VALIDITY OF RESULTS FROM THE DIRECT SHEAR TEST

LA VALIDITE DES RESULTATS DE L'ESSAI DIRECT DE CISAILLEMENT

ДОСТОВЕРНОСТЬ РЕЗУЛЬТАТА ИСПЫТАНИЯ НА ПРОСТОЙ СДВИГ

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SYNOPSIS. A series of comparative tests have been performed using the commonly accepted triaxial and direct shear methods. The object was to evaluate the direct shear test as a practical tool for measuring peak, effective stress, shear strength parameters for design purposes. A broad spectrum of typical cohesive fill materials was used. The results are discussed with reference to the theoretical work of Rowe and laboratory data obtained by other workers. It was found that for the various materials compacted at between 92% and 96% of Proctor density the differences between the two test methods are not significant in the design of large embankments.

INTRODUCTION

Over the past two decades it has been clearly demonstrated from consideration of field failures that the triaxial effective stress shear strength parameters are in close agreement with back calculated values, provided the tests are carried out correctly (Bishop and Bjerrum, 1960). Certain materials, for example stiff fissured clays, do present problems but on the whole the standard triaxial test is useful in design work.

In the routine design of road fills and earth dams in a dry climatic region, such as Southern Africa, effective stress shear strength parameters are invariably required. For such routine work the triaxial test does have certain disadvantages. Long testing times are needed for clayey materials, apparatus is expensive if automatic recording is required and great care is necessary in sample preparation and in following the correct testing procedure.

The direct shear test, although theoretically less acceptable than the standard triaxial test, has a number of advantages for the routine determination of drained, effective stress, shear strength parameters. The question, however, is whether the parameters determined from the shearbox are sufficiently accurate for use in the effective stress limit design of embankments.

A survey of earlier work in this field showed that while work had been done on sands to compare the strengths obtained by the triaxial and shearbox tests there was very little information on other materials. A programme of comparative tests was therefore undertaken on a number of typical fill materials used in earth dams and road fills in Southern Africa. The results of these tests and those of other workers are discussed from a practical design viewpoint.

THEORETICAL DIFFERENCES BETWEEN THE DIRECT SHEAR AND TRIAXIAL TESTS

The limit design of earth fills is based on the Mohr-Coulomb failure criterion written in terms of effective stresses as

$$s = c' + p' \tan \phi'$$

where s = shear stress on failure plane
c' = cohesion intercept (effective stress)
ϕ' = effective angle of internal friction
p' = normal effective stress on failure plane

Although this criterion has a number of disadvantages* it works well in practice.

The stress dilatancy approach of Rowe leads to a generalization of the Mohr-Coulomb expression in terms of more fundamental properties of the soil, such as the true interparticle friction (ϕ_u) and the angle of friction at the critical state. Using his stress dilatancy approach Rowe (Rowe, Barden and Lee 1964, Rowe 1969) has developed equations which, for granular materials, enable a comparison to be made in terms of the Coulomb ϕ value, between the triaxial, plane strain and direct shear tests. Rowe's laboratory results on glass ballotini, sand and crushed feldspar agree well with his theoretical predictions.

The problem in applying this theory to typical fill materials, composed of more than one mineral type, is that a parameter such as ϕ_u is impossible to measure. In addition the equations are only given for cohesion-

*

1. No account taken of σ_2.
2. The rupture line i not always straight, making it difficult to use single c' and ϕ' values.
3. Parameters c' and ϕ' are not fundamental properties of the soil.

less granular materials at relative densities of 0 and 1.

Rowe's work does, however, give some general guidance as to the relationships one may expect between the different tests for granular materials. These are :-

1. lower values of ϕ' from the direct shear test for material in the loose state (as compared with the triaxial results),

2. lower values of ϕ', from the direct shear test, at all densities for material composed of minerals having a high interparticle friction (e.g. feldspar) and a high critical state friction angle and

3. lower values of ϕ', from the direct shear test, at low densities but higher ϕ' values at higher densities for material composed of particles having a low interparticle friction (e.g. rounded silica grains).

Although there have been certain queries with regard to Rowe's approach [e.g. Skinner (1969)] the authors feel that it provides at least a basic framework against which to evaluate laboratory results.

PREVIOUS QUANTITATIVE COMPARISONS BETWEEN THE DIRECT SHEAR AND TRIAXIAL TESTS

An examination of all the commonly used British and American text books reveals little mention of comparison between the direct shear and triaxial tests. The various authors merely point out the errors inherent in the different types of tests and mention that the triaxial results give reasonable correlation with field failures. The one exception is the book by D.W. Taylor (1948). Referring to the strengths of sands Taylor states, 'comparison studies have been made which show that differences obtained by different types of apparatus are of minor importance.' Referring to the shear strength of cohesive soils he mentions the various factors (e.g. effect of σ_2, anisotropy, progressive failure) that would cause differences between the tests but notes that 'relatively little dependable data which furnish quantitative demonstration of these differences are available at the present time' (1948).

A literature survey of research papers has shown that matters have not changed much since Taylor wrote his book. The little information that was obtained is summarised below.

1. Cohesionless material (sands and crushed rock)

Tests have been done by Taylor (1939), Nash (1953) and Rowe (1969). The results of these workers all agree with the trends given by Rowe's theoretical analysis.

In the very loose state Nash found the angle of friction from the direct shear test (on a fine river sand) to be about 2° below the triaxial results, whereas Taylor found a maximum difference (four different sands were considered) of 1°. In the very dense state the triaxial results were found by both workers to be from 2° to 5° lower than those from the shearbox. At medium densities the results from the two types of test were virtually identical.

On Mersey River sand Rowe found the direct shear results to be lower, by about 4°, in the loose state but found little difference in the medium and dense states. On glass ballotini the direct shear results were 1,5° lower in the loose state but 4° higher in the very dense state.

These results are all in good agreement and justify the conclusion in Taylor's book that on cohesionless sands, at densities usually encountered in the field, the differences between the results from the two test methods are insignificant.

It must be noted that a plot of the friction angle of sand against the initial density is only valid for tests at the same confining pressure. Plotting results from different confining pressures gives a large scatter due to different dilatancy rates. This is the reason why Nash's results showed far more scatter than those of Taylor.

2. Cohesive materials (clays, silts etc.)

Extensive triaxial and direct shear tests on compacted montmorillonitic clay were performed at Harvard from 1958 to 1964 (Casagrande and Hirschfeld 1960, Casagrande and Poulos 1964). Two types of direct shear test were employed. A special apparatus was built for large diameter, thin samples held between serrated porous carbon plates. The conventional shearbox was used for high normal stresses. The two types of direct shear test agreed very well with each other, giving a ϕ' value of 31,9°. This result was in close agreement with the drained triaxial tests (ϕ' of 29,6° to 31,5°)

Tests by Schultz and Horn (1966) on a silt with a 10% clay content showed that drained, direct shear tests gave an angle of friction (32,5°) appreciably lower than that obtained from the triaxial machine (36,2°).

Consolidated undrained triaxial tests, ring shear tests and direct shearbox tests were carried out by De Beer (1967) on the stiff, fissured Boom clay. The triaxial tests gave c' = 1,5 tonne/m2 and ϕ' = 22°. The torsional direct shear apparatus gave c' = 1,5 tonne/m^2 and ϕ' = 24,3°. The shearbox samples were pre-cut so no peak results were obtained. It is, however, invalid to compare directly the triaxial and torsional direct shear results as the latter include work done against volume change.

From the above brief discussion it can be seen that the literature appears to offer no definite guide as to the validity of the results from the drained direct shear test on materials other than clean sands. In order to provide some guidance to practising engineers the authors undertook a programme of comparative tests. Details of the materials considered, the test methods and the results obtained, are given below.

LABORATORY TEST PROGRAMME

Using the triaxial and direct shear methods, the drained shear strength parameters were determined on a range of typical fill materials at densities corresponding to those usually achieved in high road fills (92% to 96% of Proctor maximum). Tests were also carried out on a normally consolidated, remoulded, decomposed shale and on two sands. The properties of the materials tested are given in Table 1.

Table 1

Index Test Results, Grading and Mineralogy

No.	MATERIAL DESCRIPTION	GRADING			MINERALS* (Minus No.300 fraction)	ATTERBERG LIMITS		PROCTOR COMPACTION	
		% Clay	% Silt	% Sand		LL	PI	OMC %	MDD gm/cm^2
1	Decomposed Granite	0	6	64		20	1	9,5	2,03
2	Highly Decomposed Granite	30	14	56	Kaolin, Felspar, (microline), Quartz	47	22	26,0	1,55
3 & 4	Decomposed Dolerite (coarse)	2	8	76	Felspar, Montmorillonite	37	3	14,5	1,85
5	Decomposed Ecca Shale	19	19	57	Illite, Chlorite, Montmorillonite	44	24	20,0	1,62
6	Reef Silty Sand	14	26	40	Quartz, Kaolinite, Muscovite	31	3	16,0	1,90
7	Decomposed Beaufort Shale	45	31	24	Illite, Kaolinite, Muscovite	55	35	19,0	1,64
8	Decomposed Dolerite (fine)	12	26	56	Quartz, Montmorillonite	47	19	25,4	1,54
9	Crushed Quartzite (coarse sand)			100					
10	Cape Beach Sand (fine)		1	99					

* X-ray and microscopic analysis

(a) Triaxial Tests

Other than for the normally consolidated shale and the sands, all samples were prepared by static compaction in a double piston mould. The specimens of decomposed shale (material 7) were first consolidated uniaxially from a slurry in a split mould and then consolidated isotropically in the triaxial cell. Sand samples were either poured, tamped or vibrated into split moulds to give samples at different densities.

The triaxial specimens were either 38 mm or 50 mm in diameter. Internal load cells (Imperial College type) were used on the 38 mm diameter specimens and rotating bushes with proving rings on the 50 mm specimens. Side filter-paper drains were used on the specimens of materials 2, 3 and 4 (Table 1). In general the testing procedure was as set out by Bishop and Henkel (1967). Back pressures of up to 850 kN/m^2 were used to achieve high degrees of saturation ($B > 0,9$) with effective confining pressures of up to 500 kN/m^2. Drainage of all specimens was from one end only. On the 38 mm specimens pore pressures were recorded continuously at the undrained end of each specimen. Utmost care was taken to ensure that testing rates were slow enough to allow near-full drainage at peak deviator stress. Other than for six dead-load tests on material 7 all tests were run at a constant rate of strain.

(b) Direct Shear Tests

The drained direct shear tests were performed in commercially available equipment (Wykeham Farrance Model 2500) using 60 mm- and 102 mm-square shearboxes and specimens were 25 mm thick. Normal stresses up to 700 kN/m^2 were used. Data recording was automatic. All tests, other than those on the sands, were run slowly over periods of up to four days. Except for the normally consolidated shale and the sands, all specimens were prepared directly into the shearboxes by static compaction to the same initial densities as the triaxial specimens. This method works well at densities of up to about 96% of Proctor maximum but at higher densities, the high load required to mould the specimens appears to cause high "locked-in" horizontal stresses. This aspect requires further investigation.

The normally consolidated, remoulded shale specimens were consolidated in the shearboxes from a thick slurry. The sands were poured and tamped into the shearboxes for low and medium densities while for high densities the boxes were placed on a vibrating table.

In processing the results from these tests, the effects of area changes on the normal and shear stresses were taken into account in all cases where large strains were required to reach failure.

(c) Commercial Laboratory Tests

Materials 1, 2, 3, 5 and 6 (Table 1) were submitted to a commercial laboratory for the determination of the triaxial drained shear strength parameters at 95% of Proctor maximum density.

These tests were done on 38 mm diameter specimens using internal load cells and automatic data recording. Side drains were used and testing rates were sufficiently slow. The chief differences between the commercial tests and those performed in the research laboratory were :

1. specimens were prepared by simultaneously pushing four 38 mm tubes into the material previously compacted in three layers into a 152 mm diameter mould (C.B.R. mould). The specimens were then extruded into split moulds for trimming to the required length before mounting in the triaxial machines. With this method of preparation the final samples are usually at least 5% denser than the material as compacted in the 152 mm mould,

2. a standard back pressure of 200 kN/m^2 was used,

3. no volume change readings were taken (no automatic method was available) and hence the area corrections were in error and

4. no stress strain curves were plotted; the maximum value of the deviator load was taken and divided by the "corrected" area.

DISCUSSION OF RESULTS

The results of the direct shear and triaxial tests on the materials given in Table 1 are summarised in Figures 1, 2 and 3.

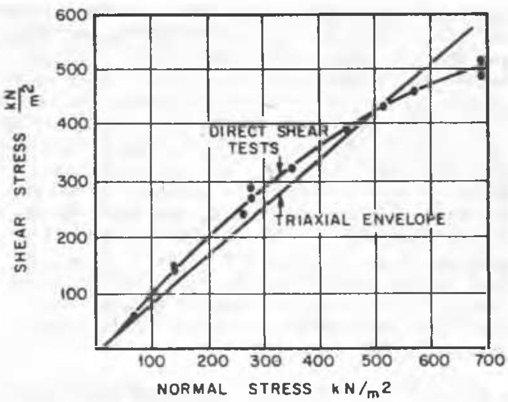

Fig. 1 Mohr envelopes for Cape Flats Sand

No clear trend as to the differences between the results from the two test methods emerged from this study. For the wide range of cohesive fill materials considered the maximum difference in the angle of friction was just under 3°. The cohesion values from the direct shear tests were in general higher than those determined from the triaxial tests.

It must be emphasized that all specimens were tested at densities of between 92% and 96% of Proctor maximum, except for the coarse decomposed dolerite (material 3/4) which were at 95% and 101% of Proctor maximum. The triaxial test gave the same angle of friction at both densities with a higher cohesion value at the higher density. This trend agrees with that found by Glynn (1948). The direct shear tests however showed an increase in the angle of friction at the higher density, with only a small increase in the cohesion intercept. These data fit in with the pattern given by Rowe's theoretical work, as well as the tests on sands discussed earlier, namely that at medium density the two test methods give similar results but at high densities the direct shear test gives higher values of ϕ. The good comparisons obtained, between the two test methods, in this program of work are thus possibly the result of the density range considered. However as this density range is the one most often considered in the field of high road fills the results are of great practical importance.

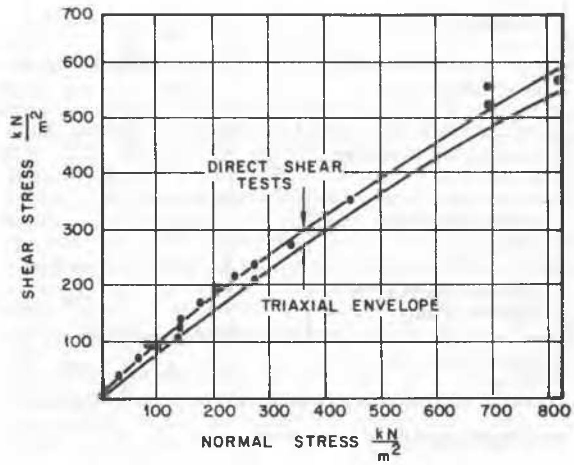

Fig. 2 Mohr envelopes for crushed Reef Quartzite

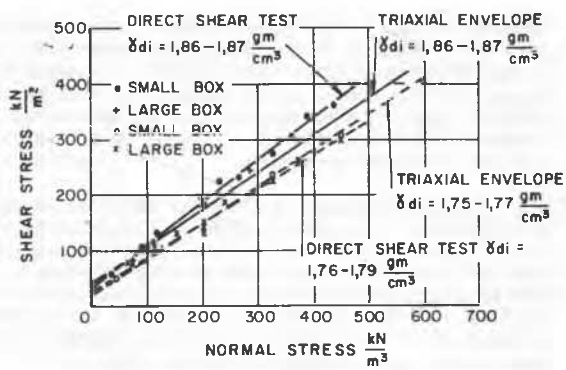

Fig. 3 Mohr envelopes for decomposed dolerite

The results of the tests on the two sands are given in Figures 1 and 2. These results are for the sands in a medium dense state (relative densities of between 0,66 and 0,71). The Mohr envelopes are curved, with the direct shear results showing greater curvature than the triaxial results. The higher strengths from the direct shear tests in the stress range of 70 to 250 kN/m^2 agree with the trend found previously by Taylor, Nash and Rowe who worked in the same stress range.

The important question that remains to be considered is the significance of the differences given in Table 2 which can best be evaluated from the Mohr envelopes (Figure 3) for the worst case, the coarse, decomposed dolerite (materials 3/4). In the range of stresses that applies to the stability analysis of fills from 10 metres to 50 metres in height the strength difference between the two test methods, for material placed between 92% and 96% of Proctor maximum, is likely to be less than 5%. As the field condition is usually one of plane strain, and plane strain shear strengths are higher than those obtained from the standard triaxial approach, it is obvious that use of direct shear results in the analysis of such a fill would be acceptable. In stability analyses involving very low stresses (e.g. shallow surface slides) the differences between the two test methods would be significant.

the factors of safety obtained for the upstream slope were :

Circular arc analysis (Bishop method)

- triaxial results 1,52
- direct shear results 1,57

Critical non-circular surface (Morgenstern-Price method)

- triaxial results 1,43
- direct shear results 1,44

Considering $r_u = 0$ throughout the embankment, results for the circular analysis are as follows :

- triaxial results 1,91
- direct shear results 1,94

Table 2

Effective Stress Shear Strength Parameters

No.†	DRAINED TRIAXIAL TESTS				DRAINED DIRECT SHEAR TESTS			
	Initial Density	Initial M.C.	c'	ø'	Initial Density	Initial M.C.	c'	ø'
	gm/cm^3	%	kN/m^2	degrees	gm/cm^3	%	kN/m^2	degrees
1	1,87-1,89	8	17	35,3	1,88-1,90	8	15	37,5
2	1,65	21	47	27,5	1,69-1,70	22,5	45	27,7
3	1,75-1,77	15	17	33.7	1,76-1.79	13,7	32	31
4	1,86-1,87	13,7	43	33,6	1,86-1,87	13,3	38	36
5	1,56	23,7	33	25,2	1.55	24,5	34	26
6	1.7 -1,75	21	0	37	1,73	23,6	5	34,2
7	*	*	0	18,4	*	*	7	16
8	1,49	28	32	27,8	1,47	33	35	28,5
9	1,44-1,48	See Figure 2			1,44-1,48 (RD ~ 0,67)	See Figure 2		
10	1,68-1,71	See Figure 1			1,68-1,71 (RD ~ 0,71)	See Figure 1		

† Descriptions given in Table 1

* Samples normally consolidated from a slurry at the liquid limit

In order to evaluate, quantitatively, the effect of the differences shown in Table 2 the results from materials 1, 2, 3 and 7 were used in the analysis of the embankment dam shown in Figure 4. Considering the end-of-construction case*, with r_u values as shown,

* As effective stress parameters are not markedly stress path dependent, the drained parameters can be used for this case although results from consolidated undrained tests are usually used.

Finally it is of interest to note that a comparison between Tables 2 and 3 shows that the triaxial results from the commercial laboratory differ from those determined in the research laboratory by as much as the research laboratory direct shear results. These differences are partially due to initial density differences (the Proctor maximum densities determined by the two laboratories were significantly different in some cases) but are mainly due to the different sample preparation and testing techniques, as discussed earlier.

Table 3

Drained Triaxial Tests

Soil No.*	DRAINED TRIAXIAL TESTS (Commercial Laboratory)			
	Initial Density	Initial M. C.	c'	ø'
	gm/cm^3	%	kN/m^2	degrees
1	2,08	8,2	9	40
2	1,52	24,5	25	33
3	1,72	15,5	17	35,5
5	1,63	20,6	20	28,5
6	1,81	14,1	17	36

* Descriptions given in Table 1.

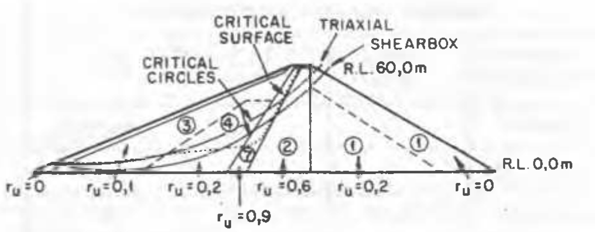

Fig. 4 Trial Embankment Dam

CONCLUSIONS

1. In the design of medium sized embankments (10 to 50 metres in height), composed of typical cohesive fill materials, compacted between 92% and 96% of Proctor maximum density, the differences between the effective stress shear strength parameters, as determined from the triaxial and direct shear tests, are not significant.
2. For cohesionless sands placed at medium densities, in the stress range of 70-700 kN/m^2, little difference is found between the strength parameters given by the direct shear and triaxial tests.
3. Further work is required to evaluate the differences between the two test methods, for cohesive and cohesionless materials, at high densities (above 100% Proctor maximum) and at high stresses (greater than 750 kN/m^2).

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