# Earth and Rockfill Dams

These dams can use a wide range of materials and can be constructed on weak, compressible and pervious foundations.

The factors which allow this are

1. Ability of plastic earthfill to deform without rupture
2. Large base width - reduces shear stress in $f^{dn}$.
3. Large base width. - lengthens the seepage path in the case of pervious foundations and enables easier control of leakage and uplift.

Functions of a dam as a structure

1. To provide a relatively impermeable barrier against water
2. To transmit the water load to the foundation

In general because impermeable plastic soils are weak in shear an economical design is usually one of different soil types in zones.

Factors influencing the choice of a dam type.

1. Range of borrow pit materials available.
2. Foundation conditions - the depth to bedrock, and if this is large then the shear strength, permeability and consolidation characteristics of the soil
3. Climatic conditions - influences the control of placement water content and also subsequent moisture content changes in the core eg drying out on prolonged drawdown.
4. Type of labour and plant available
5. Relative costs of excavating, hauling and compacting the various materials available. The length of haul is important. Prices and transport of concrete materials must be considered.
6. Relative costs of diversion works

Classification of different types.

These are classified on the basis of the nature, and position of the impervious core.

The functions of the core may be twofold.

a. To limit seepage to economic values.
b. To localise the head loss through the embankment so that seepage does not endanger the downstream slope.

3. To provide a more plastic zone in relatively more pervious and rigid fills. This zone should be able to adjust itself to deformation without damage.

In very general terms dams may be divided into

a. Homogenous fill dams.
b. Zoned earth dams.
c. Rockfill dams.

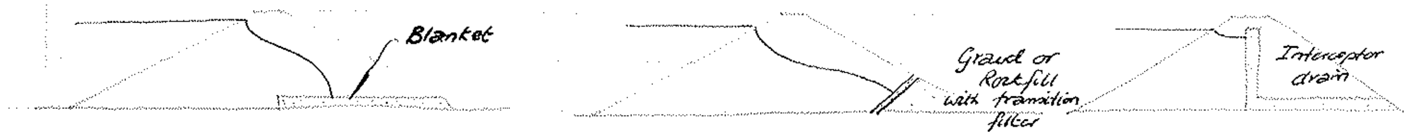
Homogenous Fill dams.

The fill serves both to resist the flow of water and to transmit the loading.

It is necessary to provide some sort of filter in the dam in order to prevent flow from the downstream ~~filter~~ slope.

The filter may be

1. Under drain blanket
2. Filter toe
3. Interceptor drain



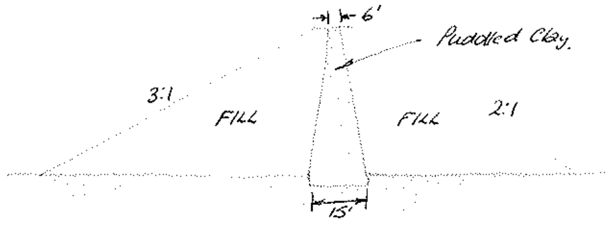

In types 1 and 2 the filter may be ineffective if the horizontal permeability is much greater than the vertical. If a toe of gravel is used then it must be sufficiently large to allow for the $\frac{Kh}{Kv}$ ratio. The interceptor drain works regardless of $\frac{Kh}{Kv}$. The interceptor drain might include a transition filter to guard against piping.

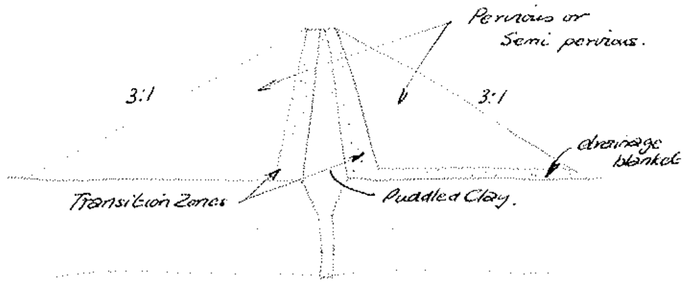
In general flatter slopes are needed for a homogenous dam than for a zoned dam of the same height.

Zoned Dams.

In Britain this first had a puddled clay core which has been used fairly successfully until quite recently (Selset).

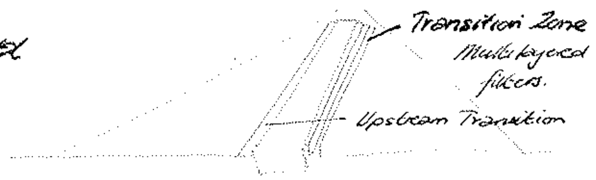
The following shows the 1827 Birmingham canal embankment and the modern equivalent of the puddle.



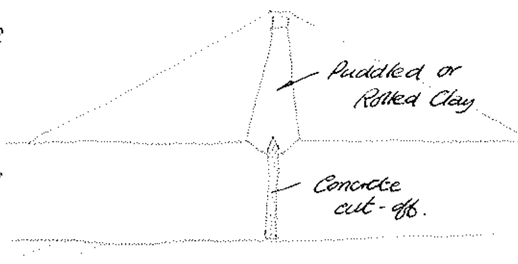

The modernised rolled clay core dam is very similar to the above puddled core section. If the core material is cheap then the core is enlarged and the pervious gravel or rockfill used as a weighting and filter element rather than a structural zone
eg Sasumua dam

For high dams of the zoned type the most economical from the point of view of quantity of material is a cross-section is a sloping core at a fairly steep angle. However for a zoned dam of this nature the foundation must be fairly strong.
It is often easier to build the core at the angle of repose of the downstream shell material




The use of concrete.
Can be used successfully as a cut-off and can be used in small dams which have a strong rock foundation as a core wall.


If used for the core then one can either have a thin R.C wall or else mass concrete cast in bays.




It is possible to use a thin mass concrete wall together with what clay is available. The clay is used to control the leakage while the concrete controls internal erosion

## Dams with a Partial Cut-off.

A complete cut-off is desirable to

1. Minimise loss of water
2. Guard against piping
3. Ensure stability by reducing the downstream uplift pressures.

This is sometimes impossible because of

1. Great depth of overburden.
2. Great quantity of water that would have to be handled in an excavation

Examples of cut-offs of greater than 200' are on record but very often not economical.

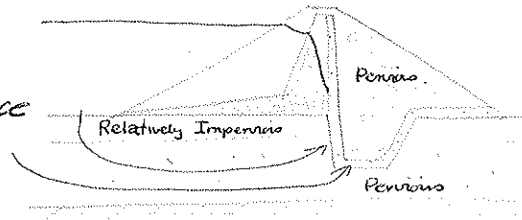
Dams without complete cutoffs are liable to failure by piping or shearing due to excess pore pressures.

These possibilities are guarded against by.

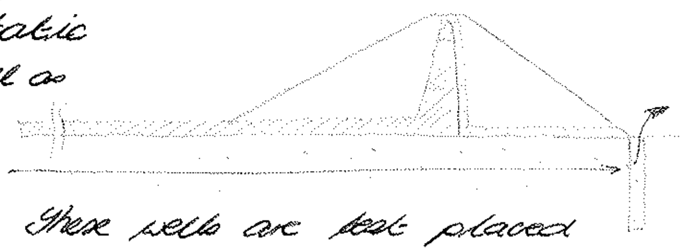
i/ Collection of all the seepage water by drainage blankets, trenches and relief wells - which are in turn protected by filters to prevent progressive removal of solid material from beneath the dam.

ii/ The reduction of excess pore pressure under the downstream slope by
a. Drainage measures.
b. Grout curtains
c. Sheet piling
d. Clay blanket.
} Also reduce piping danger.

A drainage trench can be successfully used as shown. This may be very effective if the surface strata upstream is left intact.




A clay blanket upstream of the dam reduces the hydrostatic pressures downstream as well as decreasing the seepage.



Can be successfully used together with relief wells. These wells are best placed at a wide spacing initially. If piezometers show more to be necessary then extra wells can easily be placed.

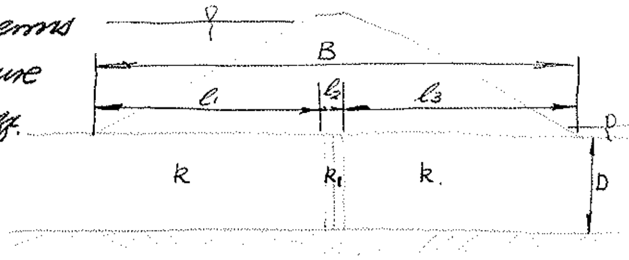
## Grout curtains

These achieve both a reduction in downstream pressures and a seepage reduction provided the particle size of the pervious stratum is suitable.

Grouting of alluvium is done with a mixture of cement, clay and chemicals and it is possible to reduce the value of $k$ to $\pm 10^{-5}$ cm/sec. If the permeability of the ground is not much greater than $10^{-5}$ cm/sec then a narrow grouted zone will achieve very little

eg Mangla. Initial value of $4 \times 10^{-1}$ reduced to $\pm 5 \times 10^{-5}$

The efficiency of any form of partial cut-off cannot be measured in terms of water flow. One must measure the head loss across the cut-off.




$$\eta = \frac{\Delta H_2}{\Delta He + \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta He}$$

where

$\Delta He$ = exit or entry head loss.

It can be shown that from the above eq$^n$

$$\eta = \frac{1}{1 + \frac{K_1}{K}\left\{\frac{B}{l_2} - 1 + 0.88\frac{D}{l_2}\right\}}$$

If $B/D = 10$ and $l_2/D = 0.2$ we have for $K_1/K = 0.02$ ie $\frac{2\times10^{-4}}{1\times10^{-2}}$

$\eta = 48\%$

and $l_2/D = 0.1$ $\eta = 32\%$

However if $K_1/K = \frac{2\times10^{-5}}{1\times10^{-4}} = 0.2$ then for $l_2/D = 0.2$

$\eta \doteq 10\%$

## Sheet Piling

This forms only a partial cut-off.

For $\eta = \frac{\text{Loss in head across piling}}{\text{Total loss in head.}}$

it was found that when new $\eta = 8-18\%$

with age $\eta = 20-38\%$

The improvement of efficiency with time is due to rusting and the migration of fines

Factors controlling the stability of slopes & foundations of E.D.

The main factors that must be considered are

i. Overtopping during and after construction.

This is principally a matter of hydrology - grassed earth slopes and rockfill are virtually as susceptable as bare earth.

ii. Instability due to shear failure.

This may be of the bank fill alone or the bank fill plus foundation. It may occur

a/ During or at the end of construction
b/ At full reservoir - under steady seepage.
c/ On drawdown of the reservoir.

• Failure during construction is more common in wet climates due to large construction pore pressures, or where there ~~are~~ a saturated clay foundation. It is generally the least catastrophic and probably occurs while the contractor is on site

• Failure at full reservoir under steady seepage is the least likely to occur provided reasonable filter zones have been provided

• Drawdown failure usually only dangerous if there is a rapid refill. Usually the crest is not broken by the slip. It may be a problem if an old reservoir is incorporated in a pumped storage sceme.

iii. Instability due to piping

This may take place very rapidly after a number of years with very little warning

iv. Wave action.

Adequate rip rap must be provided. A sandy fill - insufficiently protected may be breached during a storm just due to wave action

The Factor of Safety against shear failure - General.

Based on Elastic Theory.

The stress distribution within the dam is calculated together with the foundation, under various conditions of loading and stresses are compared with the strength of the soil.

Stresses in triangular embankment continuous with the foundation


Contours of maximum shear stress plotted as a ratio $\frac{\tau_{max}}{\gamma H}$.

In addition the maximum average shear stress for the $\phi = 0$ analysis was determined.

From elastic analysis $\frac{\tau_m}{\gamma H} = 0{\cdot}258$

$\phi = 0$ analysis $\tau_{av} = 0{\cdot}148\,\gamma H$.

Thus there is local overstress when $F = \frac{0{\cdot}258}{0{\cdot}148} = 1{\cdot}75$.

against complete failure on the rupture surface. Usually $F = 1{\cdot}5$ is considered satisfactory for the rupture surface and hence considerable overstress would have to be accepted even if the assumptions of elastic theory were satisfied, and actual stresses could only be satisfied if some redistribution stress was considered.

In actual fact the assumptions of the elastic theory are not satisfied

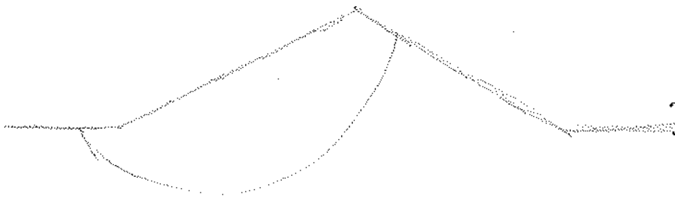
1. Hooks law is not obeyed.
2. Deformation modulus depends on mag. of average stress
3. The stresses are not fully reversible
4. Dam and foundation have different moduli.

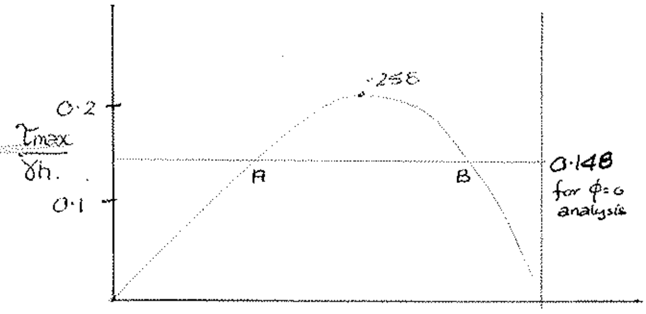
This elastic analysis is useful in that

a: Shows that taking $\gamma h$ as equal to $\sigma_1$ is not too far out in estimating pore pressures

b: Serves to illustrate the mechanism of prog. failure, Progressive failure. : Total Stresses $\phi = 0$.

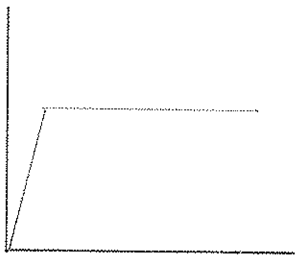
The elastic stress distribution enables an idea of the shear stress distribution across a typical slip surface to be obtained.



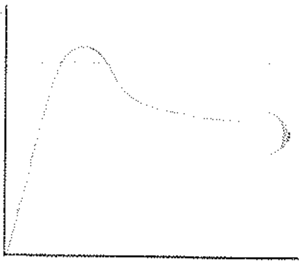



The $\phi = 0$ analysis postulates a uniform shear stress along the surface at failure. Thus according to the limit design there is a zone of overstress from $A \rightarrow B$ when the dam has $F = 1{\cdot}0$

If the stress strain curve was of the conventional plastic type redistribution of stress would occur until peak stress was fully mobilized over the full length of the slip surface and then complete rupture would occur.



In practice however the stress-strain curve (particularly when dealing with effective stresses) will have a peak. The local failure will spread from the zone of overstress into the zones of lower shear stress. While these zones approach peak value the shearing resistance of the clay in the area where failure started will drop towards the much smaller residual value.



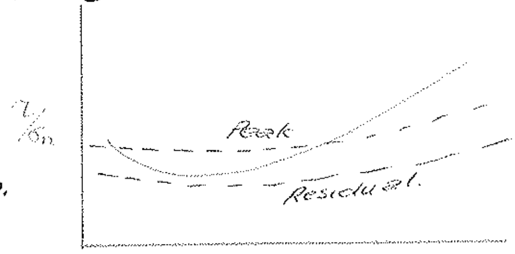
As a consequence. the total shearing force that acts along a surface of sliding at the instant of complete failure is considerably smaller than the shearing resistance computed at peak values.

$$\text{Brittleness index } I_b = \frac{\tau_f - \tau_r}{\tau_f}$$

When $I_b$ is large then progressive failure may play a significant effect.

## Progressive failure - effective stress.

Concerned not with distribution of $\tau_{max}$ but with distribution of $\tau/\sigma_n'$ ratio.

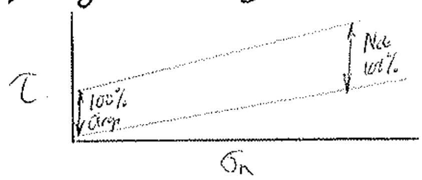



Consider values of $\tau/\sigma_n'$ along a trajectory of maximum shear stress.
For $\frac{u}{\gamma h} \approx 0.5$ ie standpipe level at surface.

In terms of effective stresses failure should start at the toe and at the top of the critical circle. By the time the strength has dropped to residual failure would have occured.

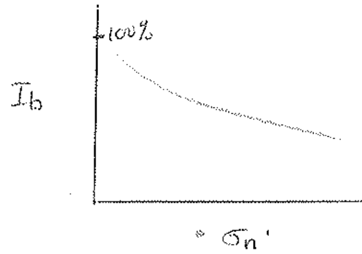
In practice the relative stress - strain curves of dam and foundation will influence progressive failure.

<u>Peak - Residual drop.</u>




At low values of $\sigma_n'$ there is percentagewise a large drop in strength peak - residual than at higher stresses.

Thus the influence of progressive failure will be more important in low embankments than in high embankments.




The redistribution of excess pore pressure is also an important factor when considering progressive failure

<u>Thus in this consideration progressive failure is</u>

1/ A function of the redistribution of stresses

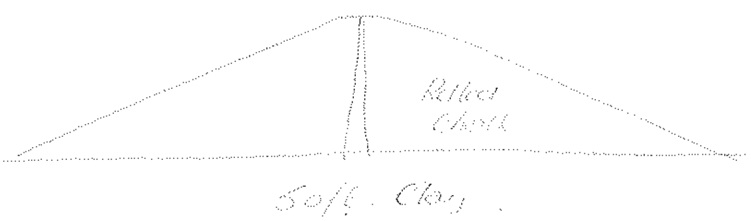
2/ Hence depends on p.w.p redistribution

3/ Hence dependant on the stress - strain curves of the constituent materials.

Because of the redistribution of excess pore pressure the process will be time dependant in practice. It does seem that reological effects are of a secondary importance

<u>Illustration of influence of stress - strain curves.</u>

Because of its rigid nature the chalk cracked
∴ no strength mobilized



## Limit Design Method of Analysis

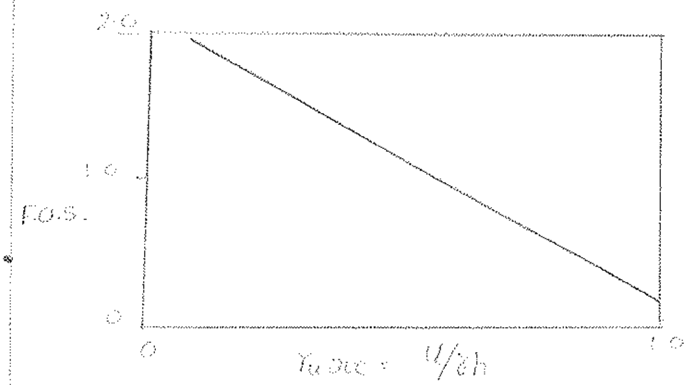
An estimate is made of the strength required to just maintain limiting equilibrium along the critical slip path. This strength is compared with the available strength of the soil to estimate the factor of safety

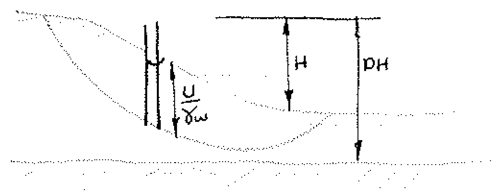
$$\left.\begin{aligned} s &= \frac{c'}{F} + (\sigma_n - u)\frac{\tan\phi'}{F} \\ s &= \frac{c_u}{F} \end{aligned}\right\} \text{mobilised shear strength.}$$

As stated earlier, it is postulated that at failure there is a uniform distribution of shear stress along the critical plane.

The effective stress analysis.

"The most important single conclusion of the effective stress analysis is that the F.O.S. is almost linearly related to the excess pore water pressure."




It was found that the value of F could be written

$$F = m - n\,r_u$$

where $m, n$ : are stability coefficients - not slopes > 1:2.

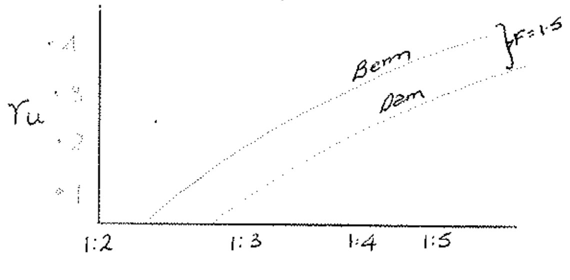
$r_u$ : average pore pressure ratio round slip.

The values of F from these coefficients are quite good enough for preliminary design and for checking a final design when a more complicated u distribution has been considered.

For Selset the stability above the berm as well as the whole dam was considered.

$\frac{c'}{\gamma H} = 0.01$ for dam

$c'/\gamma H = 0.02$ above berm.



By plotting lines of constant factor of safety the strong influence of $r_u$ could be clearly seen and it was possible to see what slopes could be maintained at the field value of $r_u$.

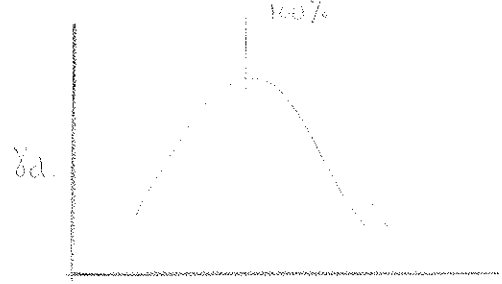
## Factors controlling the pore pressure.

### Stability during and at the end of construction.

The principal factors controlling the pore pressures are

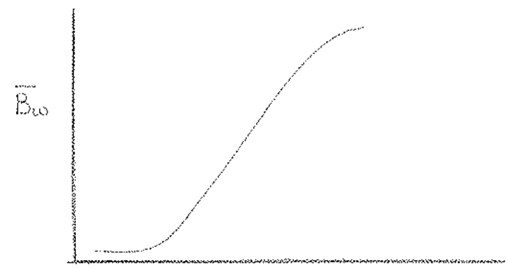
1. The placement moisture content and the degree of compaction
2. The state of stress in the dam
3. The rate of dissipation of pore pressure during construction.

### 1. Placement M.C.



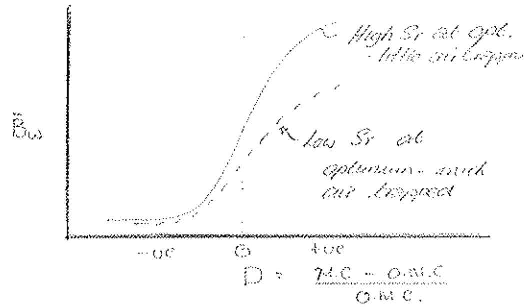

The O.M.C. values can differ greatly and so does the sensitivity of $\bar{B}_w$ to water content.

eg • If omc = 50% then $\bar{B}$ insensitive to a 1% change
• If omc = 7% then $\bar{B}$ very sensitive to a 1% change

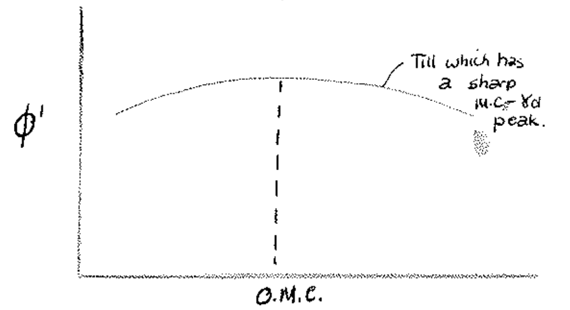


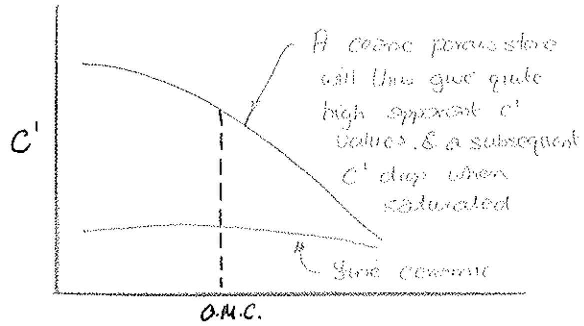

Can express the relationship of $\bar{B}$ to mc by plotting against the factor

$$D = \frac{M.C. - O.M.C}{O.M.C.}$$




The influence of M.C. on the shear strength parameters is illustrated as follows.



Placement moisture content.

<u>Dry-placement</u> – low pore pressures – good.
– too rigid and hence cracks.
– on saturation apparent c' will drop almost to zero and there will be a volume change (+ve at low loads & -ve at high loads – particularly if the compaction is poor)

<u>Wet-placement</u> – high pore pressures.
– however at least the core has to be placed in a wet condition so that it is sufficiently plastic.

<u>Note</u>

On drawdown the upstream slope will have to withstand a $r_u \approx 0{\cdot}45$ and so there is no advantage in having lower constructional pore pressures in the upstream slope.

## 2. The State of Stress.

$$\Delta u = B_w \left\{ \Delta\sigma_3 + A_w(\Delta\sigma_1 - \Delta\sigma_3) \right\}$$

In the central zone of the dam there is not much tendency for the stresses to rotate: hence –

$$\Delta u = \bar{B}\,\Delta\sigma_1$$
$$= \Delta\sigma_1\, B\left\{1 - (1-A)\left(1 - \frac{\Delta\sigma_3}{\Delta\sigma_1}\right)\right\} \quad \text{— (1)}$$

For fills in general $A < 1{\cdot}0$ thus $\bar{B} < B$

It is easy to measure $B$ under all round pressure but this will yeild an upper limit

The problem is the determination of $\Delta\sigma_1$ and $\Delta\sigma_3/\Delta\sigma_1$

From the elastic analysis contours of $\frac{\sigma_1}{\gamma H}$ show that along a typical slip surface $\sigma_1 \approx \gamma H$ although the direction of $\sigma_1$ is not necessarily vertical


In a zoned dam with a plastic core the may be a large ammount of arching in the core and $\frac{\sigma_1}{\gamma H} \approx 50\%$ in some cases.

Thus the value of $\Delta\sigma_1$ is easy to obtain

The ratio $\frac{\Delta\sigma_3}{\Delta\sigma_1}$ is more difficult and cannot be obtained from the elastic solution

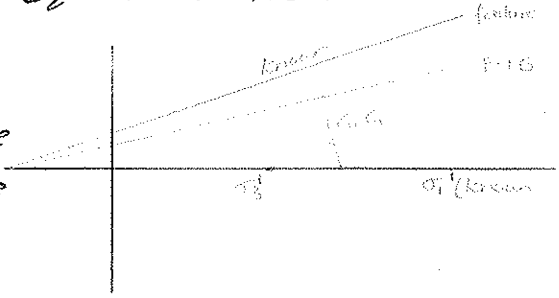
If the bank were a series of horizontal layers.

$$\frac{\Delta\sigma_3'}{\Delta\sigma_1'} = K_0$$

If some lateral yeild is allowed then the value of K will drop to a stress ratio equivalent to the F.O.S permitted

In the limit F = 1.0 and $K_f$ is obtained from the Mohr envelope

Thus it is possible to get the ratios of the effective stresses



$$K = \frac{\Delta\sigma_3'}{\Delta\sigma_1'} = \frac{\Delta\sigma_3 - \Delta u}{\Delta\sigma_1 - \Delta u} = \frac{\Delta\sigma_3 - \bar{B}\,\Delta\sigma_1}{\Delta\sigma_1' - \bar{B}\,\Delta\sigma_1}$$

$$= 1 - \frac{(\Delta\sigma_1 - \Delta\sigma_3)}{(1 - \bar{B})\,\Delta\sigma_1}$$

$$\therefore \quad \frac{\Delta\sigma_1 - \Delta\sigma_3}{\Delta\sigma_1} = (1-K)(1-\bar{B}) \qquad \text{——— (2)}$$

Now equation 1 gives

$$\bar{B} = B\left(1 - (1-A)\left(1 - \frac{\Delta\sigma_3}{\Delta\sigma_1}\right)\right)$$

from eq$^n$ 2

$$\bar{B} = B\left(1 - (1-A)\left\{(1-K)(1-\bar{B})\right\}\right)$$

$$\therefore \quad \bar{B} = \frac{B\,(1 - (1-A)(1-K))}{\{1-B)\,(1-A)(1-K)}$$

Thus knowing the value of A, B and K it is possible to determine $\bar{B}$

| Soil | $K_0$ | A | B | $\bar{B}$ $K = K_0$ | $\bar{B}$ $K = K_f$ |
|---|---|---|---|---|---|
| Sandy Clay | 0·5 | +0·5 | 0·8 | 0·75 | 0·72 |
| Clay gravel | 0·4 | 0 | 0·8 | 0·62 | 0·50 |

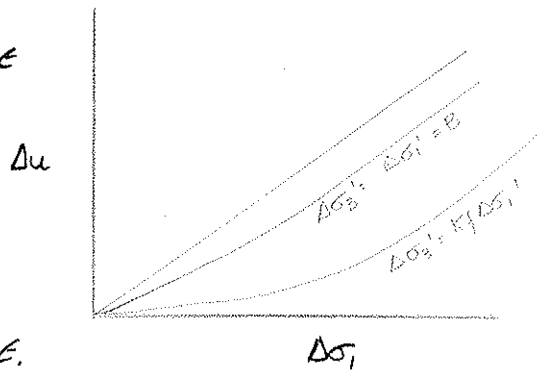
Thus the influence of the stress ratio is important in dialatant soils

For correct determination of $\bar{B}$ values

It is necessary to carry out an undrained test with pore water measurements while applying increments of $\sigma_1$ and $\sigma_3$ in appropriate ratio.

The test must be done slowly and there is the problem of air migration around the sample through the rubber membrane.

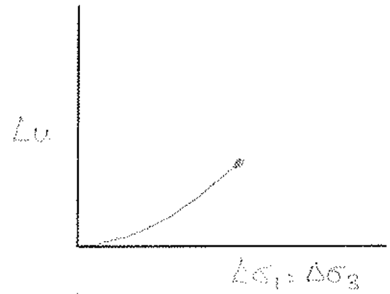
From the results of such a test the curves of $\Delta u$ vs $\Delta\sigma_1$ can be obtained




The form of the results will vary both with soil type and with the initial water content.

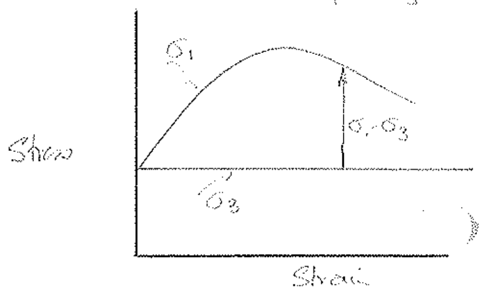
Consider the standard undrained test.

On applying the all round cell pressure we have $\Delta u = B\,\Delta\sigma_3$

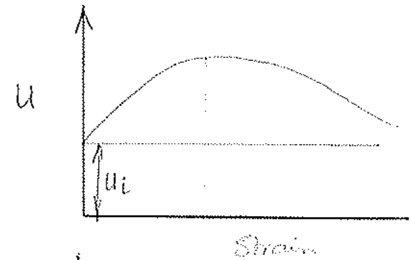



On applying the deviator stress we get a further increase in pore pressure.

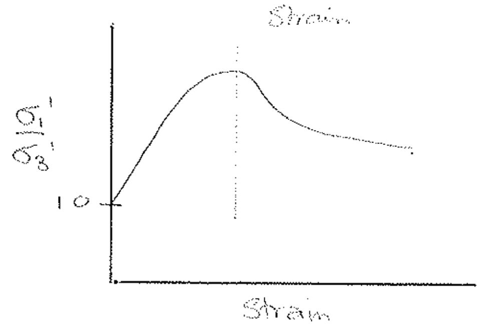
This results in a peak value followed by a falling off on further strain.



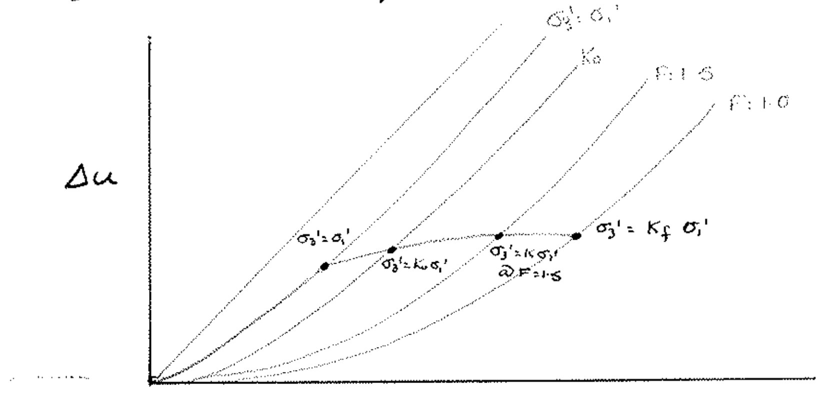

Similarly we get a peak in the $\frac{\sigma_1'}{\sigma_3'}$ curve.




Now: If one plots $\Delta u$ vs $\Delta\sigma_1$ in the usual manner for different values of stress ratio $\frac{\Delta\sigma_1'}{\Delta\sigma_3'}$ then a family of curves is obtained as shown above.



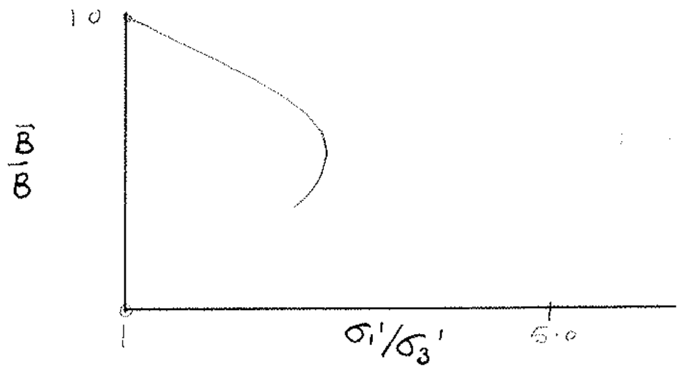

But during the process of the undrained failure the ratio $\frac{\Delta\sigma_1'}{\Delta\sigma_3'}$ is changing the whole time ie the shear stress has an effect on the value of $\Delta u$ set up at any one moment.




Thus the stress ratio has a large effect on the value of $\bar{B}$ where $\bar{B} = \dfrac{\Delta u}{\Delta \sigma_1}$

and $u = u_o + \bar{B}\, \Delta\sigma_1$

The effect can be shown quite clearly in the following plot.




Where the purpose of the test is the accurate prediction of pore pressure at states of stress other than failure a more accurate result is obtained if the stress increments occurring in practice are closely followed in the test by making simultaneous changes in the values of both $\sigma_1$ and $\sigma_3$. The test result is conveniently expressed in terms of the relationship

$$\Delta u = \bar{B}\, \Delta\sigma_1$$

$\bar{B}$ only gives the change in pore pressure due to a stress change under undrained conditions. The actual pore pressure depends also on the initial values of $u_o$

$$u = u_o + \bar{B}\, \Delta\sigma_1$$

In rolled fill the initial value is usually negative.

In cases where no dissipation of pore pressure is assumed to occur, the pore pressure ratio $r_u$ used in the stability analysis is directly related to $\bar{B}$

$$r_u = \frac{u}{\gamma h} \qquad \text{but} \quad u = u_0 + \bar{B}\,\Delta\sigma_1$$

we assume that $\Delta\sigma_1 = \gamma h$.

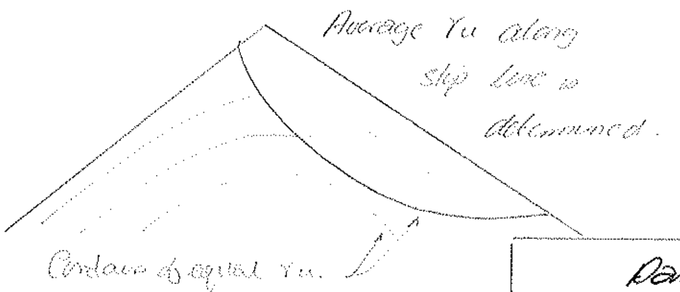
$$\therefore \quad r_u = \frac{u_0}{\gamma h} + \bar{B}$$

In dams compacted wet of optimum - material of low plasticity

$$\frac{u_0}{\gamma h} \text{ is small} \qquad \therefore \ r_u = \bar{B}$$

iii/ <u>The Dissipation of pore pressure.</u>

Some dissipation occurs, even in large dams without drains, especially near the edges and contacts with pervious strata.

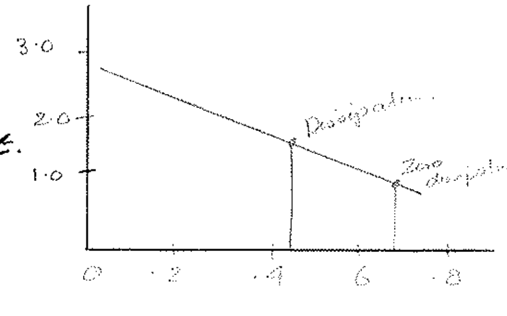



The effect of dissipation reduces the ave $r_u$ by a significant amount and cannot be neglected for an economical design

| Dam | $\frac{r_u \text{ ave}}{r_u \text{ max (no dissipation)}}$ |
|---|---|
| Anderson Ranch | 0·54 |
| Green Mtn. | 0·67 |
| Granby | 0·66 |
| Vallecito | 0·43. |

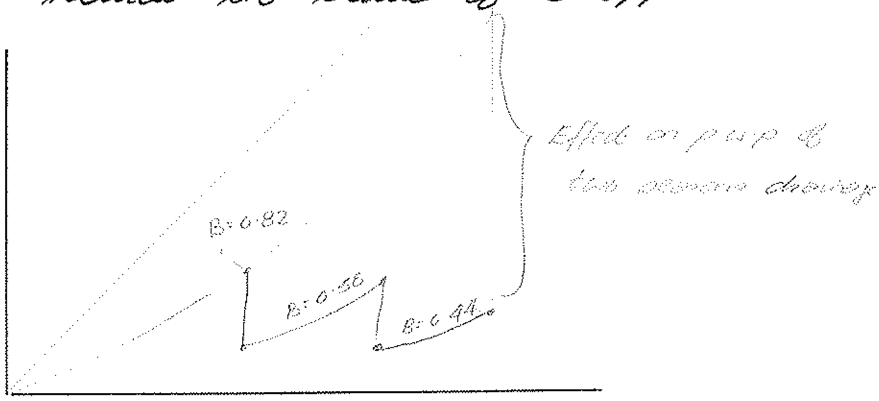
Now the factor of safety is linearly dependent on the value of $r_u$.

∴ As shown by the graph the value of $r_u$ can be very important.

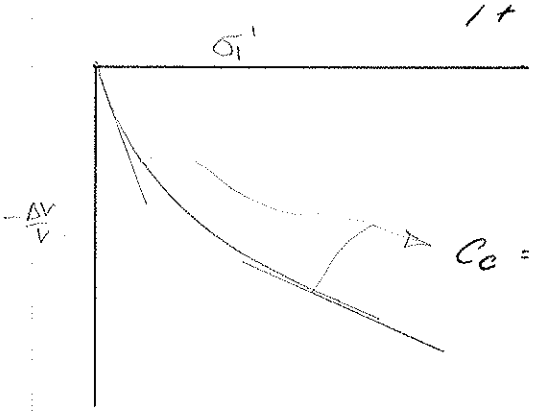


The effect of pore pressure dissipation has two effects.

1. It reduces the existing pore pressure
2. It reduces the value of $\bar{B}$ applicable to the next lift



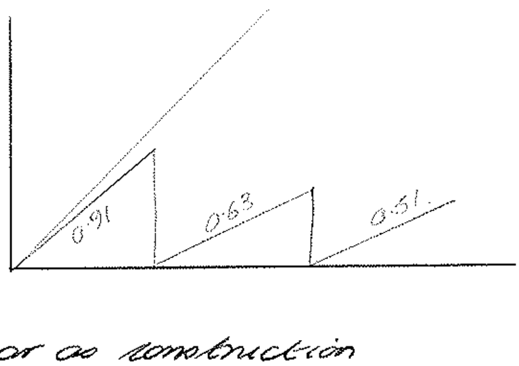

In general $B = \dfrac{1}{1 + n_0 \frac{C_w}{C_c}}$ where $C_w$ = compressibility of the fluid phase. This is approximately independent of drainage in a wet soil because air and water leave together.




$C_c$ = compressibility of the soil structure, and this decreases with increase in effective stress.

Example : Usk Dam.

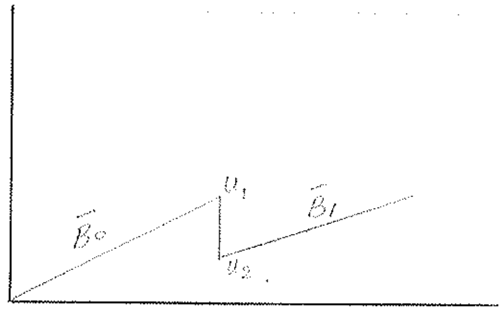
Laboratory tests showed results typically as follows.

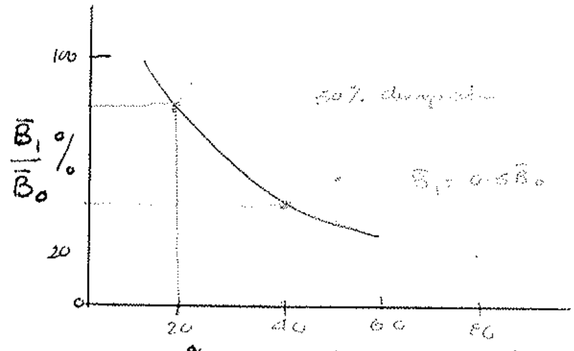



The actual decrease in the value of $\bar{B}$ after partial dissipation exceeded the predicted drop. This probably represents the additional influence of both

1. Change in state of shear as construction proceeded
2. The continuation of the dissipation during the construction periods as well as during shut-down.

The influence of the ammount of dissipation in the preceeding stage on the subsequent stage value of $\bar{B}$ is illustrated as follows – Usk piezometers.




Drainage Blankets.

Usk : 110' : blankets @ 40' spacing
$C_v \approx 10\ ft^2/mth$

Selset : 130' : blankets @ 15' spacing
$C_v \approx 1\ ft^2/mth$
50% dissipation in 3 mths.

Diddington : 70' : 5' spacing
$C_v \approx 0{\cdot}1\ ft^2/mth.$

---

Stability when reservoir full

The principal risks are due to erosion and piping due to

1. Settlement cracks in the fill
2. Absence of a filter layer between core and rock or coarse gravel fill.
3. Concentration of flow at the junction between the earth core and :
   1. Concrete or steel culvert.
   2. Bedrock – fissured or poor concrete cut-off.
4. Incomplete cut-off and lack of filter protection for the drains in the foundation strata.

Stability under steady seepage.

Excess pore pressures set up in this case are usually less critical than during construction and drawdown and can be controlled by various drainage measures

Methods of determining the flow net.

a. Sketching
b. Relaxation
c. Analog solutions
d. Finite element methods.

One is usually so uncertain as to the permeability that the use of elaborate methods is not warrented.

Effect of $K_H > K_V$.

Let $\bar{y} = y\sqrt{\frac{k_x}{k_y}}$ ie increase measurements on the Y axis

or $\bar{x} = x\sqrt{\frac{k_V}{k_H}}$ ie decrease scale of X axis.

When the problem has been solved the figure is corrected back to its original scale.

The expression for the seepage is given by

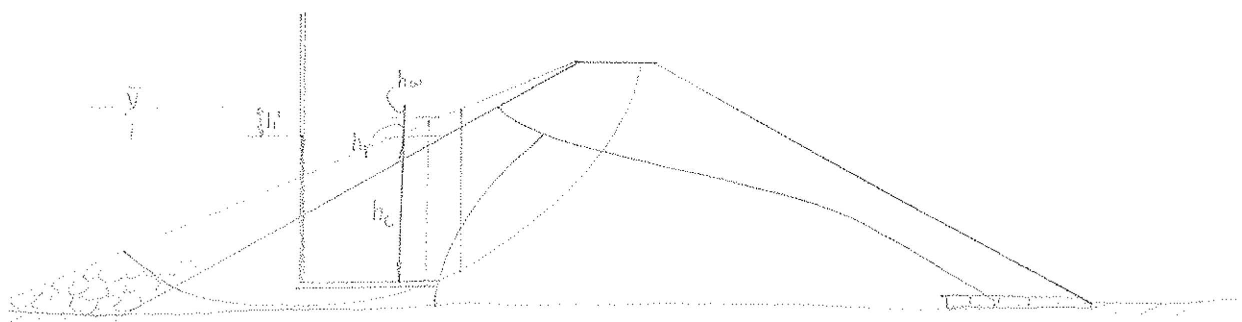
$$Q = \sqrt{k_x k_y} \cdot \frac{N_f}{N_d} \cdot h.$$

In analysing the stability of this state one must consider $c'$ and $\phi'$ values of samples that have been saturated.

Stability under rapid drawdown.

a. Compressible soils.

The changes in total stress on the removal of the water lead to changes in pore water pressure.



Initial pore pressure at A:

$$u_0 = \gamma_w \{ h_c + h_r + h_w - h' \}$$

After drawdown this will change to $u = u_0 + \Delta u$.

Now $\Delta u = \bar{B}\, \Delta\sigma_1$

$$\therefore u = u_0 + \bar{B}\, \Delta\sigma_1$$

Major total principal stress before drawdown

$$(\sigma_1)_0 = \gamma_c \cdot h_c + \gamma_r \cdot h_r + \gamma_w \cdot h_w$$

after drawdown

$$\sigma_1 = \gamma_c h_c + \gamma_{rd}\, h_r$$

where $\gamma_c, \gamma_r$ = saturated densities

$\gamma_{rd}$ = drained density

Thus $$\Delta\sigma_1 = -\{ (\gamma_r - \gamma_{rd}) h_r + \gamma_w h_w \}$$

now $\gamma_{rd} = \gamma_r - n_0 \gamma_w$   $n_0$ = specific porosity = volume of water drained from unit volume.

$$\therefore \Delta\sigma_1 = -\{ n_0 \gamma_w h_r + \gamma_w h_w \}$$

$$= -\gamma_w \{ n_0 h_r + h_w \}$$

Thus $$u = u_0 - \bar{B} \gamma_w ( n_0 h_r + h_w )$$

$$= \gamma_w \{ h_c + h_r + h_w - h' \} - \bar{B} \gamma_w \{ n_0 h_r + h_w \}$$

$$u = \gamma_w \{ h_c + h_r(1 - \bar{B} n_0) + h_w(1 - \bar{B}) - h' \}$$

Note that this final equation is the same as the initial equation but with $h_r$ and $h_w$ multiplied by reduction factors.

Discussion

The smaller the value of $\bar{B}$ the greater will be the residual pore pressure.

$$\bar{B} = B \left\{ 1 - (1-A)\left(1 - \frac{\Delta\sigma_3}{\Delta\sigma_1}\right) \right\}$$

For saturated soils $B = 1$

$A \to 1.0$.

For $A < 1.0$   $\bar{B} \nless B$ and may hence be ~~greater~~ than unity. ie have a ~~good~~ reducing value

poor

It must be remembered that changes in total stress on drawdown consist of a decrease in $\Delta\sigma_1$ and an even larger decrease in $\sigma_3$.

As a convenient rule for saturated fills $\bar{B} = 1.0$

Thus the equation simplifies to

$$u = \gamma_w \{ h_e + h_r(1 - n_0) - h' \}$$

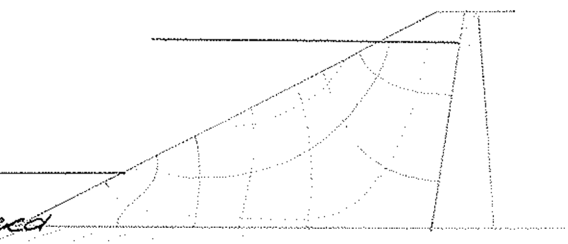
The value of $u_0$ should be taken from the pore pressures measured before drawdown and not a value from the flownet. The $h'$ term is important and should be as accurate as possible.

b. <u>Incompressible soils</u>

These are relatively pervious soils in which the volume of water flowing during drawdown is large compared with the volume change of the soil due to stress change.

With these soils it is necessary to use a flow net with a moving boundary.

<u>The limiting case is instantaneous drawdown.</u>



The flow net will be somewhat as shown.

The effect of a finite rate of drawdown can be considered by a finite step by step analysis.

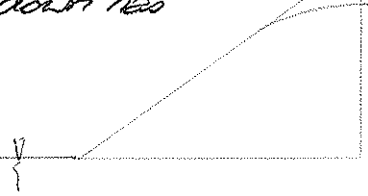
The position of the free surface and the shape of the flow pattern is a function of <u>drawdown rate, permeability and specific porosity</u> - for a given geometry

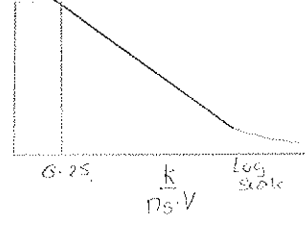
$$\text{Position \& shape} = f\left(\frac{k}{n_s \cdot V}\right)$$

$V$ = Vertical rate of drawdown

<u>Reinius did much work on a 2:1 slope.</u>

This has been used by Bishop to enable easy determination of the phreatic surface when the drawdown has been completed.





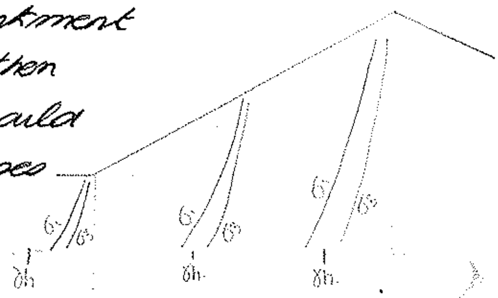

The position of the free surface can thus be determined.

## Stability under reservoir full conditions

The principal risks under this condition are effects of erosion by piping through cracks in the fill.

### Settlement cracks.

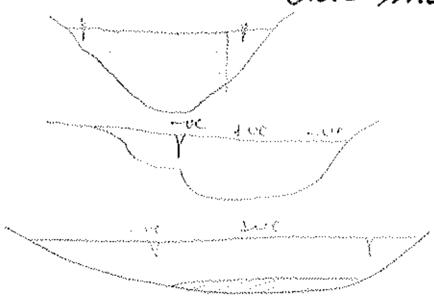
If one considers the variations in $\sigma_1$ and $\sigma_3$ with depth in a homog embankment under conditions of plane strain then it is apparent that cracking should not occur as both principle stresses are always positive



Thus any cracking is associated with non-uniformity of properties or departures from plane strain

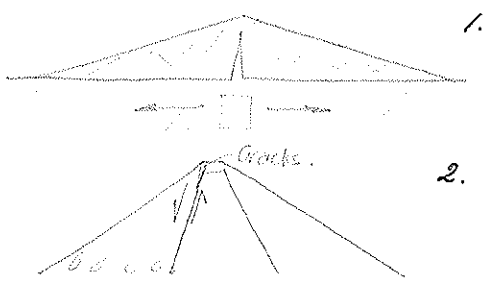
a: Transverse cracks

These may be due to



1. Steep sided valley - departure from plane strain
2. Discontinuities in valley slopes. Sherard Phd
3. Compressible foundation in valley floor Leonards & Noray.

b. Longitudinal cracks.



1. Rigid fill on a soft foundation
   If one builds fast then there is no time for dissipation
2. Differential settlements
   - rockfill and rolled clay core.

c. Horizontal cracks.

As a plastic core settles between rigid walls of compacted material there is a tendency for arching to take place ie the vertical stress can be considerably less than the overburden pressure.

If the sides of the core were nearly vertical then the general equation

$$\sigma_v = \frac{B(\gamma - c'/B)}{K \tan \delta'} \left\{ 1 - e^{-K \frac{z}{B} \tan\phi'} \right\} + S e^{-K \frac{z}{B} \tan \phi'}$$

Now during construction we have virtually undrained conditions in the dam and no surcharge

$$\therefore \quad \sigma_v = \frac{B(\gamma - c_u/B)}{K \tan \phi_u^*} \left\{ 1 - e^{-K z/B \tan \phi_u} \right\}$$

Using L'Hopitals rule we have

$$\sigma_v = \left\{ \gamma - \frac{c_u}{B} \right\} z$$

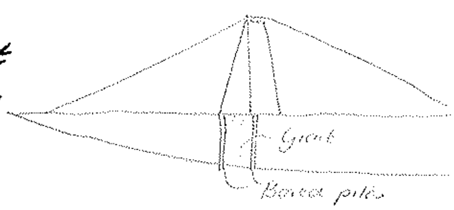
If on reservoir filling $\sigma_v$ is low enough such that

$$\sigma' = \sigma_v - \gamma_w H = 0$$

then there is the possibility of hydraulic cracking. It does however seem that the opening up of a horizontal crack will be associated with poorly bonded layers rather than be formed in a homogenous mass.

d. General cracking due to non-uniformity of foundation

The Shek Pik dam is an example of this. The rigid wall virtually acted as a pivot about which the dam bent and cracked on settling



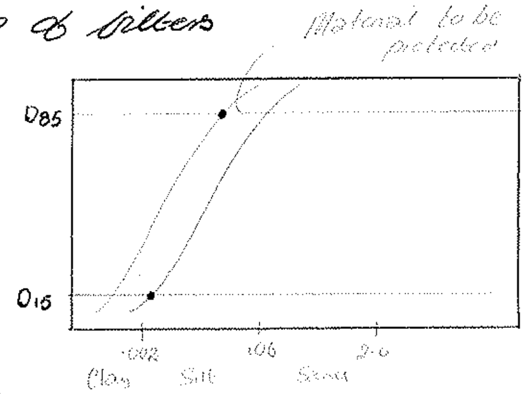

## Protection against Piping

Protection is afforded by means of filters

The Terzaghi - Bertram criteria

Uniformly graded material.

Two criteria - one aimed at preventing the base being washed through the filter.




Piping Ratio $\dfrac{D_{15}\ \text{filter}}{D_{85}\ \text{finest soil}} \leq 4.$

The second criteria is to insure that the water shall be able to drain from the filter

$$\therefore \quad \frac{D_{15}\ \text{filter}}{D_{15}\ \text{coarsest soil}} \geq 4.$$

Terzaghis criteria includes a built in factor of safety.

Filter Criteria : US Corps.

They specify $\frac{D_{15}\text{ filter}}{D_{85}\text{ fine base}} \leq 5.$

and $\frac{D_{15}\text{ filter}}{D_{15}\text{ coarse base}} \geq 5.$

In addition they have a specification for the ratio of the $D_{50}$ sizes

$$R_{50} = \frac{D_{50}\text{ filter}}{D_{50}\text{ base}} \overset{?}{\geq} 25$$

This $R_{50}$ specification is to allow for the effects of segregation and is specified in the USBR criteria in greater detail.

Design of Relief Wells.

The screen or slot opening in a relief well shall not be $> \frac{1}{2}\, D_{85}$ size of the material surrounding it.

ie $D_{hole} \leq \frac{1}{2}\, D_{85}$

It may thus be necessary to use a few layers of filter material - all obeying the relevant criteria around the hole. Finest material effectively used is a fine cement sand.

The USBR Criteria

These are the most comprehensive. These apply to well graded materials as well as uniformly graded.

Uniformly graded material $u\left(\frac{D_{60}}{D_{10}}\right) = 3\text{-}4.$

$$R_{50} = \frac{D_{50}\text{ filter}}{D_{50}\text{ base}} = 5-10$$

$R_{15}$ not specified - not necessary

Well graded filters - subrounded grains

$$R_{50} = 12-58$$
$$R_{15} = 12-40.$$

Well graded filters of angular particles

$R_{50} = 9 - 30$

$R_{15} = 6 - 18$

The $R_{15}$ ratios are strict while the $R_{50}$ values replace Terzaghi's piping ratio.

Also

1/ If the material grading ranges from Gravel >10% passing N°4 to silt (over 10% passing N°200) then the limits should be based on material passing N°4.

2/ Maximum size of filter ≯ 3"

3/ Filters should not contain >5% passing N°200.

---

Rockfill Dams.

There are four principal types.

a: Upstream membrane dams.

These are built on a rock foundation and may have very steep slopes at the natural angle of repose of the rock. Previously were of dumped rock but nowadays using at least some compacted rock eg Venemo. Face preparation - hand placed or dumped.

1. Concrete deck

- on the higher dams - especially if dumped fill - large movements - drawdown to repair

- low dams can be constructed to have little movement eg Quiock.

2. Asphalt deck

- prepared surface - thin layer of asphalt - it is more plastic - upstream slope 1:7 for ease of working

3. Wood or Steel

- usually use steel only in cases of extremely high construction rate requirements - special care over joints - can take deformation

b Central Membrane types.

These have only been used on low to medium height dams.

The membrane may be concrete eg Crane Valley Dam.
asphalt eg Duhm Valley 35m.
steel or wood.

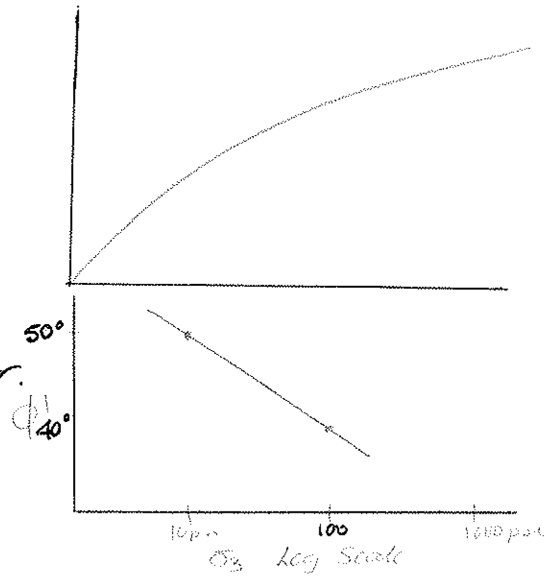
In the case of steel, wood and sometimes concrete some earthfill may also be used in the core.
The concrete membrane may be flexible R/C or pannels with joints.

c/ Sloping Core Dams – Growdon Type.
Core built at angle of repose – upstream acts as a weighting zone. – usually built on a rock or a strong alluvium foundation (Brownlee).
Careful filter control

d/ Narrow central core dams
May prefferably have a slightly sloping core – shoulders of gravel or rock-fill. Careful filter controll.

Properties of rockfill.

Under the pressure in a 1000' dam (900 psi) the rockfill will tend to breakdown and crush. There is thus a possibility than under earthquake shocks large pore pressures may be set up.
This breakdown of the particles affects
1/ Volume change and pore pressure on shearing
2/ The shape of the failure envelope which is very curved.
Under high stresses Rockfill tends to have a lower value of $\phi'$ than gravel.



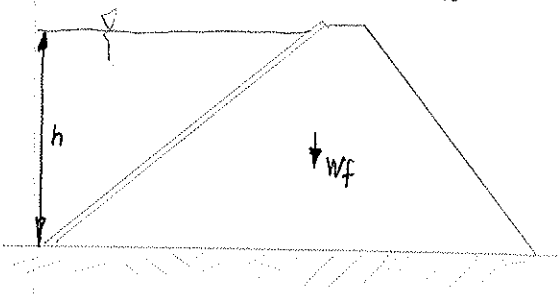

Effect of saturation
Rockfill placed in a dry condition will undergo large deformations on saturation.
Discussed later.

NB: $\sigma_3 = 10\,psi \approx H = 20'$

The stability of Rock-fill against shear.

There are two possibilities

1. Shear along the base.
2. Passive failure of downstream fill




Horizontal Force = $\frac{1}{2} h^2 \gamma_w$.

Vertical Force = Wt of fill + vertical water load.

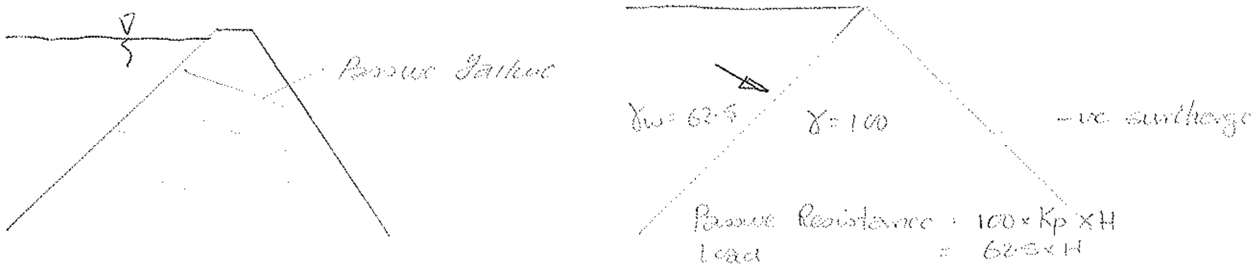
$$V_F = W_f + 1.3 \times \tfrac{1}{2} h^2 \gamma_w$$

Thus $$\frac{H_F}{V_F} = \frac{\frac{1}{2} h^2 \gamma_w}{W_f + 1.3 \times \frac{1}{2} h^2 \gamma_w} = \tan \phi'_R.$$

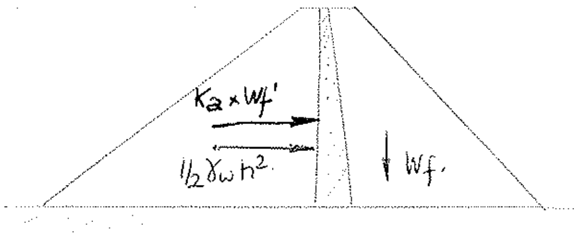
Thus the value of $\phi'$ required for limiting equilibrium can be calculated.

Passive Failure:

From tables the passive pressure of a rock-fill wedge full to the crest can be determined




In the case of a central core dam a similar operation is performed.




Horizontal force = Water + active pressure of submerged fill

Vertical force = Weight of downstream fill

$$\frac{H_f}{V_f} = \tan \phi'_r$$

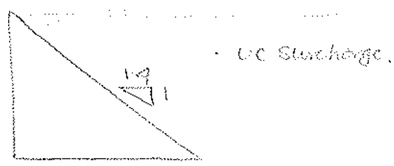
In general a good base angle of friction is required and flatter slopes will be used.

Passive failure.

Passive Resis = $K_p \times \frac{H^2}{2} \times \gamma$

Load = $H^2 \gamma_w$

$\therefore F = K_p \dfrac{\gamma}{\gamma_w}$




For F = 1.0 require $\phi' = 45°$.

Settlements and Deformations

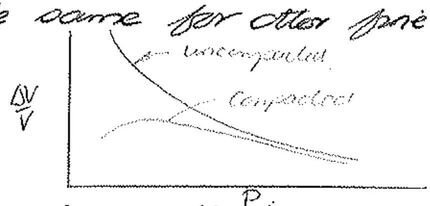
This is particularly important where there is a concrete upstream membrane. The deformation is normally three dimensional and hence design is difficult.

The magnitude is influence by three important factors.

1/ On first loading rock fill undergoes considerable compression accompanied by subsequent long term structural creep.

2/ The magnitude of the compressibility decreases with decreasing load. This is the same for other fine grained materials.

Thus the compressibility can be reduced by compaction

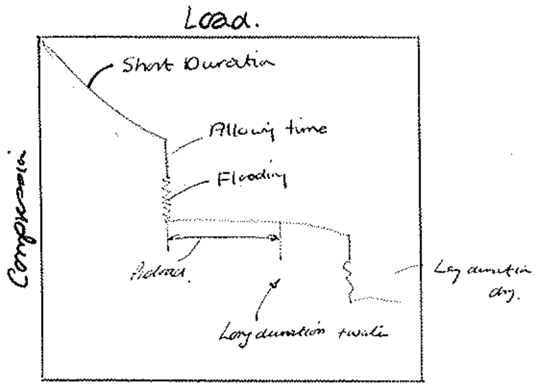



This compaction also has a marked effect on creep

3/ Material under high loads, if placed dry, undergoes a sudden decrease in volume on saturation. This applies to compacted soils under high loads as well.

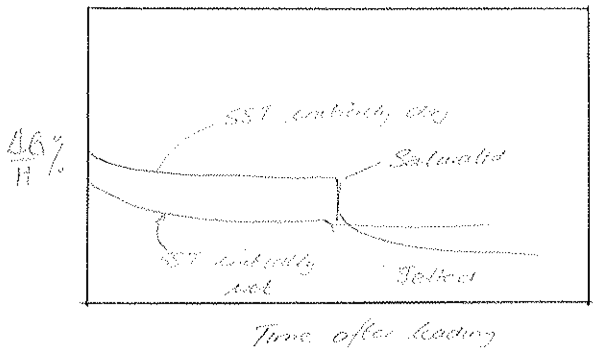
The effect of saturating rock fill

- A fill will compress in relation to the load upon it.
- Under constant load the movements will continue with time
- Adding water will accelerate and increase the compression.




If at this stage water is no longer added and the contact points are allowed to dry the yield stress at the points will increase to that of dry rock. Thus on further loading the fill will have greater resistance and the compression of the dry fill will be reduced in the same way as if the fill had been preloaded. If the fill is loaded above the preload the contact points will again be adjusted to the dry strength of the rock. Thus the cycle will be repeatable.

Results illustrating these effects have been given by Bjerrum and also Sowers et al.




Cogswell Dam:

15" rain - 11 ft.

Additional sluicing to 6%

Settlement with time

| | | | | | | |
|---|---|---|---|---|---|---|
| 85m | Dix River | after | 35 yrs | > 0.02% | per year. | ≤ 1.97 m. |
| 100m | Kenney. | | 9½ yrs | 0.016% | " " | ≤ 0.6 m |
| 110m | Paradela | after | 3 yrs | > 0.02% | " " | ≤ 1.0 m |
| 100m | Lewis Smith | after | 7yrs. | ≪ 0.02% | " " | ≤ 0.15m. |

Relationships - settlement vs height.

$$S = 0.001 \times H^{3/2} \qquad \text{Lawton}$$

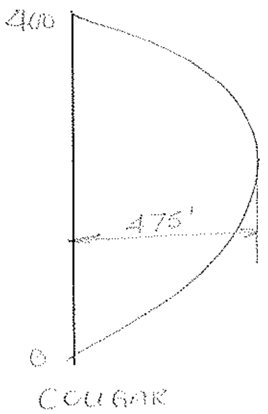
$$S = k_s H^{2.1} \qquad \text{Kjaernsli}$$

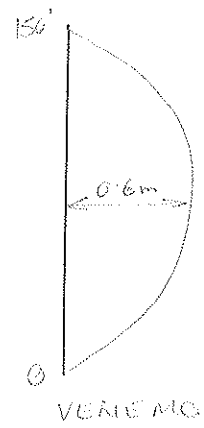
$k_s$ .00005 → .00011

In actual fact construction procedure more important than height eg Lewis Smith 0.15%.

Kenney. 0.55%

Compressibility of Rockfill in Dam.




Placement of Winterfill

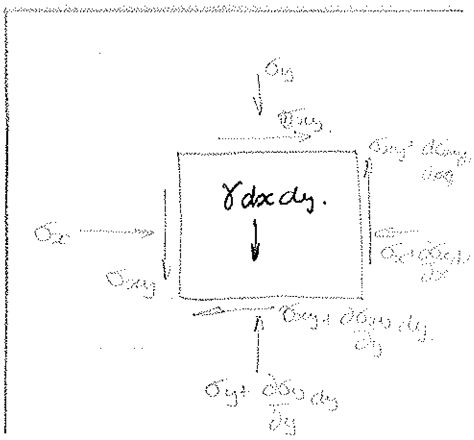
1/ Place in < 2m layers

2/ Sluice in spring when temp rises

3/ Lower $\phi$ value in winterfill

## Stress distribution in a narrow plastic clay core.

Consider an element of the core which is considered to be in plastic equilibrium.




### For vertical equilibrium

$$-\frac{\partial\sigma_y}{\partial y}\,dy\,dx + \gamma\,dx\,dy - \frac{\partial\sigma_{xy}}{\partial x}\,dx\,dy = 0$$

$$\therefore \quad \frac{\partial\sigma_y}{\partial y} + \frac{\partial\sigma_{xy}}{\partial x} - \gamma = 0.$$

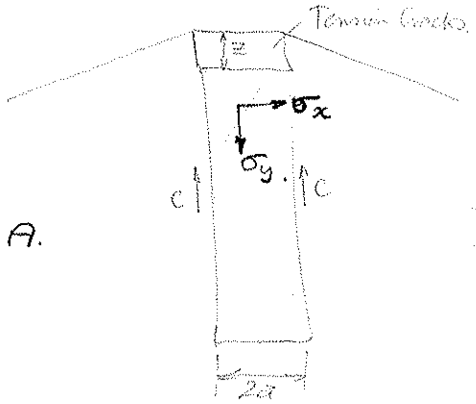
### For horizontal equilibrium.

$$\frac{\partial\sigma_x}{\partial x} + \frac{\partial\sigma_{xy}}{\partial y} = 0$$

Assume that there is no drainage and $\phi_u = 0$
Thus in plastic equilibrium we must satisfy the equation

$$\sigma_1 = \sigma_3 + 2c_u$$ in terms of ~~total~~ principal stresses.

$$(\sigma_x - \sigma_y)^2 + 4\sigma_{xy} = 4c^2$$




### For a parallel sided core:

$$\sigma_x = \gamma y - c_u\, y/a + A.$$

$$\sigma_y = \gamma y - c_u\, y/a \pm 2c\sqrt{1 - \frac{x^2}{a^2}} + A.$$

$$\sigma_{xy} = c\,\frac{x}{a}.$$

This solution satisfies the boundary conditions

$$\sigma_{xy} = c \quad @\ x = a$$
$$\sigma_{xy} = -c \quad @\ x = -a.$$

To evaluate the constant A: Consider state of equilibrium at bottom of tension crack zone. Assume no side shear until the bottom of crack is reached.

$$\therefore \quad \sigma_y \ @\ z = \gamma z$$

Now the average vertical stress across the width of the core in the plastic zone is

$$\overline{\sigma_y} = \frac{1}{2a}\int_{-a}^{+a} \sigma_y\,dx.$$

$$\therefore \; \bar{\sigma}_y = \frac{1}{2a}\int_{-a}^{+a}\left\{\gamma y - C_u\frac{y}{a} \pm 2c\sqrt{1-\frac{x^2}{a^2}} + A\right\}dx$$

$$= \frac{1}{2a}\left[\left(\gamma - \frac{C_u}{a}\right)y \;\overset{\text{action}}{\pm 2c}\left[\frac{x}{2}\sqrt{1-\frac{x^2}{a^2}} + \frac{a}{2}\sin^{-1}\frac{x}{a}\right] + Ax\right]_{-a}^{+a}$$

$$= \underline{\left(\gamma - \frac{c}{a}\right)y \pm \frac{\pi}{2}\cdot c + A.}$$

<u>Using St Venants Principal</u>

$$\bar{\sigma}_y = \gamma z \quad @ \quad y = z$$

$\therefore$ Taking positive values for active we get

$$A = c\left\{\frac{z}{a} - \pi/2\right\}.$$

Thus the expression for $\sigma_x$ is

$$\underline{\sigma_x = \left(\gamma - \frac{c}{a}\right)y - C_u\left[\pi/2 - z/a\right].}$$

Thus $\quad P_A = \int_z^H \left\{\left(\gamma - \frac{c}{a}\right)y - c\left(\pi/2 - z/a\right)\right\}dy.$

$$= \tfrac{1}{2}\left(\gamma - c/a\right)(H^2 - z^2) - c\left(\pi/2 - z/a\right)(H - z).$$

<u>The factor of safety is put on $C_u$.</u>

$$\therefore \; P_A = \tfrac{1}{2}\left(\gamma - c/aF\right)(H^2 - z^2) - \frac{c}{F}\left\{\pi/2 - \frac{z}{a}\right\}\left\{H - z\right\}.$$

<u>Using the horizontal strip approach.</u>

$$\sigma_v = \left\{\gamma - \frac{C_u}{B}\right\}z$$

$$\therefore \; P_a = \int_{z_0}^{H}\left(\gamma - \frac{C_u}{B}\right)z\,dz$$

$$= \left[\gamma - \frac{C_u}{B}\right]\frac{z^2}{2}\Bigg]_{z_0}^{H}$$

$$= \tfrac{1}{2}\left\{\gamma - \frac{C_u}{B}\right\}\left(H^2 - z_0^2\right)$$