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Laboratory measured parameters of well graded soils

Mesure en laboratoire des paramètres des sols hétérogènes

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SYNOPSIS

In the development of modern concepts of soil behaviour there has been extensive use of model soils that obey certain constitutive relationships. There have been detailed investigations into the behaviour of clays, particularly kaolin, and of sands. Information regarding the behaviour of well graded soils which contain a wide range of particle sizes is minimal. Well graded soils such as some glacial tills and some residual deposits occur frequently in engineering problems in various parts of the world. Current research work at Trinity College, Dublin, is aimed at developing an understanding of the behaviour of such soils and measuring their fundamental properties. Direct comparisons can then be made with existing data from tests on clays and sands, and the differences between the behaviour of well-graded soils and theoretical models can be quantified. The relevance of index tests in relation to well graded soils is examined together with consolidation, strength and deformation characteristics. The triaxial test results indicate that the undrained stress path in p' , q space is not elliptical and that experimental results are at variance with currently accepted theories.

INTRODUCTION

Research studies into the behaviour of well graded soils are not common, despite the fact that such soils are frequently encountered in engineering problems in different parts of the world. In Ireland, a stiff dense glacial till is particularly common, it has a high bearing capacity and negligible deformation under normal working loads. As a consequence there has been no demand for an explanation of its behaviour. Well graded soils are being encountered in construction projects in many developing countries where little or no information regarding their behaviour is available. This paper presents the results of laboratory tests to measure the fundamental properties of two distinctly different well graded soils. The test results are compared with those of typical clays and sands and methods of relating the various properties are examined.

THE SOILS

Two soils of similar grading but different origin have been examined. The particle size distributions of these two soils are shown in Fig. 1.

Soil 'A', a brown glacial till, widely distributed in Ireland is a well graded deposit consisting of particles of all sizes. Electron microscope and x-ray diffraction studies show that the clay fraction is composed of angular grains with quartz and feldspar being the only minerals present (Fig. 2). The till is commonly called "Boulder Clay" but this is not considered to be an appropriate definition since the soil comprises similar proportions of gravel, sand

and silt with normally less than 15% clay sized particles. Unlike many other tills it does not consist of large stones dispersed in a stiff matrix of silt and clay. The till has a high dry density, often in excess of 2.0 Mg/m^3 with all particle sizes randomly dispersed within the soil mass. It is normally regarded as being heavily overconsolidated however comments in this paper are confined to normally consolidated, saturated specimens and are therefore relevant to the behaviour of the soil in a remoulded state. The till has a natural water content in the range 11%-14%. A large number of excavations, cuttings and trial pits have been examined and fissures have not been observed. This is contrary to the assertion by McGown (McGown, 1975) that the till found in Ireland is fissured, further there is insufficient evidence to suggest that a random packing of stones of various sizes should contain fissures.

Soil 'B' is a recent residual deposit of volcanic origin with the same range of particle sizes as the till. It only exists to shallow depths of 2m-3m and it is underlain by the parent rock which is weak and severely weathered. Quartz is the predominant mineral present in the soil above $2 \mu\text{m}$ (Fig. 3) but unlike the till, there are significant quantities of iron making this soil red-brown in colour. Clay minerals including kaolinite and illite have been identified (Fig. 4) and the dispersed clay sized particles tend to be flat rather than angular. It has not been possible to estimate the amount of clay minerals as a proportion of the clay sized fraction. Values of natural water content of 50% have been measured and as a result the in-situ dry density can be as low as 1.2 Mg/m^3 .

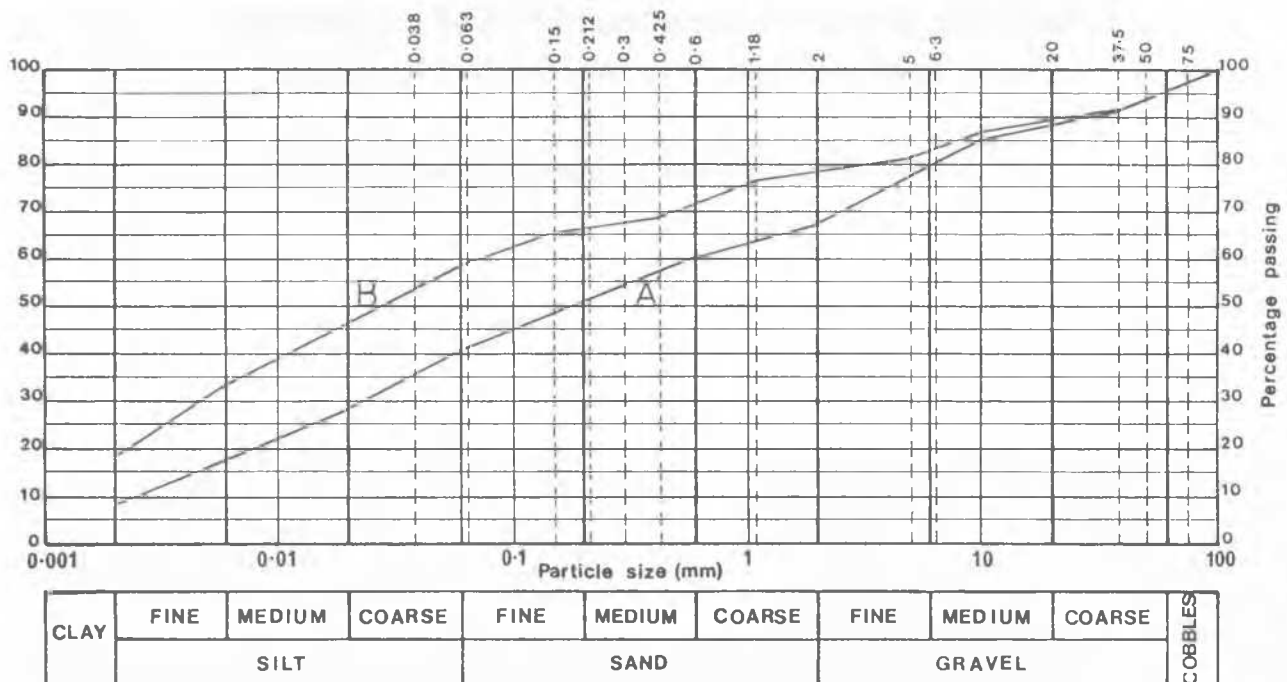


Fig. 1: Particle Size Distribution

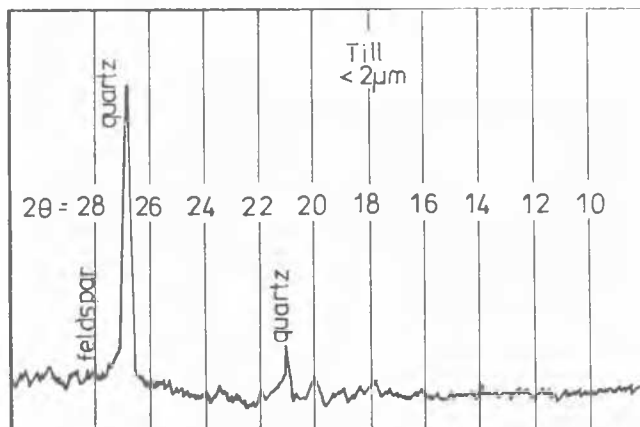


Fig. 2: XRD of Clay Fraction of Till

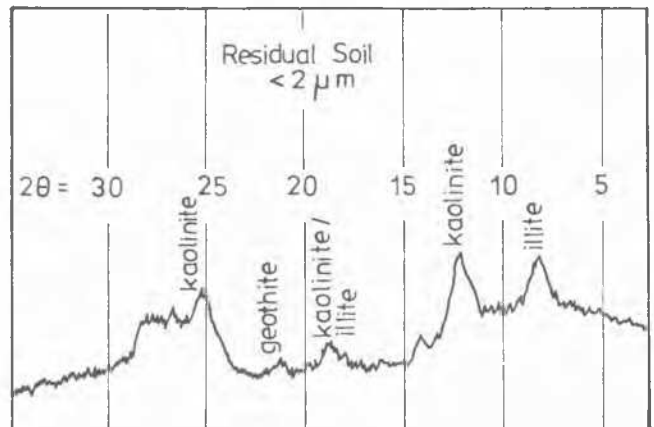


Fig. 4: XRD of Clay Fraction of Residual Soil

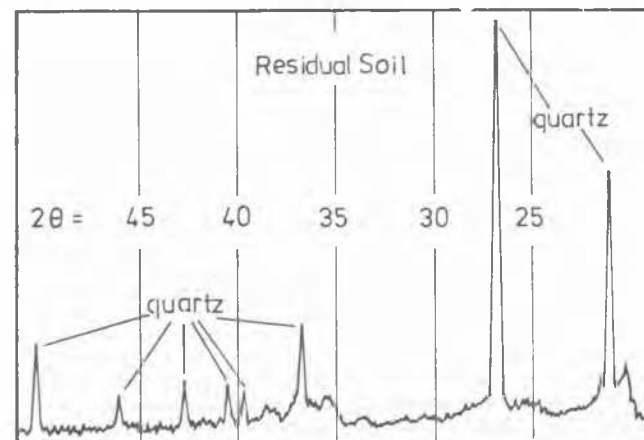


Fig. 3: XRD of Residual Soil

Seasonal variations in water content induce changes in the effective stress state of the soil mass. The stability of a natural slope is dependent upon the intensity and amount of rainfall.

LABORATORY TESTING PROGRAMME

A series of standard laboratory tests has been carried out on both soils to determine their properties, a summary of which is given in Table 1. The presence of large stones makes sampling extremely difficult, consequently it has not been possible to obtain undisturbed samples of either soil. The testing programme is confined to tests carried out on reconstituted samples with a maximum stone size of 6mm. Of the tests performed, the consistency limits, undrained triaxial tests with pore pressure measurement and isotropic consolidation tests are of greatest importance.

TABLE 1

	w_l	w_p	% clay size	G_s	M	w_{nat}
Till	.33	.22	8	2.68	1.32	.11-.14
Residual Soil	.70	.28	18	2.73	1.48	.40-.50

CONSISTENCY LIMITS

The liquid and plastic limits measured on the fraction of the soil finer than 425 μ m are not directly applicable to the consistency of the whole soil mass. Methods of adjusting the values measured in the test to describe the consistency of the soil mass have been proposed in the past (e.g. Arrowsmith, 1977). It is not possible to remould specimens of the till near to or above the plastic limit. The liquidity index of the soil mass can be as low as -1. In contrast, the residual soil occurs naturally at water contents above the plastic limit and has a liquidity index greater than zero. If an adjustment to the consistency limits based on the proportion of the soil mass not included in the test is made then the adjustment for both soils will be approximately the same since they have similar grading curves. A correction of this nature is inadequate when due consideration is taken of the range of water contents over which the two soils can be remoulded. Hence the consistency of the soil mass cannot be directly inferred from that of the fraction tested. The concept of activity (Skempton, 1953) does not clarify the problem because it is extremely sensitive to small changes in the percentage of clay in soils where the total proportion of clay is small.

ISOTROPIC CONSOLIDATION TESTS

Isotropic consolidation tests were performed to determine the normal consolidation and swelling lines of the two soils. These results are plotted in Fig. 5 together with the results of similar tests on London Clay and Weald Clay performed by Parry and reported by Schofield and Wroth (Schofield and Wroth, 1968). Also plotted are the results of isotropic consolidation tests on sand performed by Vesic and Clough and reported by Atkinson (Atkinson, 1978).

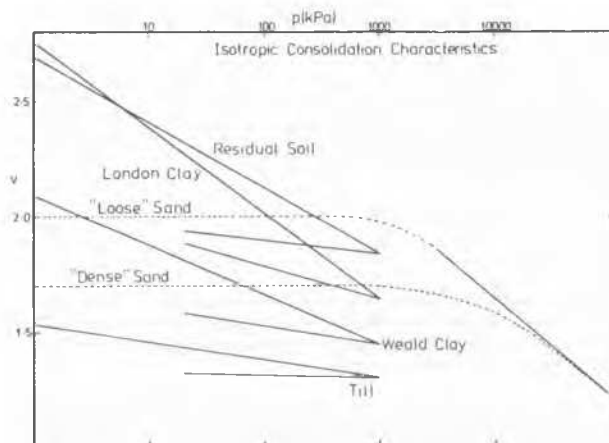


Fig. 5

It is clear from Fig. 5 that the well graded soils have consolidation characteristics similar to clays despite the large proportion of granular material present. A summary of the relevant consolidation characteristics is given in Table 2. Of particular interest is the high value of λ measured for the high plasticity residual soil and the correspondingly low value of λ measured for the low plasticity till.

TABLE II

Property	London Clay	Weald Clay	Residual Soil	Till
v_{wl}	3.144	2.180	2.911	1.884
v_{pl}	1.715	1.495	1.764	1.590
v_{pi}	1.429	0.688	1.147	0.295
N	2.752	2.088	2.689	1.534
v_{pi}/N	0.520	0.329	0.427	0.192
λ	0.161	0.093	0.124	0.034
K	0.062	0.035	0.026	0.005

CONSOLIDATION CHARACTERISTICS IN RELATION TO CONSISTENCY LIMITS

Various attempts have been made in the past to relate consolidation test data to the consistency limits (Skempton, 1957, Schofield et al, ibid). The empirical relationship of Skempton:

$$C'_c = 0.7(w_l - 0.1)$$

is equivalent to

$$\lambda = 0.304(w_l - 0.1)$$

This compares well with the equation of Schofield and Wroth

$$\lambda = 0.36(w_l - 0.09)$$

These two relationships are plotted in Fig. 6 together with the test results of Parry and the authors. In view of the suggestion by Schofield (Schofield et al, ibid) that the plastic limit may be the more appropriate parameter to use, the measured values of λ are plotted against the plastic limit (Fig. 7).

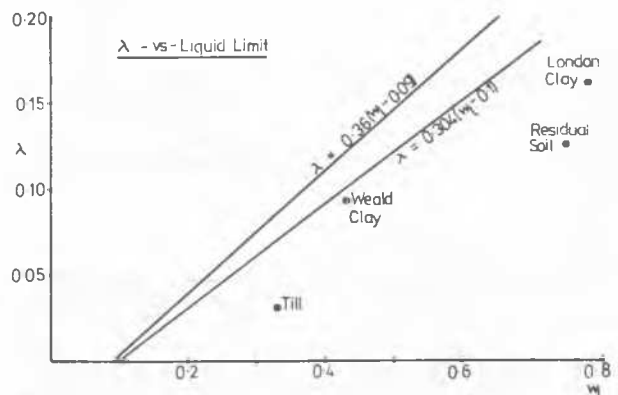


Fig. 6

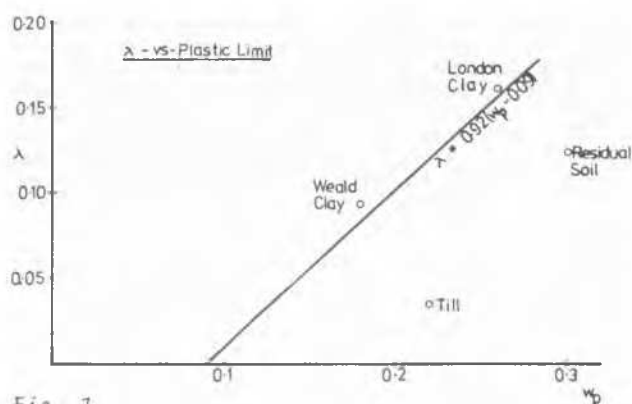


Fig. 7

Examination of Figs. 6 and 7 suggest that neither of these two relationships are particularly suited to the purpose of relating consolidation test data to the consistency limits of well graded soils.

Since consolidation is the process of squeezing of water from the voids, it seems reasonable to take due account of the amount of water within the soil mass, as well as the properties of the fine fraction, when relating consolidation characteristics and consistency limits. With the view to relating 'λ' to consistency limits it is convenient to express the plasticity limits in terms of the specific volume at the liquid and plastic limits

$$\text{i.e. } v_{pi} = v_{wl} - v_{wp}$$

The water content within the soil mass can be expressed in terms of the specific volume for a mean stress of $P = 1.0 \text{ kPa}$. This value is chosen since the natural logarithm of 1.0 is 0. This specific volume is denoted by 'N' in this paper.

$$\text{i.e. } N = v(P = 1.0 \text{ kPa})$$

The consistency limits of the fine fraction can

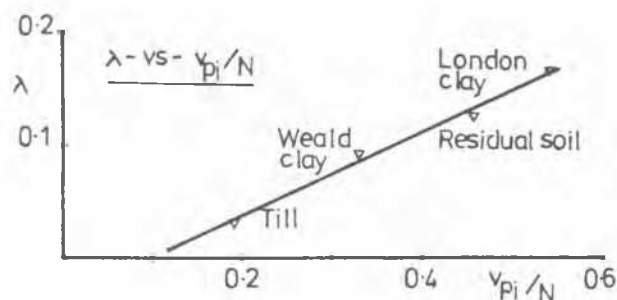


Fig. 8: Relationship between the slope of the normal consolidation line of four soils and their plasticity characteristics.

be combined with a possible specific volume of the soil mass in the form of a ratio

$$\frac{v_{pi}}{N}$$

If the slope of the normal consolidation line is plotted against this ratio as shown in Fig. 8, a straight line fit is obtained. This is particularly significant in view of the fact that the four soils considered have been formed under different conditions in different parts of the world and have widely varying characteristics.

TRIAXIAL TESTS

A series of undrained triaxial tests with pressure measurement have been performed on remoulded specimens of both soils. It has been suggested that low plasticity glacial soils are insensitive to remoulding and a sensitivity of 1 is normally assumed for the till (Vaughan et al, 1975). The sensitivity of the residual soil to remoulding has not been established. Although a sensitivity of 1 implies agreement between the peak strengths of remoulded and undisturbed samples it does not mean that the deformation characteristics prior to failure are the same. The effects of remoulding larger stones from the bulk sample can be estimated from trends observed by testing specimens with different uniformity coefficients. The soil properties as measured in the triaxial apparatus and presented in this paper were derived from tests performed on 75mm x 38mm remoulded specimens with a maximum stone size of 6mm. Side drains were not used since they can lead to a variation in stiffness across the diameter of the specimen.

For the purpose of this paper a suitable selection of test results are presented to indicate the general behaviour and they are compared with the results of similar tests performed on sands and clays.

The results of tests carried out on specimens of the two soils which were isotropically consolidated to an initial mean effective stress of 50kPa are given in Fig. 9. The results are very similar in many respects, in particular, peak deviator stress is reached at large strains and does not occur at the same strain as peak pore water pressure. A similar behaviour has been observed for a loose sand (Bishop, Henkel) as shown in Fig. 10.

It is clear from the results of both the residual soil and the sand that there are two values of axial strain at which the criterion for failure at constant pore pressure (constant volume) can hold.

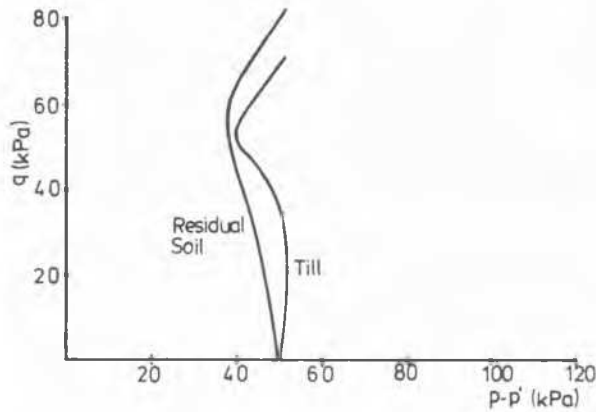
$$\frac{du}{d\epsilon_1} = 0$$

Examination of Fig. 11 indicates that the strain to failure and the post peak reduction in pore pressure that occurs with this strain depend upon the confining stress.

Further examination of the results of the tests on the residual soil show that before peak pore water pressure is attained there is a linear relationship between pore water pressure and the

ratio of the deviator stress to the mean effective stress (Fig. 12)

$$\text{i.e. } u \propto q/p'$$



Undrained Triaxial Test Results

Fig. 9: Results of consolidated undrained triaxial tests on 38mm diameter specimens of both soils. Rate of loading 0.0024mm/min. Specimens assume to be saturated ($B > 0.98$).

Owing to the dependence of the pore water pressure on the mean stress it is appropriate to normalise the pore pressure by dividing the

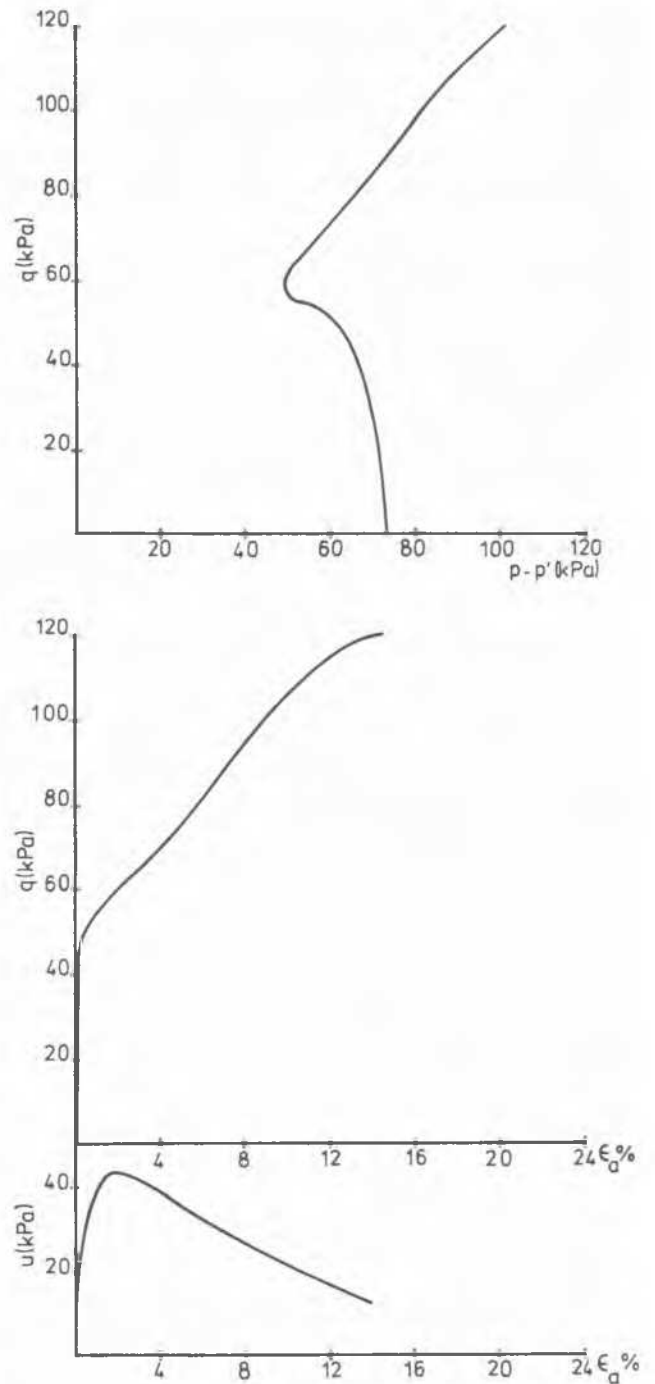
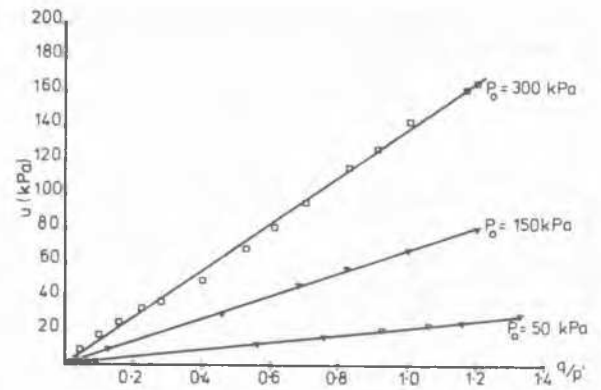
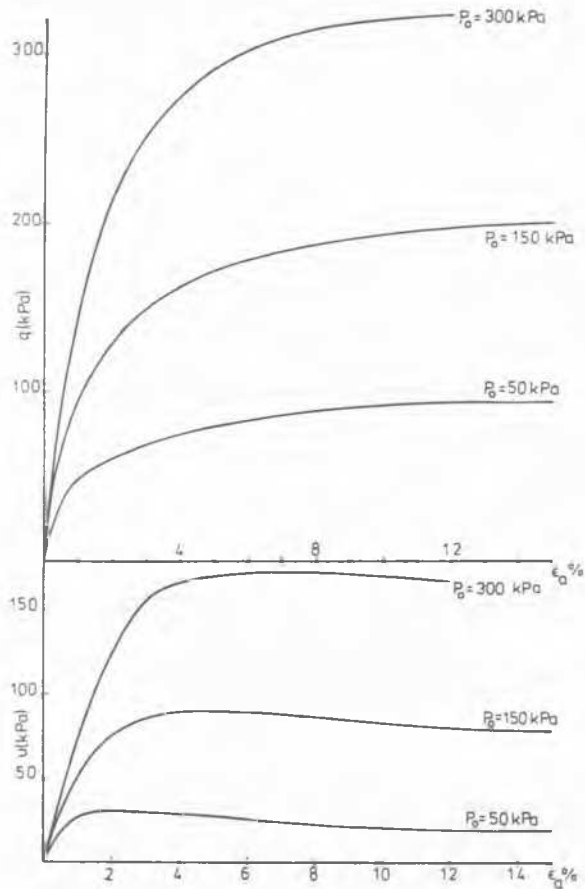
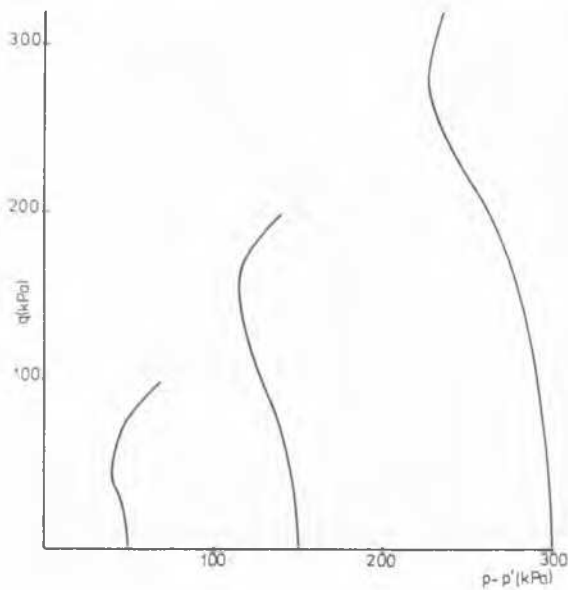


Fig. 10: Undrained triaxial tests on loose sand (Bishop and Henkel).

measured values of ' u ' by ' p_0 ', the initial mean stress. The results are re-plotted in Fig. 13 and all the points lie generally on a straight line. A similar behaviour is observed for tests on Boston Blue Clay by Taylor and Clough and reported by Bishop (Bishop, 1971), and Loose Sand (Bishop et al, 1965) (Fig. 14).



Variation of pore pressure with stress ratio

Fig. 12

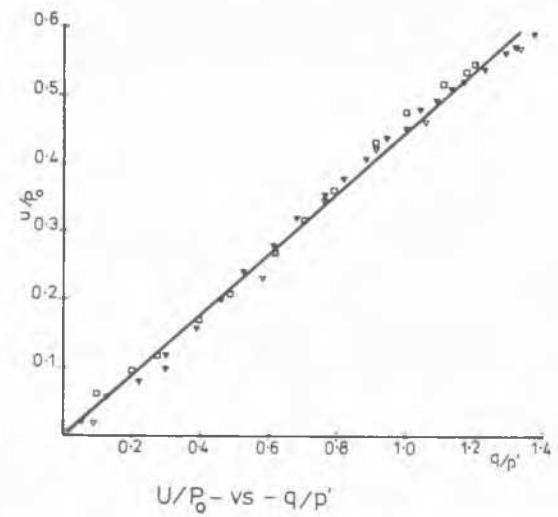
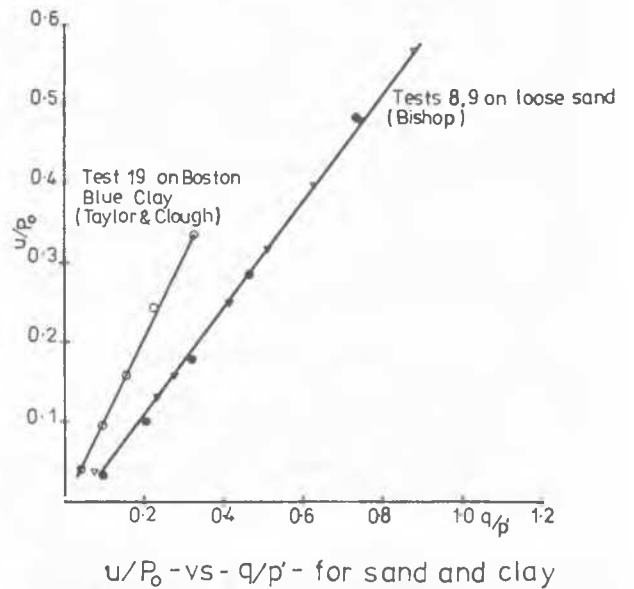


Fig. 13



Triaxial Tests on the Residual Soil

Fig. 11

Fig. 14

It is possible to define the shape of the stress path as:

$$\frac{u}{p_0} = Kq/p'$$

where K is a constant.

The following statement can then be made regarding the portion of the stress path in q/p' space where the pore water pressure is increasing and the mean effective stress is decreasing.

"The pore water pressure is directly proportional to the product of the confining stress and the ratio of the deviator and mean effective stress".

It is therefore possible to define this portion of the effective stress path mathematically as the locus of points whose distance from the total stress path is proportional to the slopes of the lines connecting the points and the origin. It has been shown (Hartford, 1984) that this curve is part of a conic section of the form

$$\frac{X^2}{a^2} - \frac{Y^2}{b^2} = 1$$

This is the equation of a hyperbola and thus the results of tests on well graded soils are at variance with the modified Cam clay model (Burland, 1965). Since sands and clays also show a similar linear relationship between pore water pressure and stress ratio the author would suggest that many naturally occurring soils exhibit a stress path that is part of a hyperbola and not an ellipse as required by consideration of plasticity theory.

CONCLUSIONS

A more general relationship between the consistency limits and the consolidation characteristics is obtained when the saturated water content of the soil mass is included.

The results of undrained triaxial tests on two well graded soils show stress strain characteristics similar to sands.

There is a linear relationship between pore water pressure (u) and effective stress ratio (q/p') at small strains for the two well graded soils, sand (Bishop) and clay (Taylor & Clough). The stress path defined by this relationship is a hyperbola.

The authors show that the elliptical stress path used in the modified Cam clay does not adequately describe the stress path followed by a range of natural soils tested under undrained triaxial conditions.

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