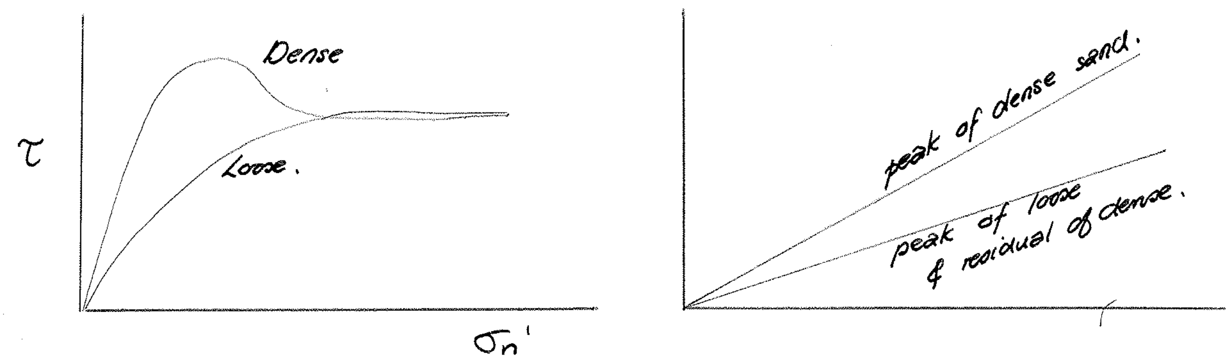
Stability of Natural Slopes.

Natural slopes in Sand

a: **Shear strength of sand.**

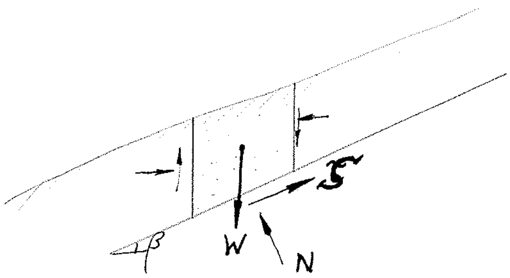


For Dense sands $\phi'_{ave} = 38° - 40°$

Loose sands $\phi'_{ave} = 33°$ $= \phi'_R$ of dense.

b: **The infinite slope analysis**

i/ **Dry Sand.**



All the elements are the same and so the side forces are all the same

Resolving normal to the plane

$$W\cos\beta = N$$

Resolving parallel to the plane

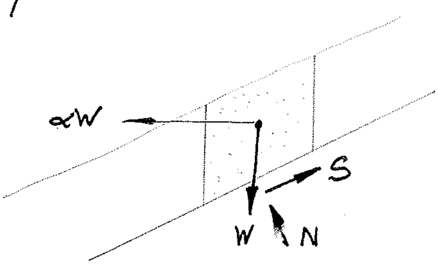
$$W\sin\beta = S$$

where $S = N\dfrac{\tan\phi'}{F}$

Thus $$W\sin\beta = W\cos\beta\,\frac{\tan\phi'}{F}$$

$$\therefore \quad F = \frac{\tan\phi'}{\tan\beta}$$

ii/ **Effect of a horizontal force.**



Resolving horizontally:- parallel to plane

$$W\sin\beta + \alpha W\cos\beta = S$$

Resolving normally to plane

$$W\cos\beta - \alpha W\sin\beta = N.$$

Again $\quad S = N \dfrac{\tan\phi'}{F}$

$$\therefore \quad W\sin\beta + \alpha W\cos\beta = (W\cos\beta - \alpha W\sin\beta)\frac{\tan\phi'}{F}$$

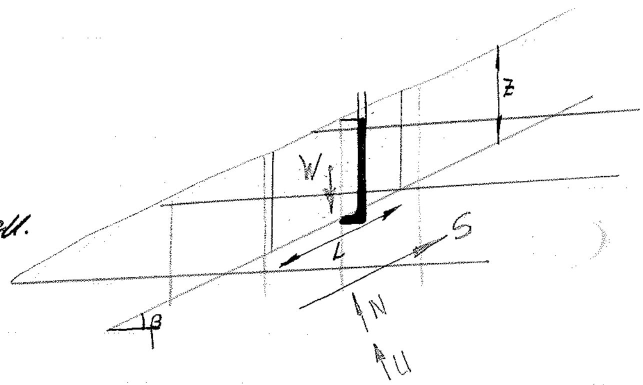
$$\therefore \quad F = \frac{1-\alpha\tan\beta}{\tan\beta + \alpha}\tan\phi'$$

If $F = 1.0$ then $\tan\beta_L = \dfrac{\tan\phi' - \alpha}{1 + \alpha\tan\phi}$.

iii/ <u>Effect of Seepage Forces.</u>

<u>Resolving parallel to the slope</u>

Assuming that pore pressures on either side of the wedge cancel.



$$W\sin\beta = S$$

$$\therefore \quad \frac{S}{L} = \frac{\gamma L\cos\beta Z}{L}\sin\beta$$

$$\therefore \quad s = \gamma z\cos\beta\sin\beta.$$

γ: full density

Resolving normal to the surface (this resolution of total stress is the same error as involved in Jellinius).

$$N = W\cos\beta$$

$$\therefore \sigma_n = \frac{W\cos\beta}{L} = \gamma z\cos^2\beta.$$

Now $\quad s = (\sigma_n - u)\dfrac{\tan\phi'}{F}$

$$\gamma z\cos\beta\sin\beta = \frac{[\gamma z\cos^2\beta - u]\tan\phi'}{F}.$$

$$\therefore F = \left[1 - \frac{u}{\gamma z\cos^2\beta}\right]\frac{\tan\phi'}{\tan\beta}$$

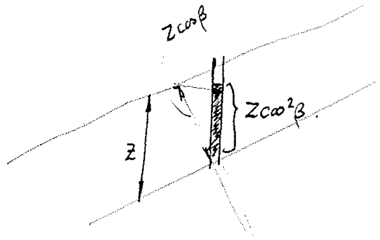
Now $\quad r_u = \dfrac{u}{\gamma z}$

Thus $\quad F = \left[1 - \dfrac{r_u}{\cos^2\beta}\right]\dfrac{\tan\phi'}{\tan\beta.}$

Case a: <u>Flow parallel to the slope.</u>

$$u = z\cos^2\beta\,\gamma_w$$

$$\therefore r_u = \frac{u}{z\gamma} = \frac{\gamma_w\cos^2\beta}{\gamma}.$$



Thus $F = \left[1 - \frac{\gamma_w}{\gamma}\right] \frac{\tan\phi'}{\tan\beta}$.

$= \frac{\gamma'}{\gamma} \frac{\tan\phi}{\tan\beta}$.

Case b Horizontal flow.

The equipotentials are vertical

$\therefore \quad u = z\gamma_w$

$\therefore \quad r_u = \frac{\gamma_w}{\gamma}$

$\therefore \quad F = \left[1 - \frac{\gamma_w}{\gamma\cos^2\beta}\right] \frac{\tan\phi'}{\tan\beta}$.

<u>Example:</u> Consider a sand of $\phi' = 35°$

Stable slopes ($F = 1.0+$) are thus

i/ Pore pressure $u = 0$ $\beta = 35°$

ii/ Parallel flow $\beta = 20°$

iii/ Horizontal flow $\beta = 18°$.

<u>Note:</u> The above analysis assumes that the flow lines are parallel ie ∞ slope.

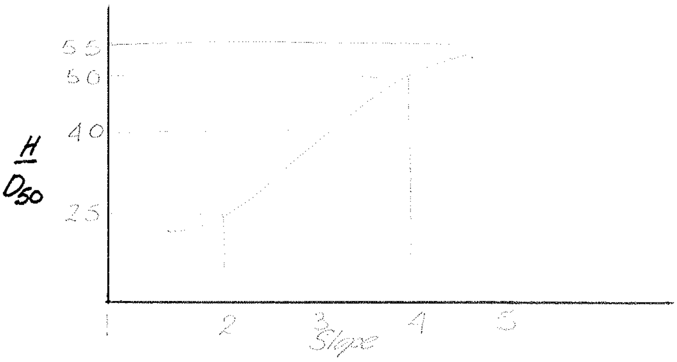
<u>Slope protection</u>

Size of rip rap. Burgess & Hicks

H = Ht of wave

D_{50} = 50% size of rip rap

The chart applies for $t = 1.5 D_{50}$



<u>example</u>: $H = 4'$ on 3:1 slope

$\therefore \quad \frac{4'}{D_{50}} = 4$

$\therefore \quad D_{50} = 1'$

<u>NOTE:</u>

If $t = 2.0 D_{50}$ then H increase by 15%

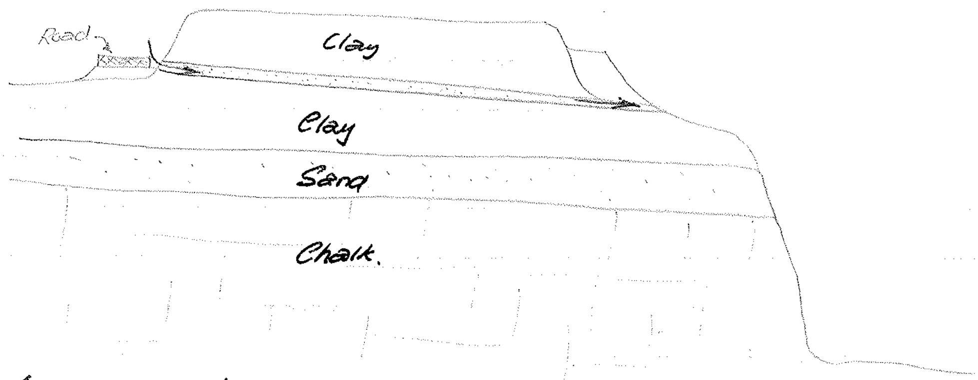
$t = 3.0 D_{50}$ " H " " 40%

<u>Ex</u>: Same as above but $t = 3.0 D_{50}$

$\therefore \quad \frac{H}{D_{50}} = \frac{140}{100} \times 4.0 = 5.6 \quad \therefore D_{50} = \frac{4}{5.6} = 9''$

Effects of Internal Erosion

Newhaven



Water from drain of road ran down the sand layer and there was internal erosion where it appeared above the cliff. Clay was under-cut and hence failures occured.

Process halted by placing a reverse filter against the sand.

CLAYS.

There is a fundamental difference between normally consolidated and overconsolidated clays.

A: Normally Consolidated.

Found round coasts as a result of last rise in the sea after ice caps had begun melting.

Found inland in areas where there has been considerable uplift.

$$\text{Sensitivity} = \frac{\text{Undisturbed unconfined str.}}{\text{Remoulded strength.}}$$

B: Overconsolidated Clays.

Heavily overconsolidated clay. similar to mudstone.

Boulder Clays are also overconsolidated and can be very hard. They are fully remoulded structureless materials.

Structural discontinuities in O.C Sedimentary Clays

Fissures.

• Occur in most clays of London type

• 1"-6" in scale and randomly orented to a first approximation

• The surfaces are never polished but can show some texture eg radial lines etc.

• Seems to be a concentration of fairly horizontal and fairly vertical fissures but no orentation in plan.

• Cracks are tight closed in the ground.

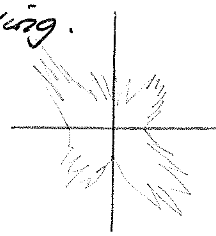
• The fissures are more abundant and smaller near the surface

Joints

• Found in the London Clay & may be > larger than 1'.

• Nearly all are virtually normal to bedding.

• Show strong prefered orentation. NW-SE
and minor NE-SW



• Some joints at about 45°.

• The nature of the surface seems to be a brittle fracture

Bedding Planes

Very common but may be difficult to find in some clays.

Faults Rather special. - find shear zones of tectonic origin.

Old Slip Surfaces

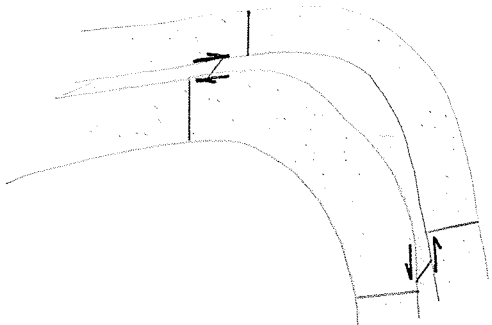
These are very important and must be located.

Glacial Tills

Dont have fissures or faults or joints of any consequence.

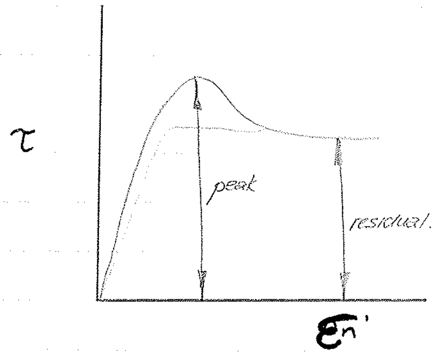
Shear Zones due to folding

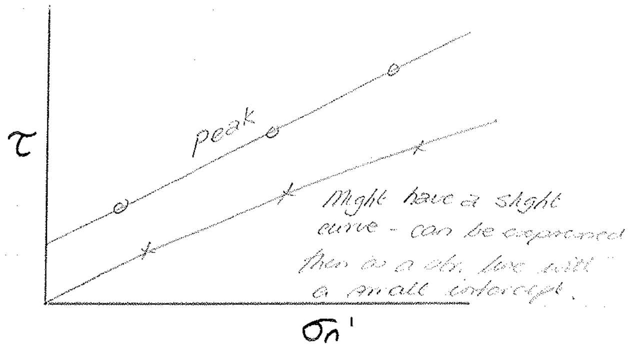
The shearing movements in all such cases are sufficient to reduce the strength to residual.



Relevant Properties of Clay.

Intact Clay.





If one tests an old slip surface then there is no peak to the curve at all.

Material from the joint in a clay might have a small peak.

Undrained strength.

Changes in total stress have no effect on the effective stresses and hence $\phi = 0$
and $\tau = C_u$.

Note that if the clay is failed in different ways one gets slightly different answers due to anisotropy. There is no unique value of C_u. but the differences are small.

N/B: The fact that C_u increases with depth does not contradict the $\phi = 0$ idea. The reason for the increase is change in water content ie change in material.

Short Term Stability in Saturated Clay

1/ Congress Street

$\phi = 0$ analysis $F = 1.1$.

N/B: 1/ The $\phi = 0$ analysis does not give the correct critical slip surface. It will give a deeper critical surface than in reality.

Thus to apply the $\phi = 0$ analysis to the actual failure surface is incorrect.

2/ The value of C_u usually is measured in compression ie σ_1 increases. In a cut σ_1 is const.

and σ_3 decreases. Thus perhaps a passive type of failure would be a better test for a cut.

2/ Huntspill cut.

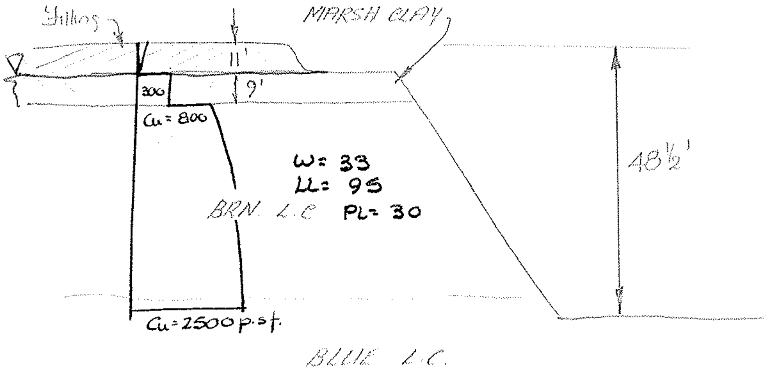
The samples were somewhat disturbed. Hence the measured Cu was low. Obtained F = 0.9.

3/ Skattmanso : Sweden.

A number of failures which on average gave a factor of safety F = 1.10.

Short term in Overconsolidated fissured Clay

Bradwell 1957.



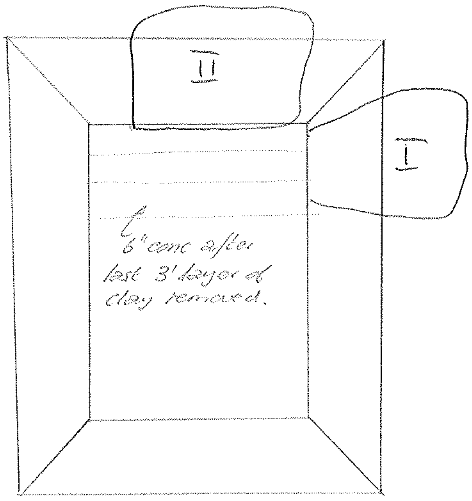
N°1 5 days after middle strip removed.

N°2 19 days after N°1.
(ht @ N°2 = 47')

Analysis:

It was considered that all the strength in the marsh clay was mobilized.

Calc. Cu of L.C. for F = 1.0



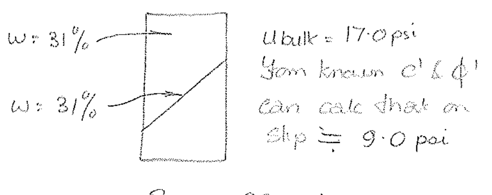
Slip N°1 Cu req. = 0.56 Cu meas.
N°2 Cu req. = 0.52 Cu meas.

This is in marked contrast to the results obtained with N.C. clays.

Considered time effects on measuring of Cu.

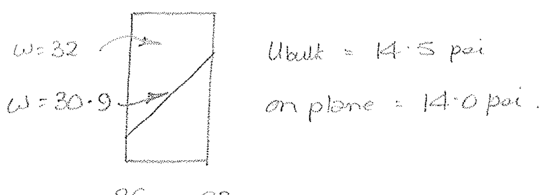
Time effects were found and are due to pore water migration

15 min test



2Cu = 29 psi

7 day test.



2Cu = 23 psi

Due to dilation there is a relative pore pressure depression on the plane in the quick test.

Thus out of total error of 50% there is ± 15% due to time rate of testing.

ie Lab conventional = 100%

Lab slow = 80%

Field = 55%

The difference is due to the presence of fissures. It seems that strength of mass ≈ 75% lab strength on normal samples.

Also investigated slips that had not failed.

N°1: $\bar{C}r$ ≑ 0·38 Cmeasured.

N°2 $\bar{C}r$ ≑ 0·4 Cmeasured.

Leda Clay. High sensitivity

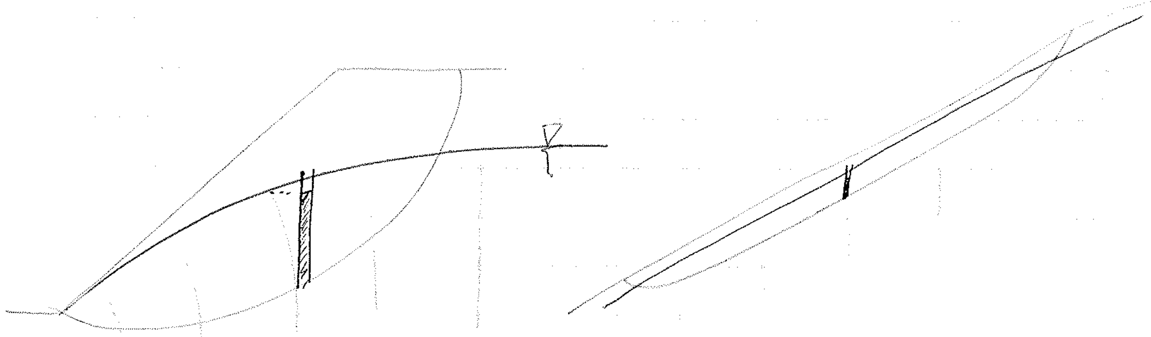
ω = 60 LL = 57 PL = 26.

Obtained Creq = 0·75 Cmeas.

This material is recorded as a fissured clay.

LONG TERM STABILITY.

Pore pressures are governed by the flow net. There are seasonal variations in the flow net as the water table varies with vegetation requirements.



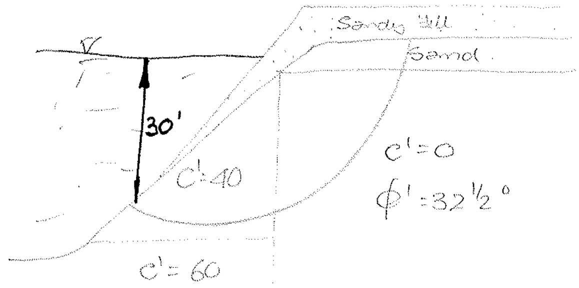
Cuttings Natural Slopes.

In a natural slope the slip often takes place through the weathered zone ε ± 5'. Often one finds a major pore pressure discontinuity in the slope. In the weathered material the piezometric level may be near the surface whereas in the unweathered material it may not be up to the level of the weathered zone even.

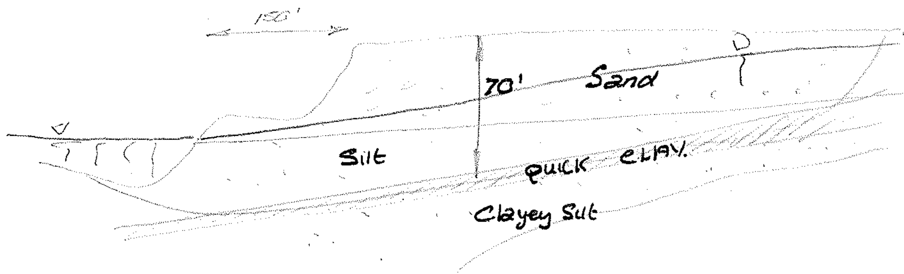
Normally consolidated clay.

2) Drammen

Calculated $F = 1.01$
using peak values.



Quick Clays.



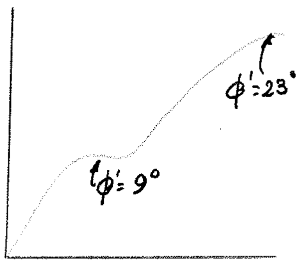
$S_t > 100$

$w = 43$

$LL = 32$

The materials are nearly normally consolidated.
Calculated that for $F = 1.0$ $c' = 0$
$\phi' = 8°$

Large scale in-situ shear box test $\phi' = 9°$
All previous results had given $\phi' \doteq 20°$
After some very careful sampling obtained stress strain curves as shown.

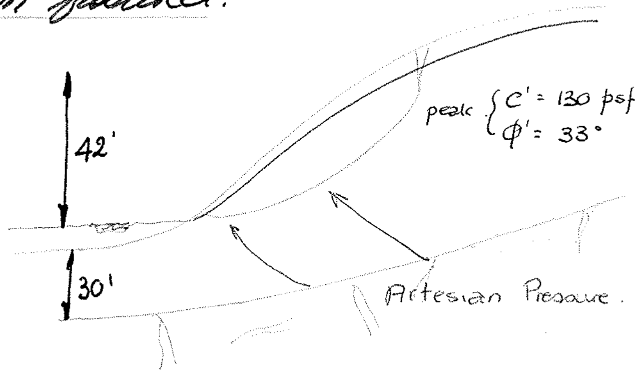


When the sample is sheared there is a collapse at about $\phi' = 9°$. It is difficult to say whether at this stage the material is drained or undrained.

Overconsolidated clays - non fissured.

Glacial Till at Selset.

Slip triggered off by slightly larger erosion at a corner in the river



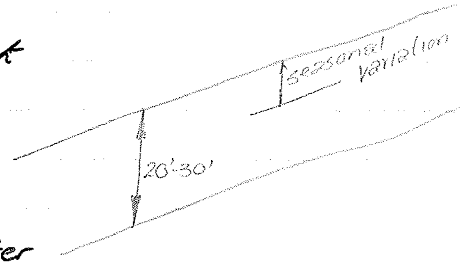
Bishops method $F = 1.03$ on peak.
$= 0.7$ on residual

Lightly O.C clay at Lodalen. - cutting

The actual slip surface was determined by borings.
Analysis for $c' = 200$ psf
$\phi' = 27°$
gave $F = 1.05$.
Because c' fairly small and ϕ' quite high these parameters are close to the Hvorslev parameters and hence the critical surface is very close to the actual surface.
The residual was not measured but with $c' = 0$ & $\phi' = 27°$ $F = 0.70$.

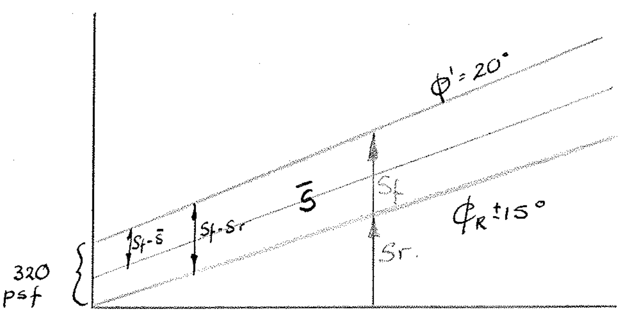
Overconsolidated Fissured Clay.

LONDON CLAY.



- Weathered clay 20'-30' thick
- Zone of seasonal variation in temp & moisture ±10'
- Ground water 0-6'.
- Brown clay occurs below water table due probably to oxidation by dissolved oxygen.
- Apparently no brown clay below the terrace gravels - none formed since placement of gravels.
- Often have an upper zone of reworked London clay.
- Mechanical properties. of upper 30', with the exception of the zone of seasonal variation.

Peak $c' = 320°$
$\phi' = 20°$
Residual $\phi' = \pm 15°$



Residual Factor.

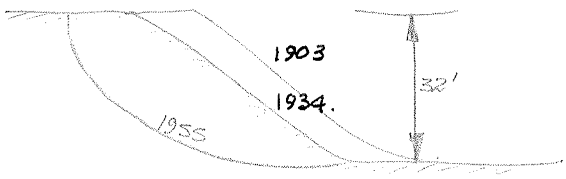
$$\bar{S} = R\,S_r + (1-R)\,S_f$$

$$\text{ie}\quad R = \frac{S_f - \bar{S}}{S_f - S_r} = \frac{S_f/\bar{S} - 1}{S_f/\bar{S} - S_r/\bar{S}} = \frac{F_f - 1}{F_f - F_R}.$$

$$F = \text{Factor of Safety} = \frac{\text{Available Shear Strength}}{\text{Required Shear Strength}}.$$

CUTTINGS.

a: Northolt.



$F_f = 1{\cdot}57$

$F_r = 0{\cdot}57$

Hence $R = 0{\cdot}56$ ie required $c' = 140$

$\phi' = 18°$

b: Sudbury Hill.

Cutting 1900 - Failure 1949.

$F_f = 2{\cdot}08$

$F_r = 0{\cdot}74$

$\therefore R = 0{\cdot}80.$

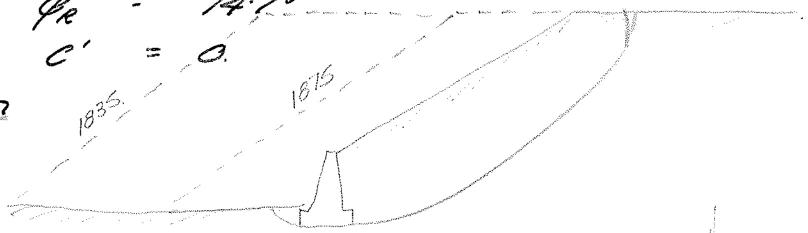
Because this slip was never cleared up but just had material removed at the toe when necessary it was possible at the time of the investigation to use it as a "giant shear box" test.

ie $F = 1{\cdot}0$ and hence could calculate

$\phi_R' = 14{\cdot}7°$

$c' = 0.$

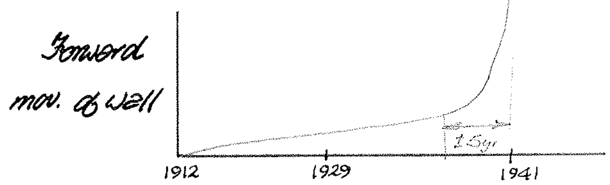
c/ Kensal Green



$F_f = 1{\cdot}6$

$F_r = 0{\cdot}6$

$\therefore R = 0{\cdot}61$



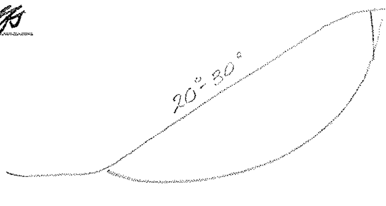
The measurements on the crest of the wall showed that some form of progressive failure was taking place.

This is due in the case of cuttings to a decrease in $\frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$ with a resultant opening of the fissures. This allows softening of the material and hence failure due to a loss in strength.

NATURAL SLOPES.

It is possible to divide the natural slopes of short time into four divisions

a/ Actively eroding cliffs



This means a rate of erosion of about ± 5' per year.

b: <u>Eroded Cliffs</u> where the rate of retreat is such that the removal of material by the sea is less than the max possible rate of supply by mud flows.

Average slope $15° - 25°$

c: <u>Defended Cliffs</u>

Protection works have been carried out to prevent marine erosion.

Have usually not protected for more than 100 years.

d: <u>Abandoned Cliffs</u>

The sea has retreated and the slopes have been free of erosion at the toe for centuries.

eg Hadleigh Castle:

Degregation slope $12°$ after $\pm$ 700yr.

<u>Long time scale slopes</u>

These are inland valley slopes the majority of which have not been actively eroded for a great time. These slopes may be divided into

a: Unstable - varying from $8°$-$12°$

b: Stable - there are no stable slopes of ~~more~~ than $10°$. Usually stable below about $8\frac{1}{2}°$.

<u>Analysis of these results.</u>

Consider that failure of these slopes can take place at $\phi_R = 15°$.

Assume that GWL = 1' below surface.

Thus using the infinite slope analysis we have.

// <u>Flow parallel to slope.</u>

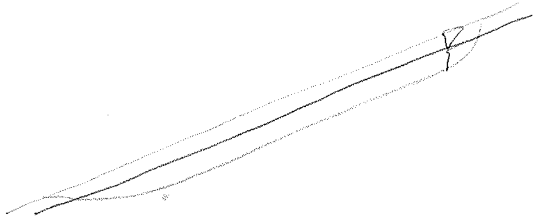
$$F = \frac{\gamma'}{\gamma} \frac{\tan\phi'}{\tan\beta}$$

$$\tan\beta \doteq \frac{1}{2} \tan 15°$$

<u>Thus $\beta = 10°$.</u>

Another factor that must be considered is that all these slopes have been subjected to solifluction processes. Most of the slopes are similar to what they were after the last glaciation. The slipping is in material that has been reworked and thus in stable slopes one finds shear surfaces which must have been formed in glacial times.

Difference between rapidly eroded slopes and inland slopes.



piezo ht ≈ head of clay

r_u = smaller.

The steeper the slope the lower the p.p ratio. The steep angles of say some of the defended slopes can be explained with reasonable accuracy using usual values of ϕ an the probable pore water distribution

For defended slopes ϕ' is probably close to residual.

Cause of slopes flatter than ±10°.

The reasons have nothing to do with mechanics and are compounded of

1/ Solifluction
2/ Soil Creep

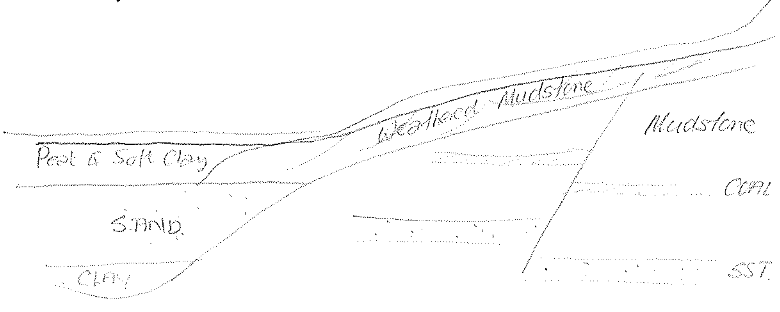
Movements on existing slip surfaces.

a: Sudbury Hill - already considered.

$F = 1.0$

$\phi'_R = 14.7°$ $c' = 0$.

b: Waltons Wood.



Many old slip surfaces and once the embankment for M6 was placed, slides occured.

<u>For $F = 1.0$</u> . If c' taken $= 0$

then $\phi' = 14°$.

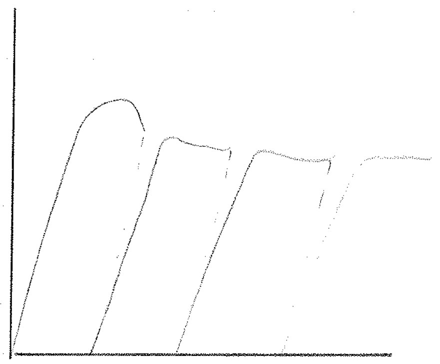
<u>Samples taken across the slip surface</u> yeilded

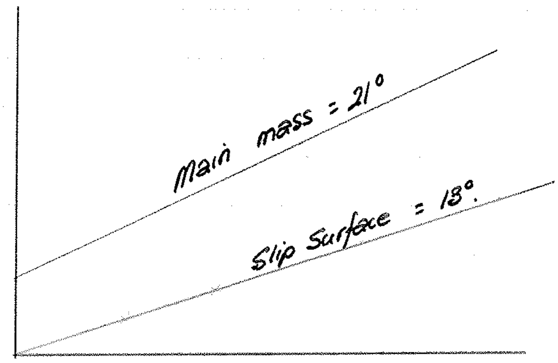
$\phi'_{ave} = 13°$.

The strengths across the slip surfaces were $1/3$ of those in the surroundings.

In a narrow band of $\pm 50\,\mu$. the clay particles were orented. as opposed to the rest of the mass.

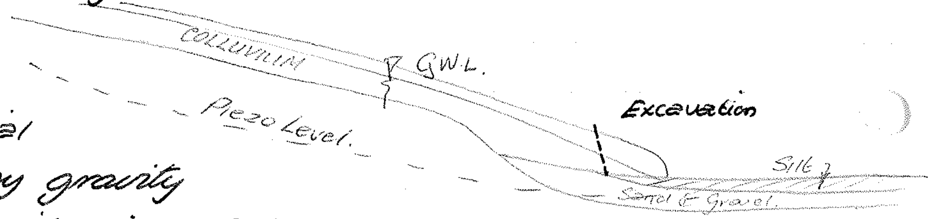
<u>Residual testing</u> using the reversing shear box





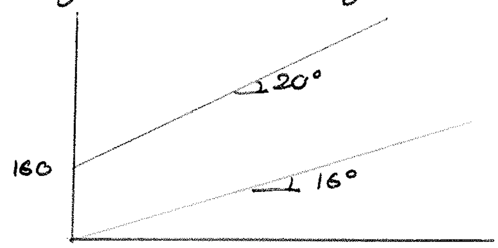
The shear box results lay exacctally on the 13° line obtained by testing the actual slip material.

c: <u>Weirton : W. Virginia</u>



<u>Colluvium</u> is materal put in place by gravity

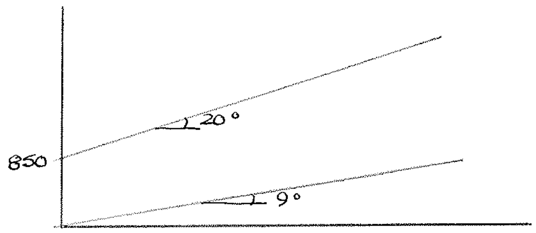
The material in which the old slip surfaces were found was rock fragments in a matrix of silt. However at the slip surfaces there was a band of clay possibly produced by the grinding action of the movements.



$F = 1.05$ on residual.

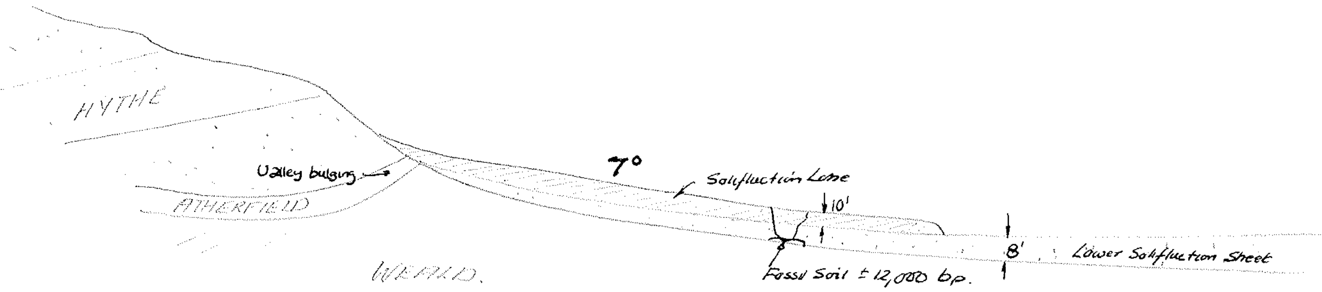
d: <u>South Saskatchewan Dam.</u>

An old slip through a soft shale reactivated due to a new cutting



Measured residual ± 9°.

e: <u>Sevenoaks.</u>

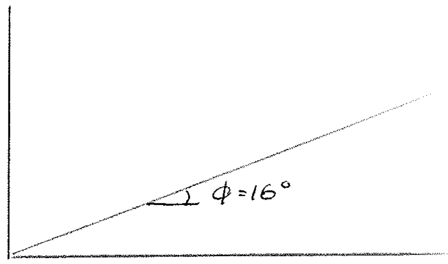


<u>The solifluction material.</u>

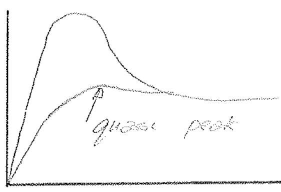
- fragments of Hythe beds + considerable portion of chert which is found further up the scarp. The angular chert lumps were up to 18".
- The main sheet (of Weichselian origin) has no topographical expression but extends at least 3/4 m. from the scarp.
- The upper material (Zone III of late glacial) occurs in tongues stretching out some 1200' from the scarp.
- Origin is intense frost shattering of the Hythe beds followed by slumping & sloughing down the slope on melting

<u>Strength.</u>

Above the old soil and underlying the main chert was a heavily sheared clay layer.



In general tests on the slip surfaces have shown $\phi_R = 14°$. with virtually no peak to the stress strain curve.



<u>The FOS of the lobes</u> using $\phi = 16°$ and the winter piezo levels = 1.08

<u>NB:</u> Shears found in Weald Clay below the lower sheet. with strength at residual. No movement for 10000 yr. Thus no build up of strength with time

Remedial Measures.

Remedial measures may be classified as follows.

1/ Those that decrease the disturbing moments

2/ Those that increase the shear strength.

3/ Those that prevent erosion

A: DECREASE DISTURBING FORCES.

a: Flatten slopes

In redesign of a sliped area one must use ϕ_r.

b: Reduce the height or add a toe weight.

B: INCREASE SHEAR STRENGTH

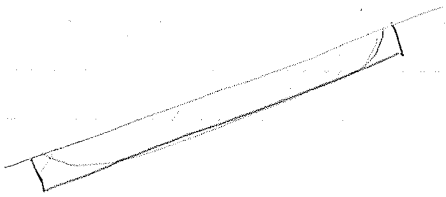
a: Drainage

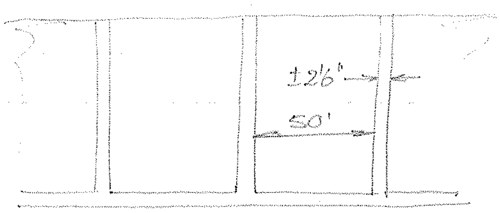
b: Mechanical means eg piles.

Drainage.

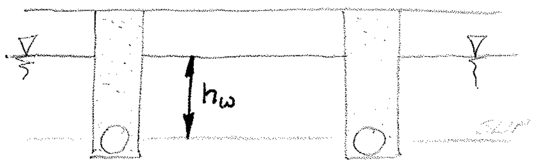
In natural hillside slips this is about the only method available. Not much aid from a toe weight.

If possible the drains should extend below the slip surface. but this is not necessary.

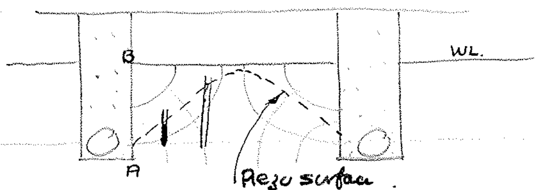




These drains work even if the ground water is not lowered because the change the flow pattern and hence the pore water pressures.



BEFORE



AFTER.

The water level afterwards still remains the same but AB is now a surface at atmospheric pressure (neither flow surface nor phreatic surface) and so the pore pressures on the slip surface drop.

Example : If the water level is near the surface then

$$s = (100 - 50) \tan \phi'$$

If drains are installed so that the average pore pressure, now approximate parabolic, = 2/3 previous then

$$s = (100 - \tfrac{2}{3}50) \tan \phi'$$

$\therefore$ Gain in shear strength ≈ 1.3.

In practice there will be some ground water lowering as well and this will help even more.

It is because the flow pattern is changed that drainage can be used in clays. Usually the mass permeability of a clay is less than that of the intact clay $\therefore$ ground water lowering as well.

b/ Mechanical strengthening

- May need very heavy piling to resist the deformation forces.

- Grouting : It is necessary to know the position of the slip surface. It then seems that a thin layer of grout is injected along the slip surface. This probably results in an increase in the angle of friction

There are other specialized techniques

eg Electro osmosis

Electro chemical hardening

c: PREVENTION OF INTERNAL EROSION.

This is in the case where flow from a sandy material is removing soil. eg Newhaven.

Solved by reverse filters

The validity of the $\phi = 0$ analysis is restricted to saturated ~~clays~~ soils (stiff fissured clays under a reduction of normal stress are an exception) in which insufficient time has elapsed, after the stress change considered, for an increase or decrease in water content to occur. It is there-for an end of construction method. Whether the factor of safety subsequent to construction will have a lower value depends on the sign and magnitude of the stress change. Essentially this method is ~~restricted~~ correct only when the field conditions correspond to the lab conditions ie where the shear stress tending to cause failure is applied under undrained conditions. There is an error due to the fact that the principal stress directions in most practical problems differ from those in the laboratory. This is not usually worried about and has been discussed by Hansen and Gibson.

Stability of cuts.

The pore pressure equation

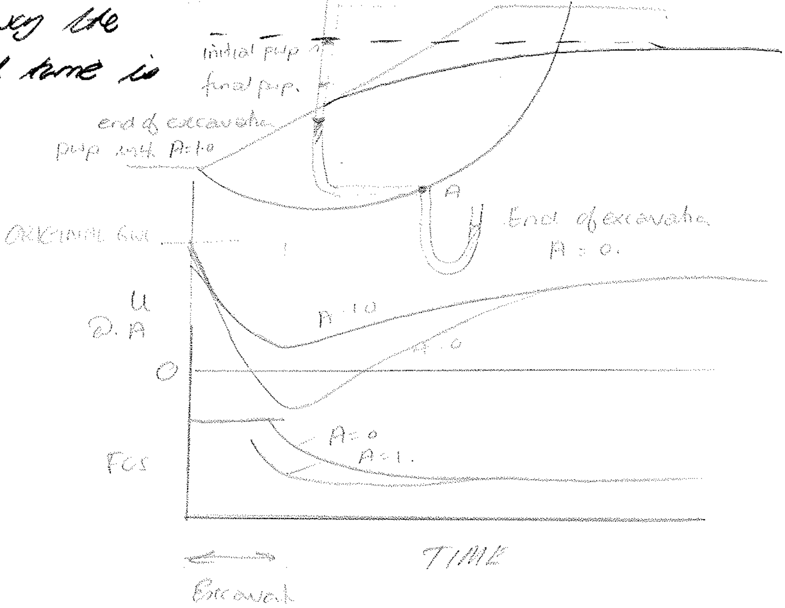
$$\Delta u = B\{\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\}$$

may be written for $B = 1.0$

$$\Delta u = \frac{\Delta\sigma_1 + \Delta\sigma_3}{2} + (A - \tfrac{1}{2})(\Delta\sigma_1 - \Delta\sigma_3)$$

This shows that the reduction in mean principal stress will lead to a decrease in pore pressure and the shear stress term will also lead to a reduction unless $A > \frac{1}{2}$, if the unknown effect on pore pressure of the changes in principal stress direction can be neglected.

A typical example showing the variation of the FOS with time is thus as follows.



In the majority of cases unless special drainage measures have be taken to lower the final GWL the FOS reaches its minimum value under the long term equilibrium

In tempory work where the end of construction condition is of primary interest the factor of saffy may be calculated using the $\phi_u = 0$ analysis and the undrained shear strength. This method may also be used to check that the initial factor of saffy is not lower than the long term factor of saffy, as it avoids the necessity of explicitly determining the stress distribution and pore pressure values at the end of construction

The method however does not give satisfactory results with stiff fissured clays when ordinary methods of testing are employed. The F.O.S is far to high and it is due to the fact that the incorrect s^{u} is measured under normally procedure

The measured strength is too high because of
1. Rate of testing too fast
2. Size of sample is not representative

b. Natural Slopes:

It may be thought that the factor of saffy for a long term slope stability problem could be calculated using the $\phi = 0$ analysis using the undrained strength of samples from the slope where pore pressure and water content equilibrium have been attained.

A large number of case records indicated that this is very unreliable because:

The fundamental difference is that in the undrained test the pore pressure is a function of the stress applied during the test and is not necessary equal to the pore pressure in situ. To obtain a factor of saffy of 1.0 for a slope in limiting equilibrium using undrained tests would require that the same pore pressure should be set up in the sample when the in-situ normal and shear stresses were replaced. This is in general prevented by the irreversibility of the stress strain characteristics of the soil and by the changes in the principal stress directions

The position is made worse by the fact that the water content changes both in overconsolidated and in sensitive clays are very localised at failure and samples which do not pick up these layers have very little bearing on the stability analysis

Inter-relationship between $\phi = 0$ and effective stress analysis

1. Both total and effective stress methods of stability analysis will agree in giving a F.O.S = 1.0 for a soil mass brought to limiting equilibrium by a change in stress under undrained conditions
2. Although the F is the same the position of the critical sliding surface will be different in each case. The closer the value of ϕ approximates to the true angle of internal friction the more realistic is the position of the failure surface

The choice of $\phi = 0$ method for short term stability is thus generally a matter of convenience as
1. Pore pressures don't have to be explicitly calculated.
2. Forces between slices can be neglected unless the slip surface is non-circular.

For factors of safety greater than 1.0 the two methods will not in general give numerically equal values of F. In the effective stress method the pore pressure is predicted for the stresses in the soil under the actual loading conditions and the value of F expresses the amount of c' and ϕ' necessary for limiting equilibrium. The total stress method implicitly uses a value of pore pressure related to the pore pressure at failure in the undrained test. The high F shown by the $\phi = 0$ analysis on a slope of o.c. clay where the pore pressures tend to drop during shear will not be shown to the same extent in c' ϕ' analysis.

A comparison between $c'\phi'$ and ϕ_u methods can only be made when the shear stress tending to cause instability has been applied under undrained conditions

Measurement of undrained shear strength

The undrained strength is obtained from undrained triaxial tests on undisturbed samples (or from unconfined compression tests, except on stiff fissured clays) and from vane tests in situ.

It is also determined from plate loading tests and large in-situ shear box tests.

It cannot be obtained, without risk of error on the unsafe side, from consolidated undrained tests where the sample is reconsolidated under the overburden pressure. This effect is serious in N.C. clays of low plasticity.

It is probably more realistic to calculate the value of the pore pressure parameter A for undisturbed soil from the relationship between the undrained strength of undisturbed samples and the values of c' & ϕ' rather than to measure it in a C.U. test.

The value of A measured on an undisturbed sample in an undrained test is very different from that the in-situ value under the same change in stress. This is because of the stress history given to the sample on removal from the ground. The release of the deviator stress existing in samples normally consolidated under no lateral yield plays a large part in this alteration.

Tests have shown that the effective stresses in a sample when unconfined or under an all round pressure may be $< \frac{1}{2}$ the in situ values. However when the sample is failed the strength obtained agrees closely to that measured in the field.

This is consistent with the observations that for a limited number of soils and stress paths the strength and the water content are uniquely related. It is this fact that provides the empirical justification for the use of the $\phi = 0$ analysis. Then to reconsolidate the samples under the existing overburden pressure will inevitably lead to an overestimate of the in-situ strength of the soil.