Practical use of clay fills in reinforced soil structures

Michael Dobie
Asia Pacific Regional Manager, Tensar International Limited, Indonesia
E-mail: tensar@attglobal.net

ABSTRACT: Clay fills have been used successfully to build reinforced soil structures in many countries and climates. For design purposes it is commonly assumed that the pore water pressure within the fill is zero, whereas in reality the actual long term equilibrium pore pressure in a clay fill above the phreatic line is likely to be negative (suction). In addition, during placement and compaction of clay fill to typical earthworks specifications, pore pressures generated in the short term under undrained conditions are also likely to be negative. This may be seen in numerous case studies of clay fills, either in laboratory tests or in actual structures where instruments have been installed to measure pore water suction. There is a popular misconception that compaction of clay fill always generates positive pore pressures, however pore water pressures only become positive towards the base of clay fills if the fills are very high, or at lower heights if the fill was purposely placed on the wet side of optimum with low undrained shear strength. The suction present in a well compacted clay fill provides an additional safety margin which is generally not taken into account during design. Design details for reinforced soil structures normally include internal drainage measures, intended to ensure that design water pressure conditions are achieved during the life of the structure. In the case of well compacted clay fill which is in a state of suction, it is important that drainage features are designed to prevent the compacted soil mass coming into prolonged contact with free water, which will tend to soften the fill resulting in swelling and reduction in undrained shear strength. In simple terms it must be appreciated that drainage features designed to let water drain out of a structure may, if incorrectly configured, let water into the structure. In extreme cases this could result in failure.

Keywords: reinforced soil, clay fill, suction, drainage measures.

1 INTRODUCTION

1.1 *Use of clay fills in reinforced soil structures*

Clay fills have been used successfully to build reinforced soil structures in many countries and climates Although generally less preferable than granular fills, clay fills often provide significant cost savings and in some locations may be the only fill available, especially in many regions of Indonesia. Provided that the requirements and limitations of their use are understood then reinforced clay fill structures may be built successfully, and provide adequate performance after construction. It must always be appreciated that clay fills have the ability to change their volume after completion (increase or decrease) to a significantly greater extent than granular fills, so if structures are highly sensitive to post-construction movement, then clay fills may not be usable. Engineering assessment of these aspects of a structure is an important part of the design process.

As with the design of any soil structure it is vital that the properties of the soil fill are well understood. The principle soil parameters required for design are generally the soil shear strength in terms of effective stress (c' and ϕ') and unit weight. It is also necessary to establish the likely pore water pressure in the fill. For reinforced soil structures which are not submerged, it is commonly assumed that internal drainage measures placed at the base and back of the fill will ensure that pore water pressures (u) within the reinforced fill and backfill are zero. Therefore design calculations are carried out using $u = 0$ in the fills, so that effective stress equals total stress. In coarse granular fills (sands and gravels), this assumption is well justified, but in clay fills it is very unlikely that pore water pressure in the fill will be zero, and in a well compacted clay fill, they are far more likely to be negative (suction), although this is generally ignored and the $u = 0$ condition is normally assumed for design.

1.2 *Outline of paper*

The aim of this paper is to examine the pore water pressure distribution likely to exist in compacted clay fill, both in the long term, as well as during and after construction in the short term undrained condition. Based on basic soil mechanics principles a general relationship is developed between the total vertical stress in a compacted clay fill and the resulting pore pressure, assuming typical compaction specifications based on achieving a target undrained shear strength. This relationship is compared to data measured both in laboratory tests as well as from instrumented structures. The results are then used to examine the issue of drainage provisions to be used in conjunction with clay fills, and examine two failures where drainage provisions may have contributed to the observed behaviour. The discussion is restricted to true clays, which have very low permeability ($k < 5 \times 10^{-10}$ m/s) and PI > 20 or so, and once compacted, become effectively impermeable.

2 LONG TERM PORE PRESSURE IN CLAY FILL

A simple cross section of a clay embankment resting on a clay foundation is shown on Figure 1. This shows the long term condition when any excess pore pressures in the foundation have dissipated, and the pore water pressure distribution has reached long term hydrostatic conditions. The embankment is 4m high and the phreatic line is 2m below the original ground level.

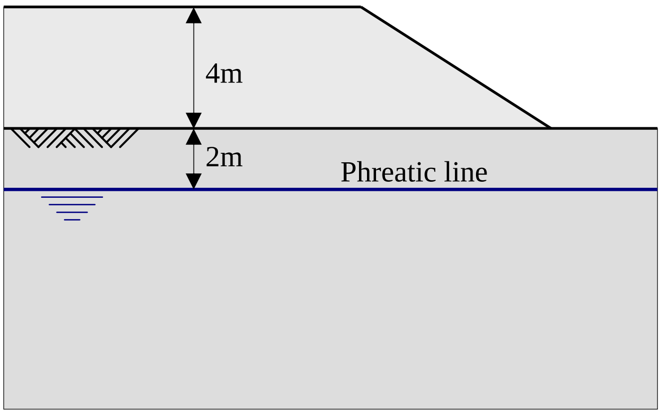

Figure 1 Clay embankment over clay subsoil

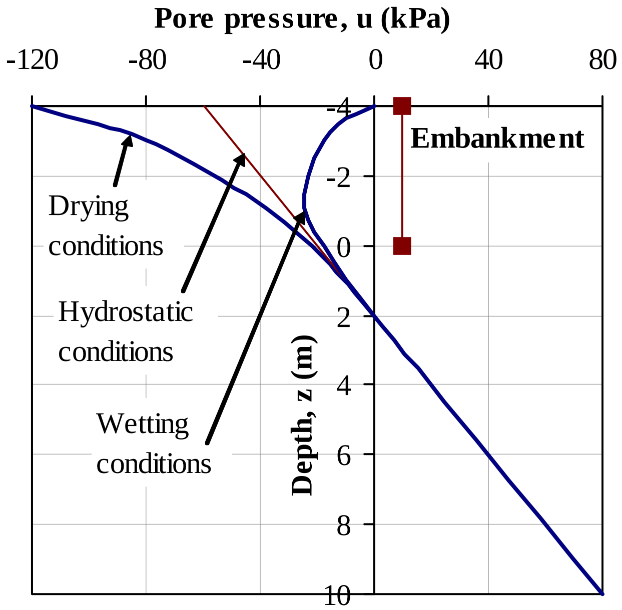

Figure 2 Pore pressure profile in clay embankment over clay subsoil

Figure 2 shows the likely distribution of pore water pressure against depth in relation to the embankment and supporting subsoil, with the phreatic line at a depth of 2m below original ground level. Below 2m depth, it can be assumed that the pore pressure profile is hydrostatic, which would be the case for any soil. Above 2m depth, for design purposes, it would be common to assume that the pore pressure value is zero. This would be the case for coarse granular soils like gravels and coarse sand which become unsaturated such that the voids above the 2m depth become mainly air filled. But clay soils are able to remain saturated (or close to saturated) for significant heights above the phreatic line. In the case of no flow (hydrostatic) the pore pressure profile must continue upwards at the same slope as the lower hydrostatic line. This is an essential requirement of assuming that no flow occurs, and implies that the pore water pressure above the phreatic line must be negative, as shown on Figure 2.

However in reality, the actual condition is unlikely to be perfectly hydrostatic, and pore pressure may fluctuate between "wetting conditions" when rainfall wets the embankment resulting in zero pore pressure at the embankment surface accompanied by downward flow and "drying conditions" when evaporation from the surface creates even greater suction resulting in upward flow. However, whichever case prevails, the long term pore water pressure distribution within the majority of the embankment is always negative.

3 PORE PRESSURE IN CLAY FILL DURING AND AFTER CONSTRUCTION

3.1 *Establishing a theoretical relationship*

According to the principle of effective stress, the shear strength of a soil is related to the effective stress on the failure plane. However when a standard UU undrained triaxial shear test is carried out on a specimen of clay, only the total stresses are known, presented on Figure 3 as the larger Mohr's circle labeled as "failure condition" and defined by σ_{3f} and σ_{1f} which are the principal total stresses at failure. The radius of this circle is the undrained shear strength, s_u. In the presentation of standard UU triaxial test data, s_u is apparently not related to the effective stress, but of course it is, and the

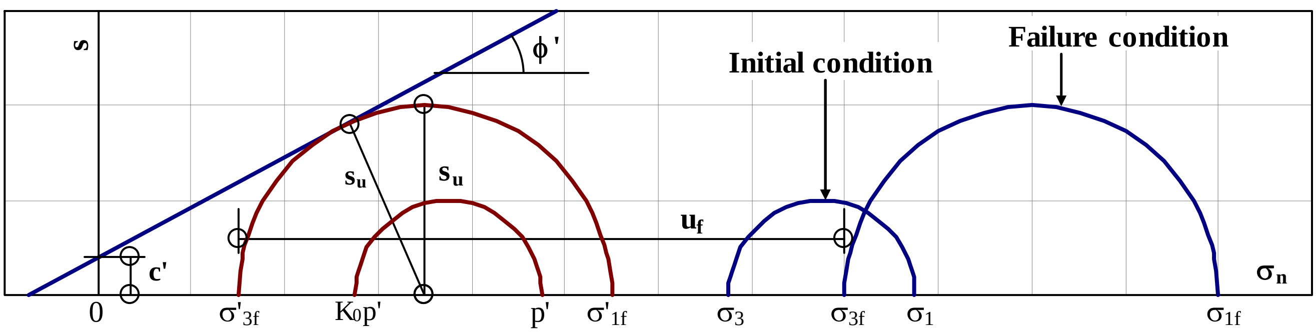

Figure 3 Relating drained and undrained shear strength in terms of total stress and effective stress

real failure criterion is given by the effective stress circle on the left coming into contact with the Mohr Coulomb failure criterion defined by c' and ϕ'. The distance between the two circles is defined by the pore water pressure at failure, u_f.

Figure 3 indicates that there must be a relationship between the undrained shear strength s_u and the effective stress parameters by c' and ϕ'. Indeed there is, and to be truly helpful, this relationship needs to be defined from the initial stress conditions in the soil element given by σ_1, σ_3 and the initial pore pressure u (as shown on Figure 3, $u = \sigma_1 - p'$). In order to create this relationship, it is necessary to make use of the following basic soil mechanics principles:

$\sigma' = \sigma - u$ (definition of effective stress)

$s = c' + \sigma' \tan\phi'$ (Mohr Coulomb failure criterion)

$\sigma'_h = K_o \sigma'_v$ (definition of earth pressure at rest)

$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$

The final expression defines the increase in pore water pressure in relation to an increase in total stress under undrained conditons using Skempton's pore pressure parameters A and B, and in the case of saturated soil, B = 1.0.

Using the geometry of the Mohr diagram in Figure 3 and some algebraic manipulation we can derive the following expression:

$$s_u = \frac{c'\cos\phi' + p'[K_o + A_f(1 - K_o)]\sin\phi'}{(1 + (2A_f - 1)\sin\phi')} \quad (1)$$

Equation 1 gives a general relationship between undrained shear strength and c' and ϕ', taking into account the initial effective overburden pressure, p', K_o and A_f, which is Skempton's A parameter at failure.

In the case of a soil specimen resting on the laboratory bench or an excavated lump of soil waiting to be compacted into a fill, we know that K_o = 1.0. In this case Equation 1 may be re-arranged as follows:

$$s_u = \frac{c'\cos\phi' + p'\sin\phi'}{(1 + (2A_f - 1)\sin\phi')} \quad \text{or}$$

$$p' = s_u\left[\frac{1}{\sin\phi'} + (2A_f - 1)\right] - \frac{c'}{\tan\phi'} \quad (2)$$

The second expression above is very useful, because it predicts the effective stress in a soil specimen of known c', ϕ' and A_f necessary to give a particular value of undrained shear strength. For a soil specimen resting on the laboratory bench, total stress is zero, so that $p' = -u$, or namely the soil suction.

In order to make use of this expression the value of A_f is required. This may be found by carrying out consolidated undrained triaxial tests, and typical values are:

Soft clay, $A_f \approx 1.0$

Elastic conditions, $A_f = 1/3$

Very stiff dilatant soils, A_f may be 0 or –ve

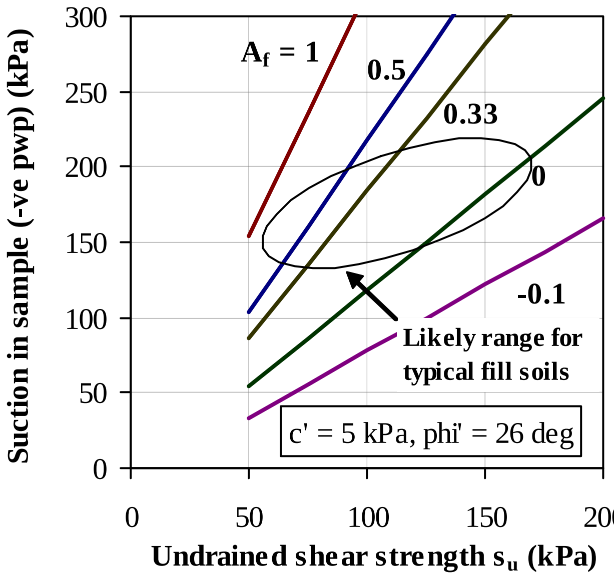

Figure 4 Relationship between suction and undrained shear strength of clay sample

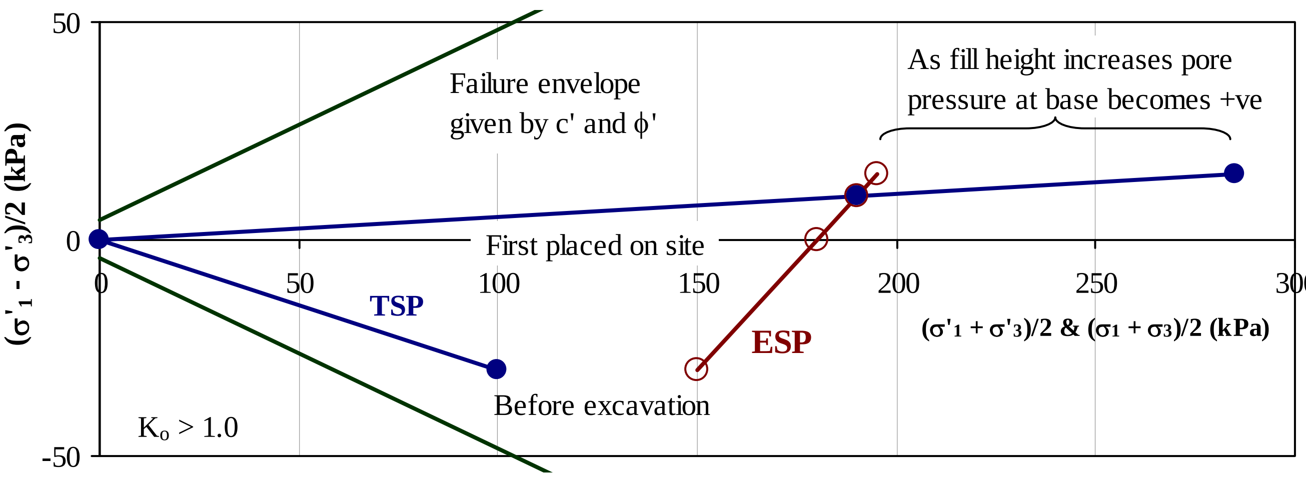

Figure 5 Total and effective stress paths during excavation, placement and compaction of clay fill

Equation 2 may be plotted as shown in Figure 4 above, in this case for c′ = 5 kPa and ϕ' = 26°. This shows the relationship between suction in a soil sample and undrained shear strength for various values of A_f for the case of saturated soil. Bearing in mind that A_f tends to become higher as undrained shear strength becomes lower, then the likely range for typical fill soils is shown by the oval area, so that suction in a laboratory specimen (or lump of soil ready to be compacted on a clay fill embankment) is relatively insensitive to the undrained shear strength and is likely to be in the order of 150 to 200 kPa.

Based on the results given in Figure 4, a stress path may be developed for the case of a clay fill excavated from a borrow pit, then placed at the base of an embankment, followed by compaction and further fill placement. In order to develop this stress path, it is assumed that fill s_u = 150 kPa and A_f = 0. The stress path is shown on Figure 5. Before excavation from the borrow pit, at about 3 to 4m depth but above the phreatic line, the likely starting stresses are shown as "before excavation" on Figure 5, which requires that $K_0 > 1.0$ and u is negative. On excavation, the total stresses both reduce to zero, and the effective stresses follow a path at 1:1 as shown, because A_f = 0 (referred to as TSP and ESP on Figure 5). This indicates a suction of 180 kPa when the clay fill is first placed, based on Figure 4. As further fill is placed the total stress increases rapidly, but the effective stress path continues to follow the 1:1 path. Assuming that total vertical stress is slightly higher than total horizontal stress, then the TSP direction is slightly upwards. When the TSP crosses the ESP, then pore water pressure becomes zero (u = 0), and on further increase of fill height, positive pore pressure is generated, in this case when p = p′ = 200 kPa, or about 10m of fill. The argument and values given above assume that the fill remains saturated.

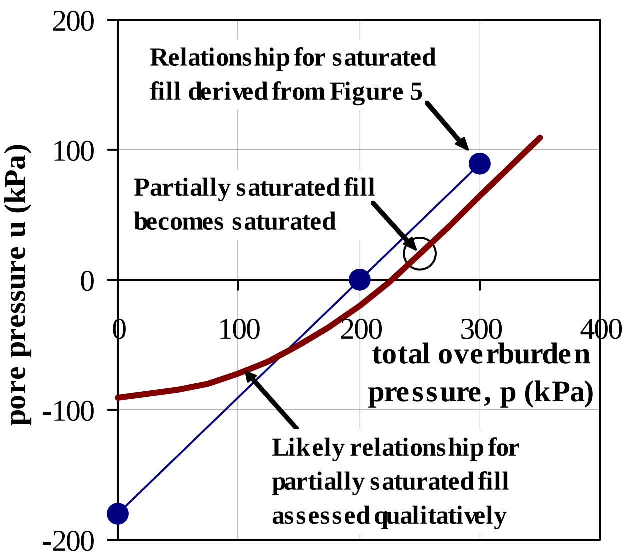

Figure 6 Relationship between pore pressure and total overburden pressure in clay fill embankment

Using the stress paths on Figure 5, it is possible to derive a relationship between total vertical stress (total overburden pressure, p) in the clay fill and pore water pressure, u. This is shown in Figure 6, by the solid black symbols. This relationship is correct for a saturated fill, and is mainly sensitive to the target undrained shear strength (s_u) of the compacted fill. In reality compacted clay fills in this situation are likely to be partially saturated on placement. This will have a tendency to reduce the suction on placement, but slightly flatten the path of the relationship as p increases. This has been assessed qualitatively on Figure 6, and is shown as the thick curved line. When p has reached a sufficiently high value, the increase in pore water pressure will cause any free air to go into solution, and the direction of the relationship above this point will be similar to the saturated case.

The relationship shown on Figure 6 is very useful because it predicts the likely pore pressure in a well compacted clay fill, depending on the depth below the fill surface. In the following section this relationship is compared to measured data.

3.2 *Comparing the theoretical relationship with measured pore pressures in compacted clay fill*

There are many published papers providing information about actual pore pressures measured in clay fill embankments. An excellent source of information is still the ICE Conference on Clay Fills held in 1978, and a number of the cases cited here are taken from the proceedings of this conference.

Penman (1978) summarises pore pressure data measured in compacted clay fill used to construct the Chelmarsh dam in UK. Clay properties and s_u are not reported, but the clay was placed on the dry side of optimum and appeared "strong" on rolling. This clay fill was used in the shoulders of the dam, so the required s_u is likely to have been high. The measured pore pressure versus overburden pressure is shown in Figure 9.

Farrar (1978) presents pore pressure data from a highway embankment constructed using compacted London Clay. A simple section is shown on Figure 7.

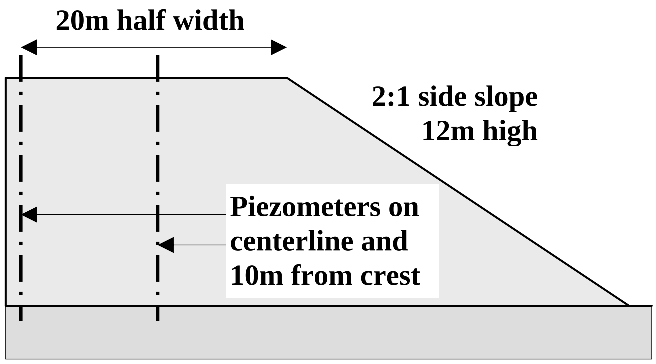

Figure 7 Section through London Clay highway embankment after Farrar (1978)

The reported properties of the clay fill are: w_P = 24, w_L = 73, unit weight = 20 kN/m^3 and water content on placement was 2 to 4% over optimum. Undrained shear strength was not reported, but the clay fill is described as being wetter than desired, so it might be expected that s_u would have been on the low side. The fill was constructed over an 18 month period, and pore water pressures in the fill were measured at end-of-construction, after 2 years and after 4 years. A detailed profile of these measurements is shown on Figure 8. This demonstrates the principles discussed in previous sections very clearly, with suction in the upper 8m of the fill, and positive pore water pressures below this level. Evidence can be seen of the "wetting condition" as indicated on Figure 2, yet the distribution of suction measured over four years is sensibly constant. However below the 8m depth, the excess pore water pressures are seen to be dissipating, albeit slowly, presumably towards a basal drainage layer. This data (except for the upper and lower points which have been affected by drainage) are plotted on Figure 9 in terms of pore pressure versus total overburden pressure.

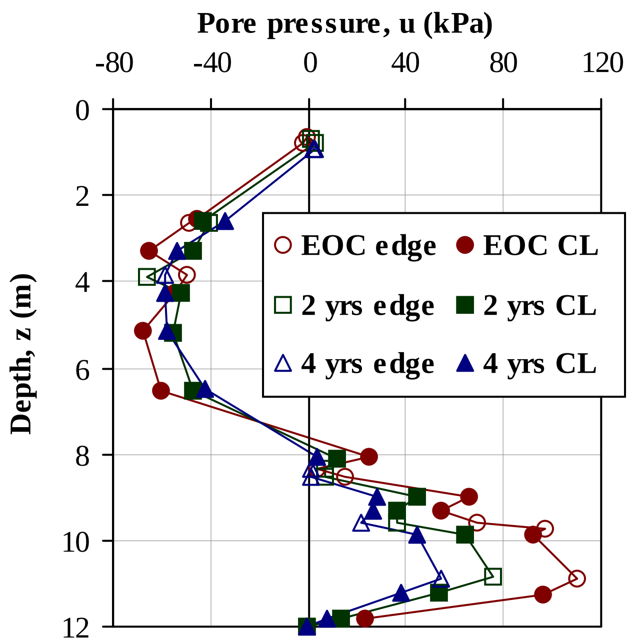

Figure 8 Profile of pore pressure versus depth in London Clay highway embankment

Vaughan et al (1978) summarise measured pore pressure data for various clay dams, as shown on Figure 10. Pore water pressure was measured in dam shoulders as shown by the solid round symbols, where s_u would be relatively high, typically around 150 kPa, as well as in dam cores where s_u would be around 50 to 75 kPa. This data comes from several earth dams, all constructed using plastic clays, but of varying properties, so not surprisingly there is quite a lot of scatter.

Both Figure 9 and Figure 10 demonstrate that the theoretical relationship between pore water pressure and overburden pressure in a compacted clay fill agrees well with measured data, and most importantly confirms that a well compacted clay fill is generally in a state of suction.

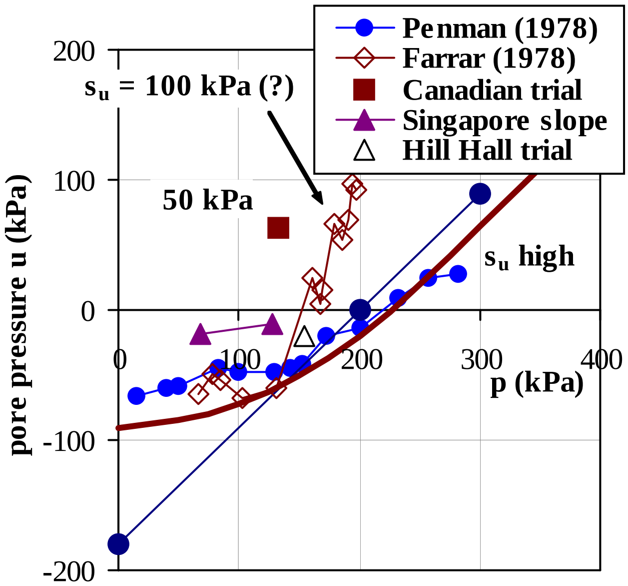

Figure 9 Predicted pore pressure in clay fill embankment compared to field data

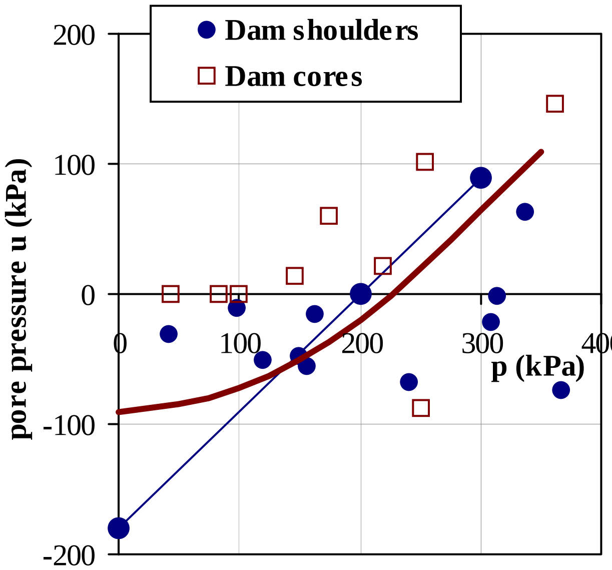

Figure 10 Predicted pore pressure in clay fill embankment compared to field data from various dams

3.3 *Comparing the theoretical relationship with data measured in the laboratory*

In addition to field measurements of pore water pressure in compacted clay fill, laboratory testing may be used to investigate this relationship. The ICE Clay Fills conference includes such data, but generally for clays of alluvial or glacial origin. Wesley (2006) presents the results of laboratory tests on compacted samples of residual soil, with properties w_P = 31, w_L = 70, and w_{opt} = 24.5%. The results are presented in terms of the water content of each compacted specimen. At w_{opt} the overburden pressure must be very high to achieve u = 0 in the specimen. As water content becomes higher than optimum, the relationship moves further to the left as may be seen on Figure 11. Although negative pore water pressures were not measured in these experiments, the results again confirm the theoretical relationship, in this case for a residual soil.

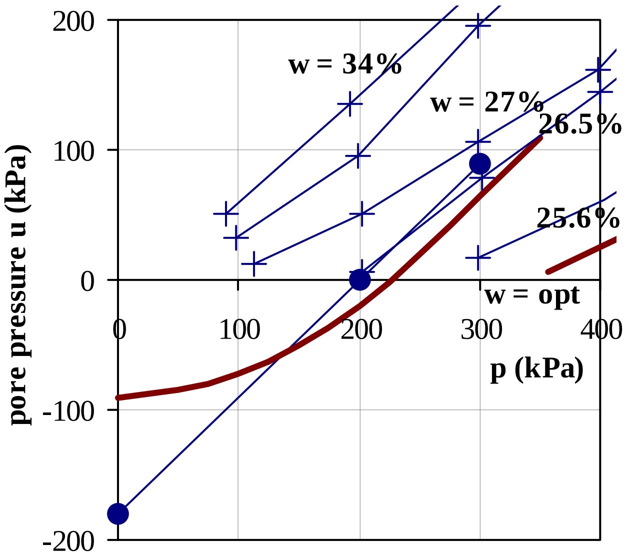

Figure 11 Predicted pore pressure in clay fill embankment compared to laboratory data after Wesley

4 PORE PRESSURE IN REINFORCED SOIL STRUCTURES BUILT USING CLAY FILL

4.1 *Canadian trial embankment*

This trial reinforced soil steep slope was built in Canada in the mid 1980's and is reported by Liu et al (1994). The structure was built specifically as a trial slope in such a way as to encourage performance approaching failure. The published paper is a good account of such a trial, providing extensive information about performance of a clay fill reinforced soil slope. Aspects summarised here are only concerned with compaction of the clay fill and pore water pressure generated during construction. A typical section through the trial embankment is shown on Figure 12.

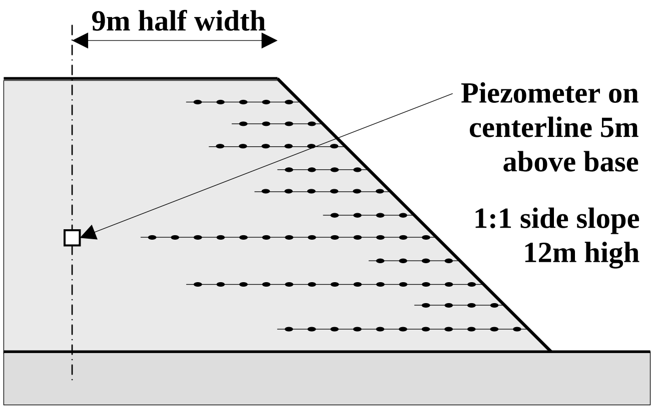

Figure 12 Section through Canadian trial embankment after Liu et al (1994)

An unreinforced control section was included in the trial, which experienced surface instability, mainly due to freeze thaw cycles. The clay fill had properties w_P = 18, w_L = 42, and w_{opt} = 22%. The compaction curve is shown on Figure 13 together with the relationship between undrained shear strength and water content. Based on this information, the compaction water content was restricted to the range 22.5 to 24% so that s_u would be in the range 30 to 80 kPa.

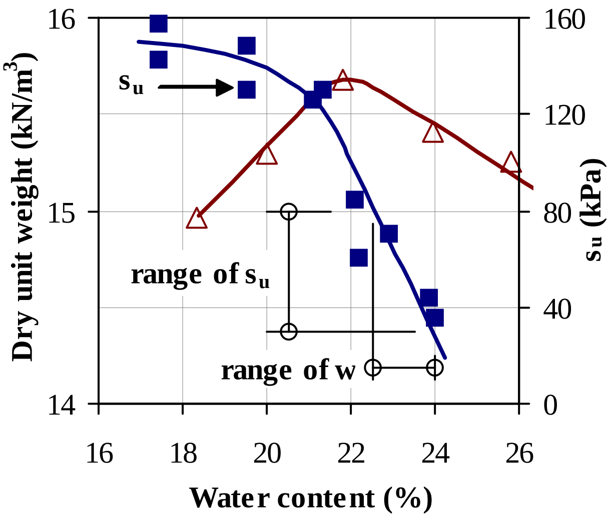

Figure 13 Compaction characteristics of clay fill

A number of piezometers were installed, and the results from one of them, installed 5m above the base on the centerline, are shown on Figure 14. It can be seen that a maximum positive pore pressure of 64 kPa was generated, but at an overburden pressure of 133 kPa. This data point has been plotted on Figure 9, and matches the general trend of the relationship, if the low s_u is taken into account.

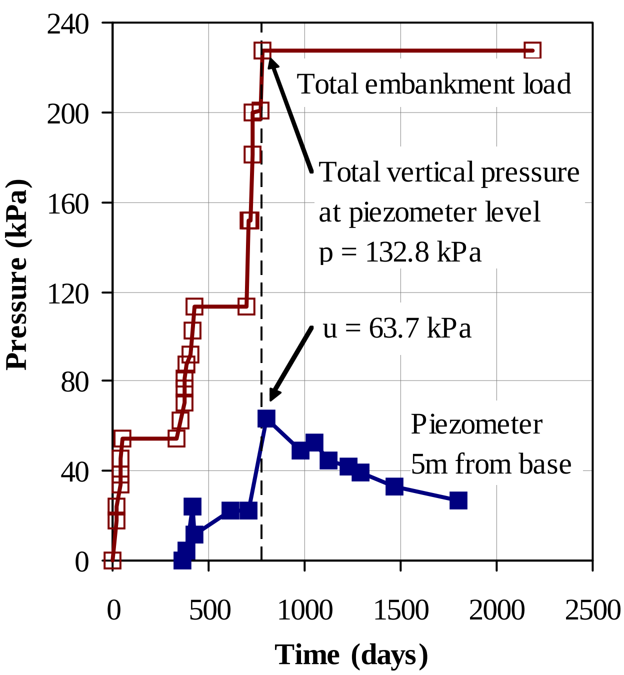

Figure 14 Pore pressure in Canadian trial embankment

4.2 *Singapore test slope*

The test slope built at Bukit Panjang in Singapore has been reported in a number of papers, but the reference used here is Wei et al (2002). This slope, like the Canadian trial embankment, is 12m high, but unlike the Canadian case it is part of an actual housing development, rather than a dedicated trial embankment. The slope is shown in section in Figure 15 which gives the principal dimensions and it was heavily reinforced. The fill used was a residual soil, most likely derived from the weathering of granite. The index properties of the fill material are not reported, but it was most probably a sandy silty clay. The fill was heavily compacted, but s_u achieved is also not reported.

One of the main aims of the test slope was to investigate the performance and benefits of using a geocomposite reinforcement, consisting of both high strength reinforcing fibres and a non-woven geotextile drainage layer.

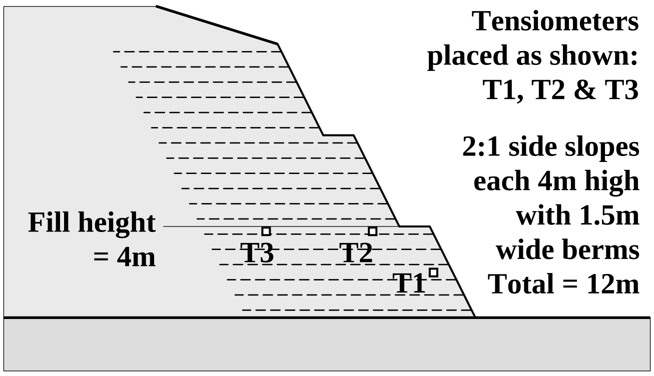

Figure 15 Section through Singapore test slope after Wei et al (2002)

A large array of instrumentation was installed to measure total stress, pore pressure, deformation and reinforcement strain. Of particular interest to this study are tensiometers installed to measure pore water suction. These are shown on Figure 15: T1, T2 and T3.

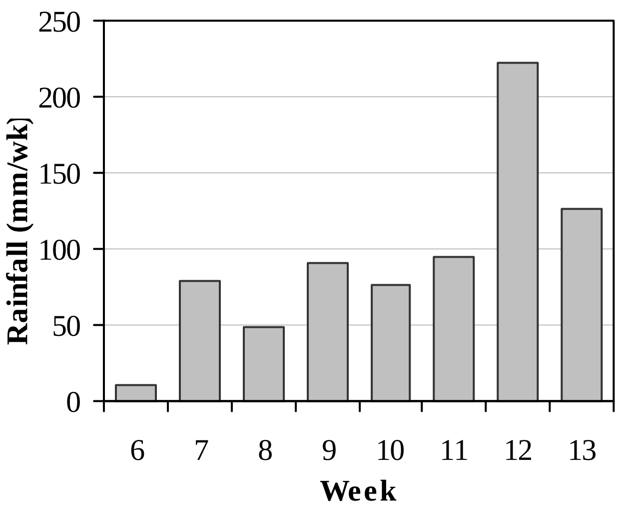

Figure 16 Rainfall at Singapore test slope site

The construction history of the slope is shown on Figure 17 (fill height versus time). The slope was built up to the 4m height, then stopped for approximately 10 weeks, during which heavy rainfall occurred, as shown on Figure 16, with a total of over 750mm of rain falling over the 10 week period. At location T1, both a tensiometer and a total stress (p) cell were installed. The suction measured in T1 reduced very slightly during this period. Over this period the data for T3 (8m from the face) is not reported. After Week 13, construction resumed, with little further effect on the total stress or suction at T1. For T3, suction values are only reported once the slope reached full height. However despite the severe inundation for 10 weeks and the addition of a further 8m of fill, T3 continued to indicate suction in the fill. Piezometers were also installed, but no positive pore water pressures were recorded. The suction and total stress values taken from Figure 17 are plotted on Figure 9, and follow the general trend of the other reported data.

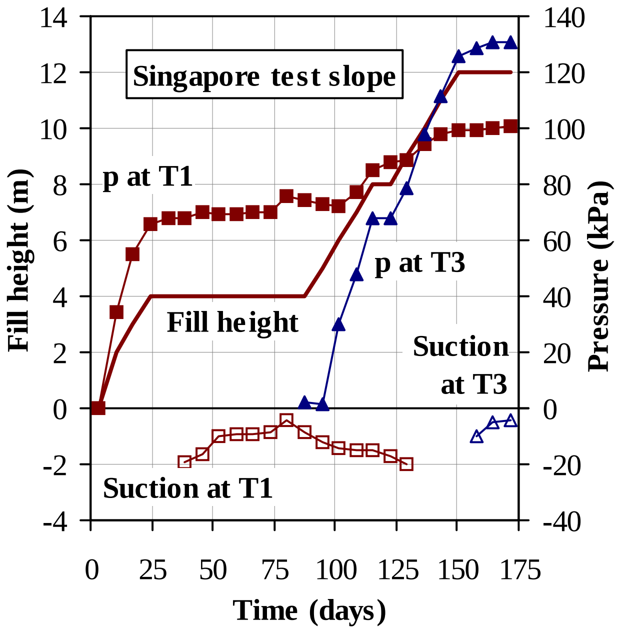

Figure 17 Fill height and pressure versus time

The first conclusion of the Wei et al (2002) paper states that “inclusion of a geotextile with high in-plane drainage capability is critical for the rapid dissipation of excess pore pressure in the soil. This is supported by the reasonably high suction values in the soil despite heavy rainfall and high ground water.” The suggestion here is that the presence of the geotextile drainage layer is responsible for the measured suctions. This seems to be very unlikely: the water pressure in the geotextile will be atmospheric or higher (if water is draining out) so that flow must be towards the tensiometer rather than away from it. If flow is towards the instrument, then the geotextile layer cannot be responsible for maintaining the suction, and in fact at the best it would only maintain pore pressure at zero or slightly higher. The second part of the second conclusion to the paper states that "the low permeability of residual soil prevents a rapid infiltration of water into the soil, hence preventing drastic decrease of suction". In the opinion of the author, this is the important conclusion, and the suction in the fill was created during placement and compaction as outlined in Section 3 of this paper.

5 EXAMINATION OF TWO EARTHWORKS FAILURES ASSOCIATED WITH CLAY FILL

5.1 *Hill Hall embankment, United Kingdom*

The Hill Hall embankment failure occurred in 1981, and is reported by Finlayson et al (1984), although much important information is given in the discussion published in 1985. The main issues arising are very relevant to the subject of this paper, and some of the important results are summarised below. The author was part of the team which investigated the failure. Plate 1 shows a general view of the failed embankment, and Figure 18 shows a simplified cross section. The failure occurred during construction of a London Clay embankment over a London Clay foundation, separated by a drainage blanket. As can be seen on Figure 18, the failure took place along a very gently inclined failure surface (the embankment was built over gently inclined sidelong ground). Conventional analysis of this section would predict high factors of safety.

Plate 1 Failure of the Hill Hall embankment

As part of the investigation, an incomplete (4m high at the time) nearby embankment was used as a trial, built to the same specifications, but with piezometers installed in the foundation, the drainage blanket and the base of the clay fill. The results of the trial embankment are shown on Figure 19 below. This figure deserves careful study. The records of the embankment construction and the pore water pressures are all given in terms of their elevation. The three small square symbols indicate the elevations of the three piezometers (P6 in the foundation, P7 in the drainage blanket and P8 in the base of the clay fill). The water pressure traces for each piezometer are also labeled P6, P7 and P8. Therefore if the pore pressure elevation drops below the elevation of the relevant piezometer, then the pore pressure is negative (suction) at the piezometer.

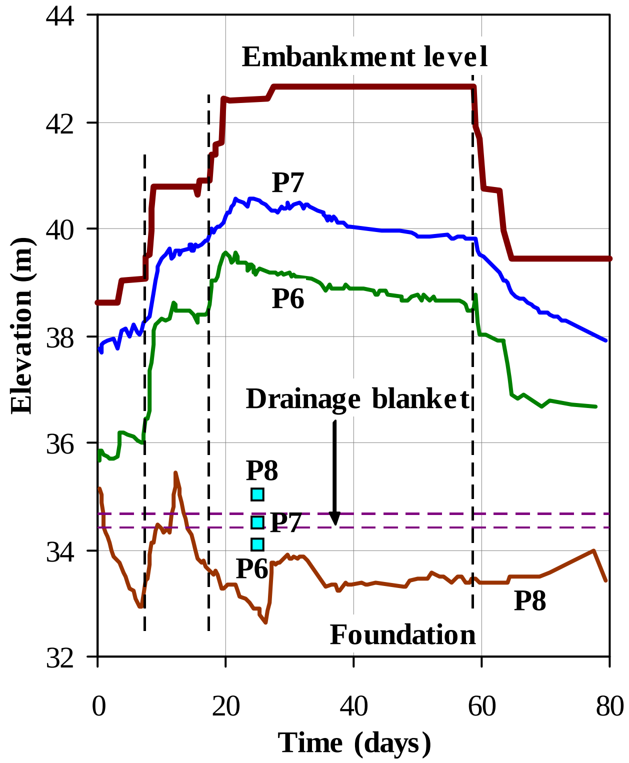

Figure 19 Pore pressure from trial embankment

Examination of this data indicates that the base of the clay fill is largely in a state of suction, the top of the foundation has relatively high excess pore water pressure, but the highest pore pressure of all is in the

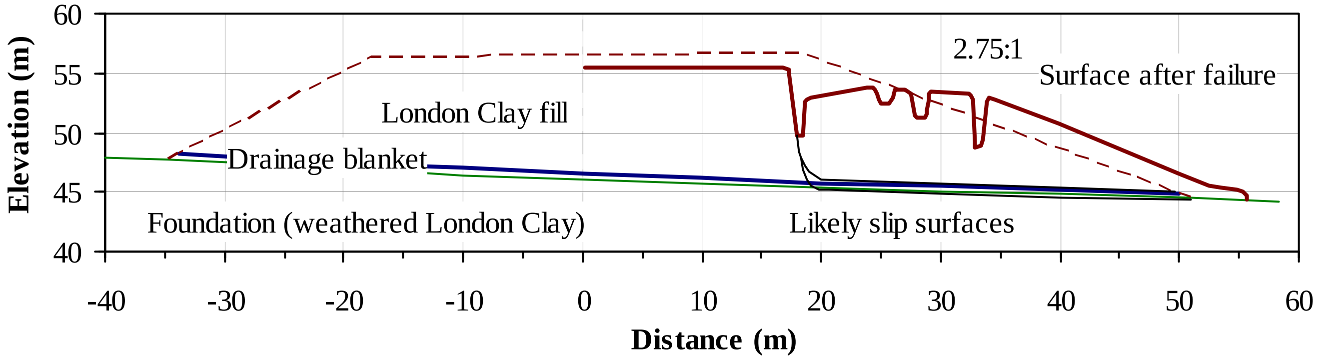

Figure 18 Section through Hill Hall embankment on the M25 near London, UK

drainage blanket. These results appear to be completely the reverse of what would be expected. Investigation of the failure established that the drainage blanket was a poorly graded material with a high content of fine sand and had very low permeability. The sequence of events was as follows:

- The foundation soil was a field, probably with low suction values near ground surface (as outlined in Section 2.0 of this paper) before construction started.
- A low permeability drainage blanket was placed over the field, but coarse enough to become saturated by heavy rain, so pore pressure u = 0 before any fill was placed.
- Very well compacted fill in a state of high suction was placed over this drainage blanket very rapidly.
- Pore water pressure in the saturated drainage blanket rose rapidly in response to the undrained loading and acted as source of water to soften both the fill and the foundation soils. This excess pore pressure dissipated slowly (see Figure 19), but was still over 40 kPa excess after 40 days. This was due to the low permeability of the drainage material and the long drainage path for dissipation to take place.
- Sliding occurred in the base of the fill and the top of the foundation, due to the high pore water pressure generated within the drainage blanket causing softening of the clay material.
- A geotextile had been used to separate the fill from the drainage blanket. It did not contribute to instability directly, but its installation created a relatively planar surface at the base of the fill helping to encourage the planar slide which took place.

As suggested by title of the published paper, important lessons were learned from this failure, in particular that drainage blankets must be genuinely free draining and should be arranged so that they can drain easily (ie remain empty). Another lesson is that having free water in contact with well compacted clay (which is in suction) can result in rapid softening at the edges of the clay fill, reducing expected safety margins of the earthworks. In this case the clay fill had properties w_P = 20-26, w_L = 45-72, and w of the compacted fill was normally = 17-33%. However, detailed investigation of water content close to the drainage blanket after the failure indicated values of 40% or higher.

5.2 *Jalan Veteran, Jakarta*

Photographs similar to Plate 2 appeared in the news papers in Jakarta in March 2008. The supported road is the Jakarta Outer Ring Road, near Tanah Kusir, in Jakarta. The retaining wall visible between the sandbags is a reinforced soil structure built using a modular block facing technique (MBW). Behind the blue and orange sheets, the facing has fallen off the wall. The fill is almost certainly "tanah merah" (tropical red residual clay soil derived from the weathering of volcanic ash deposits). The author is aware of many similar retaining walls used to support other sections of the Jakarta Outer Ring Road very successfully, but in the case of this section, had no involvement at all. However, the structure is in a public place, and for a while could easily be inspected. Based on observation at the site, the following could be seen:

- The reinforced soil structure as a whole has performed as intended. Although the left-hand lane of the highway was coned off at the time this photograph was taken, there was no sign of significant deformation to the road pavement, and no sign of any general collapse.
- The main issue is that the facing had become detached from the fill.
- Facing details: based on inspection of the site, there was no sign of mechanical connectors being used to attach the facing to the layers of geogrid which form the reinforced soil structure. Therefore it appears that the connection was mainly frictional, namely with the geogrid resting between the blocks and relying on the weight of the column of blocks above to "clamp" the ends of the geogrid and form the connection.
- Drainage details: it could be seen clearly that the gravel drainage aggregate placed behind the facing (which is a normal detail for MBW systems) extended up to the top of the wall, and could be seen daylighting behind the tops of the facing blocks. A concrete channel drain had been installed, but of relatively small dimensions. The concrete channel was resting within the top of the gravel drainage material, so if water overflowed, it would permeate into the gravel drain.

Plate 2 Jalan Veteran MBW facing failure

The author had no formal connection with this project, and is not aware of any investigation which may have been carried out. However, the failure was highly visible and clearly such pictures are alarming. Making use of the ideas and information presented in this paper, the following mechanism is suggested as one possible reason for the failure taking place (and is depicted on Figure 20):

- The exposed height of the wall at the point of failure was about 6m. A 6m thickness of well compacted tanah merah would be in a state of suction (see Section 3 of this paper).
- Rainfall running off the road pavement (the failure occurred during the rainy season) would run into the side drains, but due to the arrangement of the gravel drain, much of this water would tend to enter the gravel drain and run down between the compacted clay fill and the back of the facing blocks.
- The high suction in the tanah merah fill would result in flow into the fill from the water within the drain. This flow would result in softening of the clay. The softening would be accompanied by an increase in volume, causing deformation both outwards and upwards.
- This deformation would tend to reduce the downward force in the column of facing blocks (thereby reducing the frictional connection strength), and the outward force would push the blocks forwards, in this case sufficiently to cause the facing to collapse.
- Based on visual inspection at the site, it was clear that the front of the tanah merah backfill was very wet.

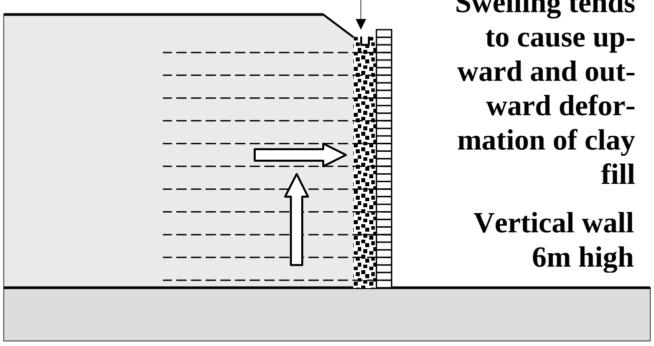

Figure 20 Suggested mechanism for facing failure at Jalan Veteran

Reinforced soil structures with clay backfill are used widely in Indonesia, so it is vital that lessons learned from such a failure are taken into account in future designs. In this case, one issue is connection type and strength between the geosynthetic reinforcement and the facing, which is outside the scope of this paper. Issues (and lessons learned) relating to drainage details are outlined in the next section.

6 OUTLINE OF IMPORTANT DRAINAGE FEATURES FOR REINFORCED SOIL STRUCTURES

The aim of the final section of this paper is to outline important drainage features for reinforced soil structures, with special reference to the use of clay fills. This is not intended to be a specification, but it gives the main aims and features that need to be considered as part of the design of a reinforced soil structure, with reference to Figure 21:

- Internal drainage: should be designed to intercept internal ground water flows (in this case there is considerable difference between back-to-back highway embankment structures where the only likely source of water is run-off and structures built up against an existing hillside, where ground water flows may exist within the retained soil mass).
- Internal drains (including back-of-facing drains) should not daylight at the upper surface of the structure, which would then permit ingress of run-off into the fill. This is a common mistake, but is undesirable.
- Internal drainage: should be installed and maintained as free-draining and arranged so that water can drain out easily and the drain can stay essentially "dry". It is important that internal drains should not remain full of water for prolonged periods, which might be in contact with clay fill resulting in softening. Consideration may be given to outlet pipes.
- In relation to the previous point, in cases where the embankment may settle, then settlement at the toe may well be less than under the centre of the embankment, so that an initial cross-fall running outwards could later reverse itself and run back into the fill.

Internal drains should not allow inflow of surface run-off and should be arranged to remain empty

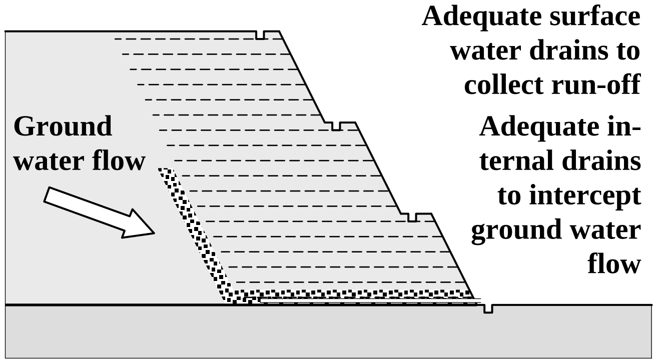

Figure 21 Section through reinforced soil slope indicating important drainage features

- Surface water drainage: should be adequate to intercept and remove surface water run-off. Cross falls and gradients should be adequate and areas of ponding avoided. Leakage from damaged pipes or tanks should be considered in maintenance procedures.

7 DICUSSION AND CONCLUSIONS

This paper summarises a considerable amount of data about the performance of clay fill earthworks and structures, in particular the pore water pressures likely to exist in such fills after placement. Economy and fill availability are often likely to dictate that such fills are considered for use in both general earthworks and reinforced soil structures. Based on the information presented, the following should be taken into account when designing earthworks using clay fill.

- Clay fill compacted to a normal earthworks specification is likely to be in a state of suction (negative pore water pressure) up to considerable heights. Pore pressures are only likely to become positive at the base of very high fills (in excess of 10 to 15m), or at lower heights if the clay was on the wet side and therefore soft during placement.
- The suction in the clay fill is generally ignored in design, and provides an additional margin against failure or poor performance of the structure. However it is highly desirable to maintain this suction in the long term, and to avoid or minimise situations or conditions which would result in loss of these suctions. By the principle of effective stress, loss of suction implies a reduction in effective stress, therefore a reduction in (undrained) shear strength and an increase in volume.
- Therefore once established, it is desirable that the boundary conditions around a mass of compacted clay fill should be arranged so that suctions are maintained. It is fortunate that both the short term and long term equilibrium pore water pressures in a compacted clay fill are likely to be negative, which means that the potential for long term disturbance of the initial conditions is very small.
- The principal boundary condition of concern is the access of free water to the surfaces of the fill. Drainage measures of various types are likely to be required around any earthworks, but in the case of clay fills, it is important that details are arranged so that free water has little chance to come into contact with the outer surfaces of the compacted mass of fill.
- The final point relates to the advisability of using geosynthetic reinforcement which incorporates a geotextile drainage layer in clay fill (see Section 4.2). This idea has been promoted widely, and has even been suggested as being “essential” in clay fills “to dissipate the excess pore water pressure”. It is the opinion of the author that any design situation should be assessed on its merits. There may well be cases where very wet fills may be used to construct earthworks, and in this case having a regular pattern of thin drainage layers may well be of great benefit. However in the more common case of a clay fill compacted to normal earthworks specifications, where the pore water is likely to be in a state of suction, then a regular pattern of fine drainage layers may well provide ready access for external water to penetrate well into the fill and exacerbate any swelling and softening which might take place. In these situations, a geosynthetic reinforcement of this type is better avoided.

ACKNOWLEDGEMENTS

I would like to acknowledge the influence of two people in preparing this paper: firstly the late Professor A W Bishop, whose excellent lectures on soil properties during my soil mechanics course at Imperial College provided the background to Section 3.1; and secondly Dr Laurie Wesley who is well known to many Indonesian engineers and with whom I have held many fruitful discussions about soil mechanics and in particular clays and clay fills. I am also very grateful to Dr Wesley for reviewing the paper.

REFERENCES

Farrar, D.M. 1978. Settlement and pore-water pressure dissipation within an embankment built of London Clay. In Institution of Civil Engineers, *Clay fills*: 101-106, 14-15 November 1978: ICE, London.

Finlayson, D.M., Greenwood, J.R., Cooper, C.G. & Simons, N.E. 1984. Lessons to be learnt from an embankment failure. Proceedings Institution of Civil Engineers, Part 1, 1984, 76, Feb, 207-220, and later discussion in Proceedings Institution of Civil Engineers, Part 1, 1985, 78, 1211-1235.

Liu, Y., Scott J.D. & Sego, D.C. 1994. Geogrid reinforced clay slopes in a test embankment. Geosynthetics International, Vol 1, No 1, 67-91, Industrial Fabrics Association International, USA, January 1994.

Penman, A.D.M. 1978. Construction pore pressures in two earth dams. In Institution of Civil Engineers, *Clay fills*: 177-188, 14-15 November 1978: ICE, London.

Vaughn, P.R., Hight, D.W., Sodha, V.G. & Walbancke, H.J. 1978. Factors controlling the stability of clay fills in Britain. In Institution of Civil Engineers, *Clay fills*: 205-217, 14-15 November 1978: ICE, London.

Wei, J., Chua, K.C., Lim, S.K., Chew, S.H., Karunaratne, G.P., Tan, S.A. & Seah, Y.T. 2002. Performance of a geosynthetics reinforced steep slope in residual soil. In Delmas, Gourc & Girard (eds). *Geosynthetics*: Proceedings 7th International Conference on Geosynthetics, 325-328, Nice, France, 22-27 September 2002.

Wesley, L.D. 2006. Comments on the use of clay in geosynthetic reinforced retaining walls. Proceedings of Technical Meeting of the Indonesian Chapter of the International Geosynthetics Society, Jakarta, Indonesia, 12 September 2006.