ISSN 1476-1580

North West Geography

Volume 10, Number 2, 2010

The use of a field open-sided direct shear box for the determination of the shear strength of shallow residual and colluvial soils on hillslopes in the south Pennines, Derbyshire.

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July 2010

Abstract

Engineering drawings are provided for a field open-sided direct shear box with a basal shearing area of 300mm x 300mm and two 300mm x 150mm side shearing areas, following the design of Chandler *et al*. (1981). Lead blocks are used to provide normal loads simulating natural geostatic pressures and a shearing force is provided by a winch/cable system. The field open-sided direct shear box was used to determine the effective angle of shearing resistance and the effective cohesion of shallow residual and colluvial soils on hillslopes in the south Pennines, Derbyshire. The field open-sided direct shear box is appropriate for determining the effective shear strength of shallow residual and colluvial soils on hillslopes as it incorporates the pedogenic and biogenic structural discontinuities and compositional inhomogeneities which are not represented adequately in standard laboratory direct shear strength and other conventional in situ field shear strength testing apparatus.

Introduction

Measurement of the strength of soil materials and modelling of the mechanics of failure in limit equilibrium analyses are fundamental to the geotechnical investigation of slope stability (Kirkby, 1987; Selby, 1993). Various techniques of stability analyses have been used in engineering geomorphological research to determine the relationship between soil strength parameters and slope conditions. This type of geomorphological research is important and underpins the quantitative study of the morphology, processes and evolution of natural hillslopes (Dietrich *et al*., 1992; Brooks, 2003; Claessens *et al*. 2007). However, the suitability of standard engineering tests and testing equipment for the measurement of soil strength parameters together with case examples as part of geomorphological research in the UK has received limited attention. This paper draws upon the previous research of Chandler *et al.* (1981), provides engineering drawings for the construction of an open-sided field direct shear box and examines its application on shallow residual soils on hillslopes in the south Pennines, Derbyshire. The use of the open-sided direct shear box enabled the sampling and testing of shallow heterogeneous residual and colluvial soils located on steep hillslopes. The shear box design incorporates the pedogenic and biogenic structural discontinuities and compositional inhomogeneities which are not represented in standard laboratory and *in situ* field tests.

Limitations of Conventional Shear Strength Testing Equipment

The accepted use of the Terzaghi-Coulomb failure criterion as a means of estimating the effective cohesion (c') and the effective angle of shearing resistance (ϕ') requires the measurement of normal stress, shear stress and pore-water pressure at failure (Bishop and Skinner, 1977). Application of this criterion precludes the use of simple vane shear and penetrometer test equipment which, may be used to determine the *in situ* undrained shear strength but not values of c' and ϕ', and not the normal stress which can, at best, only be estimated. Recognized laboratory methods of determining effective shear strength parameters under strictly controlled conditions include direct shear, ring shear and triaxial compression tests which, for civil engineering purposes, were designed to test small samples of relatively homogeneous undisturbed or remoulded soil materials (Head and Keaton, 2010; Cerato and Lutenegger, 2006; Nakao and Fityus, 2009). In standard engineering practice pedological soil materials and organic material are specifically excluded from use and testing because of the influence any structural discontinuities or compositional inhomogeneities may have on shear strength (Skempton and Hutchinson, 1969; Head and Keaton, 2010). Similarly, samples containing particles larger than a specified size either cannot be tested due to scale effects, or the large particles are removed before testing (Bauer and Zhao, 1993). In laboratory shear tests it is important to control sample disturbance prior to undertaking the shear strength test.

All these factors present important limitations of the use of conventional laboratory shear strength testing equipment for some particular geomorphological research applications.

Limitations of Previous Research on the Shear Strength of Hillslope Regolith Samples

A number of studies have involved taking hillslope regolith samples for shear testing using direct shear strength apparatus to yield angles of internal shearing resistance (ϕ'), and these were subsequently used in Infinite Stability Analyses (Fukuoka, 1957; Franklin *et al.* 1974; Chandler *et al.* 1981). These analyses initially assumed regolith saturation, with the water table at the surface and drainage along flow lines parallel to the surface (Skempton and DeLory, 1957). Under these conditions the stability model predicts the threshold angle of stability for a hillslope (β_t) from:

$$\tan\beta_t = \left[\frac{\gamma s - m.\gamma\omega}{\gamma s}\right]\tan\phi' \approx 0.5\tan\phi'$$

where:

- β_t = Threshold angle of stability.
- γs = Saturated unit weight of soil (kN/m^3).
- $\gamma\omega$ = Unit weight of water (kN/m^3).
- ϕ' = Angle of internal shearing resistance.
- m = Unity, the relative height of the water table above the shear plane.

Various research workers have proposed that the results of such stability analyses suggest that mechanical properties of regolith may partially explain the presence of threshold angles (Chandler, 1973; Rouse and Farhan, 1976; Cross, 1987; Francis, 1987; Doornkamp, 1990; Cross, 1998). A number of studies have been undertaken to show the relationship between the effective angle of internal friction (ϕ') and material sorting (Holtz, 1960; Vucetic, 1958; Kawakami and Abe, 1970). Vucetic (1958) tested for changes in the shear strength of a clayey schist, with upper limits of particle size at 30mm, 15mm, 8mm, and 4mm. The first three mixtures all approximated to the same value of the effective angle of shearing resistance ($\phi' = 39°$) and the value of ϕ' suddenly fell to 25° in the case of the 4mm particle size sample. Vucetic (1958) concluded that the decrease in effective shear strength with decreasing particle size was discontinuous rather than gradual. Holtz (1960), similarly tested mixtures of gravel and soil and found no change in ϕ' at first, and then a sudden drop, and then no further change when decreasing the fraction of gravel in the mixture. With sandy gravel mixtures, the discontinuity occurred at about 20% gravel, and with clayey gravel, at approximately 35% gravel in the mixture.

Carson and Petley (1970) conducted laboratory direct shear box tests on samples of regolith material overlying Shale Grit on Derbyshire hillslopes. They determined peak shear strength, residual shear strength and maximum stable slope angles for two characteristic straight hillslope inclinations of 21° and 26.5°. The results of their tests are shown in Table 1.

Carson and Petley (1970) used conventional laboratory direct shear box apparatus to test regolith samples of 60mm x 60mm and 20mm thick. They acknowledged that this type of apparatus was only suitable for material consisting of particles smaller than 2.0mm. They also acknowledged that it was problematic to extract and transport samples for laboratory direct shear box testing which could be described as undisturbed. Both these factors can be regarded as significant limitations of standard laboratory direct shear box testing apparatus.

Two questions of sample representativeness should be considered for effective shear strength testing of shallow residual soils:

(a) Are the samples extracted truly representative of the material controlling the stability of the slope mantle? This is a significant problem for some hillslope materials, for example, the characteristics of regolith on south Pennine slopes vary with depth through the weathering profile, with a marked increase in the coarse fraction towards bedrock the bedrock interface.

(b) For non-cohesive materials, laboratory direct shear box tests may give unrepresentative test results. When coarse material greater than a critical size (i.e. greater than approx. 8.3% of the shear box length) is included

Table 1: Peak shear strength, residual shear strength and the maximum stable slope angles of regolith on Shale Grit bedrock for two characteristic straight hillslope inclinations of 21° and 26.5° (from Carson and Petley, 1970).

Slope Inclination (β)	Peak Shear Strength (ϕ')	Residual Shear Strength ($\phi'r$)	Maximum Stable Slope Angle (degrees) (when: $c' = 0$ and $m = 1$)	
21°	43°	41.5°	24.8°	23.5°
26.5°	45.9°	37.5°	27.3°	20.9°

in the sample, excessive stresses tend to develop where the coarse particles intersect the failure plane (Palmeira and Milligan, 1989).

In the absence of infinitely variable laboratory shear box dimensions, the necessary exclusion of large particles artificially improves sediment sorting, which may result in the underestimation of field strength parameters. It is normal practice in soil mechanics laboratory testing to minimise the sample variability by controlling water content and the particle size, by eliminating organic matter and often by removing structural discontinuities in the reconstitution of samples. This can result in providing widely variable values for even simple classification and index tests (Sherwood, 1970; Sherwood and Ryley, 1970). Small sample size and the accepted practice of screening variability of samples for laboratory tests can result in provision of inaccurate data used for subsequent analysis. Clearly there is a requirement for a practical alternative direct shear box design which can be used to test soils which incorporate the pedogenic and biogenic structural discontinuities and compositional inhomogeneities which are not represented adequately in standard laboratory direct shear strength and other conventional in situ field shear strength testing apparatus. The design should also seek to overcome some of the problems of disturbance and sample representativeness discussed above.

Large-scale Field Direct Shear Box Requirements

Large scale shear box tests have been designed to test large samples in the field under natural or anticipated design stress conditions in order to overcome some of the difficulties mentioned above (Bishop, 1948; Akroyd, 1957). These *in situ* tests involve direct shearing along a predetermined shearing plane, although soil or rock deformation properties are derived from plate loading or bearing tests. The preparation and testing of large-scale specimens in the field direct shear apparatus are generally very expensive in labour, equipment and time (James, 1969; Franklin *et al*., 1974). For these reasons a minimum of tests on carefully selected specimens are conducted in order to obtain optimum shear strength and sample behavioural information. Chandler *et al.* (1981) list the necessary requirements of a shear strength test for geomorphological testing purposes as follows:

(a) Sampling the structural and compositional inhomogeneities of shallow pedological soils (1 – 2m deep residual or colluvial mantles) in a statistically significant manner (i.e. sufficient tests should be conducted to determine the variability or range of shear strength response).

(b) Using a specimen large enough to include the roots, organic matter, structures and large particle characteristics of superficial deposits.

(c) Simulating the low *in situ* geostatic stresses, of the order of 0 – 20kN/m^2, typical of shallow regolith.

(d) Testing soils on steep hillslopes at approximately the same angle and depth as a natural failure zone might occur.

Important considerations in the design of field equipment for *in situ* direct shear box testing include portability to remote sites, and a rugged yet simple construction in order to minimize operating costs while optimizing the number of tests as noted under (a) above. A review of earlier large scale field shear test apparatus has been described by Chandler *et al.* (1981). They concluded that a practical alternative shear box design is required for sampling soil variability, incorporating undisturbed macroscopic soil structures, simulating *in situ* geostatic pressure depths and inclinations, and being of inexpensive, simple, robust and portable construction for difficult testing conditions.

Open-sided Direct Shear Box Specification

Chandler *et al*. (1981) suggested a practical alternative design for a field direct shear box based on the experimental studies of Endo and Tsuruta (1969), O'Loughlin (1974) and O'Loughlin and Pearce (1976), who employed a winch and simple direct shear box to conduct field tests of the effects of tree roots on soil strength. Chandler *et al*. (1981) designed an open-sided field direct shear box based on their design specifications. The shear box has two vertical side shearing planes in addition to the basal shearing surface. The design allows rapid and easy preparation of undisturbed samples incorporating lateral as well as vertical roots, large stones and other pedological structures that would not be adequately included in any standard laboratory or conventional shear box test. Field pro-basal shearing areas should necessarily resemble the same proportion in a natural landslide, where the area of side shearing is relatively minor. The dimensions (300mm x 300mm x150mm) were chosen to ensure adequate sampling representation of heterogeneous soil, provided sufficient tests are conducted, but at the same time the dimensions were limited by practical considerations of portability and partly to simplify calculations in data analysis.

Normal loadings were applied directly to the top of the soil specimen by a set of weights; in the case of tests conducted by O'Loughlin and Pearce (1976) these were in the order of 1.5 – 8.0kN/m^2. Shear forces were applied by an

anchored winch and cables and were measured through a calibrated proving ring and dial gauge.

For shallow residual soils on hillslopes, drained, effective stress parameters c' and ϕ' are considered to be obtained by the open-sided field direct shear box apparatus as drainage is effectively achieved along the shearing plane after peak strength is achieved. Testing partially saturated soil to obtained drained shear strength parameters is not considered to result in the development of pore suctions or excess porewater pressures because of the likelihood of ready and immediate dissipation along microscopic pedological and biological structural discontinuities, such as root channels, and because the rate of testing for these kinds of soils permits adequate drainage to occur.

Chandler *et al.* (1981) gave brief details of the engineering specifications of the shear box in the British Geomorphological Research Group Technical Bulletin No. 27. and the author has produced detailed engineering drawings of an open-sided direct shear box based on the outline design provided by Chandler *et al.* (1981), (see Appendices B1-5). In order to construct and use the field direct shear box the design features and test procedure described by Chandler *et al.* 1981 and the detailed engineering drawings provided here should be used.

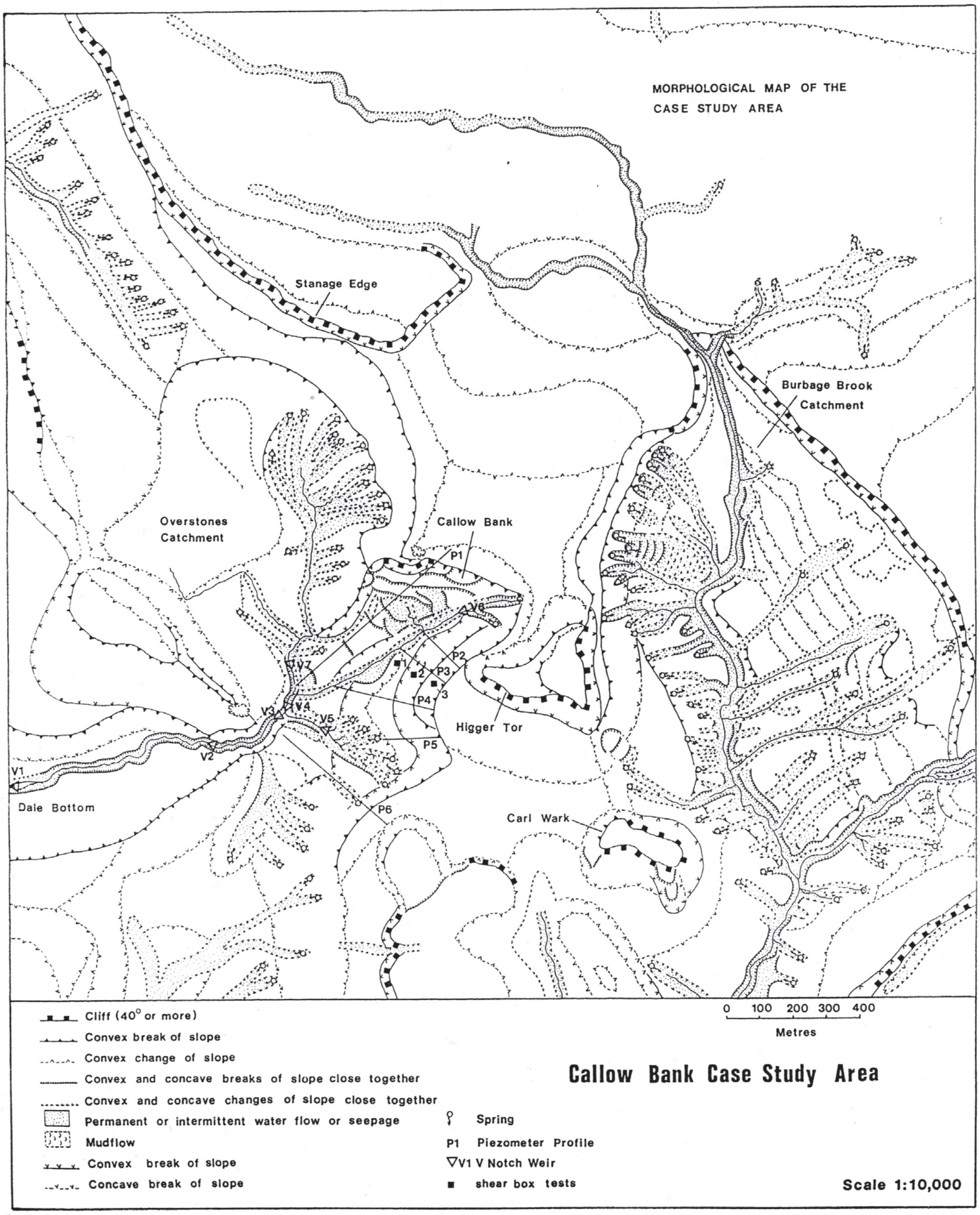

Figure 1: Map of Callow Bank case study area. The location of shear box tests for slope sections 1, 2 and 3 are shown by shaded squares.

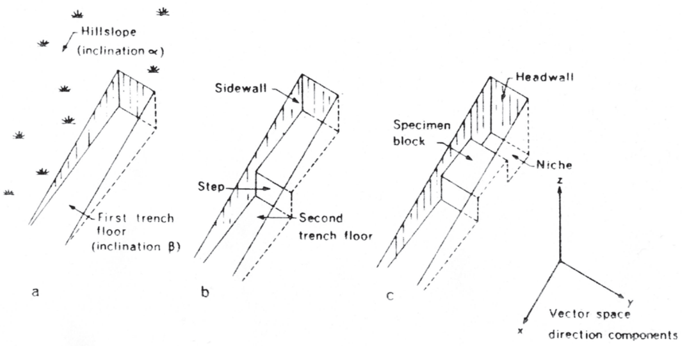

Figure 2: The preparation of an in situ *monolith for testing.*

An average of 70-90 minutes is commonly spent on each test and allowing some time to move the equipment at or between sites, about six tests may be conducted in a working day. The time spent preparing and setting up the tests is minimized if one trench is used for more than one sample.

The estimated total machine shop time for construction of the shear box, winch chassis, and accessories is 50 hours (approximate cost £1,500). The cost of the raw materials was approximately £750. The greatest single cost was the 15kN capacity proving ring. Proving rings from any of several standard soil mechanics laboratories could be readily adapted for field use (approximate cost of proving ring – £250). The lead needed for the blocks was obtained from a scrap metal dealer; it was melted down and cast into appropriately sized blocks.

South Pennines Case Study

The open-sided field direct shear box was used to test shallow residual soils on a selected slope section within a catchment area known as Callow Bank, near Hathersage in the south Pennines of Derbyshire. Fig. 1 shows the location of the case study area (Cross, 1987).

Three sample locations were selected for shear box testing, by dividing the total slope profile from the slope crest to the slope base into three equal sections – the upper, middle and lower slope sections. These locations accounted for the main morphological changes in the slope profile and corresponding regolith composition changes. There was no evidence for recent shallow landsliding along this profile (Cross, 1987). Open-sided direct shear box tests were conducted on the lower ($\beta = 16°$), middle ($\beta = 24°$) and upper ($\beta = 33°$) sections of the slope profile.

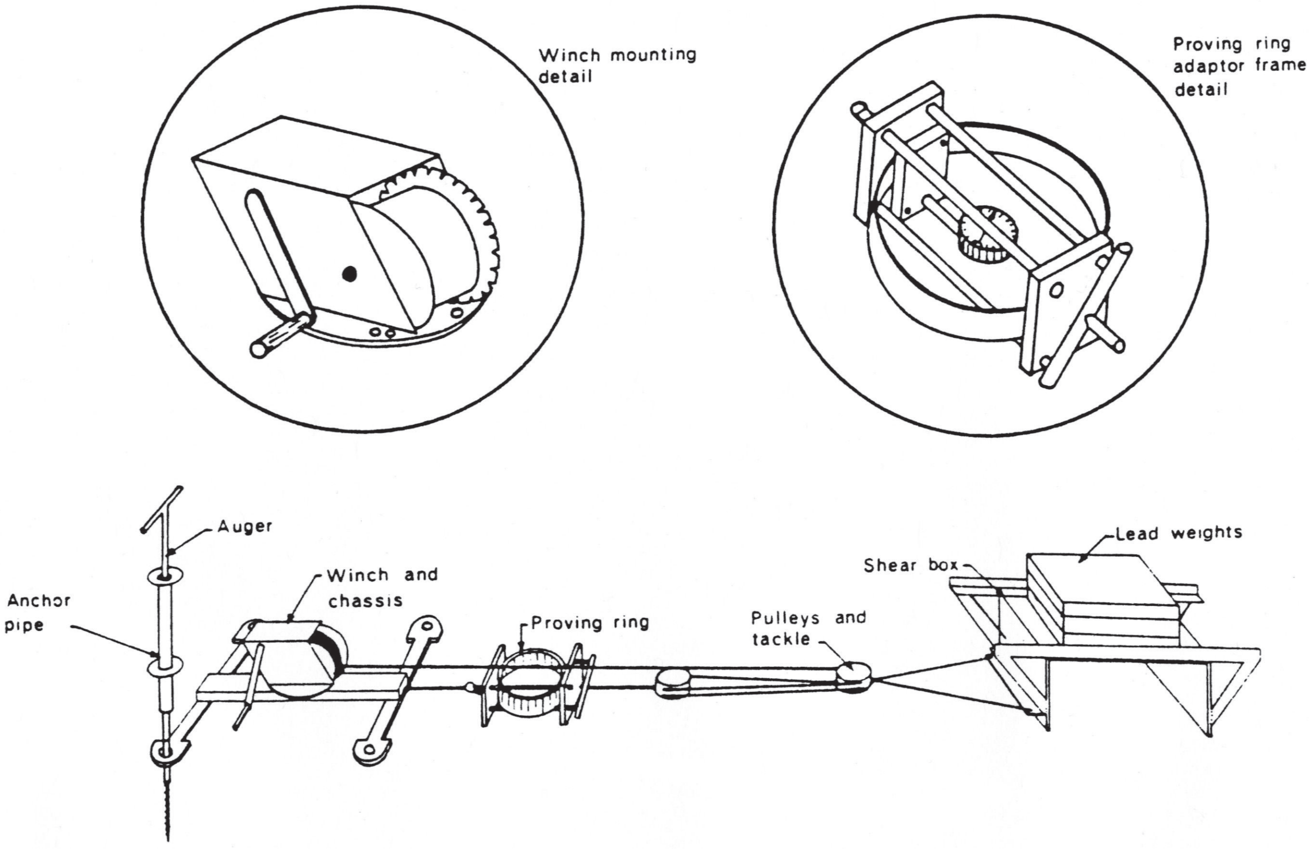

Figure 3. Schematic layout of components of the open-sided field direct shear box, showing detai ls of the winch and proving ring adaplor frame (Not to scale).

Sample Preparation

Five parallel trenches approximately 0.31 – 0.33m wide were excavated along the fall line of the slope for each section (Fig. 2). The final inclination of the trench floor (β) determined the inclination of the shear box in the test. The front portion of the trench floor was then excavated 0.15m deeper from a point 0.5m in front of the back wall, keeping the new floor parallel with the former, this created an inclined step. The top of the step was 0.75m below the ground surface. This depth of shear box testing was chosen, according to the depth of tension cracks observed above the back scar of a nearby small translational debris slide (the tension cracks were 0.7 – 1.0m) deep.

A niche was then excavated at the back of the step, again to a depth of 0.15m; this produced an isolated block (0.3 x 0.3 x 0.15m). The preparation of an in situ monolith sample for testing is shown in Fig. 2 (Chandler *et al.* 1981). The shear box was carefully placed over the pre-cut block making sure there was no contact between the lower edges of the end plates and the trench floor. A schematic layout of components of the field direct shear testing apparatus is shown in Fig. 3 (Chandler *et al.* 1981).

The winch was positioned 2.5m down-slope from the sample trench and was aligned with the central line of the trench to preclude torsional components of side friction or shear force. Following the loading of the specimen and the linking of cables to the box, the shear displacement measuring datum pin and rule were set in place. Specific site and test conditions were recorded on a proforma data sheet.

In the desire for economical, reliable and uncomplicated testing procedures the shear force, which was applied in the down-slope direction, was increased in approximately 11.4 N increments at 30 second intervals after the start of each test until failure occurred. Failure of the specimen was evident from the falling or constancy of the dial gauge readings when these were accompanied by large strain readings.

In general, it was necessary to make more than one test per site; a minimum of three tests is theoretically sufficient to define a straight line Coulomb relation, but according to Chandler *et al.* (1981) and O'Loughlin (1974) at least 15 tests should be conducted at one site in order to offset the scatter of points produced by natural soil variability. The time spent preparing and setting up the tests is minimized if one trench is used for three tests. Following completion of the first test, a new level was excavated back up the hillside in line with the original trench. Another sample block was carefully prepared as before at the same depth. Fifteen shear tests were conducted under different conditions of loading for each section of slope. With the particular gauge used each load increment was indicated by ten divisions on the dial (11.4 N) this was recorded along with the corresponding reading of shear displacement from the datum pin on the metal rule (see Fig. 3 showing shear box apparatus test set up, Chandler, *et al.* 1981).

Calculation of Shear Stress and Normal Stress

The raw data from the field tests were modified to allow shear strength (s) to be plotted against normal stress ($\sigma'n$) in accordance with the Terzaghi-Coulomb failure criterion.

Shear Stress

Shear stress values were derived from the force applied by the winch (Sa); as measured by the proving ring dial gauge, and from a component of downslope shear force (Sg) contributed by the gravitational body force of the loaded shear box and sample block. Values of (Sa), in dial gauge divisions, were converted directly to Newtons (N) from the manufacturer's calibration chart. Overall shear stress was determined from the sum of the two shear force components divided by the by the area of shearing surface which was taken to be 0.18m^2 (i.e. the sum of the base and two sides). Values of shear stress (s) were calculated using the expression:

$$s = \frac{Sa + Sg}{2A} = \frac{Sa + (w + \gamma v)\sin\beta}{2A}$$

$$s = \frac{Sa}{2A} + 0.5\left(\frac{w}{A} + \gamma d\right)\sin\beta$$

Where:

$w =$ Load of shear box and weights (kN)
$\gamma =$ Soil unit weight (kN/m^3)
$v =$ Volume of soil sample (m^3)
$A =$ Basal shearing area (0.09m^3)
$d =$ Sample thickness (0.15m)
$\beta =$ Sample inclination (degrees)

Shear stresses at failure were designated Sf for peak strength and Sr for residual strength.

Normal Stress

In order to simulate soil stresses approximately equivalent to those present at the depth of testing before the trench was excavated a range of applied loads was chosen so that there was little actual difference in surcharge between pre-excavation and test conditions. The shallow depth of testing and relatively short time which elapsed between excavation and replacement of soil surcharge with weights precluded the development of any significant stress-release phenomena, such as fissuring or base heave. The procedure

also eliminated the need for a lengthy consolidation stage in the test.

The combined magnitudes of the normal forces, divided by the total area on which they act, is the best approximation to a relatively simple single expression for total normal stress:

$$\sigma n = \frac{Pz + 2Py}{2A} = \frac{Pz}{2A} + \frac{Py}{A} = 0.5\sigma z + 0.5\sigma y$$

Where:

Pz = Vertical normal load
Py = Horizontal geostatic normal forces
A = Basal shearing area ($0.09m^2$)
γ = Soil unit weight (kN/m^3)
σz = Stresses acting on the basal shear plane
σy = Stresses acting on the side shear planes

$$\sigma n = 0.5\left(\frac{w}{A} + \gamma d\right)\cos\beta + 0.25k\left(\gamma(d+H) + \frac{w}{A}\cos\beta\right)$$

Where:

w = Load of box and weights (kN)
A = Basal shearing area ($0.09m^2$)
γ = Soil unit weight (kN/m^3)
d = Sample thickness (0.15m)
β = Sample inclination (degrees)
H = Depth normal to top of sample
k = A coefficient of lateral to vertical effective stresses.

Calculation of lateral coefficient (k)

The coefficient k is used to express lateral to vertical effective stresses; k is related to the stress history of the soil (Lambe and Whitman, 1969). An effective state of overconsolidation results as high pore-water tensions become dissipated towards an assumed maximum neutral stress. The ratio of effective stresses in respective dry and wet conditions can yield a value of overconsolidation ratio (*OCR*), for which values of k can be determined corresponding to a specified *PI* for the soil.

The over-consolidation ratio (*OCR*) was calculated from the formula:

$$OCR = \frac{P'C}{P'O} = \frac{P'd}{P'w}$$

Where:

$P'C$ = Former maximum effective overburden stress
$P'O$ = Present in situ effective overburden stress
$P'd$ = State of effective stress in dry conditions
$P'w$ = State of effective stress in wet conditions.

Example for calculation k:

γb = Maximum bulk unit weight of soil ($18.0\ kN/m^3$)
Us = pore-water tension (dry soil $Us = -40\ kN/m^2$, wet soil $Us = 0$)
w = Moisture content (32%)
z = Depth of regolith (1m)
PI = 13

Wet Soil	Dry Soil
	$\frac{18.0}{1.32} = 13.6kN/m^2$
$\gamma b = 18.0\ kN/m^3$	
$Pw + 18.0\ kN/m^2$	$Pd = 13.6\ kN/m^2$
$P'w = Pw - Us = 18.0\ kN/m^2$	$P'd = Pd - Us = 53.6\ kN/m^2$

The charts showing the relationships between *PI*, *OCR* and k are provided within Brooker and Ireland (1965) and Lambe and Whitman (1969). Hence from the charts, where $PI = 13\%$, $k = 0.7$.

Field Direct Shear Box Results

The maximum shear stress at incipient failure and the confining stress created by the weight of the applied load, the shear box and the soil block on the failure plane, were obtained from each test and plotted in order to obtain a Mohr failure envelope. The considerable variability of data points as measured in field tests reflect the expected non-homogenous pedological and biological conditions prevailing in the regolith soils. Because of the ability of the open-sided shear box to incorporate those components of the soil which contribute to variations in the shear of shallow soils (i.e. roots, fissures, spatial variations of particle size) a wider more representative range of results may be obtained.

Forty five successful tests were completed, fifteen for each slope section. The results of the fifteen successful field shear box tests conducted for each slope section are shown in Table 2. The Mohr-Coulomb failure envelopes were estimated by fitting the least squares regression line to plots of maximum shear stress (*Sf*) against effective normal stress (σn) (see Fig. 4). Peak shear strengths, not residuals, were plotted because they more closely match 'first-time' failure strength of shallow planar slides (Skempton, 1970). The soil strength parameters, cohesion (c') and the internal friction angle (ϕ') were estimated from the envelope-ordinate intercept and the envelope slope respectively (Lambe and Whitman, 1969). Estimates of the effective angle of internal friction ϕ' and effective cohesion c′ are shown at the foot of table 2. Details of the testing and analysis techniques are presented in O'Loughlin (1974) and Chandler *et al.* (1981).

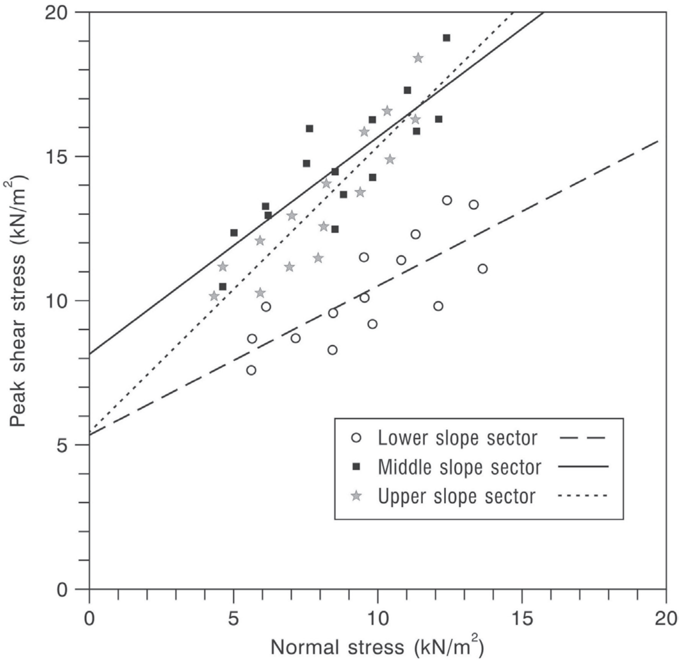

Figure 4: Plot of peak shear stress (sf) against normal stress (σ′n) for 45 open-sided direct shear box results. The Mohr-Coulomb failure envelopes shown were derived by fitting the least squares regression lines to the plots for the shear test results of each slope section. The soil strength parameters, effective cohesion (c′) and effective angle of internal friction (ϕ′) were derived from the envelope ordinate intercept and envelope gradient respectively.

Discussion

A comparison has not been made using standard laboratory direct shear box apparatus in this study. Direct comparison of standard laboratory and field direct shear box results should be made with reservation due to the influence of measured length, of differences in drainage conditions, sample size and testing rate, and the degree of sample disturbance (Skempton and Hutchinson, 1969). As a generalization it is to be expected that tests over a large range of *σ′n* would give a curved empirical failure envelope. By simplifying the curve into a sequence of straight line segments, each conforming to a Mohr-Coulomb-failure criterion for limited specified ranges of *σ′n*, it can be envisaged that tests under lower stresses could exhibit lower *c′* and higher *ϕ′* than those at higher stresses. The large shearing area may tend to even out small scale variations, however by incorporating discontinuities of a higher order of magnitude, results are produced which have greater significance to the stability of hillslopes mantled by shallow regolith.

Sample disturbance is minimized; the side support retained by the block samples for open-sided testing reduces the potential for sample disturbance which remains even under the best of conditions when full block or advance trimming sampling are involved.

Table 2. Normal and peak shear stresses at failure for field direct shear box tests for the three slope sections, Callow Bank.

Test Number	Lower Slope Section	Lower Slope Section	Middle Slope Section	Middle Slope Section	Upper Slope Section	Upper Slope Section
	Peak shear stress at failure (kN/m^2)	Normal stress at failure (kN/m^2)	Peak shear stress at failure (kN/m^2)	Normal stress at failure (kN/m^2)	Peak shear stress at failure (kN/m^2)	Normal stress at failure (kN/m^2)
1	5.6	7.6	4.6	10.5	4.3	10.2
2	5.6	8.7	5.0	12.4	4.6	11.2
3	6.1	9.8	6.1	13.3	5.9	10.3
4	7.1	8.7	6.2	13.0	5.9	12.1
5	8.4	8.3	7.5	14.0	6.9	11.2
6	8.4	9.6	7.6	16.0	7.0	13.0
7	9.5	10.1	8.5	12.5	7.9	11.5
8	9.5	11.5	8.5	14.5	8.1	12.6
9	9.8	9.2	8.8	13.7	8.2	14.1
10	10.8	11.4	9.8	14.3	9.4	13.8
11	11.3	12.3	9.8	16.3	9.5	15.9
12	12.1	9.8	11.0	17.3	10.3	16.6
13	12.4	13.5	11.3	15.9	10.4	14.9
14	13.3	13.3	12.1	16.3	11.3	16.3
15	13.6	11.1	12.4	19.1	11.4	18.4
β	**16°**		**24°**		**33°**	
c'	**5.4 kN/m^2**		**7.8 kN/m^2**		**5.5 kN/m^2**	
ϕ'	**27.3°**		**38.4°**		**45°**	

Table 3: Particle size composition for regolith samples taken at a depth of 0.75m from the three slope sections.

Slope Profile Section	Gravel >2.00mm	Sand 2.00-0.06mm	Silt 0.06-0.002mm	Clay <0.002mm	Liquid Limit	Plastic Limit	Plasticity Index	B.S. Soil Classification
Lower Slope Section	-	24%	23%	53%	30%	21%	9%	FS-CLS
Middle Slope Section	30%	28%	24%	18%	23%	17%	6%	GF-GML
Upper Slope Section	50%	35%	6%	9%	33%	24%	9%	G-F GPC

Drained, effective stress parameters c' and ϕ' are obtained by the apparatus as drainage is achieved along the shearing plane after peak strength is achieved. The development of pore suctions or excess porewater pressures are unlikely because of rapid and immediate dissipation along microscopic pedological and biological structural discontinuities, such as root channels, and because the rate of testing for shallow residual soils permits adequate drainage to occur.

In this study the values of ϕ' are related to the particle sorting; regolith samples consisting of high proportions of gravel and sand gave higher values of ϕ' (see Tables 2 and 3). Regolith samples collected from the upper and middle sections of the slope contained significant proportions of gravel and sand whereas the regolith samples collected from the lower slope contained significant proportions of clay and silt (see Table 2). The results of (ϕ') obtained appear to correspond reasonably well with those for similar regolith materials sampled from Derbyshire hillslopes by Carson and Petley (1970); Carson and Kirkby (1972); Carson (1975).

The variation in the effective cohesion (c') appears to be associated with vegetation changes causing corresponding changes in the mechanical reinforcement of soil by roots. The effective cohesion (c') was higher for the middle slope section where bracken (*Pteridiun aquilinum*) was dominant. The bracken roots extended to depths within the regolith mantle over 1.0m. Bracken was absent on the lower and upper slope profile sections, grasses (*Nardus stricta, Agrostis spp, Festuca spp*) were dominant plant species of these sections. A model of the cohesive strength imparted by roots to soil, based on the engineering theory of reinforced earth, has been presented by Waldron (1977). It appears that the direct field shear box apparatus provided a practical way of measuring the contribution of root systems of different vegetation cover to soil cohesive strength.

Conclusions

1. The open-sided field direct shear box is capable of testing relatively large in situ samples which can accommodate the range of textural, morphological and vegetative features which are characteristic of shallow residual soils.
2. The use of an open-sided field shear box allows the selection of a particular failure zone which can be affected by the structural and compositional irregularities of the soil. The preparation of in situ samples permits both abrupt and gradual visible discontinuities to be tested while preserving their relationship with adjacent materials. A consideration of both micro-topography and subsoil morphology may be used in determining the sample depth and orientation most critical for an assessment of hillslope stability. The apparatus has a unique advantage over laboratory tests in situations where shallow regolith overlies rockhead.
3. Although sample disturbance is not completely eliminated it is certainly minimized by *in situ* testing. The difficulties and costs of sample collection, application of preservative coatings, transporting, and final laboratory preparation of large samples are avoided.
4. The open-sided field direct shear box apparatus is robust and portable. Its use for measuring soil shear strength at shallow depths costs less than in time, expense and manpower compared to traditional in-situ strength tests.
5. The apparatus has been proven to be particularly applicable to determining the contribution of vegetation cover on slope stability.
6. Use of the apparatus allows the determination of strength parameters which, while different from standard laboratory engineering test results, are reasonable and realistic for shallow, low stress conditions. This has been illustrated through the results of the south Pennines case study.

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