THE BEHAVIOUR OF THE INSTRUMENTATION OF AN EMBANKMENT ON CLAY

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1. Introduction

A comprehensive research programme on the behaviour of embankments founded on soft soils, sponsored by the Brazilian Road Research Institute (IPR) has been jointly conducted by a team from the Federal University of Rio de Janeiro. As a part of this programme, full scale instrumented experiments were carried out at a site 10 km north of Rio de Janeiro where an 11 m thick deposit of soft grey clay occurs. The first field trial consisted of an instrumented embankment built on the clay deposit and taken to failure in December 1977. The analysis of the embankment behaviour is presented elsewhere (Almeida and Ramalho Ortigão, 1982 and Ramalho Ortigao, Werneck and Lacerda, 1983). Additional results concerning the instrumentation behaviour are described in this paper.

2. Site and Soil

The interest in the geotechnical properties of this clay is not recent. Early studies were carried out by Pacheco Silva (1953) and, more recently, additional investigations have been carried out under the sponsorship of IPR, as described by: Costa Filho et alii (1977), Lacerda et alii (1977), Werneck et alii (1977), Collet (1978) and Ramalho Ortigão and Palmeira (1982).

Figure 1 summarizes geotechnical properties. Liquid limit varies from 150%, near the top, to 90% near the bottom, the in situ water content being slightly higher than these values. Plasticity index is about 80%. Field vane tests results ranging from 5 to 15 kPa have shown a decrease with depth of undrained strength in the clay crust, the minimum value being recorded at a depth of 2.5 m, below which the undrained strength seems to increase linearly with depth. The mean value of the undrained strength is about 10 kPa. Field vane tests also indicated that the clay sensitivity is of the order of 2 to 4. Stress history was evaluated from

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several high-quality oedometer tests, which have shown the over-consolidation pressure to be slightly higher than the in situ vertical effective pressure, as shown in figure 1.

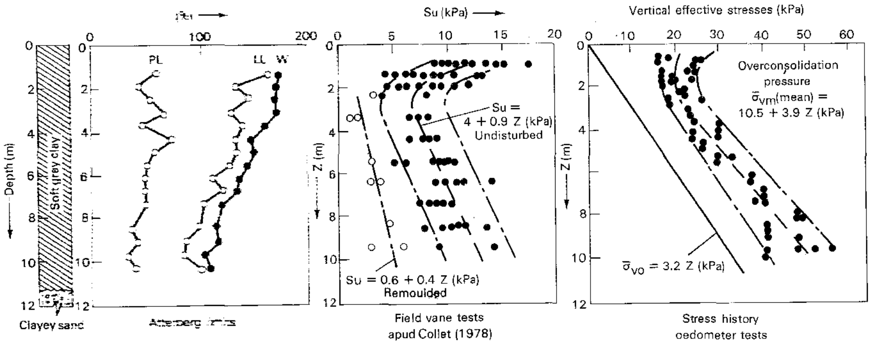


Fig. 1 Summary of geotechnical properties
Rio de Janeiro soft grey clay

3. Embankment Design

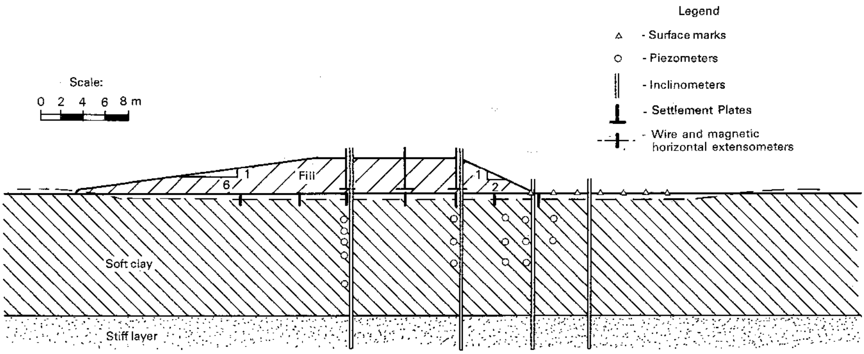
The embankment was designed as shown in figure 2 aiming at achieving plane strain conditions at the main instrumented section. In order to avoid the possibility of failure occurring outside the instrumented zone, two lateral berms for confining the main instrumeted section were adopted. In addition, the embankment height at the main section was lifted in advance of the berms, as will be discussed later in this paper.

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 failure occurring outside the instrumented zone. The three-dimensional effects caused by the adapted geometry were found not to be large (Ramalho Ortigão et alii, 1983).

4. Instrumentation

The lay-out of the instrumentation of the trial embankment is shown in figure 2. It included piezometers, magnetic and horizontal wire extensometers, inclinometers, settlement plates and surface marks. With the exception of the inclinometer sensor, all the instrumentation was specially designed and manufactured for the field experiment.

The piezometers were of the hydraulic type consisting of a porous high air entry ceramic tip, 49 mm in diameter and 140 mm long, connected by twin polyethylene coated nylon leads to mercury manometers, located at an instrument house. The method of installation consisted of opening a 100 mm diameter borehole to an elevation 0.50 m above the piezometer level, then, the porous tip was pushed into the clay and the borehole was then refilled with a bentonite-cement grout. The piezometer leads were



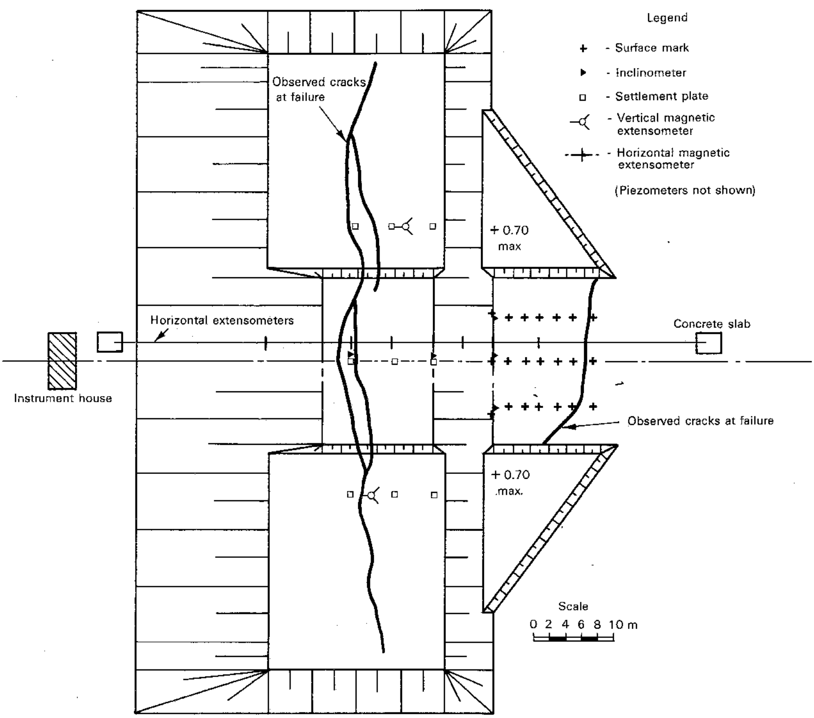


Fig. 2 Layout of the instrumentation

carefully extended in trenches and connected to the read-out units. The system was subsequently flushed with deaired water and tested for leakage.

Twenty one piezometers of this type were installed in the clay foundation. Ten of them were installed in pairs under the axis of the embankment, where little rotation of principal stresses were expected during construction. The remaining 11 porous tips were located under or near the steeper slope of the embankment, aiming at monitoring pore-pressures close to the failure surface.

Six standard aluminium inclinometer casings to be used with SINCO Digitilt Inclinometer were installed for the observation of horizontal displacements. The most important location for inclinometer installation is generally at the toe of an embankment where most of the shear deformation takes place. In addition to the casings installed at this position, two additional casings were located at a longitudinal distance of 5 m away from the main instrumented section. The remaining 3 casings were located as shown in figure 2.

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 (figure 3). It consisted of 5 mm diameter high tensile steel wires attached by means of cone clamps to the measuring targets, which were light steel plates positioned at the base of the embankment. The cone clamps, similar to those used in prestressed concrete, avoided the use of screws or welding during the assembly of the instrument in situ. The wires were introduced in a plastic casing and were extended to reach the concrete slabs located outside the embankment. Readings were taken by measuring the distance of a visible mark on the wire to a reference embedded in the concrete slab. During a reading the wire was stretched by a 50 N force applied by means of a small spring dynamometer. 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, was a horizontal magnetic extensometer which employs the same principle of that originally developed by Burland et alii (1972). Aluminium plates 400 mm long, 250 mm wide and 13 mm thick, containing a magnetic ring were laid down along the base of the embankment. A plastic access tube passes 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 could be introduced and readings could be obtained.

Horizontal ground displacements in front of the embankment's steeper slope were observed through surface marks located as shown in figure 2. Displacements were obtained from the variation of the horizontal distance of the surface marks to a line of sight, perpendicular to the displacement direction and defined by a theodolite.

As can be seen from figure 2, many instruments were located in clusters, permitting duplication in the observation of the same measurement. This duplication of efforts provides independent systems for checking the accuracy of the readings and evaluating instrumentation performance.

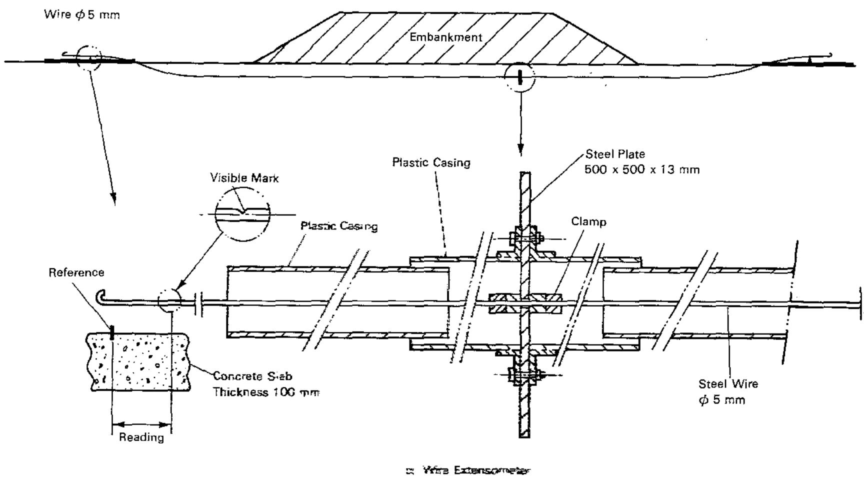


Fig. 3 Details of the wire extensometer

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

5. 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. Then, the contractor was allowed to begin the fill placement. Fill material was a silty-sand residual soil presenting, after placement, a mean unit weight of 18 kN/m^3. The embankment was lifted according to the records shown in figure 4. After placement of each layer, the construction was stopped and a new set of readings were made.

The first cracks were observed when the embankment height was 2.5 m. At a height of 2.8 m severe cracking on the embankment crest occurred and marked changes were observed in the readings of some instruments. The embankment height was then lifted to 3.1 m, the number of cracks increased considerably and failure propogated (see figure 2).

6. Results and Performance of the Instrumentation

Vertical displacement profiles under the base of the embankment are shown on figure 5. At failure, the maximum observed settlements were in the

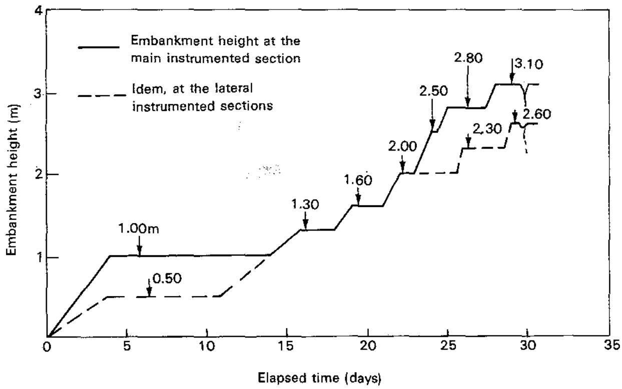


Fig. 4 Embankment height versus time

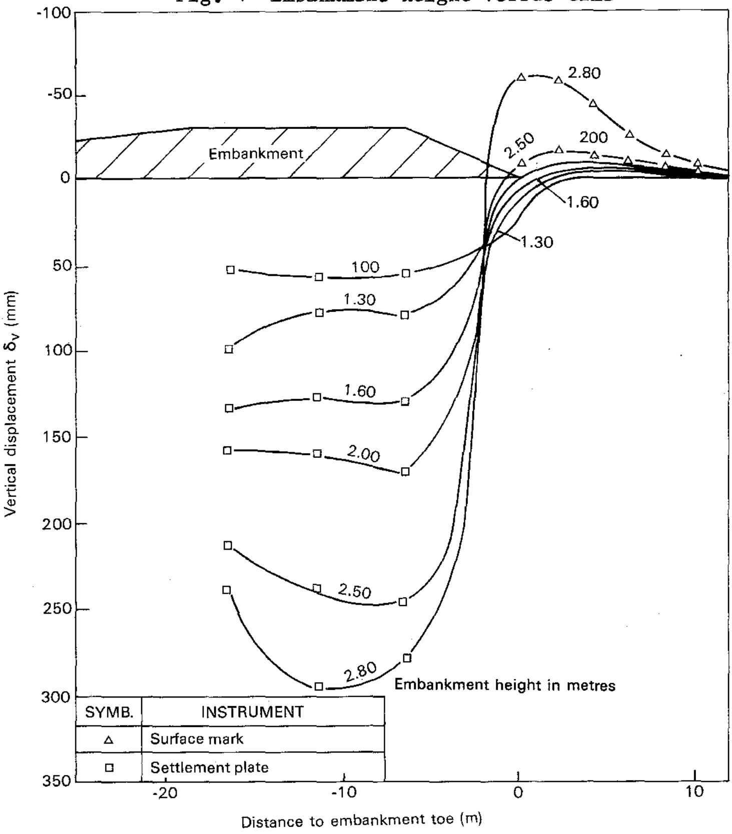


Fig. 5 Vertical displacement profiles under the base of the embankmen

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 to 2.8 m, a sudden change in heave observations was recorded. These readings show a 2 to 3 - fold sudden variation near failure. Another noticeable point on figure 8, is the high gradient of vertical displacements which occurs near the toe of the steeper slope of the embankment. At this position vertical movements at clay surface present a significant variation from positive (settlement) to negative (heave) in a few meters of horizontal distance.

Settlement monitoring at lateral or secondary instrumented sections were made through settlement plates and vertical magnetic extensometers, aiming at observing post-failure settlements. However, when the embankment failed, severe cracking also spread along the lateral sections (see figure 2), causing a sudden variation of the measurements. Thus, lateral settlement measurements have only been used for evaluating instruments performance. The experience so obtained, allow the writers to quote the accuracy of vertical magnetic extensometer in the order of ± 2 mm.

Observations of horizontal displacements by the surface marks versus embankment height are shown in figure 6. These measurements, taken at a distance of 2 m of the embankment toe, were preferred to those taken at the toe, which may have been affected by shocks during fill placement. The figure shows a sudden increase in the displacement measurements near failure preventing their use as an indication of imminent instability.

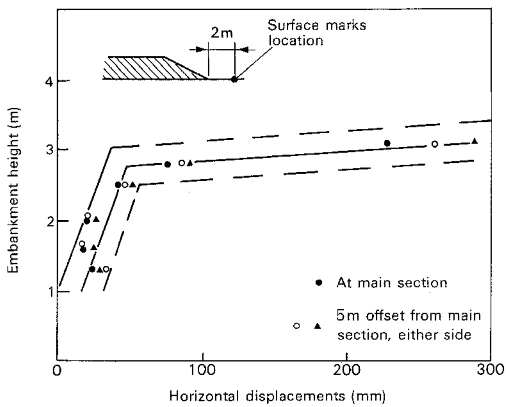


Fig. 6 Measurements on surface marks

Inclinometer observations in one casing positioned at the toe are shown in figures 7 and 8. Figure 7(a) presents the variation of angle θ, ie the observed vertical inclination with depth. The horizontal displacements are shown on figure 7(b). Figure 8 shows the backfiguring of the slip surface (assumed to be circular) employing inclinometer observations.

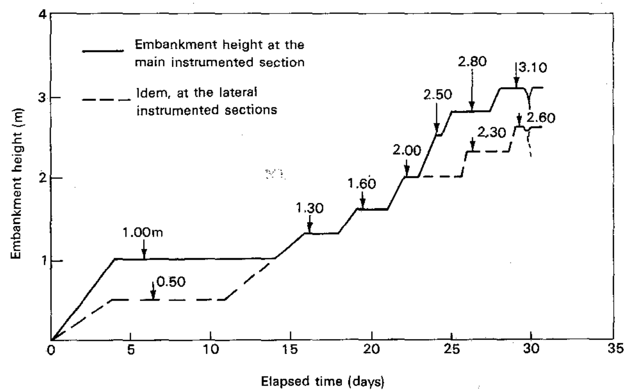


Fig. 4 Embankment height versus time

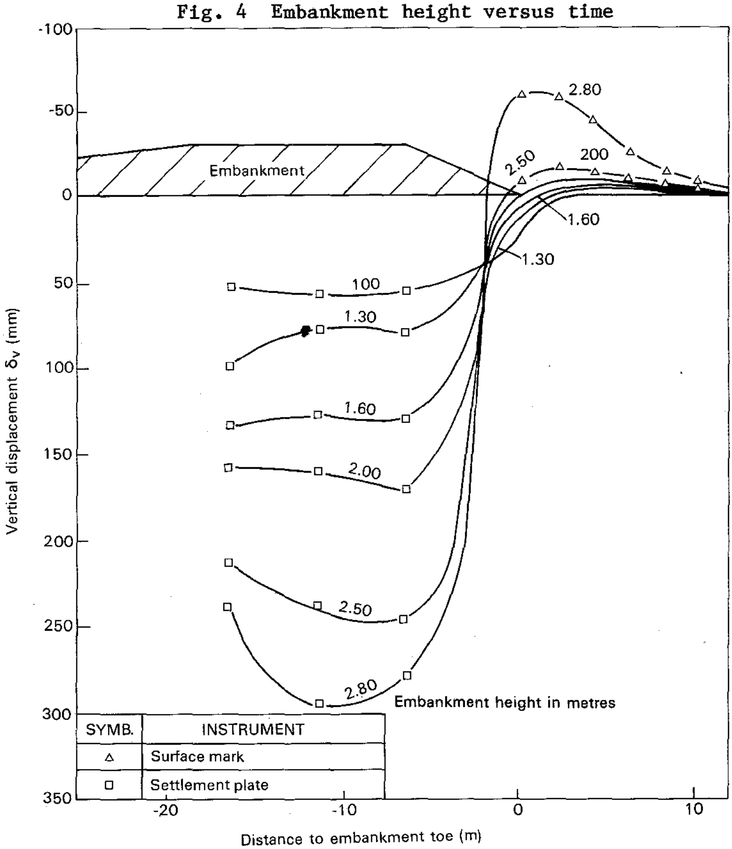
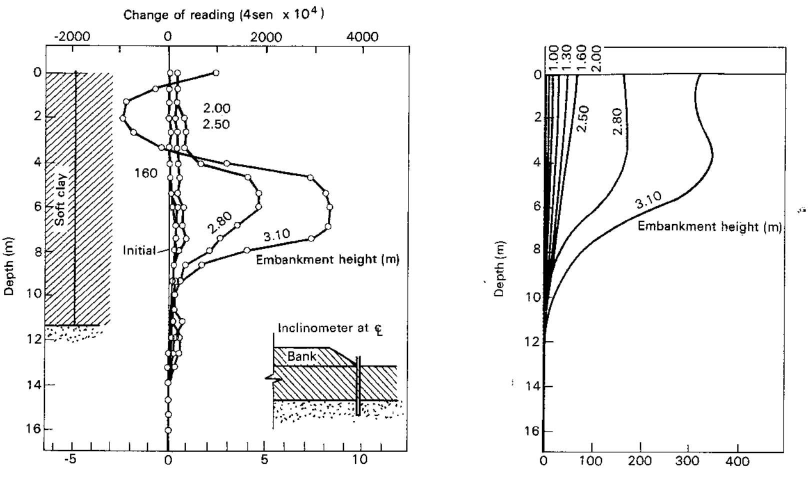


Fig. 5 Vertical displacement profiles under the base of the embankment



a) Vertical inclination angle Θ(rad. x 10^{-2})

b) Horizontal displacement (mm)

Fig. 7 Inclinometer observations

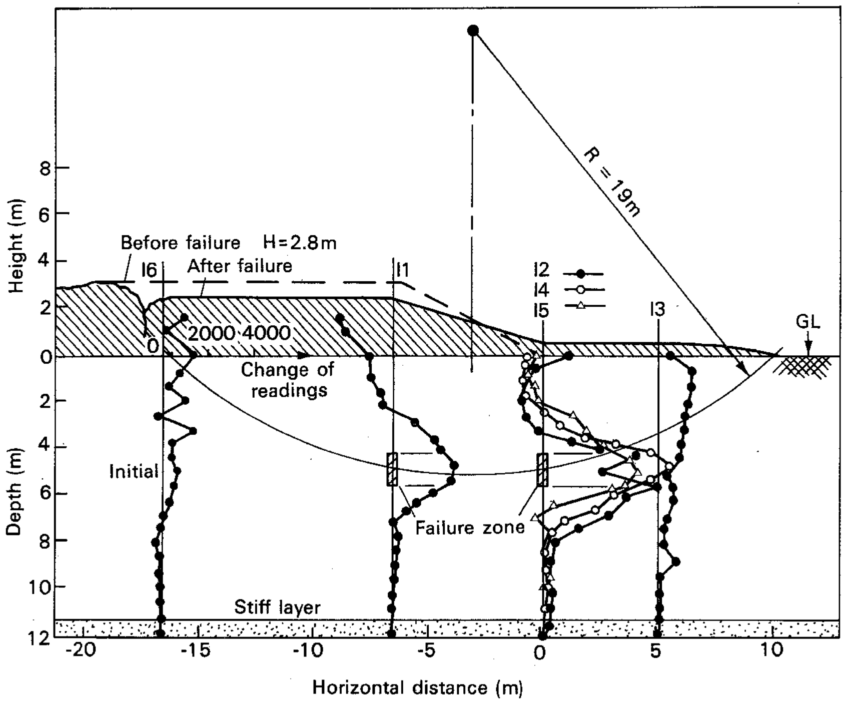


Fig. 8 Backfiguring the slip surface using inclinometer readings

Horizontal displacements measurements are summarised in figures 9, 10 and 11. At the end of fill placement, maximum toe horizontal displacements are in the range of 300 to 400 mm. Maximum values seem to occur under the steeper embankment slope, rather than at the toe. The zone of influence of ground displacements seems not to extend beyond 10 m from the embankment toe.

The horizontal magnetic extensometer performed well until the embankment was 3 m high. At this point, due to embankment deformation, the access plastic casing was obstructed, preventing further readings. This was certainly due to the use of inadequate tube joints: on the plastic access tube employed, no allowance was given to relative horizontal movements and tubes were flush-jointed by means of adhesive. Some problems were also encountered with the magnetic probe. Its moisture insulation was not initially adequate and had to be fixed during construction. Despite the above problems, the use of the magnetic extensometer was satisfactory since it is a reliable, simple inexpensive instrument. Accuracy was handicapped by tubing settlements, but comparing with other instrument results, overall accuracy was estimated to be better than ± 5 mm.

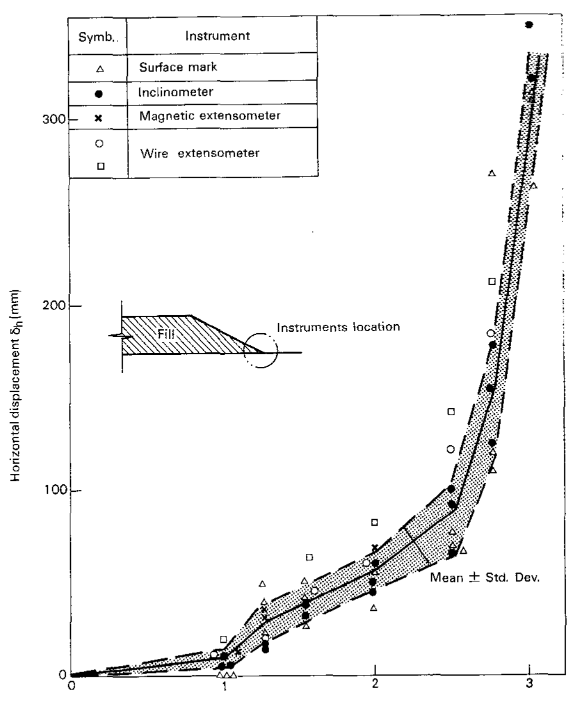


Fig 9. Observed horizontal displacements at the embankment toe

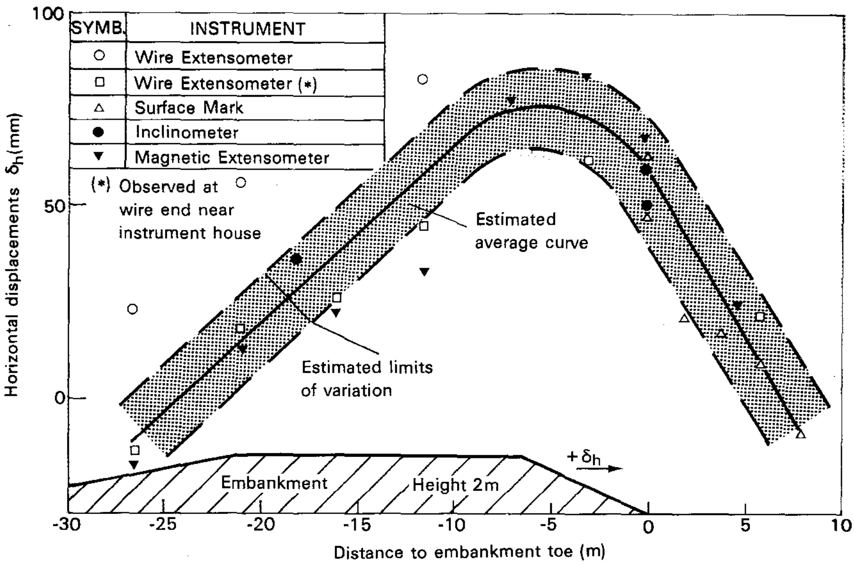


Fig. 10 Horizontal displacements at the base of the embankment

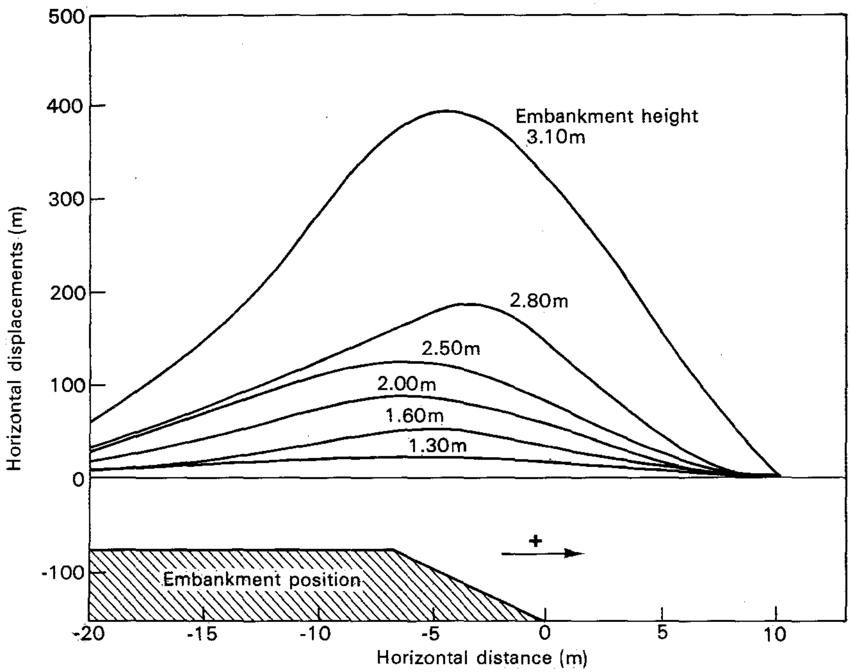


Fig. 11 Mean values of horizontal displacement profiles along the clay surface

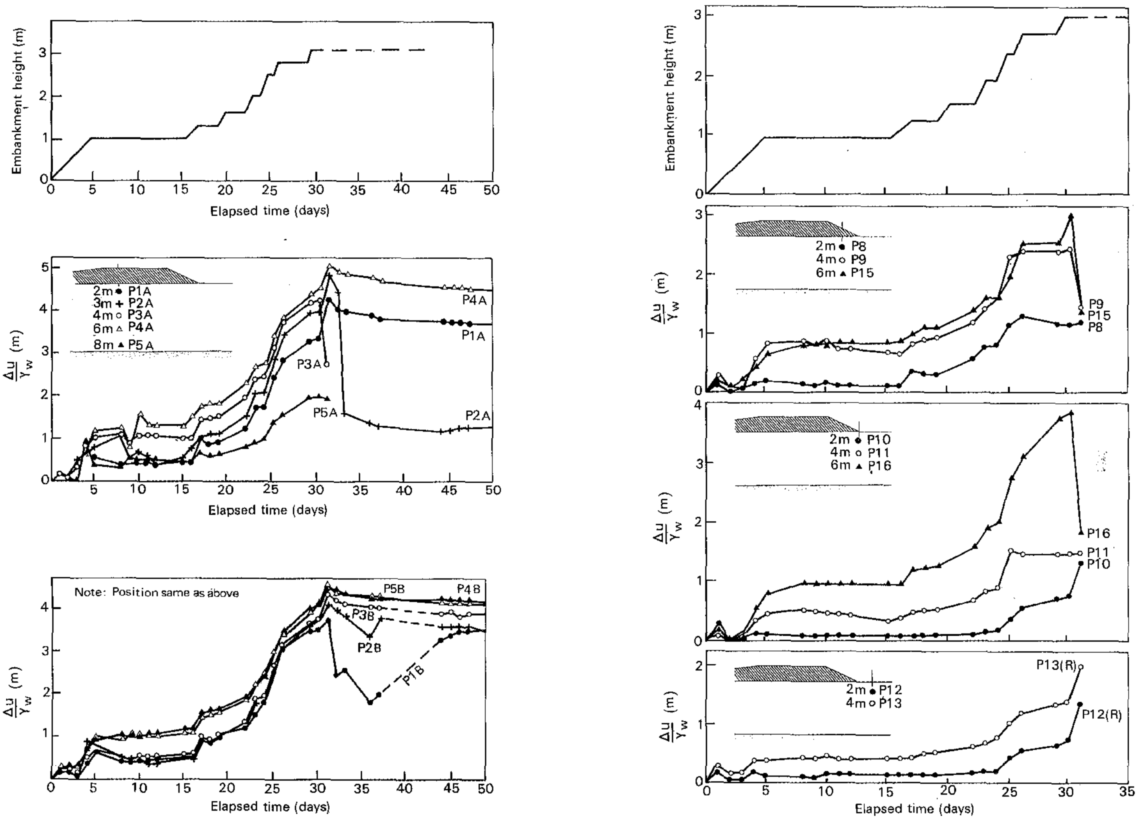


Fig.12 Piezometer observations

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 simple mechanical instrument, the overall pattern of movements presented by the measurements could be used for analysing displacement distribution along the embankment base. Its manufacture and installation were difficult and more expensive than the magnetic extensometer.

A comparison of horizontal displacements measured at the top of inclinometer casings and at surface marks at a very close position, indicates that the results are in the same range. Thus it can be concluded that the stiffness of the inclinometer casing probably did not affect the measurements. 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 seems to confirm these values. Accuracy of lateral displacement observations on surface marks was considered to be the ± 3 mm range.

The use of hydraulic piezometers in the embankment foundation was a successful experience. Local manufacturing and assembly of the piezometer system apparatus, field installation and monitoring of the piezometer response, gave to the research team the control of all phases of the work. This seems to have been an important asset for the field data evaluation and correcting 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 gauge house, making frequent deairing necessary. Scatter in initial observations permited evaluation of initial accuracy of readings to be assessed as ± 0.2 m head of water. However, during pore-pressure build-up due to loading, manometer readings became less erratic and accuracy seemed to have improved.

A summary of pore-pressure observations during embankment construction is plotted in figures 12 to 14. The piezometer response to load, as shown in figure 12, presents an initial flatter slope for the porous tip located closer to the surface, followed by a steeper slope. This typical behaviour had been previously interpreted by Lerouiel et alii (1978), who attribute the change in the rate of pore-pressure build-up to the phenomenum of clay foundation being altered from an overconsolidated state to 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.

7. The use of the Instrumentation for Stability Control

Field observations during the construction have allowed an evaluation of available methods of stability control for application to Rio de Janeiro clay (Ramalho Ortigão et alii, 1983). The experience gained has demonstrated that, among all instrument observations, the inclinometer readings were the most affected by the imminent failure. As can be seen from figure 7, a sudden change on the readings was observed just before the embankment had failed. 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 measured by the inclinometer and β values were not measured. However, at a point located near the horizontal part of a failure surface (see Gould and Dunnicliff, 1971) β may be very small. Thus, the shear strain γ_{vh} could be considered to be equal to θ_{max}, ie the maximum observed vertical inclination measured at the embankment toe. The observed behaviour has demonstrated that severe embankment cracking took place when shear deformations indicated by the inclinometer were greater than 3%. This value has been, then, suggested as a limit for construction control for stability on Rio de Janeiro clay.

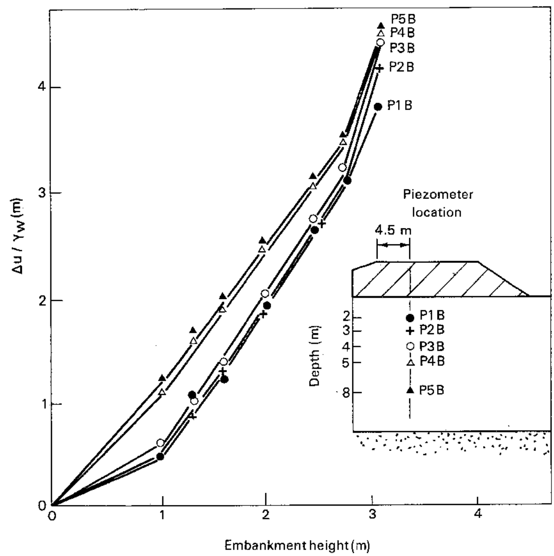


Fig. 13 Construction pore pressures

8. Stability Analysis

Stability analyses of the trial embankment have been described elsewhere (Ramalho Ortigão et alii, 1983). A summary of the results in terms of total stresses are shown on figure 15. A Factor of Safety very close to one was obtained for failure conditions when mean values of field vane tests were employed. The mean field vane results which yielded FS ≃ 1 were crude or uncorrected results, which suggests that Bjerrum's correction may not be applicable to Rio de Janeiro grey clay.

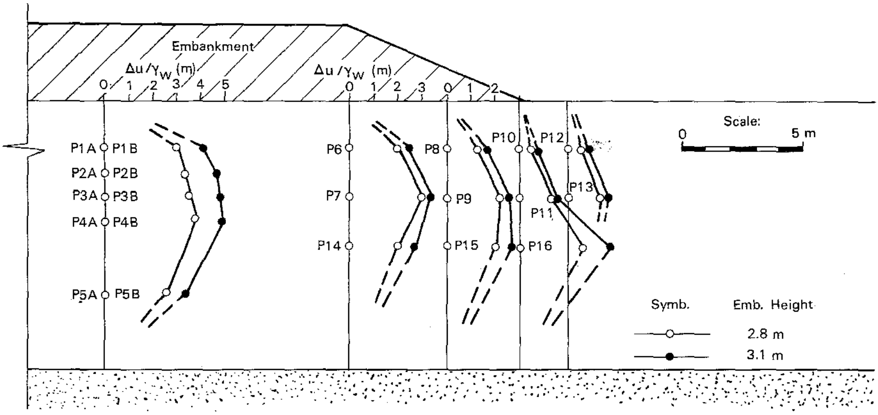
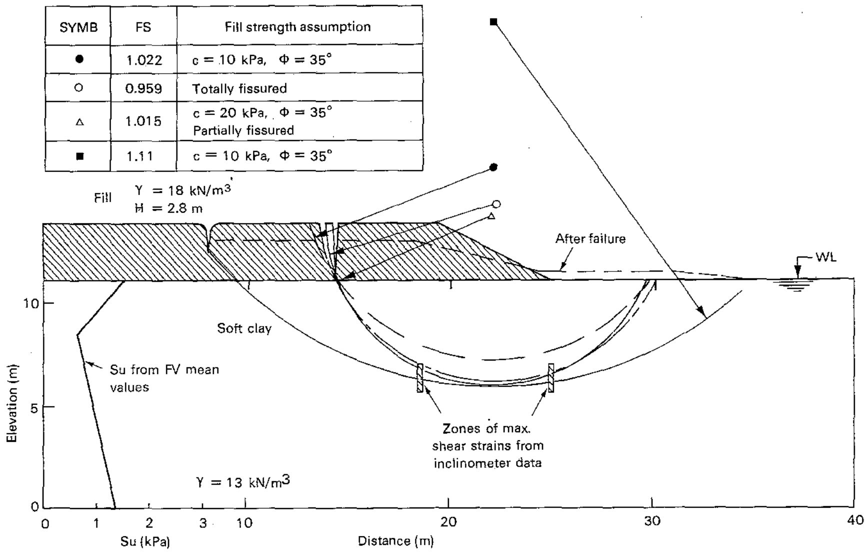


Fig. 14 Observed pore pressures



SYMB	FS	Fill strength assumption
●	1.022	c = 10 kPa, Φ = 35°
○	0.959	Totally fissured
△	1.015	c = 20 kPa, Φ = 35° Partially fissured
■	1.11	c = 10 kPa, Φ = 35°

Fig. 15 Results of the total stress stability analysis

9. Conclusions

1. Considerable experience was gained through installing and reading simple instrumentation systems, in accordance, with the needs of an under developed 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 grey clay.

3. Bjerrum's (1973) 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.

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