Stability and deformation monitoring of geogrid reinforced embankments[1]

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ABSTRACT

This paper presents the design and results of stability monitoring for embankments at the abutments of the in the BR-101 highway in south Brazil. Critical stability problems were due to a very soft clay foundation, several metres of depth and a high embankment. In order to cope with stability and settlements the design was carried out with several layers of geogrid reinforcement and prefabricated vertical drains (PVD) or geodrains. Settlement and stability control was monitored by vibrating wire piezometers, inclinometers and a settlement profiler. A stability design chart was prepared based on total and effective stresses and this enabled the selection of reinforcement tensile strength and choice of single stage or multi-stage construction. The latter reduces the necessary strength leading to geogrid savings. This chart was also instrumental for field stability control.

The results showed that the presence of the reinforcement layers reduced the soft soil lateral displacements and the damages to the existing structures.

INTRODUCTION

In July 1997 several embankments failures on soft ground took place during a highway construction in south Brazil. The project was the widening the BR 101 motorway and has strategic importance, as it links three countries: Brazil, Uruguay and Argentina. Geotechnical consultants were called in and a significant design change took place. Site investigation was enhanced by means of in situ piezocone and vane tests. The engineering solution: geosynthetics and construction control by means of instrumentation.

Geosynthetic applications for the design of embankments in Brazil on soft ground dates back from late 70's when nonwoven and woven geosynthetics dominated the market (Ortigao and Palmeira, 1982). At that time, the maximum tensile strength of a geosynthetic was low and in the order of 40 kN/m. Geogrids available today can reach twenty times that strength and prices have become very competitive. Geodrains (or PVD) to accelerate consolidation are also widely used, as modern fast installation techniques and mass production reduced its cost to one tenth of prices practiced twenty years ago.

At bridge abutments, where approach embankments on poor ground reached 3 to 5 m in height, stability was critical. The design consisted of the acceleration of settlements by means of geodrains and a temporary surcharge. The geodrains were installed on a square pattern spaced between 1.2 to 1.4 m. The smaller spacing applied to the region close to the bridges, where the required percentage consolidation was 95% of settlements occurring before paving the road. At 50 m away from the bridge, geodrains spacing increased to 1.4 or 1.5 m depending on the depth and consolidation properties of the clay layer.

Fahel et al (2000), on a paper on the same project, described in detail the behaviour of the bridge abutments and the reinforcement. This paper, on the other hand, focuses on stability analyses and control measures by means of instrumentation.

SITE CHARACTERISTICS AND FAILURES

Large, and sometimes rather deep, soft soil deposits are very frequently found in Brazil, particularly along the coastline. In the state of Santa Catarina, in southern Brazil, tropical organic soft

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soil deposits are commonly found along the coastal line of the state. For the widening of the BR-101 highway in that region failures occurred on soft ground with embankment heights in the order or less than 3 m. At this early construction stage, the Highway Department called in geotechnical consultants to analyse the problems and to propose solutions. Site investigation was then enhanced with piezocone (CPTU) and vane shear tests (VST). Due to the very soft nature of the clays, it is emphasized that the VST programme employed a frictionless vaneboring equipment (Ortigao and Collet, 1985, Ortigao, 1995).

Undrained stability back-analysis of failures (Figure 1) employing the undrained VST strength c_u yielded local field vane correction factors (μ). These studies led to the conclusion that the average empirical correction factor was 0.6.

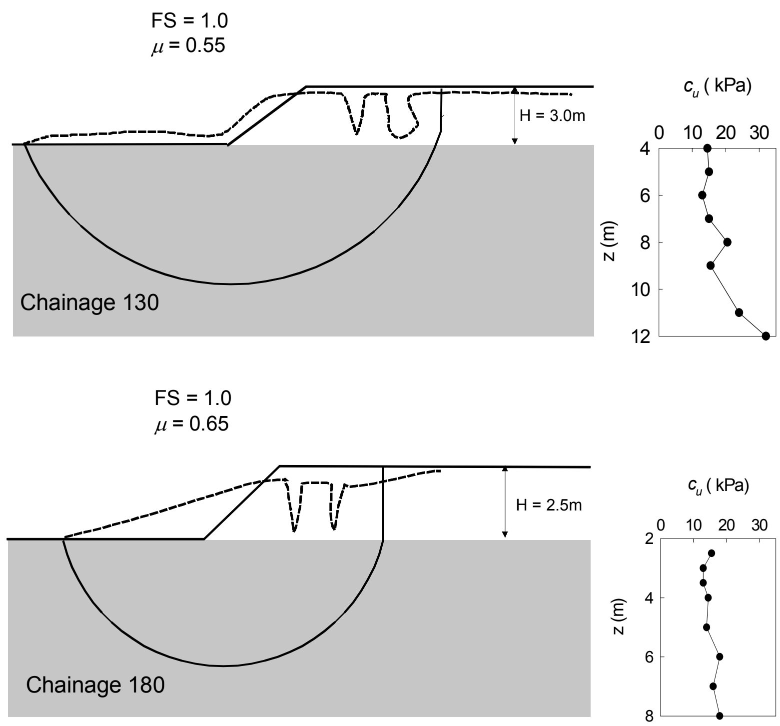


Figure 1 Embankment failures

SETTLEMENT STUDIES

Figure 2 presents a summary of settlement predictions of embankments on soft ground for a large portion of the BR 101 project. These studies led to the conclusion that values exceeding 0.5 m were expected in many cases. The engineering solution for settlements was to accelerate with geodrains and a temporary surcharge. The geodrains were designed in a square pattern with 1.3 to 1.6 m spacing, according to the site specific design. A temporary surcharge was designed with 30% of the embankment load and applied along a period of 3 to 6 months. This solution was adopted for soft soil thickness greater than 4 m, while, for soft soils less than this value, excavation and replacement by granular materials took place.

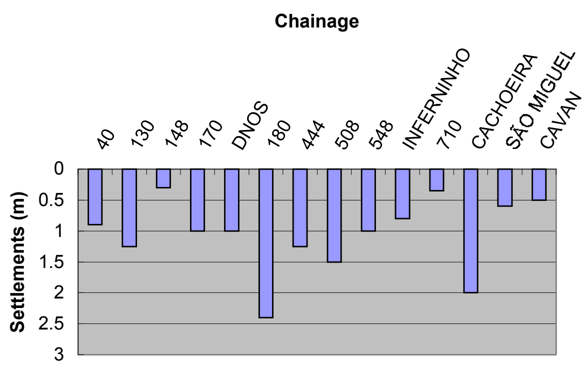


Figure 2 Settlement predictions for several embankments and bridge abutments along BR 101

STABILITY STUDIES

Stability analyses were carried out in order to check the factor of safety (FS) for unreinforced and geosynthetic reinforced embankments. The requirements were to ensure a minimum FS > 1.2. for the end-of-construction type failure where there no risk to any nearby structure. Close to bridge abutments and other structures, the minimum required FS was 1.4.

Berms were adopted to improve stability, wherever lateral space was available. Otherwise, the solution was geosynthetic reinforcement.

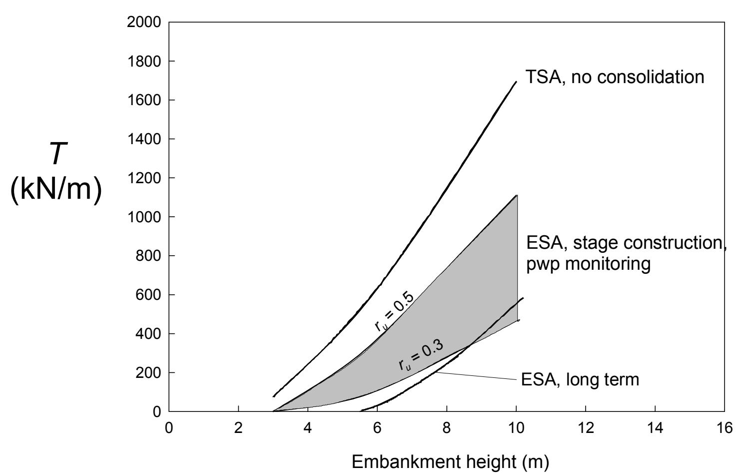


Figure 3 Stability chart for sandy embankment with 1:1.5 slopes on 10 m deep soft ground, global FS = 1.4

Stability charts

Stability charts like the one shown in Figure 3 were prepared for common situations. The aim of these charts was to enable the selection of the amount of geogrid reinforcement (*T*) and whether or not to stage the construction phases. This chart applies to polyester geogrids with a thin PVC cover for protection against environmental effects and site damages. Brazilian Highways Department specifications for this case required polyester geogrid tensile strength reduction factors 1.5 or 3, respectively for short and long term applications, to account for construction, environmental and creep.

Both total (TSA) and effective (ESA) stability analyses were carried out, as a function of the embankment height. The computer program *Rstabl* (Ortigao et al, 1995) was used employing Bishop method for circular slip surfaces. This program models a variety of reinforcement types, both rigid and flexible.

The analyses shown in Figure 3 refers to a typical case of a 10 m deep soft ground having undrained strength c_u of of 10 kPa and total unit weight of 15 kN/m^3. Embankment strength was described by a friction angle of 33 degrees and a unit weight of 16 kN/m^3.

One stage embankment construction

The upper line in Figure 3 refers to TSA for the end-of-construction case giving the amount of reinforcement as a function of embankment height for one stage construction, without any allowance for consolidation.

Multi stage embankment construction

Alternatively, this chart enables the designer to choose allowance for reducing porepressures by means of partial consolidation and construction stages. This reduces the total amount of geogrid reinforcement. This case employs the ESA approach with an effective strength for the clay layer. The effective clay strength is described by a conservative value of the friction angle $\phi' = 22^o$ and the cohesion intercept was taken as nil.

The analyses were carried out for values of porepressure parameter $r_u = u / \sigma_v$ of 0.5 and 0.3. These values were selected from past experience on similar soils. The upper r_u value was selected from analysis of an embankment failure on Rio de Janeiro clay (Ortigao et al, 1983), which yielded r_u larger than 0.5 (Figure 4). The lower bound was adopted as a value to be reached by allowing for consolidation of the clay layer with geodrains.

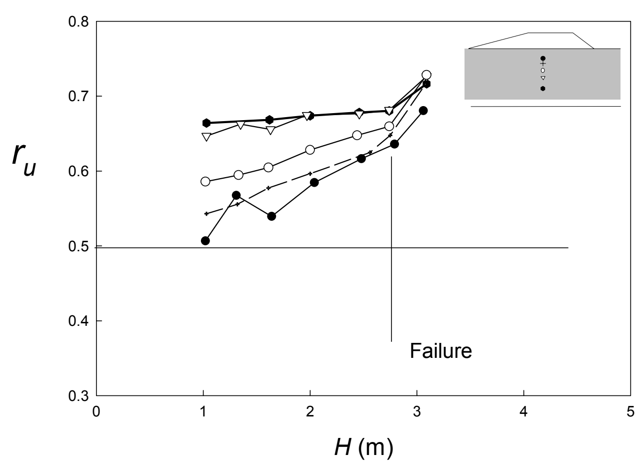


Figure 4 r_u values from embankment failure on soft ground

Therefore, the shaded area in Figure 3 corresponds to a cost-effective solution in which there is a compromise in the total amount of reinforcement and field porepressure control by means of careful monitoring. The r_u values should be lower than 0.5, if they tend to rise, time should be allowed. Geodrains should be used to reduce time span between embankment lifts.

Long term stability

The lower line of the chart in Figure 3 corresponds to long term stability. For this case excess porepressures are fully dissipated and the reduction factor of the geosynthetics complies with maximum long term values. For polyester geogrids this reduction value was taken as 3.

EXAMPLE OF A BRIDGE ABUTMENT

As an example, the data from one embankment approach are presented. It refers to the abutments of a concrete bridge over a canal known as the *DNOS Canal*, with DNOS standing for National Department of Drainage Works.

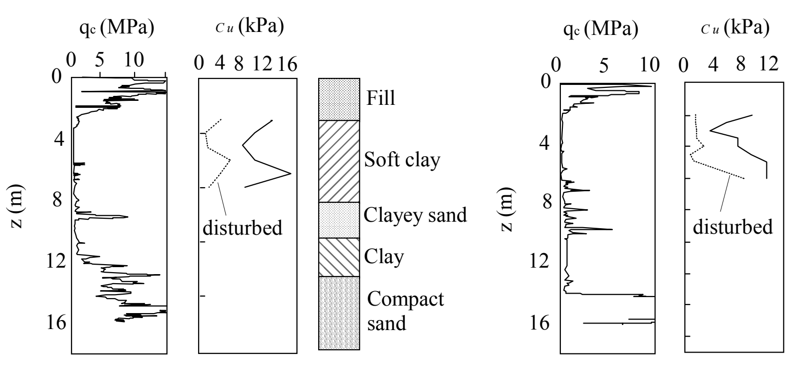
The foundation soil in the region consists of a soft organic soil layer with the presence of sand intrusions with thickness varying from a few centimetres to a couple of metres. Below the DNOS abutments a weaker clay layer, 5.5m thick, can be found separated from another slightly stronger clay layer by 2m of clayey sand, as shown in the schematic subsoil profile presented in Figure 5. This figure also shows the results of VST and CPTU performed at the site. The undrained strength varied between 4 to 15 kPa in a rather non-uniform mode. Piezocone dissipation tests performed at the site yielded values of horizontal consolidation coefficient between 10 and 28 m^2/year.

Characteristics of the Reinforced Abutments

Adjacent to the newly constructed abutments are old abutments that were built about thirty years ago and reinforced, at that time, with the use of wood branches and trees mattresses at their bases. It was expected that duplication of the highway and the construction of additional adjacent traffic lanes would cause damage to the existing embankments and structure. To minimise possible damages to those structures geogrid layers were used to reinforce the abutments in conjunction with geodrains to accelerate the consolidation of the soft soil deposit.

The two abutments described in this paper will be referred to hereafter as North and South abutments. Figure 6 shows a typical cross-section along the highway axis of the reinforced abutment (South abutment), showing the reinforcement layout. Figure 7 presents a cross-section normal to the highway axis showing the new and the old embankments.

Five layers of geogrid reinforcement were employed and the inclination of the embankment slopes were 1:1.5. The unidirectional polyester geogrids had a ultimate tensile strength of 200 kN/m in the main direction and only 20 kN/m in the secondary direction. The number of geogrid layers for this case is greater than what could be concluded from Figure 3, due to the fact that this clay location presents a strength well below the value used for this chart. Also, since this was the first abutment to be built, it was decided to be more conservative and to check for observed r_u values.



(a) North abutment (a) South abutment

Figure 5 In situ tests results

A 1.1m high surcharge fill was planned for placement on top of the embankments of the new abutments to act in conjunction with the vertical drains to minimise consolidation settlements and surface repairs after the construction of the pavement. In fact the surcharge was only applied to the South abutment, as will be discussed later in this work. The height of the surcharge was in most cases 30% of embankment height. The vertical drains used were synthetic band-shaped drains, 100mm wide by 5mm thick, comprising a plastic core with a nonwoven geosynthetic cover. The drains were installed in a square pattern with a spacing of 1.35m. A 0.4m thick sand blanket was placed on top of the foundation soil surface.

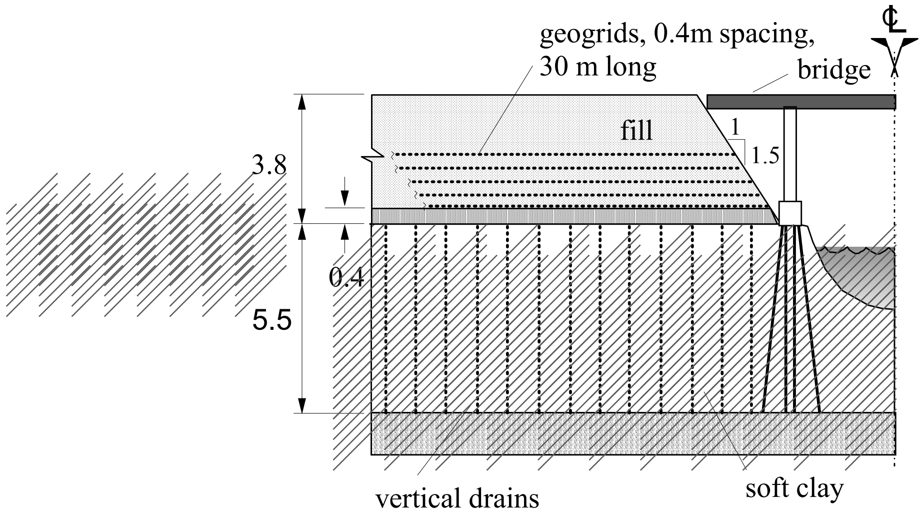


Figure 6 Typical cross-section of the abutments

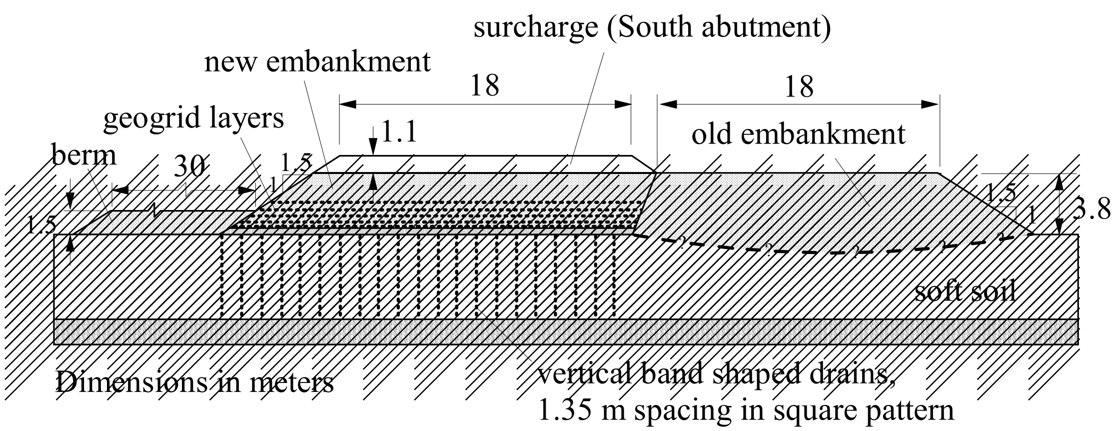


Figure 7 Old and new embankments

The soil used as fill material for the embankments was a coarse sand with a unit weight of 15.5 kN/m^3 and a friction angle of 33^o, determined by direct shear test.

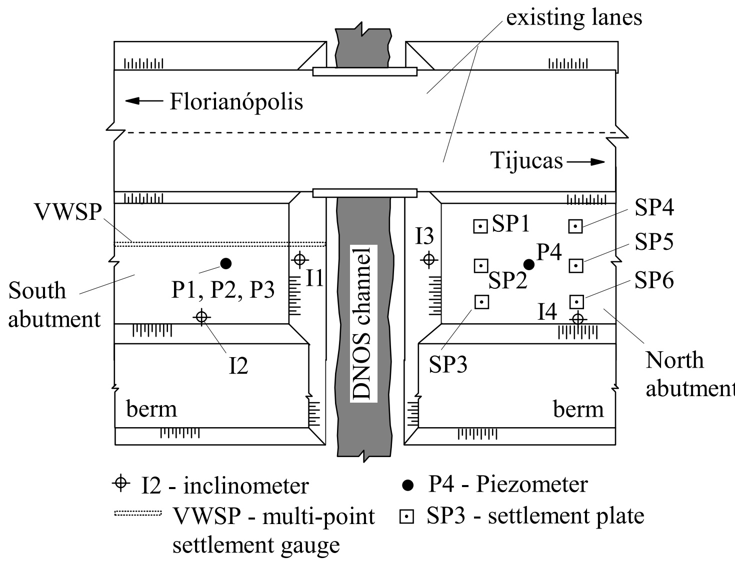


Figure 8 Plan view of the old and new embankments

INSTRUMENTATION

Figure 8 shows the location of instruments at the bridge abutments. The instrumentation consisted of four inclinometers, four vibrating wire piezometers, six settlement plates and a vibrating wire settlement profiler. The inclinometers consisted of conventional grooved casings and a servo-accelerometer based inclinometer probe with electronic read-out.

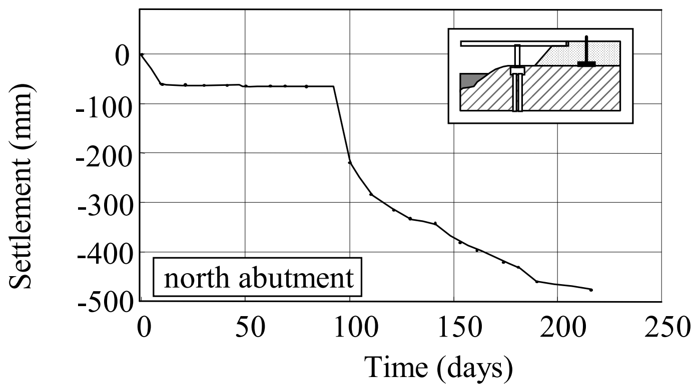
Vibrating wire piezometers were selected on account of their reliability, rapid response, long term stability and ruggedness (Buchanan et al, 1990, McRae and Simmonds, 1991). The fact that they could be read over long cables, without loss, or degradation, of the signal was also an important factor in the selection process.

Settlement plates consisted of a square plate, placed on the original ground surface, to which a riser pipe is attached which, in turn, permits optical leveling measurements to be taken.

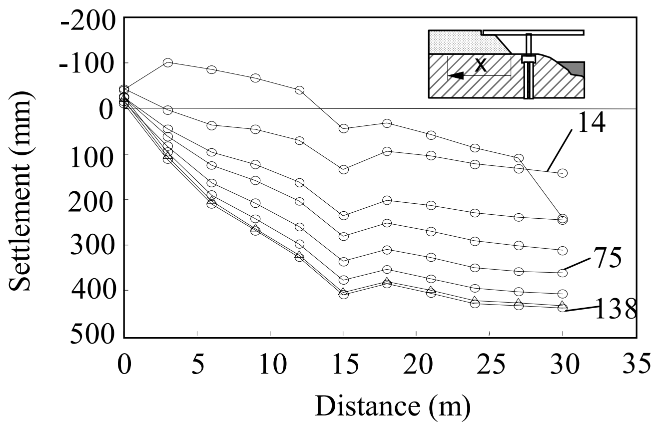
The settlement profiler consists of a vibrating wire pressure transducer mounted inside a torpedo which is connected by a liquid filled tube to a reservoir. The torpedo is pulled through a 50 mm diameter steel access tube which is placed a in a trench underneath the embankment. The profiler gives a measure of the elevation of the access tube relative to the reservoir, which is located on stable ground. With temperature and barometric corrections, the overall accuracy of this instrument is about 5 mm, which is excellent for this application.

Settlements

Figure 9 (a and b) shows settlement records with time for the abutments. The first (a) shows the settlements measured by the settlement plate SP2 and the second (b) shows the settlement profile along the embankment axis for the South abutment, as measured at different times by the settlement profiler. Settlement values as high as 0.5m can be observed in both cases.



(a) Settlements of SP5



(b) Settlements measured with the settlement profiler

Figure 9 Settlement records

Observed porepressures

Porepressure generation with time is presented in Figure 10. The embankment fill of the North abutment was placed first and very quickly and led to piezometers PE-1 to respond with a porepressure value of 41 kPa, resulting in a r_u value above upper bound of 0.5. Concerns with instability led to the removal of part of the surcharge load and to allow time for dissipation. The geodrains led to a quick dissipation and within a couple of week, the r_u value dropped to 0.1-0.15 range, well below the estimated safe limit of 0.5.

The construction of the South abutment occurred afterwards due to time need to install more instruments in the foundation. Therefore, the designers took advantage of the previous experience with the other abutment and decided upon a slow rate of fill placement. As a result, porepressures in piezometers PE-2 to PE-4 were much lower and in the order of 18 kPa and r_u value below 0.1.

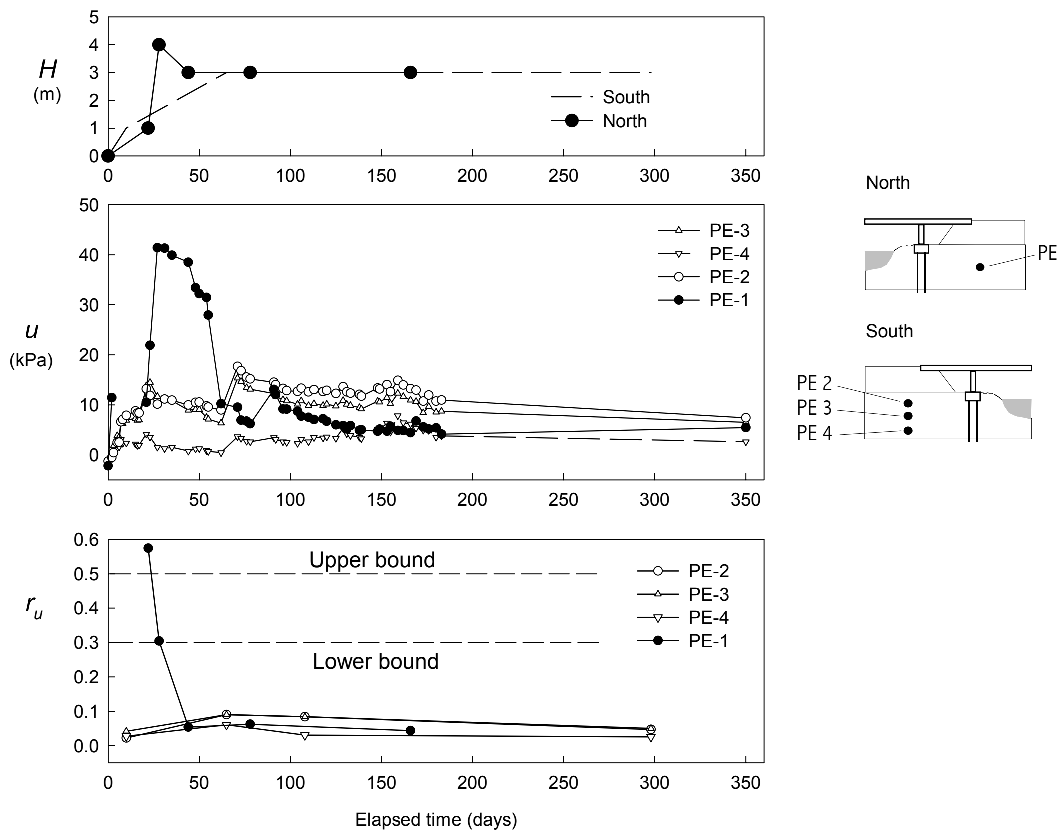
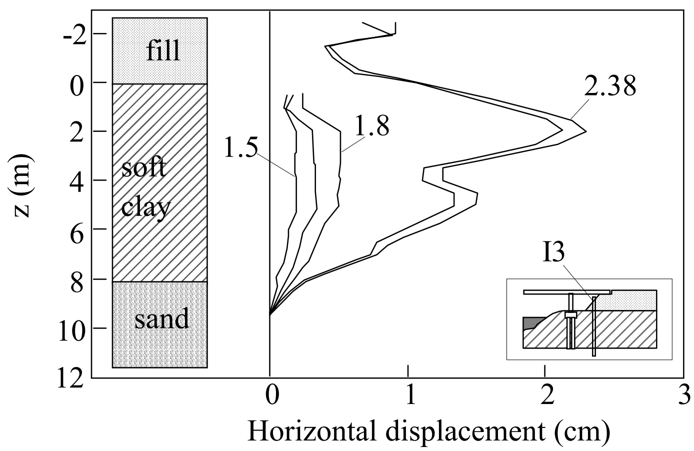


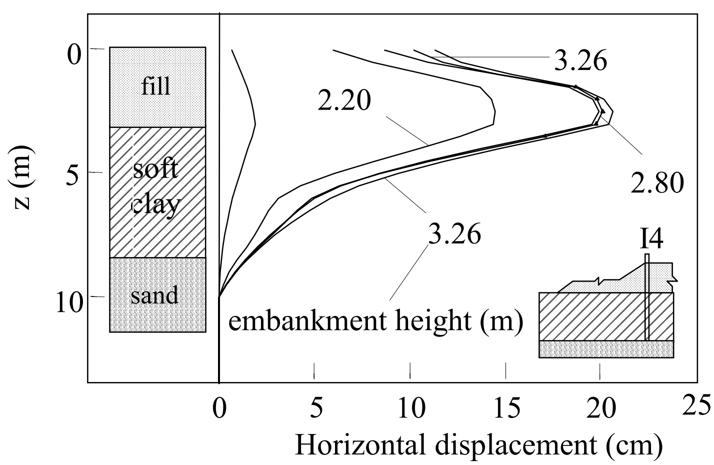
Figure 10 Observed porepressures and r_u values

Horizontal displacements

Figure 11 shows the horizontal displacements measured by inclinometers I3 and I4, in the North abutment. Inclinometer I3 was installed at mid length of the embankment slope of the North abutment, facing the canal, and inclinometer I4 was installed at the crest of the side slope. The pattern of horizontal displacement was similar in both cases, with the largest displacements occurring at the surface of the soft soil. The results also show that the fast construction of the embankment caused much larger horizontal displacements (of the order of 20 cm) along the direction normal to the embankment axis (I4) than along the direction of the embankment axis (I3). This can be explained by the fact that the embankment axis direction coincided with the geogrid reinforcement direction, as commented before. Therefore, the reinforcement was capable of reducing the horizontal displacement along that direction, in contrast to what was observed in the transverse direction.



(a) Inclinometer I3



(b) Inclinometer I4

Figure 11 Horizontal displacements in the soft ground (north abutment)

CONCLUSIONS

Failures of embankments on soft ground called the attention of the Brazilian Highway Department of the inadequacy of the design. This led to a site investigation programme through CPTU and VST, back-analyses of failures, settlement and stability predictions, which turned out in major re-design of all embankments on soft ground.

This paper described stability and deformation monitoring of bridge abutments on soft ground, close to an existing bridge which stability was main concern.

Preliminary design was carried out with stability charts based on total and effective stress analyses and the amount of geogrid reinforcement and need of geodrains to accelerate consolidation. These charts also led to upper and lower bound for porepressure and rate of fill placement control through porepressure parameter r_u.

Another independent stability control measure was the inclinometers. They have shown that the amount of horizontal displacements along the main reinforced geogrid direction was about three times less than the other unreinforced direction, despite the existing of lateral stabilising berm.

Settlement measurements indicated the time to remove the temporary surcharge load. The use of the settlement profiler, which does not interfere with the construction, proved to be very practical.

This was the first of five bridge abutments along this motorway to be instrumented and controlled in the same manner. Lessons from this one were so important that the design of the remaining structures was improved in the following ways:

- reducing the amount of reinforcement in the main direction and allow for dissipation with geodrains,
- increasing the amount of reinforcement in the transverse direction to increase stability
- replacement of the settlement plates by the settlement profiler;
- using ~~of~~ r_u parameter and inclinometer data for stability control.

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