ISSN: 0128-7680
Pertanika J. Sci. & Technol. 2(2): 215-235 (1994)

Behaviour of Soft Clay Foundation beneath an Embankment

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Received 18 October 1993

ABSTRAK

Sebagai bahan tanah liat lembut memberi banyak cabaran kepada jurutera-jurutera geoteknik. Bahan ini bertindakbalas dalam cara yang menakjubkan kepada perubahan tegasan. Di dalam rencana ini kelakuan lima buah benteng yang dibina di atas tanah liat lembut diperihalkan. Tanah-tanah ini merupakan tanah liat marin Malaysia yang terkukuh lebih sedikit. Penemuan utama yang didapati ialah tindakbalas tanah liat semasa pembinaan tidaklah tak bersalir sepenuhnya. Sedikit pengukuhan berlaku di dalam tanah liat terkukuh lebih di peringkat awal pembinaan. Tanah ini selanjutnya menjadi terkukuh normal semasa pembinaan diteruskan. Kelakuan tak bersalir hanya berlaku apabila tanah liat menjadi berkukuh normal.

ABSTRACT

As a material, soft clay poses many challenges to geotechnical engineers. The material responds in a spectacular manner to stress changes. The paper describes the behaviour of five embankments constructed on lightly overconsolidated soft Malaysian marine clays. The main finding is that the clay response to construction is not truly undrained. Significant consolidation develops initially in the overconsolidated clay, which becomes normally consolidated during construction. Undrained behaviour develops only in the normally consolidated clay during the initial stages of construction.

Keywords: consolidation, lateral displacement, pore water pressure, soft clay

INTRODUCTION

Soft clay deposits are widespread, and they present special problems. By definition, soft clays are of low strength and high compressibility, and many are sensitive, in that their strength is reduced by disturbances. Foundation failures in soft clay are comparatively common, and surface loading, e.g. in the form of embankments, inevitably results in large settlements.

In Malaysia, Quaternary erosion accentuated by climatic and sea level changes has produced widespread thick deposits of soft clays in the coastal areas and major river valleys, of varying thickness, ranging from 5 m to 30 m. Reviews of the basic and engineering properties of some of these deposits have been published by Ting *et al.* (1987) and Abdullah and Chandra

(1987). With the development of communication networks due to the increasing pace of industrialization and urbanization, the design and construction of embankments on soft clays have become problems of major importance to geotechnical engineers. The capacity to design an embankment economically on a clay foundation and to predict its behaviour are thus of great interest to the profession.

As a material, soft clay poses abundant engineering challenges. The designer must often use very low safety factors, and the decisions he takes can have large economic consequences for the project.

The usual methods for the design of embankments on soft clays have been developed from simplified assumptions and empirical approaches. The material responds in such a spectacular manner to stress changes that it offers the engineer-scientist special opportunities to evaluate the theories of soil mechanics. This evaluation process has been particularly facilitated by a number of carefully planned full-scale field trials which have been carried out in recent years, and by a series of well-documented case histories. Among major reviews of design practices are those made by Bjerrum (1972), and Tavenas and Leroueil (1980).

The present paper describes the behaviour of five embankments constructed on soft Malaysian clays. The first two embankments (designated embankment 1 and 2) were nominally 3 m and 6 m high, constructed on top of about 20 m soft marine clay. The third, fourth and fifth embankments (designated embankment 3, 4 and 5) were 2.0 m, 2.5 m and 3.5 m high constructed on top of about 14 m - 20 m of very soft to soft silty/sandy clay. All these embankments were instrumented with settlement markers, piezometers and inclinometers.

LOCATION OF SITES AND PROPERTIES OF THE GROUND

Embankments 1 and 2 are trial embankments constructed by the Malaysian Highway Authority in 1988, in the southern state of Johor, Peninsular Malaysia. The subsoil profile comprises about 20 m of a soft to very soft marine clay, underlain by a layer of loose to dense, medium to coarse sand, with SPT values of 6 - 50. The natural water content of the soft clay layer varies from 50 - 120%, liquid limit 40 to 80% and plastic limit 20 to 50%. Traces of sea shells indicate a marine origin. A summary of the geotechnical properties of the clay layer is given in *Fig. 1.* The undrained shear strength (Su) obtained from the vane test showed an almost linear increase of strength below a surface crust with an average strength of 9 kPa at depth 1 m, increasing to 36 kPa at depth 17 m, or 8 - 36 kPa if corrected with Bjerrum (1972) correction factor for anisotropy and shear rate. The Su/σ c ratio (σ c = effective preconsolidation pressure) is in the range of 0.21 - 0.29, the higher Su/σ c ratio for the upper more plastic clay. This is in reasonable agreement with correlations obtained from other sites of similar soft clays, e.g. Su = (0.24 ± 0.04) σ c (Larrson

1980); $Su = (0.24 \pm 0.04)\ \sigma c\ OCR^{0.8}$ (Jamiolkowski *et al.* 1979); $Su = (0.22 \pm 0.02)\ \sigma c$ (Ladd 1981). The above trend of $Su/\sigma c$ increase with increase in soil plasticity has also been observed at other sites by Larrson (1980) and Balasubramaniam *et al.* (1985). The clays are also known to be fairly sensitive, with a sensitivity ratio in the range of 3 to 6. Ratios of Eu/Su with Eu (undrained modulus) obtained from the laboratory were found to vary from 230 - 455, apparently higher than data from other field sites of similar clays, e.g. Eu/Su = 190 (Poulus *et al.* 1989). However, other published literature also indicates a substantial variation of undrained strength and stiffness ratio, e.g. Eu/Su varying from < 200 to 2000 has been reported by Foott and Ladd (1981), depending on stress level and soil type. Results obtained from the oedometer tests indicate that the clays are slightly overconsolidated but highly compressible. Values of cv are

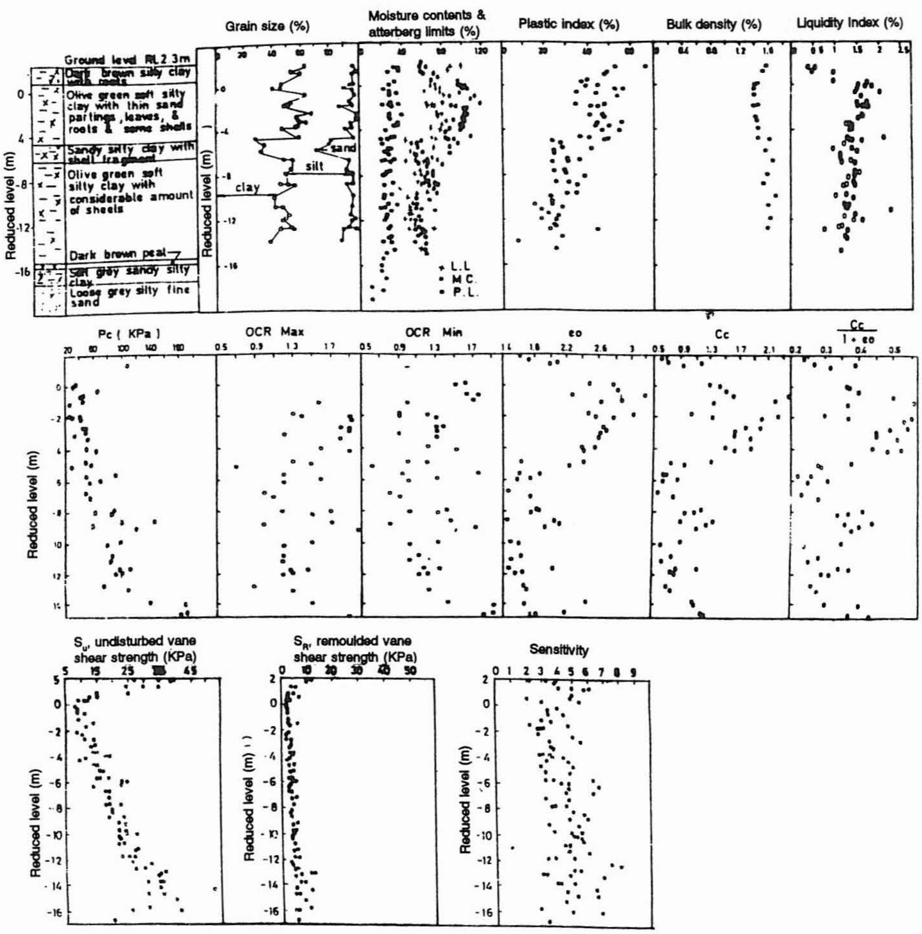


Fig. 1. Subsoil properties of embankments 1 and 2

typical low, ranging from 1 - 10 m^2/yr, and scattered, and Cc values are in the range of 0.6 - 1. The soil permeabilities (k_v k_h) are generally less than 5 × 10^{-9} m/s, with a clay fraction of the order of 50% and kaolinite as the dominant mineral present.

Embankments 3, 4 and 5 are part of the North-South Expressway, constructed in 1992-93, in the northern state of Penang, Peninsular Malaysia. The subsoil profile comprises a 14 m to 20 m thick layer of soft to very soft silty/sandy clay with thin lenses of sand and silt, and underlain by a layer of loose to dense sand. The liquid limit of the soft clay layer varies from 50% to 110%, with natural water content close to the liquid limit, and plastic limit in the range of 20% to 60%. A summary of the geotechnical properties of the clay layer is given in *Fig. 2 (a) and (b)*. In general, the undrained shear strength of the clay obtained from the vane test showed an increase of strength with depth, below a surface crust. The clays are lightly overconsolidated with OCR values in the range of 1.1 to 2.1 but fairly compressible. The Su/σc ratio is in the order of 0.25 to 0.35. Values of C_v, as obtained from laboratory oedometer tests, are typically low, ranging from 0.3 - 1 m^2/yr.

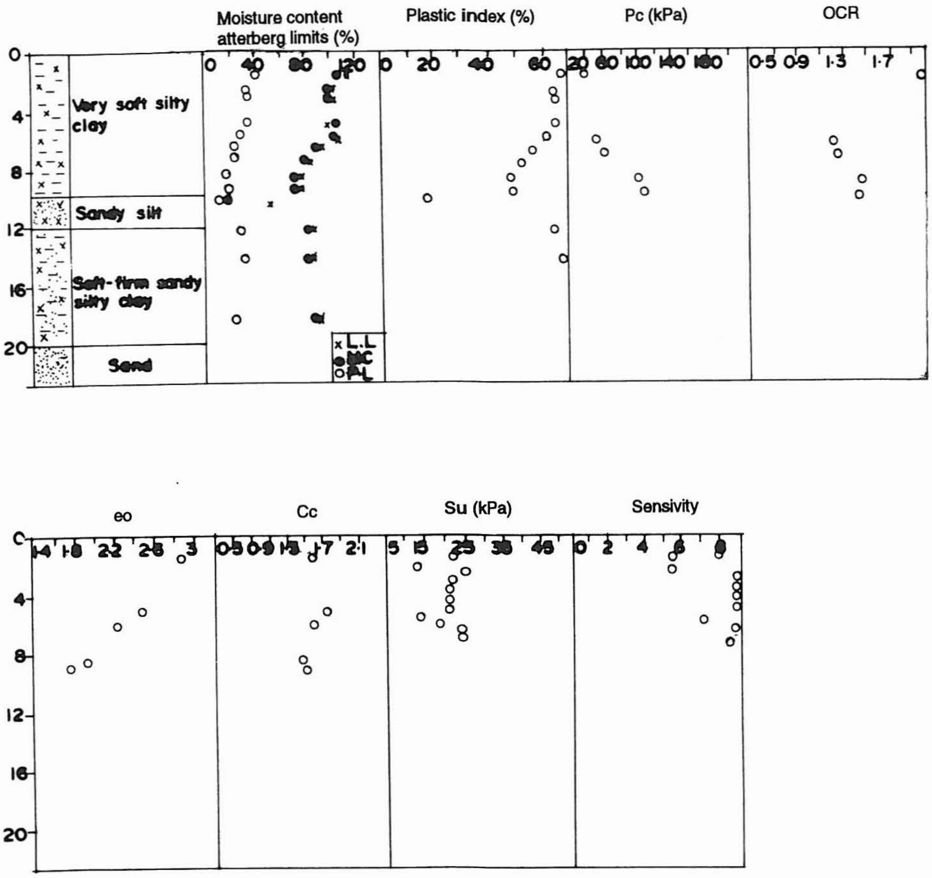


Fig. 2a. Subsoil properties of embankments 3 and 4

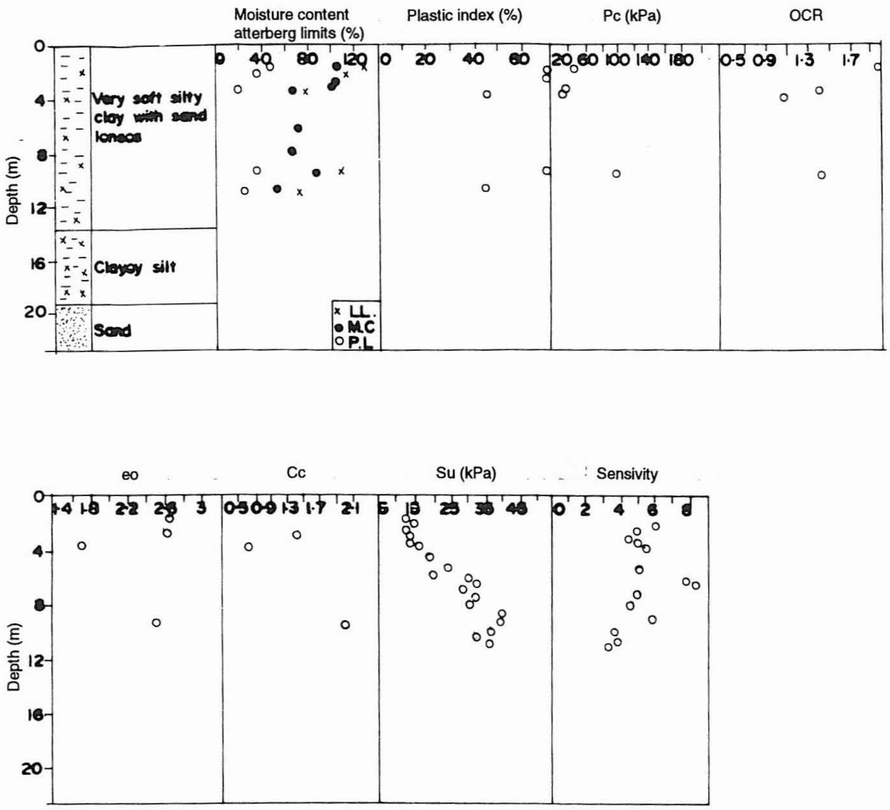
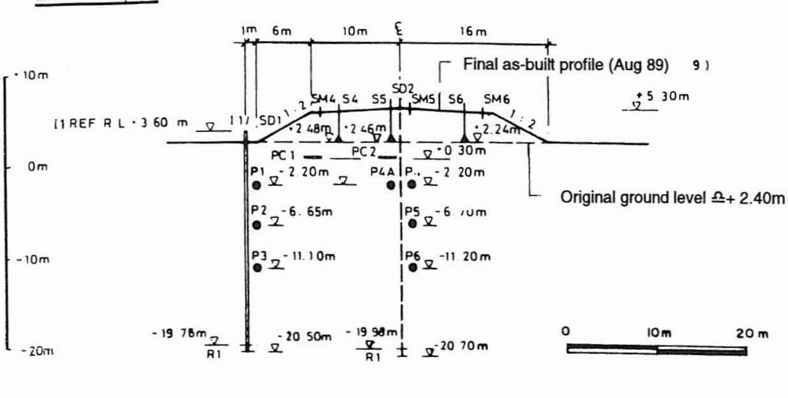


Fig. 2b. Subsoil properties of embankment 5

EMBANKMENT SECTIONS AND INSTRUMENTATION

Fig. 3 (a) and (b) show cross-sections and instrumentation of the embankments. They were instrumented with settlement gauges, pneumatic piezometers and inclinometers. Note the presence of a 50 m wide berm on both sides of embankment 2, being the highest at 6 m for reasons of stability. *Fig. 4* illustrates the construction histories of the embankments.

Embankment I



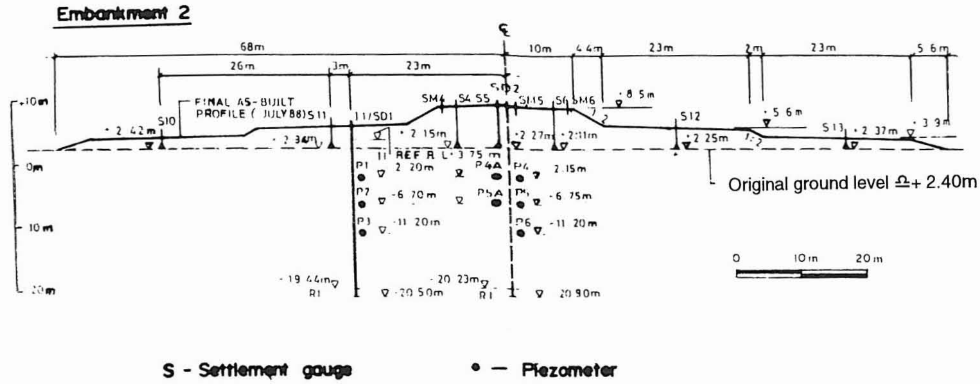


Fig. 3a. Cross-section and instrumentation of embankments 1 and 2

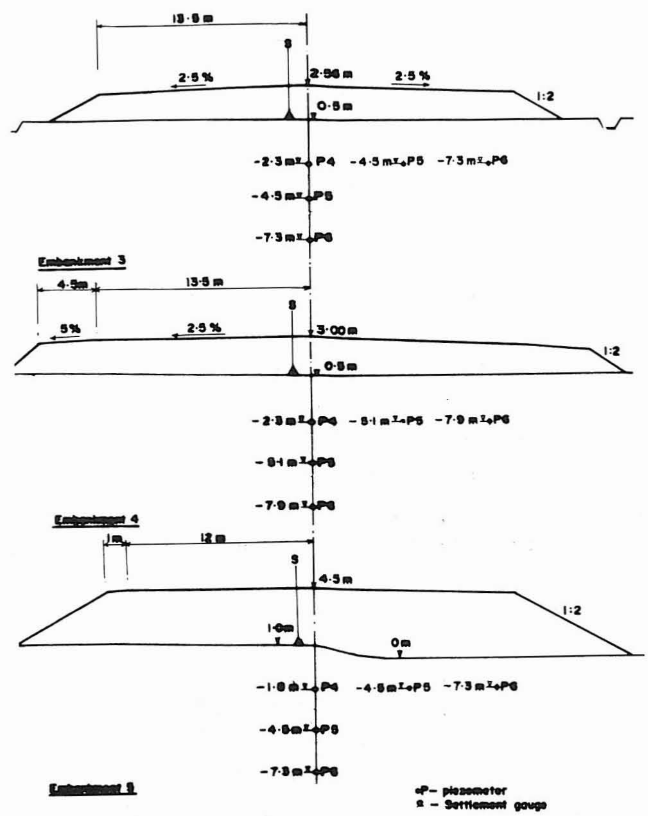


Fig. 3b. Cross-section and instrumentation of embankments 3, 4 and 5

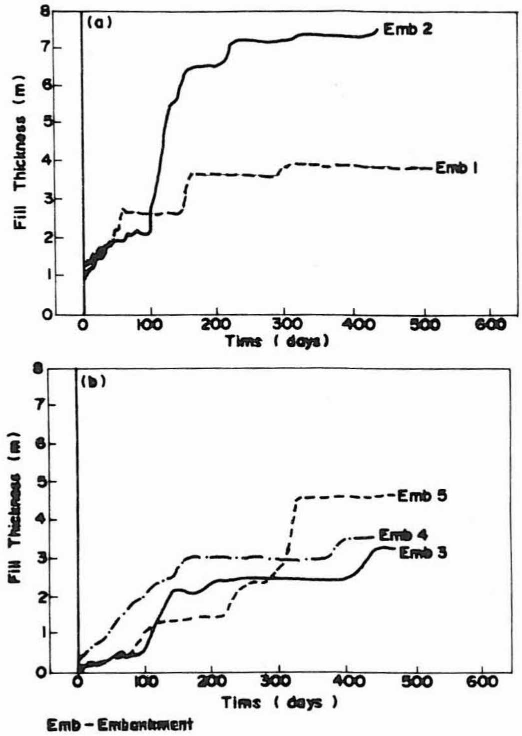
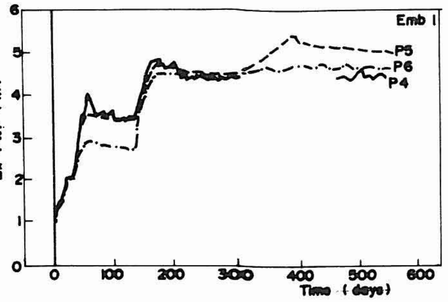


Fig. 4. Construction histories

OBSERVED BEHAVIOUR OF SOFT CLAY FOUNDATIONS

At the Beginning of Construction

Surface desiccation, groundwater fluctuation or aging nearly always causes soft clays to exhibit light overconsolidation, although they are commonly normally consolidated (Bjerrum 1967). For the subsoil of embankments 1 and 2, the values of apparent OCR estimated on conventional oedometer tests were in the range of 1.1 to 1.7. A similar range of apparent OCR was also found for the subsoil of embankments 3, 4 and 5. At the initial stage of embankment construction, the clays are expected to exhibit characteristics of an overconsolidated soil; i.e. with a small recompression index and a high coefficient of consolidation. Placement of the first few lifts of the fill layers induced total stresses to generate excess pore water pressure. A pore pressure gradient was then created between the interior of the clay foundation and its boundaries, initiating a consolidation process. Since the coefficient of consolidation of the overconsolidated clay is high, the rate of excess pore water pressure dissipation should also be high. This is shown in the initial pore pressure measurement beneath the centreline of the embankment, *Fig. 5 (a) and (c)* and *Fig. 6 (a) - (e)*. The average value of B, of the order of 0.3 to 0.6 (*Fig. 6 (a) - (e)*), is significantly below the theoretical value corresponding to an undrained behaviour.



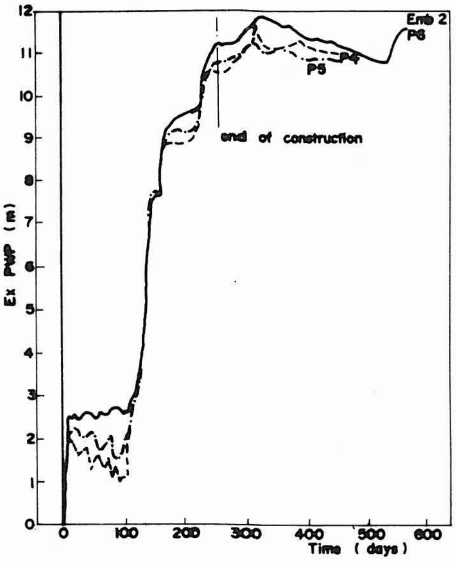


Fig. 5a. Excess pore water pressures - centreline piezometers (embankments 1 and 2)

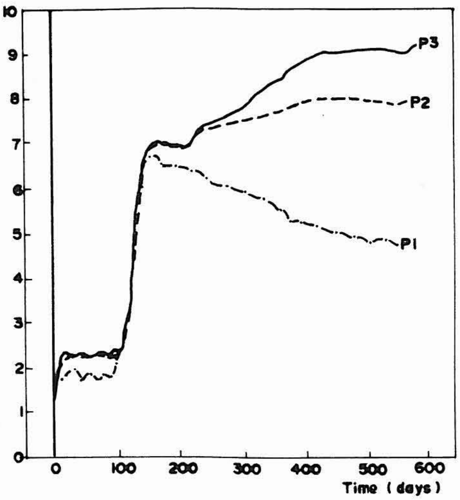


Fig. 5b. Excess pore water pressures - edge piezometers (embankment 2)

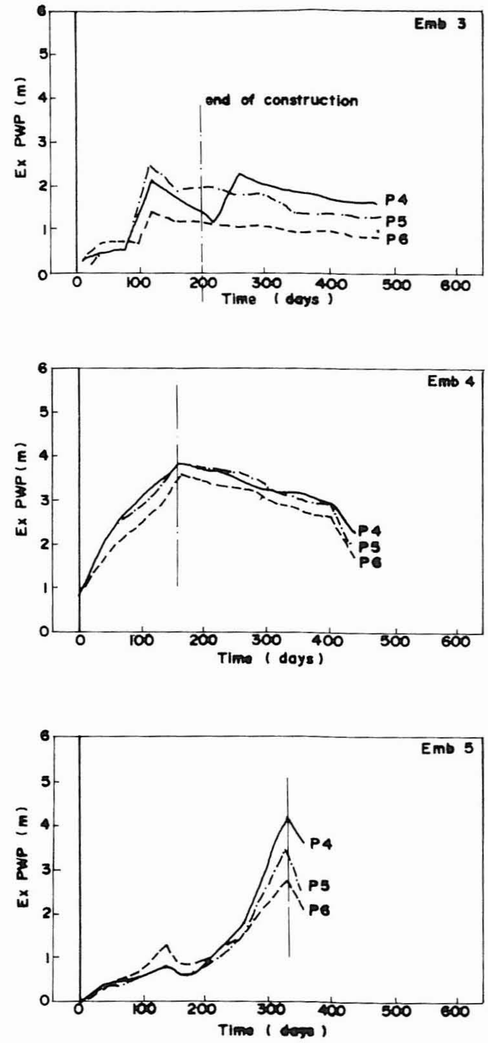
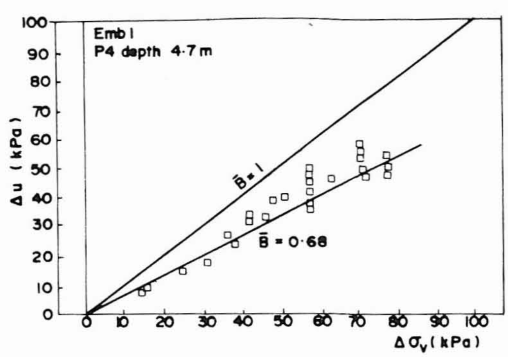


Fig. 5c. Excess pore water pressures - centreline piezometers (embankments 3, 4 and 5)



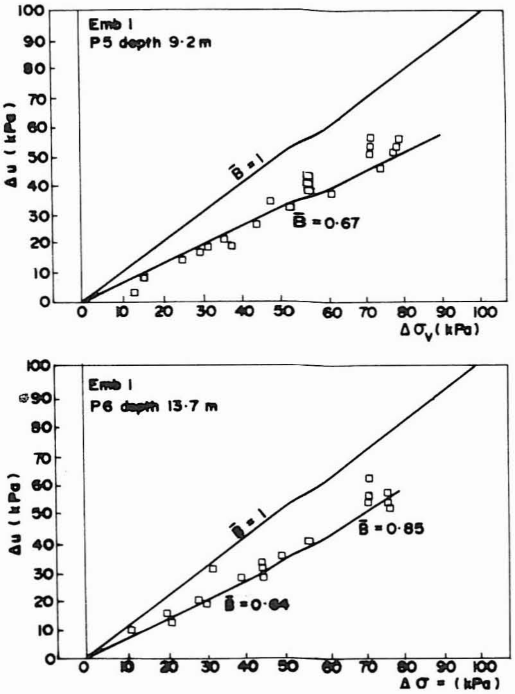
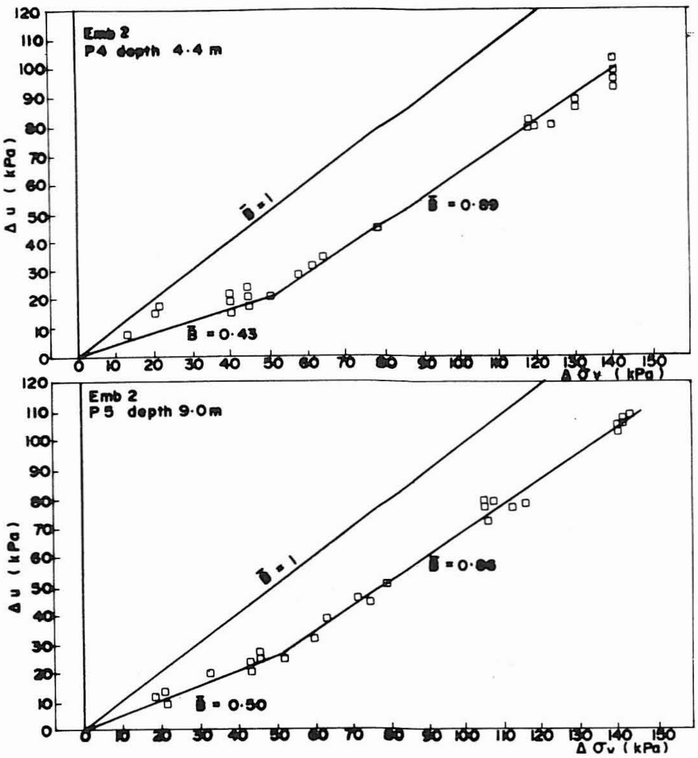


Fig. 6a. Δu - $\Delta\sigma$ *relation (embankment 1)*



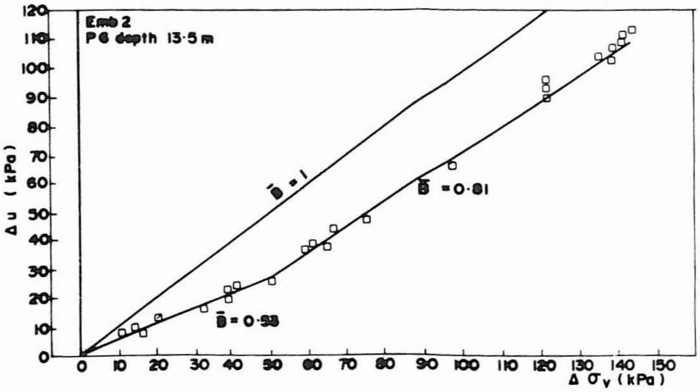


Fig. 6b. Δu - $\Delta\sigma$ *relation (embankment 2)*

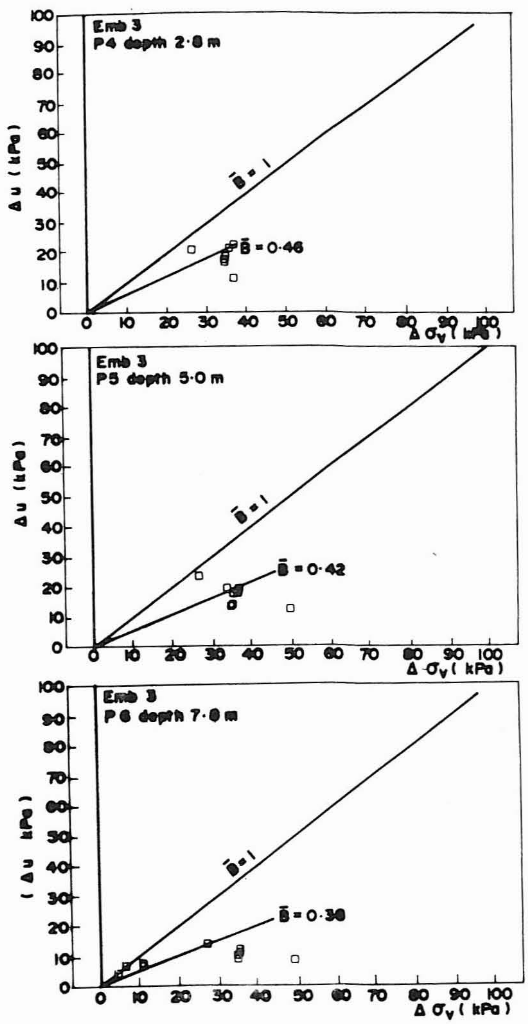


Fig. 6c. Δu - $\Delta\sigma_v$ *relation (embankment 3)*

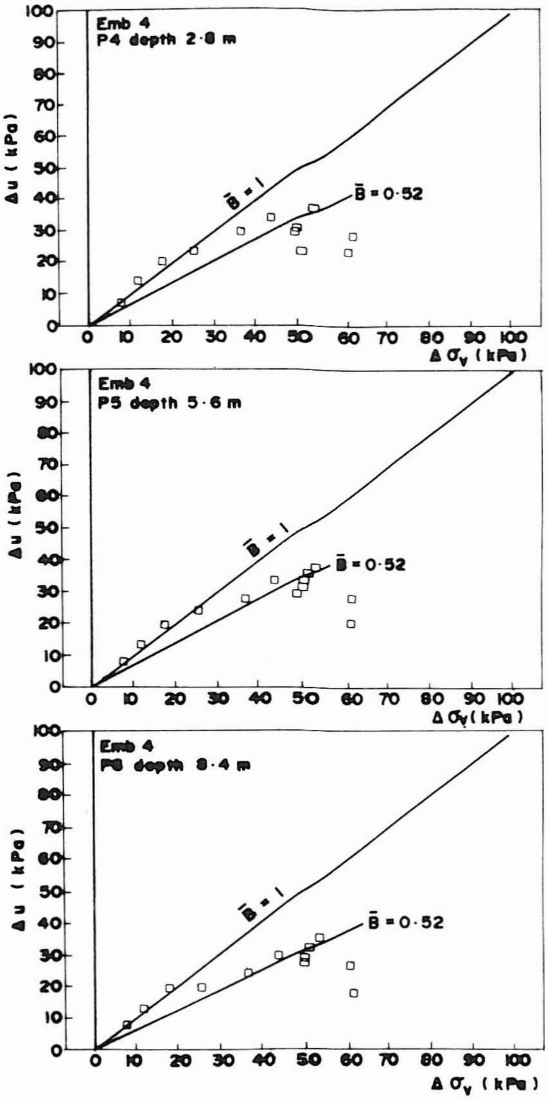
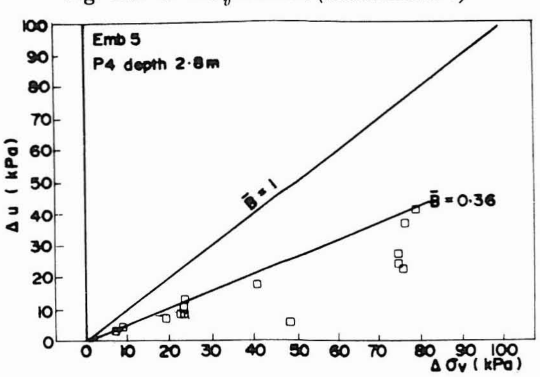


Fig. 6d. Δu - $\Delta\sigma_v$ *relation (embankment 4)*



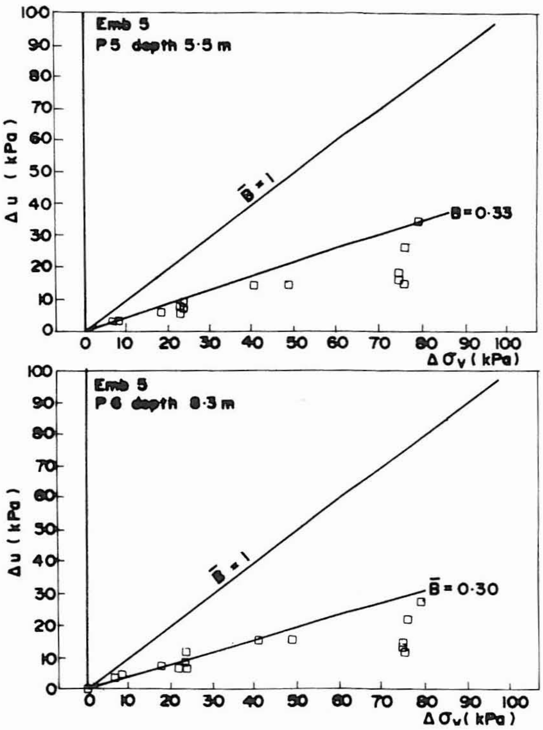


Fig. 6e. Δu - $\Delta\sigma_v$ *relation (embankment 5)*

Tavenas and Leroueil (1980) found the B = f (z) relationship assumes the shape of a consolidation isochrone, despite some scatter with relation:

$$B = \Delta u/\Delta\sigma_v = 0.6 - 2.4 \left(\frac{z}{D} - 0.5\right)$$

where z is the depth and D is the layer thickness. The plot is shown in *Fig. 7*. Superimposed on the figure are the data of the present case study. The agreement looks reasonably good. It is remarkable that such a simple relation could be found to describe the complex process of pore pressure generation and partial dissipation during construction; indeed it might be expected that B should depend on rate of construction, soil permeability and compressibility characteristics: boundary conditions, depth of the clay and other details of layering and soil properties.

At the initial stage of the construction, the settlements beneath the embankments are small (*Fig. 8 (a) and (b)*), and so are the lateral displacements. The magnitude of lateral displacement (Δy) is approximately equal to 0.08 - 0.21 centreline settlement increment (ΔS) (Table 1). The above $\Delta y/\Delta S$ relations, except those of embankments 3 and 5, are similar to those reported by Tavenas *et al.* (1979), Jardine and Hight (1987) and Suzuki (1988), who found that $\Delta y = (0.21 \pm 0.03)\ \Delta S$.

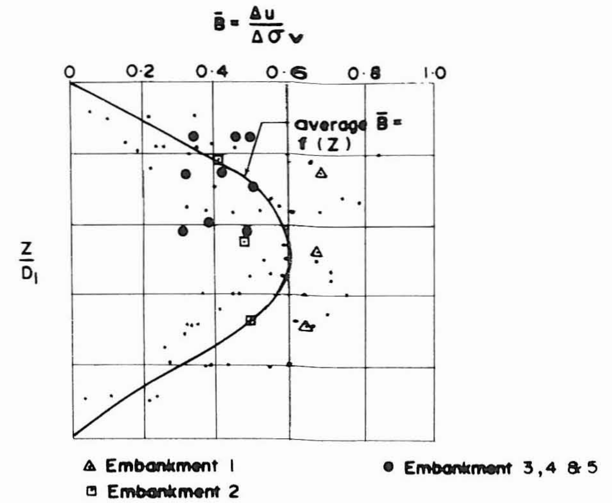


Fig. 7. Compilation of observed pore pressure in clay foundation of the early stage of embankment construction (Tavenas and Leroueil 1980)

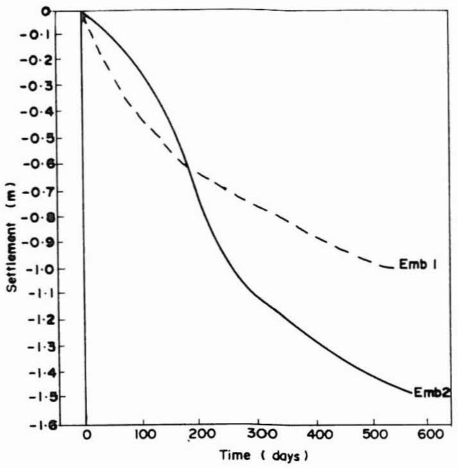


Fig. 8a. Centreline settlement of embankments 1 and 2

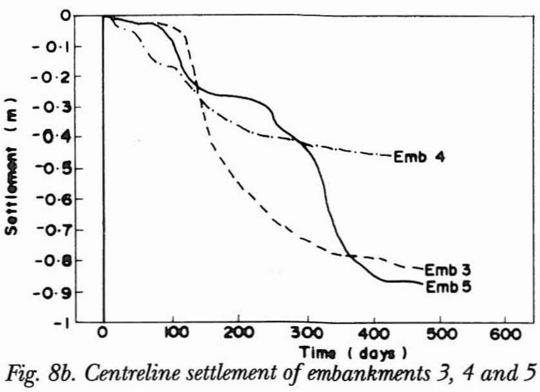


Fig. 8b. Centreline settlement of embankments 3, 4 and 5

TABLE 1
Δy/ΔS Relation

Embankment	Δy/ΔS
1	0.21
2	-
3	0.11
4	0.16
5	0.08

Embankment Threshold Height

Due to rapid dissipation of excess pore water pressure in the subsoil during the initial stage of embankment construction, the effective stresses increase rapidly to a critical state. In most cases, this condition is achieved when the vertical effective stress σ'_v becomes equal to the consolidation pressure σ c. The corresponding embankment height will be referred to as the threshold height H_c. Note that of five embankments considered, only in the high embankment (i.e., embankment 1) where $\sigma'_v > \sigma$ c, and its threshold height H_c is about 2.5 m. Data points, corresponding to σ'_v and σ_c at the threshold height H_c, are superimposed on a data base of Tavenas and Leroueil (1980) in *Fig. 9.* These lie close to the line of equality. The clay at this stage becomes normally consolidated.

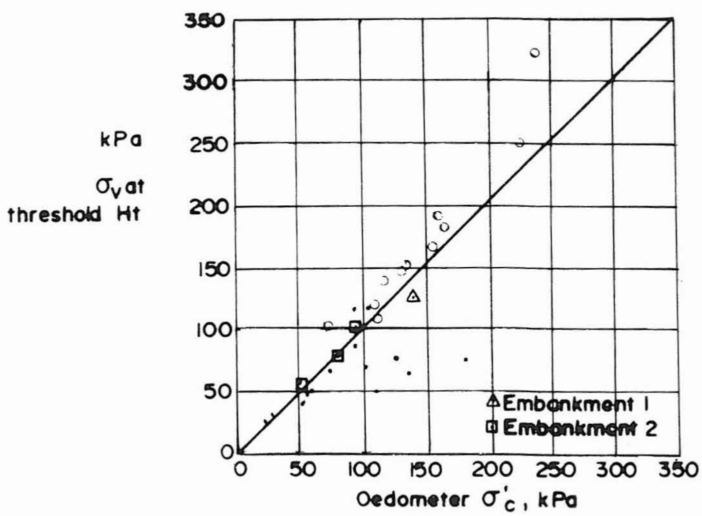


Fig. 9. Threshold efffective vertical stress from pore pressure observation and preconsolidation pressure in embankment foundation (Tavenas and Leroueil 1980)

Behaviour after the Threshold Height (during Construction)

Once part of the foundation has become normally consolidated, the properties of the clay become significantly modified. Their consolidation characteristics are significantly reduced, and further construction occurs under almost undrained conditions. According to the critical state theory (Roscoe and Burland 1968) the effective stress path then follows the critical state curve. The pore pressure increment should then be equal to the embankment load increment. This is shown in the pore pressure - vertical stress plot in *Fig. 6 (b)*. In the case of embankment 2, the pore pressure ratio after the threshold height is approximately equal to 0.8 – 0.9 $\Delta\sigma_v$.

Leroueil *et al.* (1978) found B = 1.05 ± 0.15, similar to the above. A similar observation was also made by Ramalha *et al.* (1983) and by Jardine and Hight (1987). It appears that the occurrence of B = 1.0 in the later stages of construction on clay foundation is not due to the development of confined failures as suggested by Hoeg *et al.* (1969), but merely to the passage of the clay to a normally consolidated state.

As for the deformation behaviour during this phase of construction, the clay is subjected to an undrained distortion. The compressibility of the clay is significantly increased, giving rise to large settlement and lateral displacement.

The rate of lateral displacement increase was found to increase with larger (undrained) settlement towards the end of construction. In the case of embankment 2, $\Delta y = 0.3 \ \Delta S$ (see also *Fig. 10*). This increase in the rate of lateral displacement with increase in undrained settlement has also been observed at other field sites of similar soft clays, e.g. by Marsland and Powel 1977; Tavenas *et al.* 1979; and Jardine and Hight 1987. However, in contrast to the above, Tavenas *et al.* (1979) and Jardine and Hight (1987) found $\Delta y = \Delta S$.

Behaviour after End of Construction

Embankment 2 is used to give an indication of embankment behaviour after the end of construction, as the behaviour of this embankment was observed for a longer period.

The absence of any clear break in the deformation pattern (*Fig. 10*) indicates that a shear failure is not imminent, but there are three zones of high shear strains. The post construction Δy is approximately equal to 0.33 ΔS, apparently higher than that of Tavenas *et al.* (1979) and Suzuki (1988), who found $\Delta y = (0.24)\ \Delta S$.

However, large $\Delta y/\Delta S$ one year after the end of construction has been reported by Jardine and Hight (1987). They attributed this to the effect of undrained creep.

Fig. 5 shows a pore pressure record of embankment 2. Piezometers P4, P5 and P6 installed beneath the centre of the embankment showed a continual rise in excess pore water pressure from the end of construction at Day 234 to

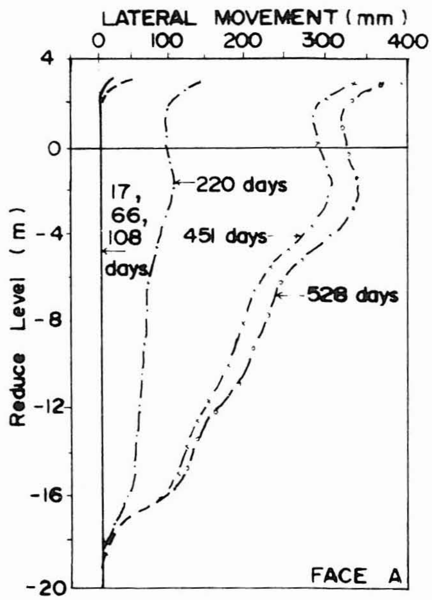


Fig. 10. Lateral deformation of embankment 2

Day 300, followed by commencement of excess pore water pressure dissipation, albeit slowly. The average degree of excess pore water pressure dissipation or consolidation U is about 15% at the end of the record, close to the value U approximated from the settlement. Piezometers P2 and P3 installed beneath the centre of the 4 m berm showed an even longer duration of excess pore pressure rise after the end of construction, from Day 234 to Day 430. This was also followed by some dissipation of excess pore water pressure, but a small rise in pore pressure just before the end of record on Day 528, as shown by piezometers P4 - P6, coincides with the addition of a little more fill at that stage. The higher degree of dissipation shown by piezometer P1, in addition to its proximity to the upper drainage layer, may also indicate a higher permeability of the upper clay layer, but this seems not to be the case, shown by piezometer P4 which was located at approximately the same depth, 4.5 m below the embankment centre.

The reason for the above continual rise in excess pore water pressure long after the end of construction is not clearly understood, but this build-up of pore pressure, notably under the embankment edge (piezometers P2 and P3), coincided with the large post-construction $\Delta y/\Delta S$ described earlier. A continual rise in pore pressure after the end of construction was also observed in 11 out of 31 case histories reviewed by Crooks *et al.* (1984), and in the centrifuge model studies of Davies and Parry (1985). Of particular interest, also, is the time taken for the piezometers to reach their peak values after the end of construction. According to Davies and Parry (1985), this increases with distance from the embankment centre. This anomalous pore pressure behaviour has been attributed by them to the effect of pore pressure redistribution. Immediately after the end of construction, the pore pressures generated in the foundation layer resulted in the formation of hydraulic gradient, in two dimensions for the case where the clay was underlain and overlain by drainage layers, resulting in flow towards these boundaries. Higher pressures generated beneath the centreline of the embankment than beneath the berm (and toe) resulted in a horizontal hydraulic gradient which led to redistribution of pore pressures in the foundation from zones of the highest excess pore pressures. Increases in pore pressure after the end of construction can also be attributed to the 'Mandel-Cryer' effect (Gibson *et al.*, 1963). This results from continuity of a consolidating layer of soil where pore pressure in the interior of the layer rises, caused by compressive force which results from the consolidation of the outer layers. However, since this effect will be most noticeable nearest to the centreline of the embankment where only a small percentage of peak value developed after the end of loading, it must be assumed that the results of Mandel-Cryer effect are, at the most, minor, and may be considered negligible. In addition, there may also be an element of progressive shearing. Embankment loading produces zones of high shear strain, which generate high excess pore water pressures. This may lower the effective stress in the zone sufficiently to permit more shear strain to develop. In turn, this shear strain results in generation of further excess pore water pressure, and strain. Unfortunately, however, there are insufficient piezometric data in the trial embankment (2) under discussion to separate the contribution towards excess pore pressure redistribution or progressive shearing. However, in the author's opinion, owing to the low permeability of the clay, progressive shearing is likely to be more dominant than that of redistribution. Of practical importance, the above indicates that delayed embankment failure can occur under these conditions. However, when noticeable dissipation began at all transducer locations from Day 430 onwards, progressive strengthening of the foundation must have resulted, giving an increased factor of safety against shear induced failure. Any local failure that may have occurred in the foundation close to the embankment shoulder should have been contained by the wide loading berm. *Fig. 11* shows an increase in soil strength with time beneath the centre of embankment 2.

Piezometric Response away from Centreline

No attempt was made by Tavenas and Leroueil (1980) to summarize pore pressure behaviour away from the centreline. Reference to the present study indicates rapidly varying excess pore water pressures under the edge of embankment 1 where potential instability was developing. The piezometric response plotted in *Fig. 12* indicated pore pressure ratio, B 0.5 for fill height

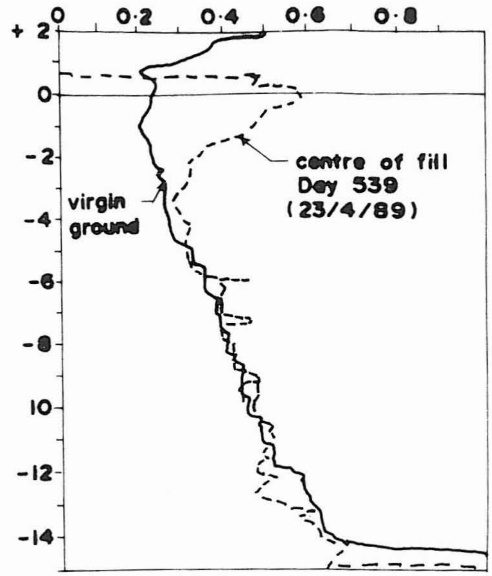


Fig. 11. Results of cone penetration testing beneath fill and in virgin ground of embankment 2

of 0 to 2.5 m (H_c), B 1.0 for fill height of H_c to completion of the berm (H = 4 m), and was followed by a substantial rise in Δu, except at the location of piezometer P1, where pore pressure dissipation apparently exceeded that of generation. Similar observations of rapidly varying excess pore water pressure under the embankment shoulder and toe have also been made by D'Appolonia *et al.* (1971), Davies and Parry (1985) and Jardine and Hight (1987).

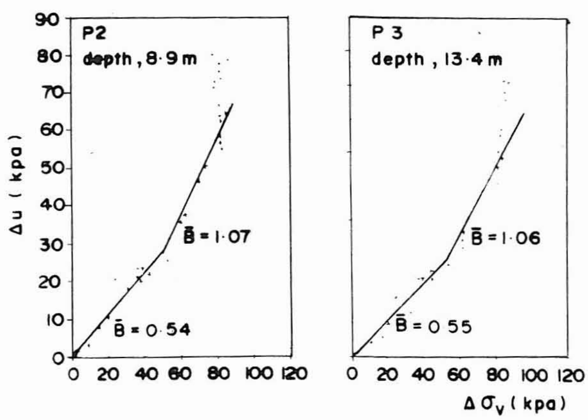


Fig. 12. Edge piezometric response of embankment 2

CONCLUSION

The main finding of an analysis of available case histories is that the clay response to construction is not truly undrained. A significant consolidation develops initially in the overconsolidated natural clay, which becomes normally consolidated during construction. An undrained behaviour develops only in the normally consolidated clay during the initial stages of the construction.

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