Geotechnical aspects of the Carsington Dam failure

Aspects géotechniques de la rupture du barrage de Carsington

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Carsington Dam is an earth-fill embankment with a length of 1200 m and a maximum intended height of 37 m above foundation level. It is situated in Derbyshire, in the Midlands of England. On Friday 1 June 1984 the dam was within about 1 m of its final height. No fill was placed during the weekend, a period of heavy rainfall. Early in the morning of Monday 4 June a slip started in the upstream bank over a length of 190 m and by Thursday 7 June it extended along a length of nearly 500 m, with a maximum horizontal displacement of 15 m.

The owner, the Severn-Trent Water Authority, immediately ordered a full investigation into the cause and mechanism of failure. The investigation has been made by the writer, with the assistance of Dr P.R. Vaughan, in association with Babtie Shaw & Morton as consulting engineers. Soil Mechanics Ltd carried out the greater part of the geotechnical site and laboratory work. Supplementary borings and tests were made by the Building Research Station, and Dr D.J. Petley of Warwick University carried out the tests on shear and slip surfaces.

An Interim Report was submitted in October 1984 and a Report on Principal Findings followed in March 1985. The latter included a paper, by Dr D.J. Coats of Babtie Shaw & Morton and the writer, which had been given earlier that month to a Symposium on Failures in Earthworks to be published by the Institution of Civil Engineers. The present paper concentrates on geotechnical aspects of the initial failure. It is based on the recently completed programme of tests, and a re-examination of the field data, but the conclusions differ in no essentials from those reached in the previous reports.

Geology

The dam, forming part of a pumped storage scheme, is located in a small valley in mudstones of Upper Carboniferous age. The adjacent higher ground, mostly above the 200 m contour (Fig. 1), is covered in places by boulder clay and glacial gravel, remnants of deposits laid down in the penultimate (Saale) glaciation. Ice-sheets of the last (Weichselian) glaciation approached within about 15 miles of the site which was therefore subjected during this period to periglacial conditions. As a result the mudstones were brecciated and crumpled to a depth of several metres and the valley slopes became mantled by Head, clayey material moving downhill by the freeze-thaw process of solifluction. In the past 10,000 years, since the return of temperate conditions, little change has taken place in the valley slopes, apart from weathering and the formation of topsoil; but the stream has slightly deepened its bed, replacing Head and the uppermost weathered mudstone by its own alluvium in a floodplain 50 to 70 m wide approximately defined by the 170 m contour.

Test pits in the lower valley slopes just upstream of the dam reveal a typical section as shown in Fig. 2. Head, containing occasional 'foreign' pebbles derived from the upland glacial deposits, averages about 1.2 m in thickness. It lies over, and (apart from the pebbles) is practically indistinguishable from, mottled yellow, brown and grey clay forming the top of the in-situ weathered mudstone. For geotechnical purposes Head beneath the subsoil (a_2) and this upper layer of weathered mudstone (b_1) are grouped together under the name 'Yellow Clay'.

Near the junction of a_2 and b_1 there are shear surfaces, formed by downslope movement of Head in the last glacial period. The shears are smooth, gently undulating surfaces, often gleyed and typically several metres in extent. On average they occupy rather more than 50 percent of the total length of 62 m examined in test pits outside the dam.

Below the Yellow Clay the weathered mudstone is dark grey in colour and changes with depth from a clay (b_3) to a brecciated material (b_4) and finally to a relatively soft blocky mudstone. Traces of the original bedding planes can be found in b_4 but no shears exist in any of the grey weathered material.

The valley slopes show no tendency to flatten as they run down to the floodplain or to the bottom of tributary gullies. Measurements between contours at 10 m intervals in the vicinity of the dam give slope angles mostly from 4° to 8° with a few values up to 10°. The slopes are stable under present groundwater and climatic conditions.

The dam

For a typical section through the core and upstream shoulder of the dam see Fig. 6. The downstream shoulder is of similar design, with a slope of 2.5:1. Construction of the embankment began in July 1982 and reached El. 182 at the end of October. The next working season began

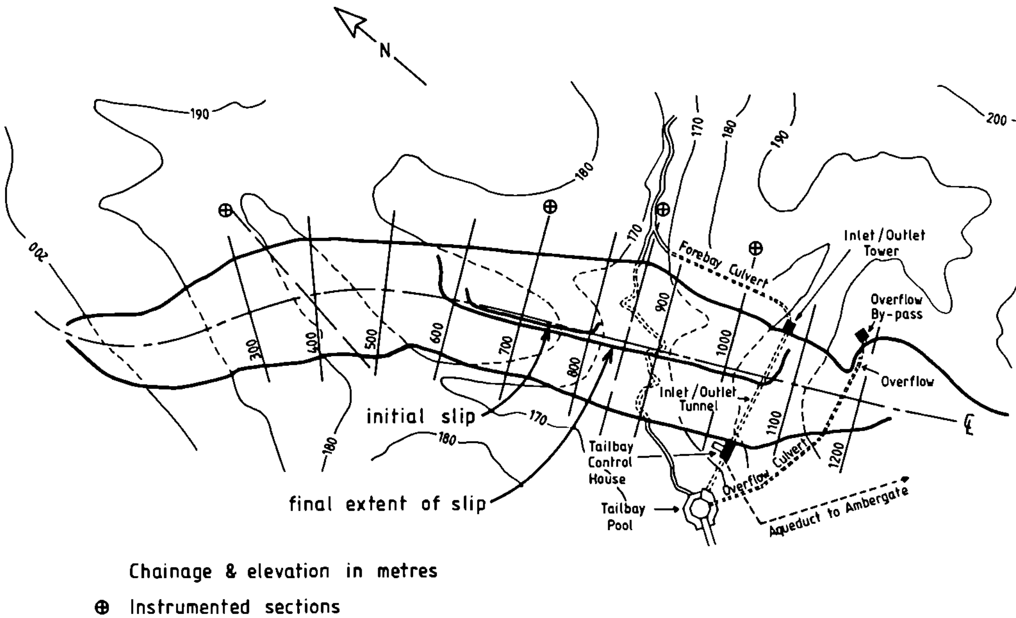

Fig. 1 Plan of Works

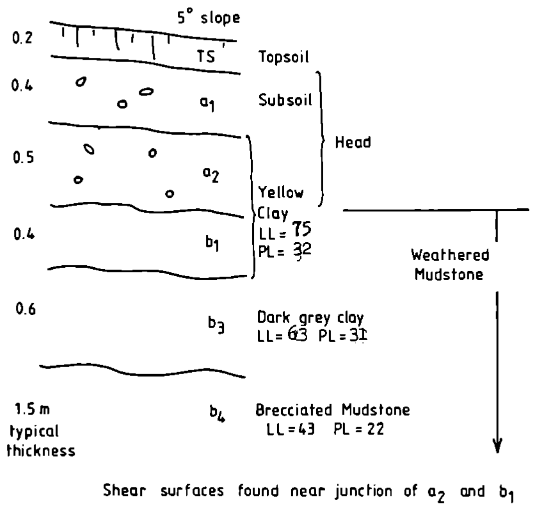

Fig. 2 Section in natural ground

in April 1983. During the summer the contractor built an upstream toe berm to El. 176 between Ch. 690 and 930 and construction reached El. 197 by the end of the season; several metres higher than had originally been planned without the berm. Placing of fill in the third season started in April 1984 and reached El. 201 at the end of May. A slip in the upstream bank began on 4 June.

The central core and its upstream extension (referred to as the 'boot') consist of rolled clay derived from strata a_2, b_1 and b_3. The shoulders are composed of brecciated and blocky mudstone, compacted at or near optimum water content, the more clayey materials being in Zone I and the somewhat harder materials in Zone II. Drainage blankets occur in both Zones of fill at about 3 m intervals and basal drainage blankets are laid on the foundation.

In the flood plain the alluvium was removed before construction and the dam was founded on brecciated or blocky mudstone. Elsewhere, top soil and subsoil were removed beneath the shoulders of the dam which therefore rest on the Yellow Clay.

The compaction equipment tended to cause rutting or small scale bearing capacity failures in the core. Examination of 250 mm diameter samples taken from a borehole revealed numerous local shears due to this effect throughout the full depth of the core, at inclinations ranging from horizontal to more than 60°. In the three large excavations made through the upstream shoulder after the failure some horizontal compaction surfaces were seen in Zone II fill, but these appear to be rare and there is no evidence to suggest they played any significant role in the initial failure.

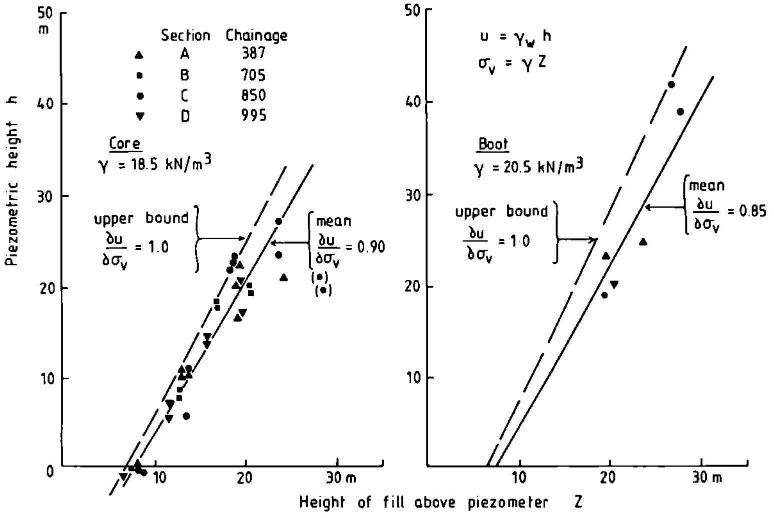

Fig. 3 Piezometer observations, 21 May 1984

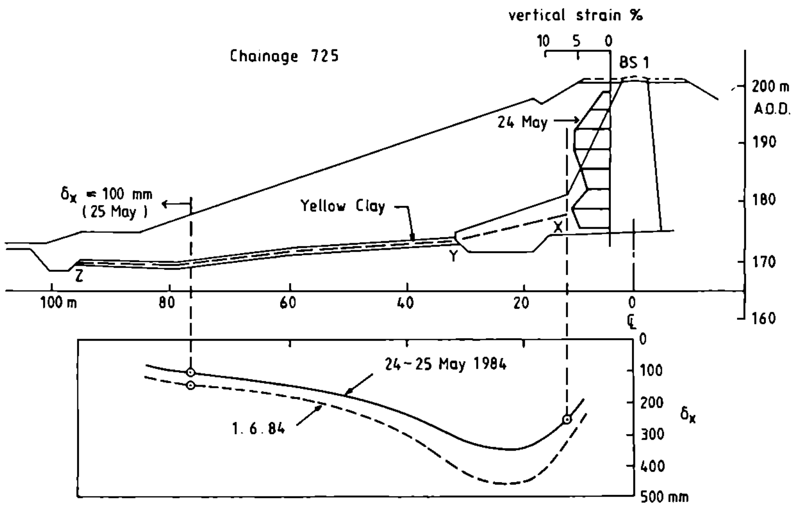

Fig. 4 Horizontal movements on line XZ and vertical strain in core

<u>Pore pressures, settlements and displacements prior to failure</u>

Systematic piezometer observations throughout construction show little if any excess pore pressures in the foundation strata or in Zone II fill, and only small pore pressures in Zone I. The relationships between pore pressure and height of fill above piezometer level in the core and boot are plotted in Fig. 3, the data being derived from the last complete set of observations at four instrumented sections two weeks before failure. The measured values incorporate the effect of a small amount of pore pressure dissipation, principally in the boot, during the winter shut-down periods.

Settlement gauge readings, also taken throughout construction at four sections, show vertical compressions in the core up to 6 percent on 24 May 1984. Values taken from the nearest instrumented section to Ch. 725 are plotted in Fig. 4. Here the average compression, ignoring the two uppermost readings, is 4.8 percent in the central core the average width of which is 12.5 m. Part of this compression would result from a reduction in air voids under load, amounting to about 1 percent. An estimate of the horizontal displacement of a vertical plane through point X is therefore

$$\frac{1}{2} \times \frac{4.8 - 1.0}{100} \times 12500 \simeq 250 \text{ mm}$$

From 18 August 1983 observations were made of horizontal displacements of the upstream face of the dam at survey points or 'pegs' in the position shown in Fig. 4. Measured displacements on 25 May 1984 are given in Table 1 together with approximate total displacements estimated by backward extrapolation to the start of construction. These movements clearly reflect the height of fill, see Fig. 5, and by interpolation the total displacement on 25 May at Ch. 725 is about 100 mm.

Table 1

'Peg' displacements on 25 May 1984

Chainage	Height of fill on centre line m	Displacement Since 18.8.83 mm	Estimated total mm
675	21.5	30	50
750	27.5	90	120
875	36	130	180
1050	19	25	40

A finite element analysis of this section by Dr David Potts of Imperial College provides a pattern of the distribution of displacements as sketched in Fig. 4.

By 1 June, about 0.5 m of fill having been placed in the preceding week, the 'peg' displacement increased from approximately 100 to 140 mm. The other displacements in the section probably increased more or less in the same proportion giving the pattern, immediately before failure, indicated by the dotted line in Fig. 4. The corresponding average vertical strain in the core is about 6.5 percent.

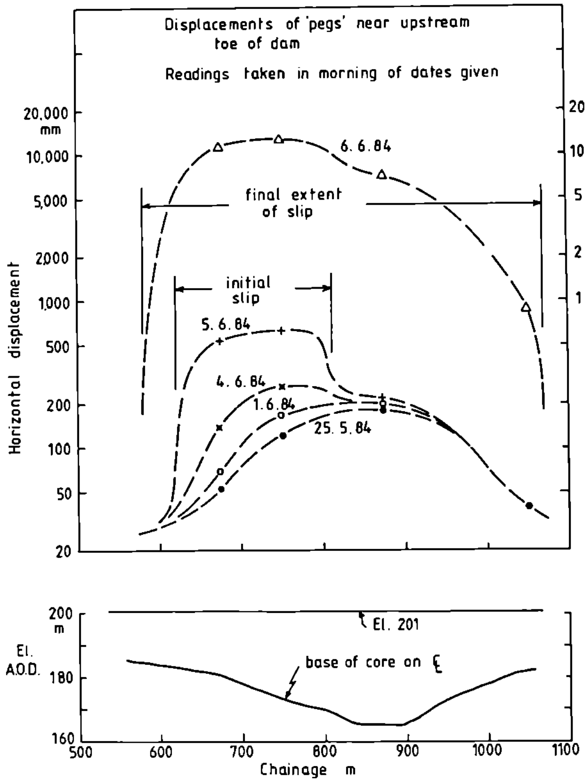

Fig. 5 Propagation of slip movements

The failure

When the work force arrived on site at 7.30 a.m. on Monday 4 June a crack was seen on the crest of the dam. It had a maximum width of 50 mm and extended from about Ch. 620 to 810, i.e. a length of approximately 190 m. By 9.30 a.m. the 'pegs' had been surveyed and showed movements of 75 and 95 mm respectively at Ch. 675 and 750 since Friday 1 June, but virtually no additional movement at Ch. 875. Thus during the weekend, no fill having been placed, the upstream slope in the area of the initial failure had been subject to plastic deformation or creep at an average rate of 25 to 30 mm/day.

Between 9.30 a.m. on Monday 4 June and the same time on Tuesday 5 June the 'pegs'at Ch. 675 and 750 moved a further 380 and 360 mm respectively, so in this 24 hour period slip had been taking place, on average, at 15 mm/hour. In this period creep movements of 9 mm/day started at Ch. 875. Total movements were now approximately 520 mm at Ch. 675, 620 mm at Ch. 750 and 210 mm at Ch. 875. These movements vividly demonstrate the initial slip: see Fig. 5.

During Tuesday morning slip accelerated to 100 mm/hour and the failure propogated southwards; as measured at Ch. 875 slip was now occurring there at 11 mm/hour. During the afternoon rates increased rapidly. By 6.30 p.m. the initial slip had moved 2.5 m and the subsequent slip (at Ch. 875) rather more than 0.5 m.

By Wednesday morning (6.6.84) the slip reached still further south; even at Ch. 1050 a movement of 0.9 m was recorded, and in the central portions the displacements were now between 7 and 12 m (Fig. 5). Meanwhile an emergency berm was being built on the upstream toe. The combined effect of the berm and the large slump at the crest eventually brought motion to a halt by Thursday morning. The slip at that time extended from Ch. 580 to 1070, a total length of nearly 500 m.

Borings and a large excavation with two pits taken down through the foundations enable the section shown in Fig. 6 to be drawn at Ch. 725, near the centre of the initial slip where maximum displacement occurred. The principal features are: a near-vertical slump and back scarp 10 m deep at the crest, backward rotation of a block upstream of the core, resulting in a deep-seated thrust and a graben in the tension zone above, and a sub-horizontal translation of the toe block of about 15 m. The slip surface passes down through the core and boot and through the Yellow Clay, finally thrusting up over the small rock-fill toe.

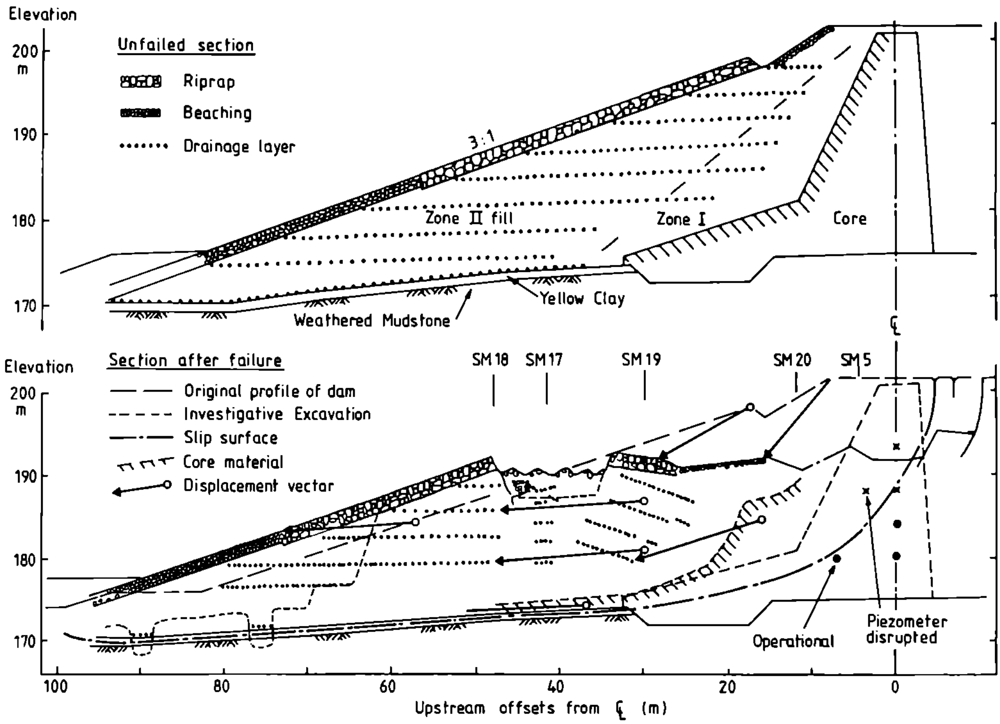

Fig. 6 Investigative section at chainage 725

A similar pattern was found in the flood plain section at Ch. 825 but here, in the absence of the Yellow Clay, the slip surface passes through the Zone II fill about 1 m above its base.

It is evident from the sequence of events that the slip in the flood plain section was subsequent to, and caused by load transfer from, the initial slip. The mechanism of load transfer, as the strength on the initial slip surface falls to its (very low) residual value, is an interesting and important matter which has been investigated by Dr Vaughan and is briefly described by him in a separate contribution to this Conference in Speciality Session 9A. Here it is sufficient to state that the effect can account for at least a 20 percent reduction in factor of safety at Ch. 850; i.e. from 1.25 to 1.0.

However, this depends chiefly on the post-peak drop in strength. Load transfer, in either direction, would have been negligible before initial failure and any three-dimensional restraints were small.

Geotechnical properties of the Yellow Clay

Index properties of the Yellow Clay have the following average values:

water content	= 40
liquid limit	= 75
plastic limit	= 32
plasticity index	= 43
clay fraction ($< 2\mu$)	= 62%

Peak strength parameters of the intact clay are

$$c' = 10 \text{ kPa} \quad \text{and} \quad \phi' = 20^{\circ}$$

the results (Fig. 7) being practically independent of sample size or type of test.

Residual strength was determined by shear box tests on slip surface samples, with the polished and striated slip surface placed exactly in the separation plane of the box. Fig. 8(a) shows a typical stress-displacement curve and the results of all these (fifteen) tests, plotted in Fig. 8(c), give

$$c' = 0 \quad \text{and} \quad \phi'_r = 12^{\circ}$$

Stress-strain curves for samples 20 mm and 100 mm thick, tested in 60 mm and 300 mm shear boxes respectively, show little scale effect up to strains of 30 percent, see Fig. 9, and it is interesting to note the way in which the strength parameters decrease with increasing post-peak strains, see Fig. 10. However, horizontal 'displacement shears' would probably have been developing by the end of the test and thereafter strength might be expected to decline to the residual more in accordance with absolute displacement than with shear strain. From ring shear tests on clays the additional displacement, after 30 percent shear strain, to produce residual strength is typically of the order 200 mm. Thus for a layer 1 m thick the stress-displacement curve for intact clay will be approximately as shown by line (a) in Fig. 11.

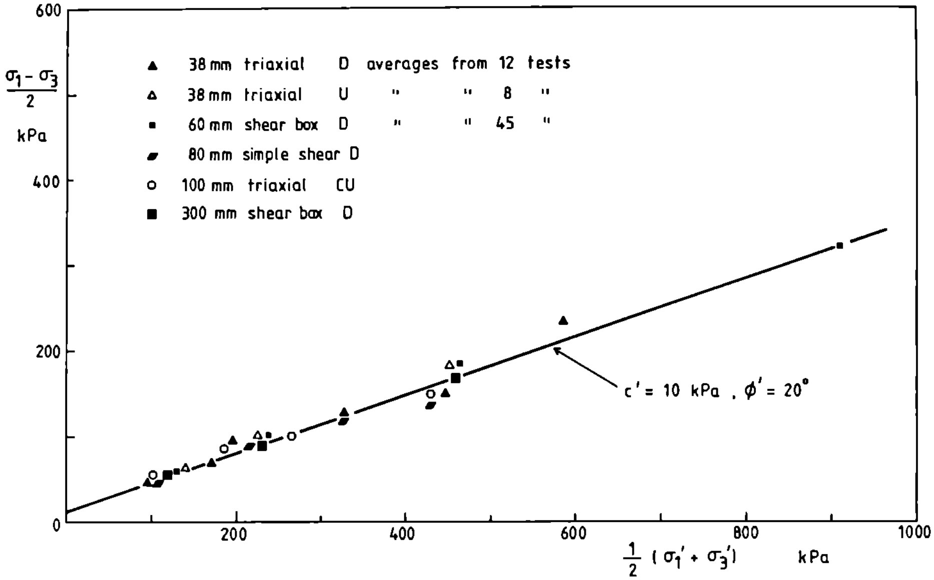

Fig. 7 Yellow Clay Shear strength (peak , intact)

Triaxial tests on samples taken diagonally across a solifluction shear, so that failure in the test occurs along the shear, show a small peak followed by a drop to residual after not more than 4 or 5 mm, see Fig. 8(b). Up to peak the displacements are strain controlled and the stress-strain curve is almost identical to that of intact clay. The drop from peak to residual, by contrast, is entirely a function of post-peak displacement as the shear is pre-existing. The stress-displacement curve for a 1 m thick layer of clay containing a (horizontal) pre-existing shear surface is therefore as shown by line (b) in Fig. 11.

The resultant stress-displacement curve for a layer containing any given proportion of sheared to intact clay, all subject to the same displacement, can now be drawn. For a proportion of 50 percent, as at Carsington, the curve is line (c) in Fig. 11.

In reality a minor correction is required to allow for the fact that the shears are undulating, not plane. This can be taken into account by increasing the 'effective' residual, as measured in a horizontal direction, to (ϕ_r + i) and it is reasonable to assume i = 2°. Thus when the intact clay reaches peak the strength in the sheared clay will be represented by φ' = 14°. At larger displacements the 'horizontal' strength of the sheared clay, as also in the intact clay, must fall to the residual.

The peak and post-peak strength parameters of the Yellow Clay thus deduced are set out in Table 2.

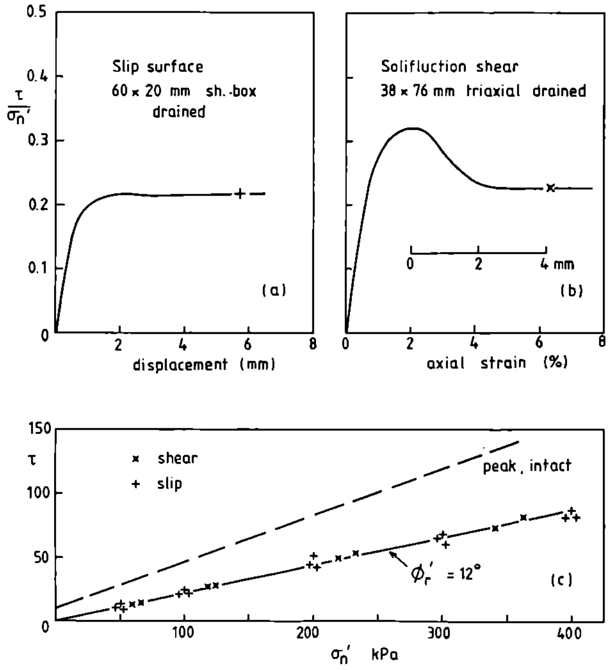

Fig. 8 Yellow Clay Strength on solifluction shears and slip surfaces

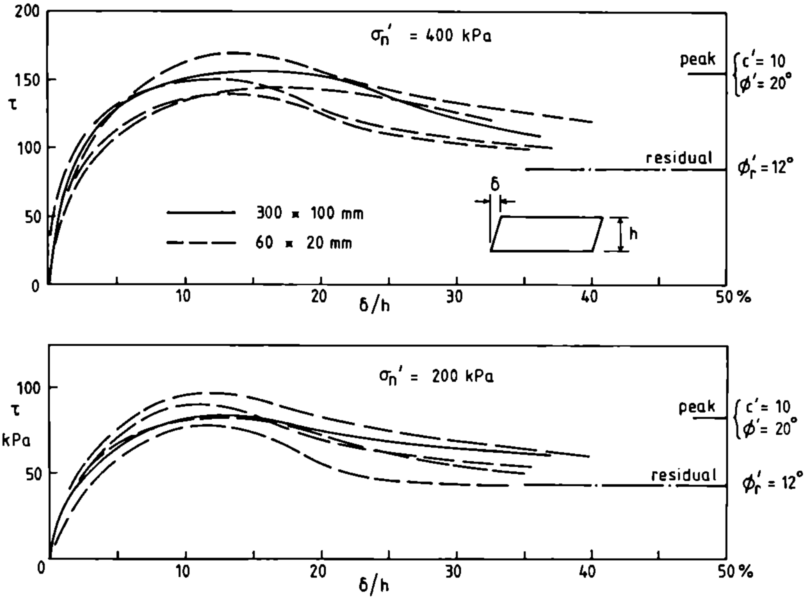

Fig. 9 Yellow Clay Shear box tests (drained)

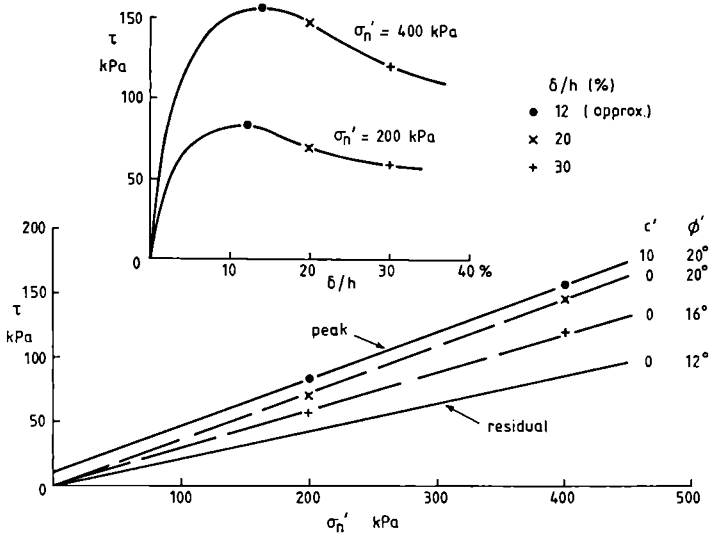

Fig. 10 Yellow Clay Post-peak strength (intact)

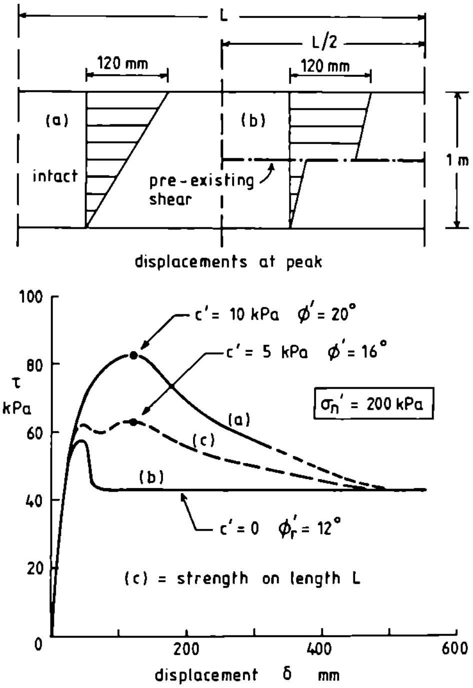

Fig. 11 Stress - displacement curves for a clay layer with a pre-existing shear

Table 2

Yellow Clay layer, 1 m thick, with 50 percent of its length containing pre-existing solifluction shears

State	Displacement	Strength parameters					
		Intact		Shears		Resultant	
	mm	c'	ϕ'	c'	ϕ'	c'	ϕ'
Peak	120	10	20°	0	14°	5	17°
'Critical State'	200	0	20	0	12	0	16
'End of test'	300	0	16	0	12	0	14
Residual	500	0	12	0	12	0	12

Geotechnical properties of the core material

Typical index properties of the core material are:

Liquid limit	= 69
Plastic limit	= 31
Plasticity index	= 38
Clay fraction (< 2μ)	= 56%
Standard Compaction { water content	= 27
Standard Compaction { unit weight	= 18.3 kN/m^3

On average the material has a water content of 34 with a unit weight of 18.5 kN/m^3 and an undrained shear strength of 65 kPa.

The relationship between pore pressure and vertical overburden pressure, derived from field observations, is shown in Fig. 3. The average coefficient of consolidation is $c_v = 2\ m^2$/year.

Tests to measure the effective-stress peak strength parameters of intact core material are plotted in Fig. 12. They give

$$c' = 15 \text{ kPa and } \phi' = 21^\circ$$

In undrained triaxial tests the peak is reached at about 4 percent axial strain, Fig. 13(a). At 6 percent strain the strength falls 15 percent below peak and the corresponding effective stress parameters, Fig. 13(b), are

$$c' = 5 \text{ kPa and } \phi' = 21^\circ$$

At the 'critical state' condition, with $c' = 0$ and $\phi' = 21^\circ$, there is a reduction of approximately 20 percent in undrained strength below the peak and this is reached at about 8 percent axial strain.

Triaxial tests on samples taken diagonally across rutting shears in the core, so that failure occurs on the shear, show (Fig. 14) 'peak' parameters $c' = 0$ and $\phi' = 16^\circ$ and a residual strength of $c' = 0$ and $\phi'_r = 13^\circ$.

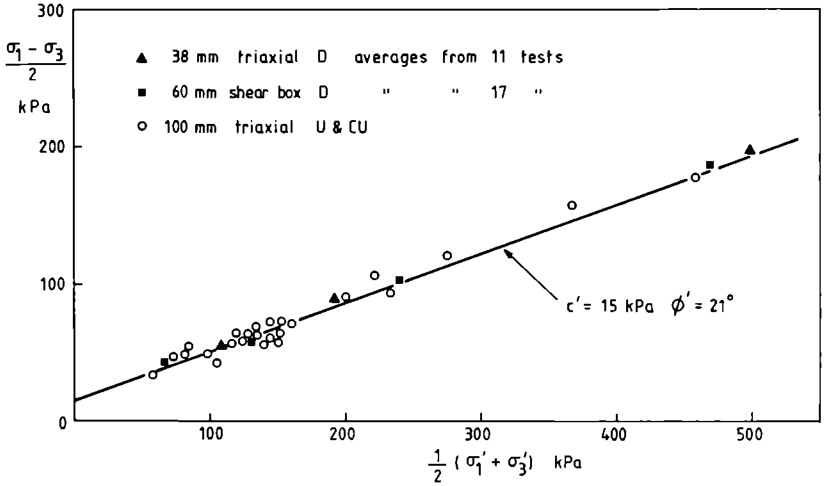

Fig. 12 Core Shear strength (peak, intact)

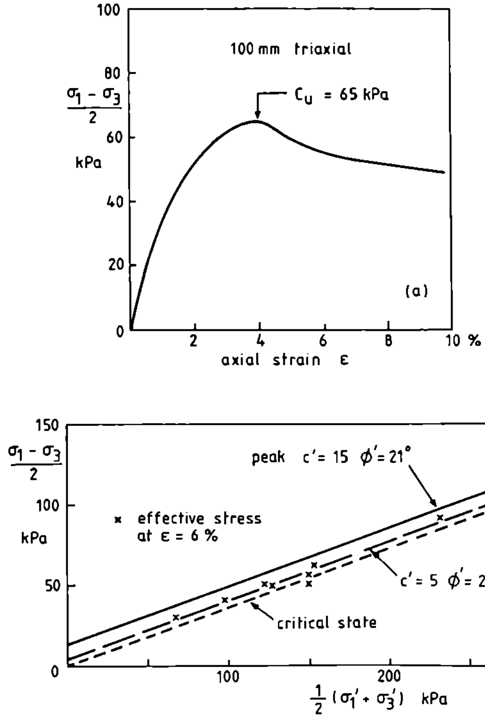

Fig.13 Core Undrained tests on intact material

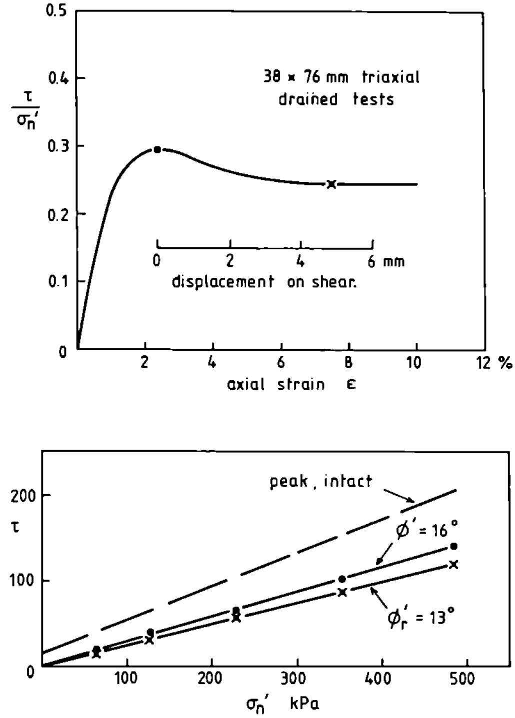

Fig.14 Core Strength on rutting shears

An analysis by Dr Vaughan (see Speciality Session 9A) leads to the following expression for the 'bulk' strength $\bar{q} = \frac{1}{2}(\sigma_1 - \sigma_3)$ of a block of clay containing relatively small randomly orientated pre-existing shears

$$\frac{\bar{q}}{q} = \frac{1}{\pi}\left\{2\psi - \sin\psi \ln\left[\frac{1-\cos\psi}{1+\cos\psi}\right]\right\}$$

where

q = shear strength of intact clay
$= c'\cos\phi' + p\sin\phi'$

strength on the shears $= p\sin\phi_s'$ (with $c_s' = 0$)

$$p = \frac{1}{2}(\sigma_1' + \sigma_3')$$

$$\sin\psi = \frac{p}{q}\sin\phi_s'$$

At peak strength in the intact clay c' = 15 kPa and ϕ' = 21°. On the rutting shears ϕ_s' may be taken as 16°, which can be regarded as the 'peak' strength or (ϕ_r' + 3°) to allow for their curved shape. Numerical solutions show, to a close approximation, that $\bar{q}$ is linearly related to p and the relationship can be expressed as

$$\bar{q} = \bar{c}\cos\bar{\phi}' + p\sin\bar{\phi}'$$

With the above mentioned parameters $\bar{q}$ is given by

$$\bar{c}' = 6 \text{ kPa and } \bar{\phi}' = 20^\circ$$

At large strains, around 8 percent, the intact clay is reduced to its 'critical state' with c' = 0 and ϕ' = 21°, and the value of ϕ_s' will be slightly less than at the peak. If $\bar{\phi}_s'$ is taken as 15° the resulting 'critical state' parameters for the bulk strength are

$$\bar{c} = 0 \text{ and } \bar{\phi}' = 19^\circ$$

This is probably a lower limit for the core strength in the vicinity of a potential slip surface immediately prior to failure.

Stability analysis

The section at Ch. 725 is selected as representing the initial failure, Fig. 15. Factor of safety is defined in the usual manner as

$$F = \frac{\Sigma \text{ available shear strength}}{\Sigma \text{ shear stress}}$$

the correct value at failure being 1.0. To evaluate strength and stress the slip surface is approximated by a series of linear segments and the sliding mass is divided into vertical-sided blocks or 'slices'. Forces between the

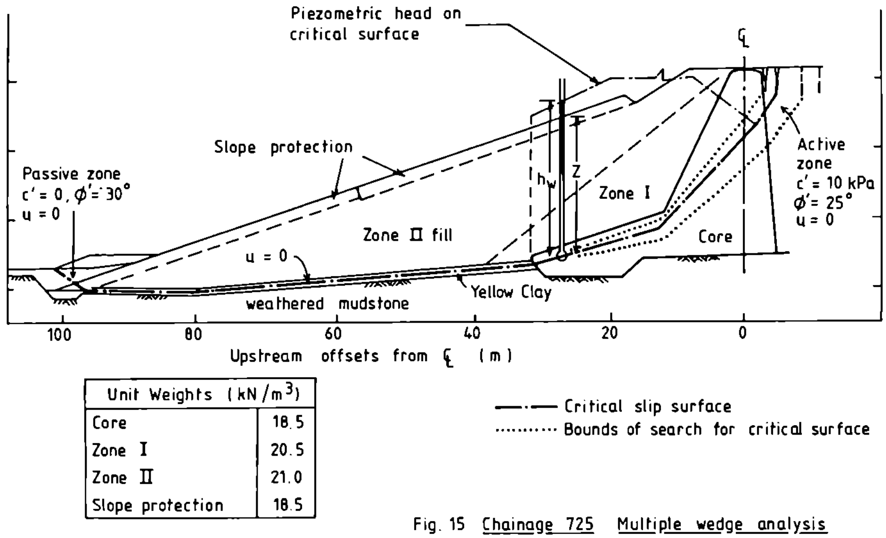

Fig. 15 Chainage 725 Multiple wedge analysis

Table 3

Chainage 725 Multiple wedge limit analysis

Assumption	Core		Yellow Clay		F
	c'	φ'	c'	φ'	($\theta = 10^{o}$)
Peak, intact	15	21°	10	20°	1.4
Peak, with pre-existing shears	6	20	5	17	1.15
'Critical State', intact	0	21	0	20	1.2
'Critical State', with pre-existing shears	0	19	0	16	1.0
Residual	0	13	0	12	<< 1.0

slices are assumed to be inclined at an angle $\theta = 10^{o}$ to the horizontal, this angle having been found to be a reasonable average from a finite-element solution of the stress distribution within the dam. Given a value of θ the calculations become statically determinate.

Pore pressures on the slip surface are taken as zero in the Yellow Clay and as given by the mean lines for the core and the boot in Fig. 3. Expressed as a pore pressure ratio

$$r_u = \frac{\gamma_w h}{\gamma z}$$

the values on the critical slip surface are

Core	$r_u = 0.42$
Boot	$= 0.53$
Yellow Clay	$= 0$

The critical slip surface, leading to a minimum factor of safety, is found by trial and error within the bounds shown in Fig. 15. It corresponds rather closely to the best estimate of the actual slip surface as sketched in Fig. 6.

Factors of safety for various strength parameters are given in Table 3. Results are quoted to the nearest 5 percent, for the sake of comparison, but in absolute terms an accuracy better than ± 10 percent cannot be expected in stability analysis.

The main conclusions from the analysis are as follows.

(i) Intact peak strengths lead to a 40 percent overestimate of stability or, in other words, to obtain the correct result a reduction of $(1 - \frac{1}{1.4}) = 29$ percent is required.

(ii) When the effect of pre-existing shears is taken into account the maximum available strengths lead to a 15 percent overestimate of stability. Thus the pre-existing shears account for a reduction in factor of safety of

$(1 - \frac{1.15}{1.4}) = 18$ percent.

(iii) The further reduction of $(1 - \frac{1}{1.15}) = 13$ percent required to produce a factor of safety of 1.0 can be attributed to progressive failure, resulting from large strains in the core and Yellow Clay prior to the slip.

(iv) Where progressive failure is likely to occur it is often allowed for, in design, by adopting the 'critical state' parameters; conventionally approximated by putting $c' = 0$ and retaining the peak values of ϕ'. This approach leads to $F = 1.2$, or a reduction of $(1 - \frac{1.2}{1.4}) = 14$ percent which, as it happens, is almost identical to the figure deduced in paragraph (iii) above.

(v) If the intact clay is assumed to be at its 'critical state' immediately prior to failure and pre-existing shears are additionally taken into account, the correct value of $F = 1.0$ is obtained within the accuracy obtainable by limit analysis. However, such a result is to some extent fortuitous as the $c' = 0$ assumption can only be regarded, in general, as a convenient approximation to the effect of progressive failure in firm to stiff clays which show a marked post-peak drop in strength.

A breakdown of the reductions in factor of safety due to pre-existing shears and progressive failure in the core (and boot) and in the Yellow Clay is set out in Table 4.

Residual factor in the Yellow Clay

The degree of progressive failure can be expressed in terms of the 'residual factor'

$$R = \frac{s - \bar{s}}{s - s_r}$$

where s = peak strength
s_r = residual strength
$\bar{s}$ = strength for $F = 1.0$

If the core parameters at failure are taken as $c' = 0$ and $\phi' = 19^\circ$ the strength of the Yellow Clay for $F = 1.0$ is represented by $c' = 0$ and $\phi' = 16^\circ$. The residual parameters are $c' = 0$ and $\phi_r' = 12^\circ$ and, with allowance for solifluction shears, the peak parameters are $c' = 5$ and $\phi' = 17^\circ$.

The average effective normal stress, acting on the Yellow Clay at Ch. 725 is 235 kPa. Therefore

$$\bar{s} = 235 \tan 16^\circ = 67 \text{ kPa}$$

while $s = 77$ and $s_r = 50$ kPa. Hence

$$R = \frac{77 - 67}{77 - 50} \approx 0.4$$

Table 4

Chainage 725. Reduction in factor of safety from $F = 1.4$ for peak (intact) strength

Effect	Reduction (%)		
	Core	Yellow Clay	Total
Pre-existing shears	5	14	18
Progressive failure	4	9	13
Combined	9	22	29

Thus the average strength at failure had fallen 40 percent of the way from peak (with shears) towards the residual.

Influence of minor variations

Small excess pore pressures may have been generated in the Yellow Clay as failure was approached, with insufficient time for full dissipation. A best estimate indicates these could amount to a piezometric head of 1 m. In that case the factors of safety are decreased by about 3 percent. On the other hand the factors of safety in Table 3 should be marginally increased to allow for small three-dimensional effects.

The 'critical state' parameters for intact core material ($c' = 0$ and $\phi' = 21^\circ$) correspond to a vertical compression around 8 percent. A better figure, just prior to failure, is 6 percent and this leads to $c' = 5$ kPa and $\phi' = 21^\circ$: see Fig. 13(b). Taking $\phi_s' = 16^\circ$ on the rutting shears the bulk strength parameters become $\bar{c}' = 2$ kPa and $\bar{\phi}' = 19.5^\circ$, leading to an increase of about 5 percent in factor of safety. This also implies a larger degree of progressive failure in the Yellow Clay in order to give $F = 1.0$ at failure, corresponding to $\phi' = 15.4^\circ$ (with $c' = 0$) and a residual factor $R = 0.45$ instead of 0.4.

Finally, a variation of $\pm 5^\circ$ in the inter-slice angle of friction θ leads to a $\pm$ 4 percent variation in factor of safety.

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