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Geotechnical Engineering in and out of the Ivory Tower

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Summary: An account is given of the results of various research activities undertaken since the writer's move from geotechnical engineering in the outside world to the "ivory tower" of Auckland University. The motivation for much of this research arose from practical situations encountered while working outside - in the employ of government agencies and an Auckland consulting company. The research described covers retaining forces for steep slopes, back analysis applications, seepage studies, and the behaviour of compacted clay and clay embankments. A large number of students from many parts of the world, mostly postgraduate, but including some undergraduates, have contributed to the research described in this paper.

INTRODUCTION

Geotechnical engineers, especially those who work in the consulting world, know only too well that time and financial constraints impose severe restrictions on the opportunities available to them to do much more than satisfy the immediate expectations of client and employer. Taking time out from the daily demands of the office to look a little deeper into some of the technical problems and issues encountered along the way is not an easy matter. As the title of this lecture implies, the writer has worked "outside" the ivory tower - in the employ of government agencies in both New Zealand and Indonesia, and for a consulting firm specialising in geotechnical engineering; that experience forms the background to this paper. The consulting world is both a stimulating and demanding environment, and I have pleasant memories of my time as part of it. At the same time I found the constraints of time and cost somewhat frustrating, because of the many occasions when I would have liked to step aside for a few days, or maybe longer, to think about and investigate issues relating to particular jobs I was involved with.

The university environment, at least until recently, has been more relaxed, and much less controlled by time deadlines and profit margins. Research has always been one of its functions, and spending time thinking about and debating issues part of its ethos. Since retreating into the "ivory tower" about 17 years ago, I have attempted to make use of the opportunity this provides to investigate some of the technical questions which challenged or interested me during my existence in the outside world, and which I always intended investigating further, but never found the time to do so. The results of some of these investigations are described in this lecture.

SLOPE STABILITY AND BACK ANALYSIS STUDIES

Retaining forces for cuts in steep slopes

A problem that geotechnical engineers come up against from time to time is how to determine the forces needed to support vertical or near vertical cuts in steep slopes. Sometimes these cuts are made simply to provide extra level ground at the rear of a house site, or to make room for extensions to a factory or some other building. In many cases there are sometimes very steep and clearly without large safety margins against slip movement, even before the cut is made. They may be very large slopes, essentially "infinite" as far as theoretical analysis is concerned. Estimating the forces needed to retain cuts in such slopes is often a difficult undertaking, especially if the slope material is not homogeneous, and groundwater conditions are uncertain and change with time. Slopes in residual soils in particular may consist partly of soil and partly of highly weathered rock, containing considerable coarse material, so that the challenge of measuring or assigning shear strength parameters to the material is quite formidable.

The particular job that made me give some deeper thought to this problem was a proposed new highway in Malaysia, from a place called Kerling (between Kuala Lumpur and Ipoh), to the Fraser Hill holiday resort area. The planned route passed through rugged hill country with steep slopes, on some of which large translational slides had occurred in the not too distant past. Any cuts made would need to be retained with tied back walls if large scale instability was to be avoided, and estimates made of the forces which could be expected to act on the walls. I suppose the orthodox way to approach this situation would be to sample the soils and measure their strength parameters in order to undertake a Coulomb wedge analysis, or use the parameters in some ready-made formula for earth pressure. Sampling and testing the soils was not an attractive proposition. Getting drilling rigs...
into the area was difficult enough, given the usual budget and time constraints, and the rather heterogeneous nature of the residual soils involved made testing and selection of representative parameters difficult.

Figure 1. The problem addressed.

Rather than attempt measurement of, or guess at, the soil strength parameters, it seemed that back analysis might prove a more productive approach, so I set out to do this. Figure 1 illustrates the problem. The slope is considered to be of unlimited extent, and the water table depth dependent on seasonal and weather conditions. The starting point of the analysis was the assumption that the slope was of infinite extent, and was in a state of limiting equilibrium. From these assumptions soil strength parameters can be obtained by back analysis; these can then be used in a conventional Coulomb wedge analysis to determine the required force. An assumption has to be made about the depth of the water table and one of an infinite number of possible combinations of $c'$ and $\phi'$ must be selected.

The analysis produced a rather surprising result, at least at first sight, namely that the resultant force became less as the assumed slope angle was increased. This didn’t seem right; along with most geotechnical engineers, I had the idea that the steeper the slope behind a retaining wall the greater the force on the wall. However, I think I persuaded myself that the analysis was essentially correct, and used its results in a report recommending force levels for the design of the retained cuts. Time did not permit a fuller investigation of the method and a thorough check of its validity. After moving to the university, I had a project student look into the method and repeat my analysis, looking at a wider range of cases. His analysis fortunately came up with results essentially the same as my earlier results, and I will describe briefly the way the analysis is done.

Figure 2. Assumed situation of an infinite slope at limiting equilibrium.

Figure 2 illustrates the basic assumption of an infinite slope, which is assumed to have a safety factor of unity. Possible failure on a plane at depth $H$ is postulated and the equilibrium of the layer of soil above this depth is investigated. The ground water level is at a depth $H_w$ below the surface, and seepage is assumed to be parallel to
the surface. Equipotential lines are thus perpendicular to the surface. Static analysis of the equilibrium of the soil mass above this possible translational failure plane at depth \( H \) does not produce a unique set of strength parameters, only a range of possible combinations of \( c' \) and \( \phi' \).

The expression for the safety factor is:

\[
S.F. = \frac{c'}{\gamma H \cos \beta \sin \beta} \left[ 1 - \frac{\gamma_w}{\gamma} \left( 1 - \frac{H_w}{H} \right) \right] \tan \phi' \tan \beta
\]  
(1)

For the case of limiting equilibrium, \( S.F. = 1 \), this becomes:

\[
\frac{c'}{\gamma H \cos \beta \sin \beta} = 1 - \left[ 1 - \frac{\gamma_w}{\gamma} \left( 1 - \frac{H_w}{H} \right) \right] \tan \phi' \tan \beta
\]  
(2)

For the case of a slope in which no water table is present, the expression becomes:

\[
\frac{c'}{\gamma H \cos \beta \sin \beta} \tan \phi' \tan \beta
\]  
(3)

And for the case of a slope with a water table at the ground surface the expression becomes:

\[
\frac{c'}{\gamma H \cos \beta \sin \beta} = 1 - \left[ 1 - \frac{\gamma_w}{\gamma} \frac{\tan \phi'}{\tan \beta} \right]
\]  
(4)

where \( \gamma \) and \( \gamma_w \) are the unit weights of the soil and water respectively.

These equations are essentially the same as those given by Taylor (1948), in a slightly different form. They show clearly that for a given value of \( \phi' \), the value of \( c' \) needed to maintain equilibrium is proportional to the depth \( H \), as pointed out by Taylor (1948). By making an assumption about the water table depth, and adopting the value of \( \phi' \), we can calculate the value of \( c' \) needed to maintain equilibrium. We now have the information needed to proceed with the Coulomb wedge analysis to determine the critical wedge and the maximum force needed to retain the wall. The forces involved in the wedge analysis are illustrated in Figure 3.

For simplicity, the “wall” required to take the force \( P \) is assumed to be frictionless, so that \( P \) acts horizontally. In practice, retention of the slope may not involve a “wall” at all; ground anchors or soil nailing may be used, involving a relatively thin facing or a segmental facing at the cut face. The anchors or nails can be installed as the cut proceeds. With such retention systems the assumption of a frictionless wall is appropriate, as there can be no tendency for the soil to move relative to the “wall”. The solution is obtained by varying the wedge angle \( \alpha \) until the maximum value of \( P \) is obtained.

![Figure 3. Forces involved in the Coulomb wedge analysis to obtain the retaining force P.](image)

From wedge equilibrium considerations, it can be shown that

\[
P = W \tan(\alpha - \phi') + U \frac{\sin \phi'}{\cos(\alpha - \phi')} - C \frac{\cos \phi'}{\cos(\alpha - \phi')}
\]  
(5)

It can also be shown that the values of \( W, U, \) and \( C \) are given by the following expressions:

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\[ W = \frac{1}{2} \gamma H^2 \frac{\cos \alpha \cos \beta}{\sin(\alpha - \beta)} \] (6)

\[ U = \frac{1}{2} \gamma H^2 \frac{\gamma \mu \cos^2 \beta}{\gamma \sin(\alpha - \beta)} \] (7)

\[ C = \frac{1}{2} \gamma H^2 \frac{\cos^2 \beta \sin \beta}{\sin(\alpha - \beta)} \left[ 1 - \frac{\tan \phi}{\tan \beta} \right] \text{ for the dry slope} \] (8)

\[ C = \frac{1}{2} \gamma H^2 \frac{\cos^2 \beta \sin \beta}{\sin(\alpha - \beta)} \left[ 1 - \left( \frac{\gamma \mu}{\gamma} \right) \frac{\tan \phi}{\tan \beta} \right] \text{ for the phreatic surface at ground level} \] (9)

In deriving the value of \( U \) it is assumed that the making of the cut does not affect the seepage condition, i.e., seepage continues toward the cut with seepage lines still parallel to the ground surface. This is a conservative assumption, as in practice some drawdown of the phreatic surface is likely to occur, at least near the face of the cut. The term \( \frac{\gamma H^2}{2} \) occurs in each of these equations (6) to (9), so that it is easier to calculate and present the results in terms of an active coefficient \( K_a \) rather than the force \( P \).

Using these equations to solve for the force \( P \) by varying the wedge angle \( \alpha \) produces the "surprising" result that \( P \) is maximum when the wedge angle becomes the same as the slope angle i.e., when \( \alpha = \beta \). In other words the critical wedge is no longer a wedge but has become an infinite slab of constant depth parallel to the surface of the slope. Figure 4 shows the result for the simplest case, a dry cohesionless material, with an assumed \( \phi \) angle of 40°.

As the reader may well be aware, analytical solutions are available for both the peak value of \( K (= K_a) \) and the critical angle \( \alpha (= \alpha_c) \) for this case of a dry cohesionless material (see for example Jumikis, 1962).

![Graph](image)

Figure 4. Determination of \( K_a \) by trial wedges for a dry cohesionless slope with \( \beta = \phi = 40^\circ \).

For the case of zero wall friction these are:

\[ K_a = \frac{\cos^2 \phi}{\left[ 1 - \frac{\sin \phi \sin (\phi - \beta)}{\cos \beta} \right]^2} \] (10)

\[ \tan(\alpha_c - \beta) = \tan(\phi - \beta) + \sqrt{\tan(\phi - \beta) \left[ \tan(\phi - \beta) + \cot \phi \right]} \] (11)
For the limiting case when $\beta = \phi$, these equations become:

\begin{align*}
K_u &= \cos^2 \phi \\
\alpha_c &= \phi \quad (\beta) \\
\text{where } \alpha_c \text{ is the critical value of the wedge angle } \alpha.
\end{align*}

Figure 5 illustrates graphically the solution given by equations (10) to (13).

![Figure 5. Values of $K_u$ versus wedge angle $\alpha$ for dry cohesionless slopes with $\phi = 40^\circ$.](image)

It is seen that as $\beta$ increases, the value of $K_u$ increases as expected, and the position of the peak value of $K_u$ moves from the right to the left of the graph. The graphs are seen to have progressively sharper peaks. In the limiting situation of $\beta = \phi$, the critical wedge angle also becomes equal to $\beta$ and $\phi$, and the peak value of $K_u$ coincides with the left axis where $\alpha$ equals $\beta$. In other words, the inclination of the critical wedge decreases as the slope angle increases, and becomes progressively closer to the slope angle. Figure 6 shows the values of $K_u$ versus slope angle for this case of the dry cohesionless slope.

![Figure 6. Active pressure coefficient $K_u$ for walls supporting dry cohesionless slopes at limiting equilibrium ($\beta = \phi'$).](image)

Moving on from the simple case of a dry cohesionless material makes the solution a little more cumbersome. Analytical expressions, such as those in equations (10) to (13) are no longer available, but the results of trial wedges can be plotted as in Figure 4 above and the intercept of the line through the points with the vertical axis gives the required value of $K_u$. In all, four cases have been investigated:

- Case (a): Dry cohesionless material
- Case (b): Cohesionless material with the phreatic surface at ground level.
- Case (c): Material with some cohesion, no seepage.
- Case (d): Material with some cohesion, phreatic surface at ground surface.

The analysis of Cases (c) and (d) requires assumption about the relative magnitudes of $c'$ and $\phi'$. For simplicity, an arbitrary assumption was made that the value of $\phi'$ is somewhat less than the slope angle, and related to it by
the relationship: \( \tan \theta' = 0.7 \tan \beta \). The cohesion component \( c' \) then takes on whatever value is needed to maintain stability, in accordance with equations (10) and (11) above. The analysis produces the somewhat surprising result that the \( K_s \) values are the same for the Cases (c) and (d).

Figure 7. Values of \( K_s \) versus slope angle for all cases.

A fuller account of the analysis of these cases is given elsewhere (Wesley, 2001). The results from all four cases are plotted in Figure 7. The figure may appear surprising or illogical at first sight as the value of \( K_s \) decreases as the slope angle increases. However, simple logic dictates that the graphs must have the form they have. If the slope is flat and is at limiting equilibrium then clearly the material has the properties of a liquid, and the horizontal stress will equal the vertical stress (ie \( K_s = 1 \)). On the other hand, if the slope is stable at 90°, then no force is required to retain it. Hence the \( K_s \) value must start at unity for a level slope and decrease to zero for a vertical slope. The analysis in no way contradicts the fact that if we are dealing with the same material then the steeper the angle behind the retained wall the greater the force on the wall. The starting assumption is that the slopes are on the point of failure, so that if the slope angles are different then we are clearly dealing with different materials, or different seepage situations.

The most relevant cases are certainly (c) and (d). It is perhaps surprising that (c) and (d) give the same result, since (c) is for a slope without seepage pressures and (d) for a slope with the water table at the ground surface. The explanation lies in the fact that the required value of the cohesion \( c' \) is different in the two cases. For a typical case, say a 35° slope with \( \gamma_w/\gamma = 0.6 \), and \( \tan \theta' = 0.7 \tan \beta \), the back analysis of the infinite slope gives the result shown in Figure 8.

Figure 8. Cohesive and frictional components of shear strength with and without seepage present.
The required shear resistance to maintain stability is 0.470H. With no seepage present the required cohesion intercept \( c' = 0.141 \gamma H \), and the frictional component is 0.329\( \gamma H \). However when seepage is present the required cohesion intercept \( c' = 0.338 \gamma H \) and the frictional component is only 0.132\( \gamma H \). Thus when the wedge analysis is carried out the relative proportions of cohesive and frictional resistance on the slip plane are different but the net result is the same.

There are a number of assumptions involved in the above analysis, but the basic method is sound and should be a more reliable indicator of required force levels than simply assumption of soil parameters or a \( K_a \) value.

**Shear strength parameters from back analysis of single slips**

Until quite recently I wrongly believed that it is not possible to obtain unique values of \( c' \) and \( \phi' \) from the back analysis of a single slip, even if full details of the geometry of the slope and pore pressures are known. I became aware that this is not the case through the work of a Japanese professor at Tokushima University in Japan (Yamagami and Ueta, 1996). Yamagami and Ueta presented a method for determining \( c' \) and \( \phi' \) from a single slip. With the help of one of my students (Leelaratra, 1999) I looked into this method, and in doing so came across several other methods which are rather more straightforward than the Japanese method. These are described below. The methods are valid only if the slope consists of homogeneous material.

(a) The slope analysed. (b) The results of back analysis.

Figure 9. Back analysis of a slope to obtain values of \( c' \) and \( \phi' \).

Figure 9 (a) shows a slope in which a slip has occurred at the position shown. The slip surface is assumed to be circular and the ground water table is taken to be at the ground surface. By carrying out conventional slip circle analysis, it is possible to obtain a range of values of \( c' \) and \( \phi' \) which satisfy the criteria that the safety factor for the slip shown is unity. This has been done using the standard Bishop method. The range of values so obtained is shown graphically as curve (a) in Fig. 9 (b). By plotting \( c' \) versus \( \tan \phi' \) the plot is almost linear; this appears to be the normal situation when the analysis is of a specific slip surface.

From this point onwards there are several methods for deciding which of these possible combinations is the correct one. Perhaps the simplest method is to now ignore the actual slip circle and carry out stability analysis of the slope using as a starting point each of the combinations of parameters shown in Fig. 9 (b). In other words we are now ignoring the slip, and treating the slope as an intact slope. This analysis produces a series of critical slip circles as shown in Fig. 10. Examination of these shows that each circle has a different location, and only one of these circles has a safety factor of unity. All the others have safety factors less than unity. Thus the true field values of \( c' \) and \( \phi' \) must be those applying to this one circle which is compatible with the field situation. The values so obtained are \( \phi' = 30^\circ \) and \( c' = 18 \) kPa.
Figure 10. Critical circles obtained using the shear strength parameters obtained from back analysis of the actual slip surface.

A second approach is to ignore the actual slip surface (and the data obtained from it), and to repeat the back analysis treating the slope as intact. This gives a new set of combinations of $c'$ and $\phi'$ which apply to the intact slope. This range of values is also shown in Fig. 9(b) as curve (b). The point where the two sets of values coincide, i.e. where the curves touch in Fig. 9(b) defines the values that must apply in the field. There are other possible methods: these are described in Wesely and Eelaratanam (2001).

**Back analysis of terraced rice-fields**

This is not a high-tech example of back analysis but I will include it as the steep terraced rice-fields of Java and other parts of Southeast Asia have long had a certain fascination for me. They are a spectacular demonstration of the very good properties of the soils on which they are built - unusually high shear strength and a resistance to erosion. They are built on slopes as steep as 40° and irrigation water permanently flows from one terrace to the next. Individual terraces are up to 3m high. It is hard to imagine local Auckland soils remaining stable in this situation. The particular aspect of soil behaviour which they demonstrate very clearly is the reality of the cohesive component of shear strength ($c'$), even in a saturated remoulded soil. At least half of the terrace height must consist of remoulded soil. The rice-fields in Java are formed on slopes of halloysite or allophane clays, which have $\phi'$ values generally between about 35° and 40°. The terraces can only remain stable if there is a significant and long term value of $c'$. This point is of some significance in view of arguments sometimes put forward by the "critical state" school that the $c'$ component of soil strength cannot be relied on.

![Diagram of terraced slope for irrigated rice cultivation](image)

Figure 11. Back analysis of terraced rice-fields to obtain $c'$ value.

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It is possible to get an estimate of the $c'$ value by analysing possible failure modes. Figure 11 shows an assumed flow net and two possible failure planes. The back analysis gives $c'$ from 8 kPa to 15 kPa. These field values are at the lower end of the range obtained from triaxial testing.

The above flow net is a simple hand drawn effort. It assumes an infinite slope, so that the seepage pattern is identical at each terrace. I doubt that this is the case in practice; because slopes aren’t infinite, and possibly also because the surface layer may be of lower permeability than the underlying material, the flow may not be as parallel to the slope as the above flow net implies. During my time in Indonesia I had intentions to install some piezometers in rice-fields to find out just what the seepage situation actually is. However I did not find time to pursue this interest, which was probably just as well because there were many more pressing and useful matters to spend my time on. Recently I have had a student using the SEEP/W programme (Hongwu Zha, 2002) to look at several seepage situations of interest to me, one of which was terraced rice-fields.

Treating the slope as infinite produces a flow net very similar to the hand sketched effort above, but if the slope is modelled a little more realistically with a source (a canal) at the top and receiving stream at the base the pattern is rather different, as shown in Figure 12.

This is a suitable point at which to move on from back analysis, and side issues like terraced rice-fields, and look into some other seepage studies of more relevance to engineering situations.

**SEEPAGE STUDIES**

**Seepage into sheet piled excavations: 3D and 2D analysis**

Between the Waikato River and the Huntly Power Station there is a large facility built almost entirely below ground level known as the cooling water intake structure. It is essentially a large concrete box, about 64m long, 24m wide and 14m deep. Its construction posed a considerable challenge, because the site consisted of deep deposits of pumice sand, of quite high permeability. An original intention to build the structure in an open de-watered excavation had to be abandoned because deep well trials were not successful. It was eventually constructed above ground level without its bottom floor and sunk into the ground like a giant caisson. Deep sheet piles were installed around the perimeter of the structure to prevent sand collapse from around the outside of the structure. Suction pumps were used to extract water and sand from within the caisson, thus inducing the sinking process. An issue in planning the operation was the extent to which the water level within the caisson could be lowered without running the risk of uplift “heave” of the base of the excavation due to excessive hydraulic gradient.
A trial of the procedure intended for the main structure was actually carried out on a smaller structure, known as the “elver (juvenile eels) chamber”; this was to be part of a special migration channel for juvenile eels. Sinking this structure showed that base heave failure was a very real threat. It occurred twice during the trial.

There were a couple of issues that stimulated my interest while involved in the planning of this project. The first was the question of 3D flow nets versus 2D flow nets. Sketching a conventional 2D flow net to predict the point at which the hydraulic gradient will reach the critical value is not difficult (provided you can remember your soil mechanics lectures on flow net sketching and the formula for critical hydraulic gradient), but how to adjust this for the 3D effect of a box structure is a little more problematical. Rather than deal with this uncertainty, it was easier to decide not to significantly lower the water level within the structure at all, and thus avoid altogether the danger of uplift failure. However this did not entirely eliminate my curiosity about the 3D versus 2D situation.

The second issue that interested me was the general properties of the sand at the site. The bulk density of the sand varied from 1330 kg/m³ to about 1840 kg/m³ with an average of only 1500 kg/m³. This is a long way below the value of about 20 to 22 kg/m³ expected with normal sands. Clearly the pumice content of the sand was very high, and one could not help wondering what the implications of this were for its engineering properties in general. I later had some involvement with pumiceous sands in the Bay of Plenty, and the use of CPT testing to investigate them. Whether the same interpretation could be put on the results of these tests as for more normal sands was an open question. Research into the properties of pumiceous sands and CPT testing is described elsewhere (Wesley et al, 1999) and will not be covered here.

When I joined the university in 1986, the geotechnical section had a computer programme called GeoFlow. It was a finite element programme of the Fortran “fixed format” variety. Not knowing very much about the joys of fixed format programmes, I very happily inflicted it on a postgraduate student (Ampuialam, 1989), with the task of investigating 2D and 3D flow into sheet piled excavations. Figure 13 illustrates the problem addressed.

![Figure 13. Geometry of sheet piled excavation.](image)

The GeoFlow programme allows 2D and axisymmetric (or radial) flow to be investigated, so the comparison made is between flow into a 2D excavation of infinite length and a circular excavation having a diameter the same as the width of the 2D excavation. The essential difference between the two situations is that in the axisymmetric case the flow is progressively forced into a smaller “channel” as the flow approaches the excavation, with the result that head loss within the excavation will be greater. Thus the exit hydraulic gradient at the base of the excavation would be expected to be higher. Various ratios of B/H, D/S and S/H were investigated. Typical results are shown in Figure 14 for the situation where D/S = S/H = 0.5, and B/H is variable.

It was found that in all cases the exit hydraulic gradient was greater by about 35% to 55% for the axisymmetric than the 2D case. This finding is for a circular excavation, so it does not provide a complete answer with respect to the situation at Huntly, which involved a rectangular structure. There is clearly the possibility that the hydraulic gradient will be greater at the corners of a rectangular structure where the greatest “crowding” of flow occurs. This effect will also be greatest when the structure is relatively shallow compared to its depth. With a deep structure, the upward flow within the structure is restrained to be essentially vertical, in which case the hydraulic gradient cannot vary across the cross section.
Flow into open excavations

Analytical solutions for this situation are not available, although some text books use the Dupuit solutions and their adaptations for partially penetrating wells or slots (e.g. Leonards, 1962). A master’s student (Visvanathan Raganathan, 1994) was looking for a project topic at the time I was interested in this situation: the results of his study were informative and of practical value and are presented fully elsewhere (Wesley et al., 1996).

Influence of seepage assumptions on stability analysis

An assumption frequently made in drawing the seepage pattern in natural slopes is that the source of seepage is external to the slope. This means that seepage enters the slope approximately horizontally and flow nets are commonly drawn with horizontal flow lines and vertical equipotential lines. This is probably reasonable for many natural situations, but may not always be so. With relatively steep “double-sided” slopes the only possible source of seepage is rainfall on the slope itself, and at the upper end of the slope the flow lines may be closer to horizontal than vertical.

Along with the terraced rice-field seepage described earlier, Hongwu Zha (2002) undertook some limited investigation of these situations using the computer programme SEEP/W. Figure 15 shows typical flow nets for the two situations: the first (a) is for the seepage source external to the slope, and the second (b) is for rainfall recharge within the slope itself. In this latter case the upper surface has been assumed to be the same as for case (a). The surface is therefore static and undergoing recharge from surface rainfall. It is evident that there is little change in the flow net at the base of the slope but considerable difference at the upper part. The equipotential lines for the slope having surface rainfall recharge are almost horizontal and the flow lines almost vertical. Pore
pressures in this zone would therefore be considerably less than those associated with the assumption of vertical equipotentials. The influence of the seepage pattern on the safety factors of the slope have been investigated using SLOPE/W. In the first case the phreatic surface was specified; when this is done the programme assumes the equipotentials are vertical. In the second case a grip of points representing the flow net was used. The results are summarised in Figure 16 (a) below. The safety factor is 1.03 in the first case and 1.25 in the second. The position of the critical circle is not greatly changed. As expected it moves towards the toe of the slope because of the increased pore pressures here and the reduced pressures higher up the slope.

![Diagram](image1)

(a) Influence of seepage conditions on Safety Factor and position of critical circles.

(b) Standpipe piezometer levels for case of seepage source entirely within slope.

Figure 16. Influence of assumptions made regarding seepage state on safety factors and piezometer levels.

While on the subject of seepage in natural slopes, and especially that represented in Figure 15 (b), it is perhaps worth noting the relationship of the seepage pattern to piezometer readings, especially in multiple stand pipes (of varying depth) at the same location. There seems to be a tendency to assume that the only explanation for different water levels in such piezometers is a perched water table (when the head decreases with depth) or artesian pressure (when the head rises with depth). This is not necessarily the case. Figure 16 (b) shows the water levels to be expected in piezometers placed at varying depths at three locations in the slope. These are taken directly from Figure 15 (b). In the upper part of the slope the head is much less in the deeper piezometer, while at the base of the slope the head is greater in the shallower piezometer. In the central part of the slope the levels are almost the same, which is to be expected as the equipotentials here are close to vertical. Where there is a difference in levels, it is simply the result of the seepage state in a uniform slope and not an indication of perched water tables or artesian pressures.

**Significance of the ground water table.**

A further point worth noting in regard to natural slopes is the significance of the water table. It is easy to assume that the water table is some sort of definitive boundary below which seepage occurs, and above which nothing happens. We lecturers are probably at fault in leaving our students with this impression. It is of course not the case that seepage is only occurring below the water table.

![Diagram](image2)

Figure 17. Water table and seepage pattern with "limited" recharge at the surface.
In many clay profiles, the water table is quite deep, but the soil may be fully saturated to within less than a metre of the ground surface. Seepage will still be occurring above the water table, and will be governed by the same laws that govern its behaviour below the water table. The only difference is that the pore pressures will be negative and the actual state of seepage more variable and less amenable to easy definition. Figure 17 illustrates this point; it represents a hillside where intermittent rainfall and evaporation maintain a zone of negative pore pressure in the upper part of the slope. The computer generated flow net has been produced by setting a series of negative pore pressures as the boundary condition at the upper part of the slope surface. The programme rightly plots the water table as the line of zero (atmospheric) pressure.

**Seepage through settled mine tailings**

Mine tailings, such as those from the Martha mine, consist predominantly of fine plastic clay material. They are generally deposited underwater and slowly consolidate with time. They thus form an artificial normally consolidated deposit. In such a deposit the effective stress will increase with depth, at least once consolidation is underway. This means that the material will be less permeable with depth, because of the lower void ratio. Various drainage conditions apply to tailings dams. In some cases, under-drains are installed before filling commences; this means that drainage during consolidation can occur towards both the upper and lower boundary. On completion, these “lagoons” can be treated in various ways. They may be left as wetlands, in which case the tailings will have a permanent recharge from their upper surface and seepage will occur vertically towards the under-drains, if these exist.

![Graph](image1.png)

(a) Experimental results. (b) Estimated permeability variation with depth.

Figure 18. Coefficient of permeability in mine tailings at Waihi Gold Martha mine.

The seepage pattern in this situation is therefore not straightforward. The increased resistance to flow in the lower part of the slope means that the vertical hydraulic gradient must increase with depth and some pore pressure will be permanently present in the slope. In other words pore pressure dissipation will not tend towards a zero situation as may naturally be assumed. Figure 18 (a) shows the relationship between permeability and effective stress from experimental tests; the values have actually been calculated from c, and m, values obtained from consolidation tests on the tailings. Figure 18 (b) shows what this means in practice within a 50m thickness of tailings. It is seen that the coefficient of permeability (hydraulic conductivity) decreases with depth by a factor of about 5.

In Figure 19 graphs are shown of the hydraulic gradient and pore pressure in the tailings assuming free under-drainage at the base and a depth of water of one metre at the surface. The hydraulic gradient has an average value very close to unity as expected, though it is less than half this at the surface and about 80% greater at the base. The maximum residual value of pore pressure is about 90 kPa, which is somewhat less than I intuitively expected when I started looking into this matter. The total stress at the centre (25m deep) is about 400 kPa, so the influence on the final effective stress is not very great though still quite significant.

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BEHAVIOUR OF COMPACTED CLAY AND CLAY EMBANKMENTS

A few years ago I became interested in the behaviour of compacted clay and clay embankments. Prior to then, I had regarded them as rather dull materials compared to undisturbed soils, especially when the latter are highly structured or contain unusual clay minerals. However, some recent interaction with the local geotechnical profession over issues involving compacted clay made me change my mind, at least to some extent. The reasons are explained in the following sections.

Parameters for the Design of Reinforced Earth Walls

A few years ago, a group of local consulting engineers approached the geotechnical group at Auckland University with a question relating to the design of reinforced earth walls, namely what does $\phi'_c$ signify? The parameter appears in various design codes, in particular British codes for the design of geogrid reinforced earth walls. Local engineers, designing reinforced earth walls using local clays, wanted information about this parameter, in particular its relation to the familiar peak $\phi'$ value, and what would typical values be for New Zealand clays. The parameter $\phi'_c$ comes from the critical state concept originating from Cambridge University. It is the value operating when a soil is loaded to such large strains that a steady state (the critical state) is reached. In the critical state it will continue to undergo strains at constant stress and constant volume – the $c_v$ stands for constant volume. The term $\phi'_{c_v}$ is also used, meaning the critical state angle. The concept in adopting it for design purposes is that it represents an ultimate value which will not decrease further with deformation, and is therefore a safe design parameter. While able to answer the question of what $\phi'_{c_v}$ meant, my colleagues and I could not say with any certainty what its value was likely to be for typical New Zealand clays.

A master’s student, Craig Davidson, looked into this question as part of an M.E. thesis several years ago. By carrying out conventional triaxial tests to large strains Craig aimed to answer the question, at least for the four local clays he did tests on. Both drained and undrained tests were conducted, and taken to a strain of about 30%. The results of his work are described in Davidson (1999) and in Wesley and Davidson (2000), and only a brief summary is included here. Figure 20 shows typical results for one of the clays. Examination of these figures shows that even at 30% strain the critical state has still not been reached. In some tests, especially the undrained tests, the deviator strength does appear to have almost levelled off, but changes in pore pressure are still occurring. Similarly in the drained tests, volume changes are still occurring.

In Figure 21 the results are plotted in the form of conventional stress paths, for both the drained and undrained tests. A failure line has been drawn through the peak values as well as the “end points” i.e. the values at large strains. These large strain values are as close to the critical state as the tests got. It is seen that the failure line at large strains is not much below the peak line, especially if the cohesion intercept is ignored. Thus if the large strain $\phi'$ value is assumed to be close to the critical state value, then adopting the peak $\phi'$ adopted for design and ignoring the $c'$ value should not be very different from using the $\phi'_{c_v}$ value.
(a) Drained tests

(b) Consolidated undrained tests

Figure 20. Triaxial tests on an Auckland plastic clay (Weathered Waitemata series) taken to large strains.

Figure 21. Stress paths from drained and undrained tests.

The conclusion from the tests was therefore very useful, namely that attempting to measure the critical state friction angle is likely to be an unproductive exercise, and that adopting the peak $\phi'$ value (and ignoring the cohesion intercept) should not be very different from using the critical state $\phi_c'$ value. Whether the arguments for using the critical state $\phi_c'$ value for design are at all legitimate is another question, which will not be addressed here.

**Total stress and effective stress analysis methods for the design of compacted clay embankments**

Investigating this topic may seem like a rather backward step - revisiting a topic long since adequately covered by earlier researchers. There may be some truth in that. However I became interested in this issue following an incident of "under-performance" in a large highway embankment of compacted clay. Involvement in the
investigation of causes and appropriate remedial measures made me think seriously about the question of undrained versus effective stress analysis for the design of clay embankments, especially those which do not involve water retention. My natural "bias" on this issue probably dates from lectures by Professor Bishop in the 1960s. Bishop, having been a pioneer in establishing the fundamentals of clay shear strength in terms of effective stress, and being very much interested in earth dams, concentrated on effective stress analysis in his lectures. At the same time, in his "classic" paper with Bjerrum (Bishop and Bjerrum, 1960), he made some pertinent comments on total versus effective stress analysis, pointing out that only in special cases could they be expected to give the same result. Bishop supported the use of total stress analysis for short term stability of undisturbed fully saturated clays.

![Diagram](image)

Figure 22. Compaction control using undrained shear strength and air voids.

The embankment in question was for a highway, so that many of the factors that favour (or dictate) effective stress analysis for earth dams were not present. In addition, its construction had been controlled using the familiar New Zealand method of specifying undrained shear strength and air voids rather than the traditional method of water content and dry density. If undrained shear strength is a specified requirement of the compacted clay forming the embankment then it would seem logical to use it as a design parameter. For those unfamiliar with the use of shear strength and air voids to control earthworks the concept is illustrated in Figure 22. Specifying a lower limit (usually around 150 kPa) for the shear strength prevents the soil being too wet, and specifying an upper limit to the air voids prevents the soil being too dry. The net result is very much the same as with the traditional method, but the specification remains the same regardless of variations in soil properties.

While it has long been known that effective stress and total stress analysis will generally not give the same safety factor, it is quite difficult to find clear guidance in the literature as to which method is preferable for designing compacted clay embankments, or which is likely to be the more conservative. Lambe and Whitman (1969) in discussing the "End-of-Construction" case, state that "from the standpoint of reliability, there is no basic difference between the two methods. The gaps in our knowledge which make it difficult to estimate pore pressures make it equally difficult to properly evaluate undrained strength. There is one major advantage to the use of the effective stress analysis: the pore pressures assumed during design of a dam can be checked by field measurements and the design can be modified during construction if necessary." Vaughan (1971) implies that either method can be used and discusses in some detail the implications of each method in terms of safety factor and strains in the embankment.

A post graduate student from Zambia, Sulani Chituta, has been looking at this question, primarily from the point of view of a designer, and seeking to answer the question of what differences in safety factor result from using an effective stress or a total stress method for "end of construction" stability, and which should be regarded as the more appropriate. His work is not yet complete and only some aspects will be described here. A large bulk sample of weathered Waitemata clay was obtained, and a conventional compaction test carried out. Samples were then prepared at different water contents ranging from slightly dry of optimum to considerably wet of optimum. Each of these samples was then subject to the range of tests that would normally be carried out as part of a routine (or conventional) design procedure. These tests covered the following:

1) Undrained shear strength
2) Effective strength parameters
3) Pore pressure response to undrained all round stress application, and the B parameter.
With the results of these tests, design in terms of effective stress and total stress can be carried out for each value of water content. The research is still in progress and only some limited results are described here.

The results of the standard compaction test and the shear strength measurements are shown in Figure 23. Undrained shear strength was measured using both hand vane and unconfined compression tests. It is evident that the vane produces substantially higher values than the unconfined tests.

![Figure 23. Standard compaction test result and shear strength measurements.](image)

The effective stress parameters \( \psi' \) and \( \psi' \) were measured using consolidated undrained tests, the soil being compacted to a density corresponding to the standard compaction curve in Fig. 24. It was found that over the water content range investigated there was only a very small variation in these parameters. All the test results are shown in Figure 26.

![Figure 24. Summary of consolidated undrained triaxial test results (water contents from 21% to 34 %).](image)

The pore pressure response to an all round stress increase (ie the parameter B) was measured by simply setting up samples in a triaxial cell and applying the cell pressure in stages, while measuring the pore pressure response. In these tests, as with any compacted sample, the initial pore pressure is negative. No attempt was made to measure the negative values accurately; only that part of the response curve with positive pore pressures was reliably established. In a field situation the pore pressure response will be somewhat different from that in these tests, as there will be different constraining conditions, the principal stress directions will not be constant, and the response will vary from point to point within the embankment. For simplicity however the B values have been used and applied as though they are also valid for applications of vertical stress equal to the depth of any soil element below the surface of the embankment. This is believed to be a reasonable representation of how designers would use this information in practice.
Figure 25. Pore pressure response for embankments of varying heights.

The pore pressure response data is summarised in Figure 25 as graphs of $B$, versus compaction water content for embankment heights of 10, 25, and 60m. The parameter $B_0$ is the ratio of pore pressure to vertical stress; it is not the same as the $B$ parameter just described, which is the ratio of change in pore pressure to change in total stress. It thus takes account of both the initial negative pore pressure and the $B$ parameter, and is numerically the same as Bishop’s $r_p$ parameter. The figure looks a little odd at first sight, but it must be remembered that the starting point for all tests is a negative pore pressure, so that for any water content the curves only become positive when the embankment height (or confining stress) reaches a certain value. With a water content of 25%, no positive pore pressures will develop in a 10m high embankment; they will just become positive when the embankment approaches 25m.

Figure 26. Safety factors for 25m high embankment with a slope of 2:1.

Stability analysis of various slopes by both total stress and effective stress methods has been done using the information in Figures 23, 24, and 25. An example of the results is presented in Figure 26. The large difference in values between the two methods of analysis is immediately apparent. The safety factors from the total stress analysis are almost double those of the effective stress analysis, except at very high water contents. The upper limit of water content likely to apply in practice is about 33%.

To understand the reasons for the difference, reference is made to Figures 27 and 28. Figure 27 shows the shear strength along a typical failure surface in terms of the total and effective stress analysis. The value to maintain stability (the required strength) is also shown. The analysis in terms of effective stress was done using the SLOPE/W programme and the Bishop method; the values of strength shown are those at the base of each slice.
The undrained strength is constant all along the failure surface, and taken as the value from the unconfined compression tests. The effective strength along the surface rises from a very small value at each end to a maximum value at the centre, just greater than the value needed to maintain stability. This highlights the reason for the low value of safety factor using the effective strength method - the strength values reflect the effective stress along the failure plane, obtained from the total stress and pore pressure.

Figure 28 illustrates the situation with respect to a typical element along the failure surface. The total stress is 100 kPa. The pore pressure using the B, value form Fig. 25 is about 105 kPa, and the effective stress is 55 kPa. This corresponds to a shear strength of only 43 kPa. The undrained shear strength is 82 kPa, and the effective stress path corresponding to this value is shown. Hence in the unconfined compression test from which the undrained strength is obtained the effective stress must have been about 90 kPa, i.e., there was a negative pore pressure in the compacted soil of 90 kPa.

Thus the essential reason for the difference in safety factors is the difference in pore pressure (and thus effective stress state) in the two methods of analysis. This arises from two factors:

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a) In the effective stress analysis, the pore pressure is obtained from the assumed B value. Only positive values of B are considered. The fact that the compacted soil may initially be subject to large negative pore pressure and that B could be negative is ignored. In the total stress analysis, the pore pressure and effective stress are not considered; the effective stress is largely governed by the negative pore pressure in the compacted soil prior to testing.

b) The stress path which the soil will follow when stressed is not taken into account. In the effective stress analysis the strength is taken directly from the Mohr-Coulomb envelope at the effective stress currently acting on the failure surface. In the undrained analysis, the pore pressure changes during testing so that failure occurs at a different effective stress state.

It is worth noting that these effects become less marked as the stress level increases. Figure 29 shows the situation with a 60m slope at 4:1. In this case, at low water contents, the effective stress analysis produces safety factors lying midway between the two values from the total stress analysis, although at higher water contents the effective stress value is again lower than the total stress values.

![Figure 29. Safety factors for 60m high embankment with slope of 4:1.](image)

The question of which is the more appropriate method cannot be given a simple answer. In theory, it appears that the total stress method should give a reliable result, especially if the undrained shear strength is actually measured when the embankment is constructed. However, there is clearly uncertainty regarding the undrained shear strength – two quite different values are obtained using a compression test and a vane test. Other methods will produce further variations. The effective stress analysis certainly appears conservative in most cases, although this doesn’t necessarily mean that it is correct. It involves assumptions about the strength that are not directly related to the actual (ie measured) shear strength of the compacted soil. Compacted clay, will initially always involve a negative pore water pressure, but it is most improbable that designers will take this into account in the design process.

It must be remembered that it is only the short term (or “end of construction”) safety factors that are being compared here. These values may increase or decrease with time as pore pressures change and bring about changes in strength. Changes in pore pressure are more easily measured than changes in undrained shear strength, and for this reason it would appear appropriate to analyse both short term and long term stability using effective stress analysis. As a matter of principle, changes in strength and consequent changes in safety factor with time should be assessed using the same formulation of shear strength, and the same method of stability analysis. Using total stress analysis for short term stability and effective stress analysis for long term stability, as is not infrequently done, does not give a reliable indication of the change in stability with time.

**GENERAL EVALUATION OF SOIL PROPERTIES**

To close I will make some general comments about the evaluation of soils as engineering materials. It seems to me that our profession is always in danger of developing an unhealthy addiction to numbers alone, especially those produced by sophisticated analytical and numerical methods, and from time to time we need to remind ourselves of some very basic aspects of geotechnical engineering, especially with regard to soil evaluation.
About 8 years of my professional life was spent in the geotechnical section of the Indonesian Public Works Department. This had responsibility for site investigation work from one end of the country to the other, so that the range of soil and geological conditions encountered was very diverse. Evaluating the properties and likely behaviour of an unknown soil at any particular site was always a challenge, and I wondered at the time, and again in more recent years, as to what are the most useful “indicators” or “pointers” readily available to engineers as to the likely behaviour of a particular soil. My list would be something like the following, at least with respect to clays and silts:

(a) Geological information. For example, in Indonesia, with any new job involving an unknown site, the first question I would try to get an answer to was whether the site was in a volcanic or non-volcanic zone. Java had reasonably good geological maps, so the question could be answered by reference to them, even if not totally reliably. The question was very important because of the stark contrast between the properties of the volcanic and non-volcanic areas. The same situation is also true to a considerable extent in New Zealand.

(b) Observation of field behaviour. While not always feasible, it is rare that there is no opportunity to observe, or fossick out information on actual field behaviour. This can be done by observing slope behaviour, exposures in cuttings, or the performance of existing structures, be they roads, houses, or major buildings.

(c) Undrained shear strength and sensitivity. Both these properties can be assessed reasonably accurately by direct manipulation of the soil, provided of course it can be sampled. Sensitivity is an under-rated property in my view, as it tells us a good deal about a soil. Highly sensitive soils owe most of their undisturbed strength to structure, in the form of bonds of some sort between particles. They can be expected to display distinct “yield” stresses – reflecting the stress level at which the inter-particle bonds start to break down.

(d) Atterberg Limits and natural water content. I am a great believer in the usefulness of Atterberg Limits, together with natural water content. The two pieces of information of greatest value that these give are: firstly, the Liquidity Index of the soil, and secondly, the position it occupies on the Plasticity Chart.

\[
\text{CLAY: Density index - Liquidity Index} = \frac{w_e - PL}{LL - PI}
\]

\[
\text{SAND: Density index - Relative Density} = \frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}
\]

Figure 30. Measures of compactness in fine grained and course grained soils.

Figure 30 illustrates the two measures of “compactness” used in soil mechanics, namely the Liquidity Index for clays and the relative density for sands. They are each a measure of the position the soil occupies in relation to reference “density states”, namely Atterberg Limits in the case of clays, and maximum and minimum densities in the case of sands. A liquidity index of zero indicates a “dense” clay, likely to be of moderate to high strength with very low sensitivity. It is very unlikely to show a marked “yield” stress. On the other hand, a clay with a liquidity index approaching or exceeding unity is likely to be highly sensitive with a very clear “yield” stress. The liquidity Index is particularly useful if earth works are contemplated, as it is an indicator of likely handling difficulties and the degree of drying required. There is of course likely to be a close connection between liquidity index and sensitivity of the soil.

The second piece of very useful information is the position the soil occupies on the Plasticity Chart, a fact which is easily lost sight of. Soil mechanics literature contains many correlations of soil properties with either plastic or liquid limit. These correlations are inherently unsound, because neither parameter on its own indicates very much about the soil. Consider the Plasticity Chart shown in Figure 31 and the position of three soils A, B, and C.
Soils A and B have the same PI but will have very different properties. Similarly, soils B and C have the same LL but will have radically different properties. The two soils most likely to have similar properties are soils A and C, and the Plasticity Chart rightly classifies them into the same category, namely high compressibility or high plasticity clays. A and C have neither the same LL nor PI. If correlations are restricted to groups of soils that occupy zones a consistent distance above or below the A-line then correlations with PL of LL may well be satisfactory, but only on this limited basis.

It is the distance a soil occupies above or below the A-line that is the most useful indicator of likely properties. (see Wesley, 1988). This is no more than confirming Casagrande's original intention when he developed the Plasticity Chart. In the writer's view, it would be useful to divide the chart into a further zone by setting up lines parallel to the A-line creating a silty clay zone as indicated in Figure 31. Soils lying well above the A-line are likely to be difficult soils, especially those with liquid limits above 50. They are characterized by low shear strength, high compressibility, and susceptibility to shrink and swell problems.

An example of the usefulness of distance above or below the A-line for indicating soil behaviour is the correlation between residual friction angle ($\phi'$) and Atterberg Limits. A number of researchers (eg Lupini et al 1981) have pointed out that correlations between $\phi'$ and PL or LL (or clay fraction) are possible within specific soil groups, especially sedimentary soils containing common clay minerals, but these have no general validity. Clays containing halloysite or allophone do not conform to these correlations. However, if the distance above or below the A-line is used, rather than PL or LL, then a correlation of more general validity is obtained. After meaning to investigate this correlation for some time, I eventually got around to doing it recently. Figure 32 is the result.
Although the correlation is rather crude it does show a consistent trend for all soils. Those lying well below the A-line generally have very high $\phi'$ values and those lying well above it have very low values. The chart is however restricted to soils with LL above 50. Those with LL below 50 do not show sufficiently stable behaviour in shear displacement tests for any correlations to be valid (shear behaviour varies between turbulent and sliding). A more complete account of the background to Figure 34 has been written up as a technical note to be published in Geotechnique (Wesley, 2003).

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