no visible crack present at the clay surface in front of the embankment. Embankment behavior was observed continuously throughout the afternoon and overnight, but a general slip did not take place.

On the following morning it was decided to accelerate embankment deformation rate by placing another layer. As soon as placement was resumed, a general slip slowly occurred and the embankment height could not be increased. This occurred on December 7th, 1977, *exactly a month after the beginning of the fill placement*. Final geometry was then surveyed and final cracks are shown on Fig. 6.

RESULTS AND PERFORMANCE OF INSTRUMENTATION

Vertical displacement profiles under the base of the embankment are shown in Fig. 8. At failure, the maximum observed settlements were in the order of 300 mm, while ground heave near the toe of the embankment was about 50 mm. When the embankment height was increased from 2.5–2.8, a sudden increase in heave observations was recorded. These readings show a 2–3-fold sudden variation near failure. Another noticeable point in Fig. 8 is the high gradient of vertical displacements which occurred under the toe of the steeper stope of the embankment.

Settlement monitoring at lateral instrumented sections were made through the settlement plates and vertical magnetic extensometers, aiming at observing post-failure settlements. However, when the embankment failed, severe cracking spread along the lateral sections (Fig. 6), causing a sudden variation of the readings. Thus, lateral settlement mea-

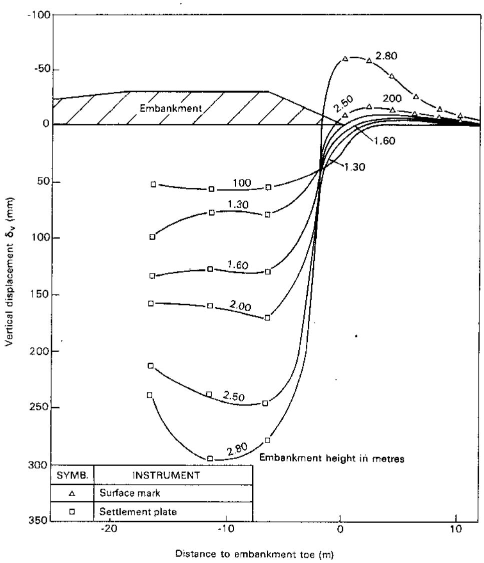

FIG. 8.—Vertical Displacement Profiles under Base of Embankment

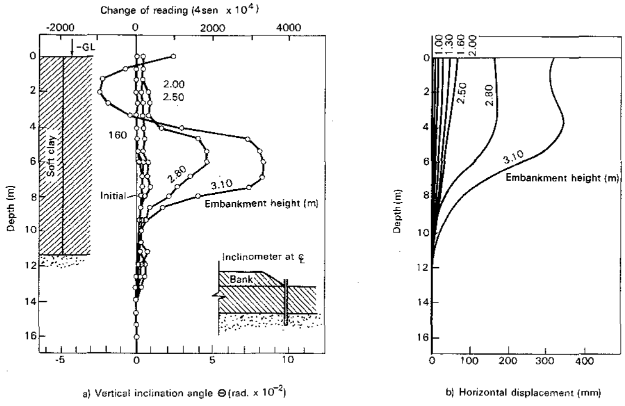

FIG. 9.—Inclinometer Observations

surements have only been used for evaluating instrument performance. The overall accuracy of the vertical magnetic extensometer was of the order of ± 2 mm.

Inclinometer observations at one casing positioned at the toe are shown in Fig. 9. Fig. 9(*a*) presents the variation of angle θ, i.e., the observed vertical inclination with depth. The horizontal displacements are shown in Fig. 9(*b*). Observations have shown that transverse horizontal displacements were negligible. The ratio of longitudinal (along the embankment center line) to transverse displacements was negligible: less than 5% in the main section and less than 10% in the two inclinometers off the main section.

Horizontal displacements at the toe are summarized in Fig. 10, and Fig. 11 presents typical results of horizontal displacement measurements along the embankment base. At the end of fill placement maximum toe horizontal displacements are in the range of 300–400 mm. Maximum values, as seen in Fig. 11, seem to occur under the steeper embankment slope, rather than at the toe. The zone of influence of ground displacements does not extend beyond 10 m from the embankment toe.

The horizontal magnetic extensometer presented a good performance until the embankment was 3 m high. At this point, due to embankment deformation, the access plastic casing was obstructed, preventing further readings. Despite these problems, the use of a magnetic extensometer proved satisfactory. Accuracy was handicapped by tubing settlements, but when compared with the results of other instruments, overall accuracy was estimated to be better than ± 5 mm.

Measurements taken by the mechanical wire extensometer were too much affected by wire settlements under the embankments and, to a lesser extent, by wire dilation due to temperature fluctuations. A tentative correction of the first effect has not yielded good results, and measurements were considered only as an upper limit of horizontal displacements. However, due to the reliability presented by such a mechanical and simple instrument, the overall pattern of displacements presented by the measurements could be used for analyzing the distri-

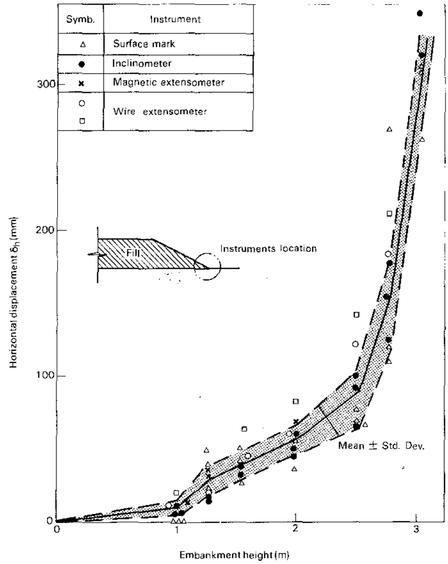

FIG. 10.—Observed Horizontal Displacements at Embankment Toe

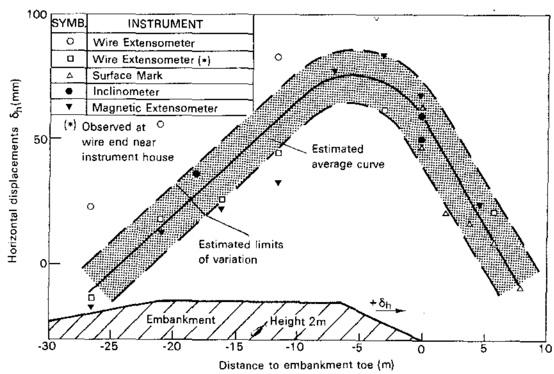

FIG. 11.—Horizontal Displacements at Base of Embankment at Height of 2 m

bution of displacements along the embankment base. Its manufacture and installation was difficult and expensive, in contrast to the magnetic extensometer.

A comparison of horizontal displacements at the top of the inclinometer casings with those of the nearest surface marks indicate that the results are in the same range. The accuracy of the Digitilt inclinometer observations, as quoted by the manufacturers, is about ±2 mm at the top of a 15 m long casing. Field experience confirmed these values. Accuracy of horizontal displacement observations on surface marks was found to be approximately ±3 mm.

The use of hydraulic piezometers in the embankment foundation was successful. Local manufacturing and assembling of the piezometer systems, field installation, and the monitoring of piezometer response, gave the research team control of all phases of the work. This was an important asset for the field data evaluation and correction of some hydraulic system malfunctioning. Observations were made during one month before embankment construction. During this period, piezometer leads were subjected to suction due to the difference in piezometric level between the phreatic level and gage house, making de-airing necessary. Scatter in initial observations permitted evaluation of the initial accuracy of readings to be assessed as approximately ±0.2 m head of water. However, during pore-pressure buildup due to loading, the mercury manometer readings became less erratic and the accuracy may have improved.

A summary of pore-pressure increment observations during embankment construction is plotted in Figs. 12–13. The piezometer response to

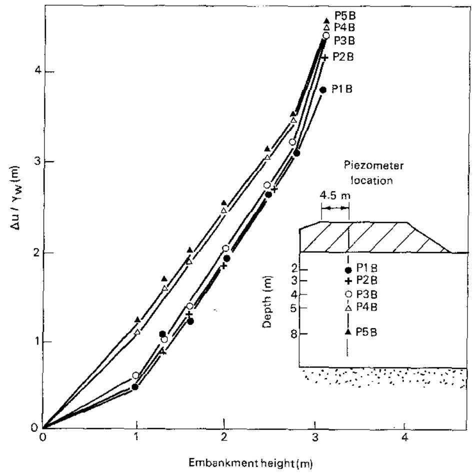

FIG. 12.—Construction Pore Pressures

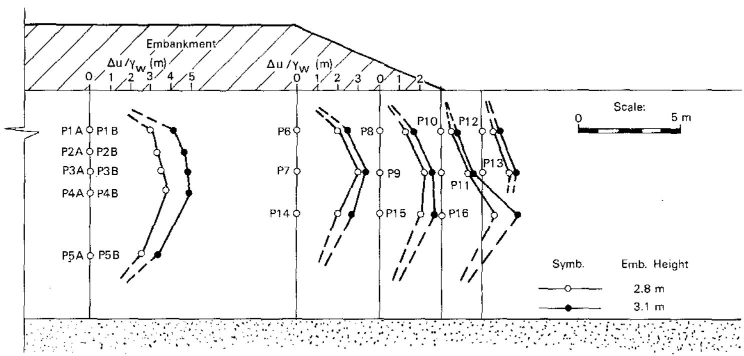

FIG. 13.—Observed Pore Pressures at Failure

load, as shown in Fig. 12, presents an initial flatter slope for the porous tips located closer to the top of the clay layer, followed by a steeper slope. This typical behavior has been analyzed by Lerouiel, et al. (13), who attributed the change in the rate of pore-pressure buildup to the fact of the clay foundation passing from overconsolidation into a normally consolidated state. At failure, as local yield occurs close to the porous tip, pore pressures seem to present a further increase in the rate of pore-pressure generation.

Stability Control During Construction

Field control of the stability of embankments on soft foundations is frequently a means of avoiding the risk of an undesirable failure, and also enables construction on a more rational and economical basis. Field observations of pore pressures, displacements, and strains may be used for that purpose. Examples of use of pore-pressure measurements are quoted, for instance, in Refs. 3, 6, 10, 16, and 25. Some of these authors have employed observed pore pressures in effective stress stability analyses, whereas others have suggested the use of control charts based on effective-stress analyses. Both procedures imply an a priori knowledge of the effective strength parameters, which sometimes may be difficult to evaluate correctly.

The use of inclinometer observations at the embankment toe as an indication of an imminent failure condition is quoted, for instance, in Refs. 10 and 30, among others. Marche and Chapuis (15) suggested a control method based on horizontal deformations close to the embankment toe. This method, however, presents a limitation: the undrained deformation modulus, E_u, of the foundation is assumed to be known, although frequently it is difficult to evaluate E_u correctly. Another approach, based on the possibility of defining a safe limit for the inclinometer observations at the embankment toe, was suggested by Vaughan (29). Matsuo and Kawamura (17) suggested the use of lateral deformations and settlements in a construction control diagram. Recent studies by Tavenas, et al. (27), however, demonstrated that the development of large lateral displacements is related to the passage of the clay foundation from an overconsolidated to a normally consolidated state, i.e., a phenomenon not directly related to the factor of safety of the foundation.

Field observations at the construction of the trial embankment on Rio de Janeiro soft grey clay have allowed evaluation of available methods of stability control. Experience during embankment construction have demonstrated that, among all instrument observations, the inclinometer readings were the most affected by the imminent failure. As can be seen from Fig. 9, a considerable change of reading was observed just before the embankment had failed. This information was subsequently used for suggesting a method for construction control on Rio de Janeiro clay. A plot of inclinometer observations against relative embankment height is shown in Fig. 14. Maximum values of vertical inclination, rather than horizontal displacements, were employed, since these measurements can be directly related to shear strains. In fact, the shear strain, γ_{vh}, is a sum of the vertical inclination, θ, plus the horizontal inclination, β, at the same point. The θ values under the embankment toe were directly mea-

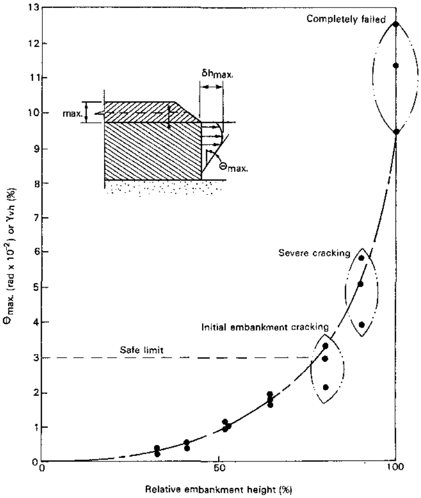

FIG. 14.—Stability Control Chart through Inclinometer Observations at Embankment Toe

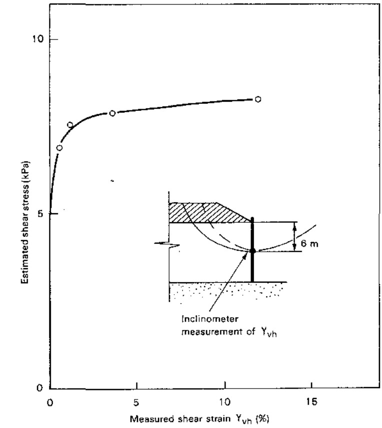

FIG. 15.—Deduced Stress-Strain Curve from Measured Strains and Estimated Stresses

sured by the inclinometer, and β values were not measured. However, at a point located near the horizontal part of a failure surface (8), β may be very small. Thus, the shear strain, γ_{vh}, could be considered to be equal to θ_{max}, i.e., the maximum observed vertical inclination measured at the embankment toe.

Data in Fig. 14 was obtained from three inclinometer casings. Correlating these observations with the observed embankment behavior, a safe limit of γ_{vh} or θ_{max} equal to 3% could be defined. Beyond this safe value, severe embankment cracking was observed in the trial embankment.

Stress-Strain Curve Deduced from Field Observations

An interesting way of exploiting strain observations is the possibility of deducing a "field" stress-strain curve. The observations of γ_{vh} at the embankment foundation were plotted against estimated shear stresses from a finite element analysis (22) yielding a stress-strain curve, as presented in Fig. 15. This information was used as input data for another FE analysis in order to simulate embankment deformation. The results of such an analysis, due to space limitations, will not be presented in this paper.

Stability Analysis

Stability analysis of the trial embankment was carried out in terms of both total and effective stresses, employing Bishop's modified circular arc analysis.

The analyses were made through a very flexible program, named BISPO, written at COPPE-Federal University of Rio de Janeiro by Moraes, Jr.

TABLE 2.—Hypotheses on Undrained Strength of Clay Foundation

Hypotheses on foundation strength (1)		Equation $S_u = f(z)$ (S_u in Kilopascals, z in Meters): At crust ($z < 2.5$ m) (2)	Equation $S_u = f(z)$ (S_u in Kilopascals, z in Meters): Below crust ($z > 2.5$ m) (3)
FV-mean		$S_u = 15.5 - 3.67z$	$S_u = 4 + 0.9z$
FV-max		$S_u = 19 - 3.67z$	$S_u = 7 + 0.9z$
FV-min		$S_u = 11.5 - 3.67z$	$S_u = 0.7 + 0.9z$
UU-1		$S_u = 4 + 0; 059z$	
UU-2		$S_u = 3.5 + 0, 24z$	
UU-3		$S_u = 4.6 + 0, 43z$	
UU-4		$S_u = 4.1 + 0.52z$	
UU-5	mean	$S_u = 3 + 1.2z$	
	max	$S_u = 5.5 + 1.2z$	
	min	$S_u = 0.8 + 1.2z$	
SHANSEP			
$\overline{\text{CIU}}$-C	mean	$S_u = 2 + 1.2z$	
Tests	max	$S_u = 2.7 + 1.3z$	
	min	$S_u = 1 + 1.0z$	
$\overline{\text{CKoU}}$-C	mean	$S_u = 1.5 + 1.0z$	
Tests	max	$S_u = 2.1 + 1.1z$	
	min	$S_u = 1.4 + 0.09z$	

(20), and recently updated by Palmeira and Almeida (21).

The total stress stability analysis consisted of testing hypotheses in order to select which set or sets of soil parameters would yield a factor of safety close to one at failure. A total of 16 assumptions on the undrained strength of the clay foundation were made, as listed in Table 2. Three of them (FV-mean, FV-max and FV-min) were based on the field vane (FV) results described earlier in this paper. The mean values were obtained from the linear regression analysis which best fitted the vane data (Fig. 4). The other assumptions (FV-max and FV-min) are, respectively, the upper and lower bound of field vane data, given by lines parallel to the FV-mean line, at a distance about two-times the standard deviation. Other hypotheses were based on the UU triaxial test results. As mentioned earlier in this paper, UU tests were performed on samples of different sizes, thus giving the assumptions presented in Table 2. The remaining hypotheses were based on the SHANSEP approach, S_u values being deduced from $\overline{\text{CKoU}}$-c and $\overline{\text{CIU}}$-C triaxial tests. Again, scatter in odometer test data, which strongly affects S_u from SHANSEP, yielded the mean, maximum, and minimum values as quoted in Table 2.

Costa-Filho, et al. (5) studied Rio de Janeiro clay's anisotropy through vane tests of different vane sizes and UU tests on both vertical and inclined specimens. The results of their study led to the conclusion that anisotropy of this clay seems not to be important. Therefore, isotropy was assumed throughout the analysis.

Fill strength was evaluated by means of laboratory direct shear tests on specimens collected in the field during construction. Strength parameters so obtained were: c = 10–20 kPa and ϕ = 35°. The small cohesion yielded by these tests may represent a total stress behavior of the em-

TABLE 3.—Hypotheses on Embankment Strength

Hypothesis number (1)	c, in kilopascals (2)	$\phi°$ (3)
1	0	0
2	0	35
3	5	35
4	10	35
5	20	35
6	30	35

bankment material, which showed many cracks at failure. Cracking of an embankment is evidence of the presence of some cohesion in its material. On the other hand, cracking would make it difficult to evaluate the amount of embankment strength actually mobilized at failure. Considering these points, six assumptions on embankment strength were made as quoted in Table 3, covering the possible range of actual embankment strength.

Considering 16 assumptions on foundation strength, 6 assumptions on embankment strength, and 4 additional embankment heights (these latter in order to observe the variation of safety factor with embankment height), 256 cases were processed. The results were subsequently "screened" to produce the assumptions which yielded a factor of safety close to one. These data were then summarized in Fig. 16. Setting the height of the embankment which produced failure equal to 2.8 m (this seems to be a reasonable assumption since severe embankment cracking took place at this height), one can obtain from Fig. 16 a value for the mobilized embankment strength at failure. While the assumption of FV-mean for foundation strength yielded a reasonable value for embankment strength, all other hypotheses did not predict failure at 2.8 m, un-

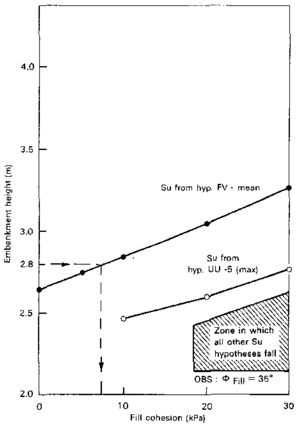

FIG. 16.—Summary of Soil Parameter Assumptions which Yielded FS = 1 in Total Stress Stability Analysis

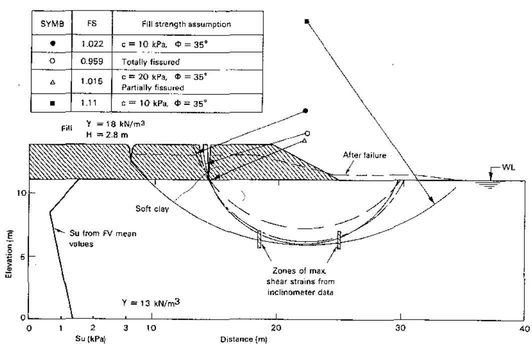

FIG. 17.—Results of Total Stress Stability Analysis

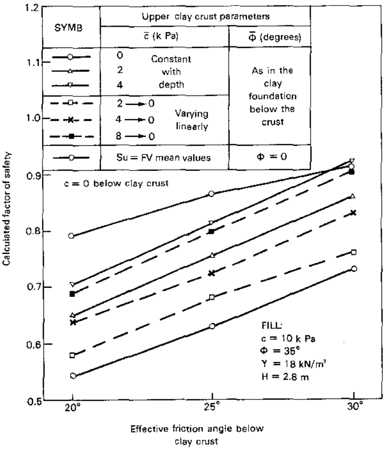

FIG. 18.—Results of Effective Stress Stability Analysis

less they were associated with very high embankment strength, incompatible with the soil type. This conclusion permitted the elimination of all hypotheses of S_u, except the one based on mean FV results. It was also observed that the increase in S_u in the clay crust was the main reason why only FV results yielded factors of safety (FS) close to one at failure. This is in agreement with Graham (9) who points out the importance of the crust strength on the stability of low embankments on soft ground.

Fig. 17 summarizes, the final results of total stress stability analyses. A FS very close to one was obtained. It can be seen that calculated and "observed" failure surfaces are in good agreement with regard to the maximum depth reached by these surfaces. The figure also shows that the calculated minimum value of FS is about 10% lower than the FS of the "observed" failure surface. The relatively small contribution of the embankment strength on the calculated FS is also shown in Fig. 17.

As the lateral berms introduced three-dimensional effects, an approximate analysis was made taking into account their influence. FS values so obtained were only 10% higher than those from plane strain analysis. Considering that and the additional uncertainty introduced in the back-analyses, such a complicated geometry should be avoided in future trial embankments.

An effective stress stability analysis was carried out employing measured pore pressures and effective strength parameters for the clay foundation. Results of pore-pressure measurements were reported earlier in this paper. Effective strength parameters were presented by Costa-Filho, et al. (5), based on $\overline{\text{CIU}}$-C triaxial tests, on samples from below the crust. Average values for $\bar{c}$ and $\bar{\phi}$ for the clay foundation were quoted as $\bar{c}$ = 0 and $\bar{\phi}$ = 25°. Later, Ramalho-Ortigão (22) carried out a series of isotropically and anisotropically consolidated undrained triaxial tests on both normally and overconsolidated clay specimens. The results seemed to confirm previous data shown in Ref. 3, and the values $\bar{c}$ = 0 and $\bar{\Phi}$ =

25° were selected for the analysis. The embankment height at failure was set at 2.8 m and fill strength parameters were chosen from the previous total stress analysis as $c = 10$ kPa and $\Phi = 35°$. These input data yielded a very low factor of safety of about 0.6. Additional analyses were then carried out varying soil parameters to account for possible errors in $\bar{c}$ and $\bar{\Phi}$, mainly at the upper clay crust. The results are summarized in Fig. 18 which shows that the calculated values of factor of safety are well below one.

Analysis of Results

The results obtained in the total stress analysis have emphasized the importance of the crust strength in the overall stability of the trial embankment. However, an increase in strength in this upper part of the foundation was not shown by UU tests nor by the SHANSEP approach. Most samples for laboratory triaxial tests were obtained from below the crust, precluding a reasonable estimate of its strength through UU tests. Also, the SHANSEP approach, as quoted by Ladd and Foott (12), does not always yield good results near the clay surface. However, the FV tests by Collet (4) have shown an increase in strength at the clay crust, and lead to a safety factor close to one at failure. These results were obtained from uncorrected FV mean values, which suggests that Bjerrum's FV correction (1) may not be applicable to Rio de Janeiro grey clay.

It should be emphasized that despite the reasonable results obtained from the analysis in terms of total stresses, piezometer data indicated that some drainage had taken place in the clay foundation during embankment construction. This was evident mainly at early stages of loading from the piezometers installed closer to the top of the clay layer. Therefore, considering the clay foundation as undrained clearly does not represent the observed behavior.

The validity of the effective stress analysis has been recently questioned by Tavenas, et al. (27), who concluded that this type of analysis is not a reliable research tool in back-analyzing clay behavior from observed full-scale failures. In addition to theoretical disadvantages, the practical problem of accurate measurement of pore pressures along the failure surface may also impose limitations on the effective stress analysis presented here. Fredlund (7), commenting on Tavenas, et al., quotes that it is better to have a "wrong" analysis that gives a "right" answer than to have a "right" analysis that gives a "wrong" answer. Indeed, despite the additional difficulty imposed by the adopted embankment geometry, the plane strain stability analysis in terms of total stresses yielded reasonable results, while very low FS values were obtained from the effective stress analysis.

Conclusions

1. Considerable experience was gained through installing and reading simple instrumentation systems, in accordance with the needs of an underdeveloped country.

2. Instrument observations during embankment construction permitted the evaluation of safe limits which may be employed for construction

control of structures founded on Rio de Janeiro gray clay.

3. Several hypotheses for the foundation soil undrained strength and fill strength were tested in a total stress analysis. The foundation strength was based on: (1) UU triaxial tests; (2) the SHANSEP approach based on $\overline{\text{CIU}}$-C and $\overline{\text{CKoU}}$-C triaxial tests; and (3) field vane tests. The reason why undrained strength deduced from UU tests and the SHANSEP method yielded very low factors of safety seems to be associated with the low strength at the upper clay crust yielded by these two methods. In fact, an increase in the undrained strength at the upper crust was only detected by means of field vane tests.

4. Bjerrum's field vane correction seems to be too conservative for application to Rio de Janeiro grey clay. Indeed, no correction seems to be necessary if mean field vane strength is used.

5. Factors of safety well below one were obtained for failure conditions from the effective stress analysis.

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