unconfined compressive strength tests when applied to varved clays, but to the disturbance produced by the type of sampler employed.

CONCLUSIONS.

1) Unconfined compressive strength tests are suitable for the determination of the shearing strength of varved clays and of other stratified cohesive soils.

2) In order to obtain reasonably correct strength values from really undisturbed specimens only modern types of thin-walled ("Shelby Tubing") samplers should be employed, especially on clays which are sensitive to remolding.

3) The comparison of the sensitivity of different clays to remolding should preferably be based on a ration of the strengths of the undisturbed and of the remolded samples at equal strains, instead of on the ratios of their respective ultimate strengths at failure.

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SUB-SECTION II e

DIRECT SHEAR TESTS

II e 1

A LARGE SHEAR BOX FOR TESTING SANDS AND GRAVELS

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I. INTRODUCTION.

The design of new reservoirs in the Thames Valley by the Metropolitan Water Board necessitated the examination of the stability of a number of proposed earth dams on sites overlain by a stratum of gravel. In addition the dams were to be constructed of rolled gravel fill.

As no satisfactory test results for gravel were available, or could be obtained with existing testing machines, a constant rate of strain shear box 1 foot square taking samples 6 inches in thickness was designed by the author and constructed by the Metropolitan Water Board.

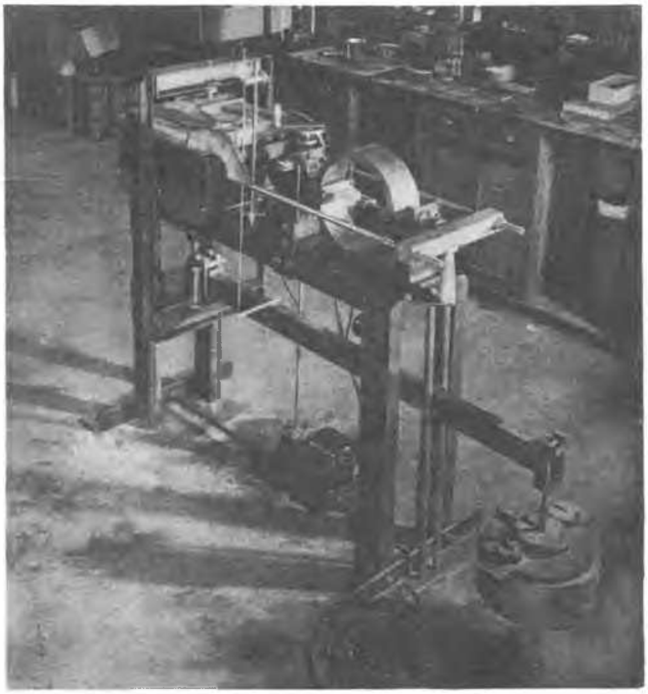
Tests are at present being carried out with this apparatus in the Soils Laboratory at Imperial College, with two main purposes in view.

a) For immediate design purposes, to determine:
1) the shear strength of in situ gravel by the correlation of field density measurements with laboratory tests over a range of densities.
2) the shear strength of samples of gravel under various degrees of compaction for the design of the rolled fill.

b) For a general research programme:
1) to examine the effects of grading and grain shape on the relation between shear strength and porosity, and extend knowlegde in this field into the range of materials containing particles up to 1½ inches effective diameter.
2) by comparative tests in the large and small (6 cm. square) shear boxes to examine scale effects in testing.

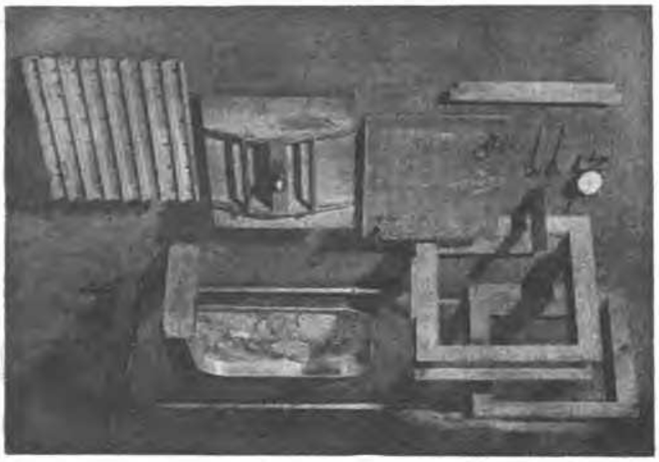
II. DESCRIPTION OF THE MACHINE.

The shear box (fig. 1 and plates I & II) is constructed of standard channel sections electrically welded and machined on the contact faces. The lower half of the box is carried in a steel tank which slides forward on two ball bearing tracks under the action of the loading jack. This provides the shearing force.



Foot Square Shear Box.

PHOT. 1



Shear Box Details.

PHOT. 2

The reaction is carried by two tension bars to the cross-head where it is measured by the deflection of the proving ring.

The normal load is applied by a simple lever system, and has a working range up to 3 tons per square foot. The loading jack is operated by a ¼ H.P. electric motor, isolated by rubber couplings and mountings to eliminate vibration. The rate of horizontal displacement is 2 inches per hour, the usual range during a test being 1 inch. The displacement is measured by a revolution counter on the jack. A correction can be made for the deflection of the proving ring, but this is only 72×10^{-4} inches per ton, and this does not make an appreciable difference to the shape of the load-deflecion curves.

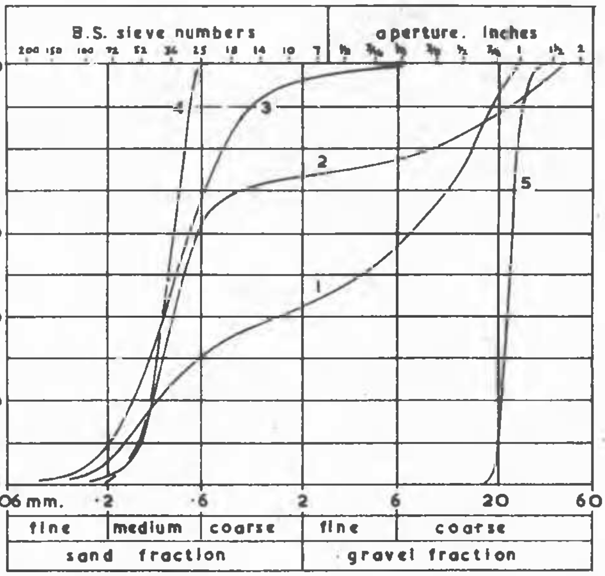
III. TESTING PROCEDURE.

Testing techniques have been developed to enable the preparation of a homogeneous sample and to ensure accurate measurement of its weight and volume. High porosities are obtained by rapid pouring, and low porosities by tamping in a number of layers by hand, though in the case of sands low porosities are also obtained by slow pouring (Kolbuszewski 1948). Sufficient tests have not yet been done to determine whether the angle of internal friction (at a given normal stress) depends only on the porosity or whether the method of packing affects it. The tests up to date indicate that the magnitude of any such effect is probably small.

IV. TYPICAL RESULTS.

Typical results are given in figs. 3 - 9 for 5 sands and gravels whose grading curves are shown in fig. 2. They are:

1) Heathrow gravel - a well graded Thames Valley gravel. (For this series of tests, particles greater than 1" were removed).



Sieve analysis.
1. Heathrow 2. Walton
3. Brasted 4. Ham River
5. Chesil Bank

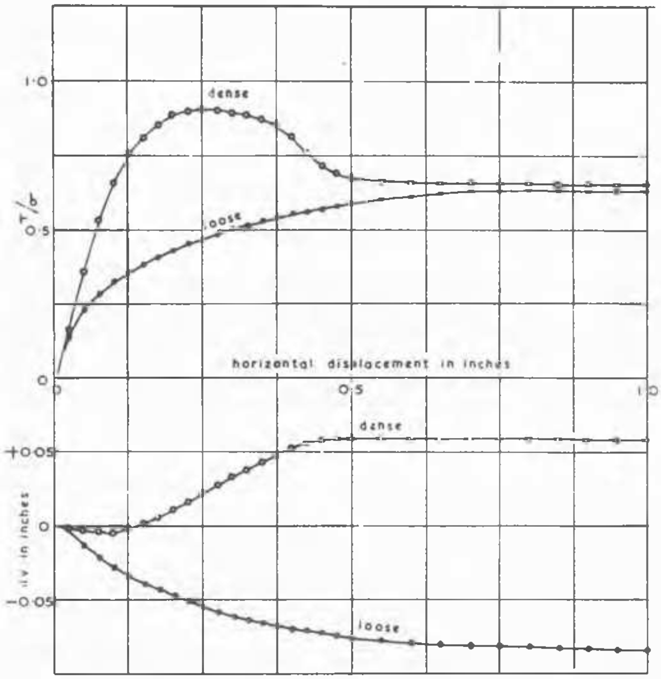
FIG.2

2) Walton gravel - a sandy gravel from the Thames Valley, possibly a mixture of thin adjacent lenses of sand and coarse gravel.
3) Brasted sand - a well graded sand of the Folkestone beds.
4) Ham River sand - a uniform sieved fraction from the Thames Valley gravels.
5) Chesil Bank pebbles - a uniform coarse beach gravel, consisting of rounded particles about 1" in diameter.

V. DISCUSSION OF RESULTS.

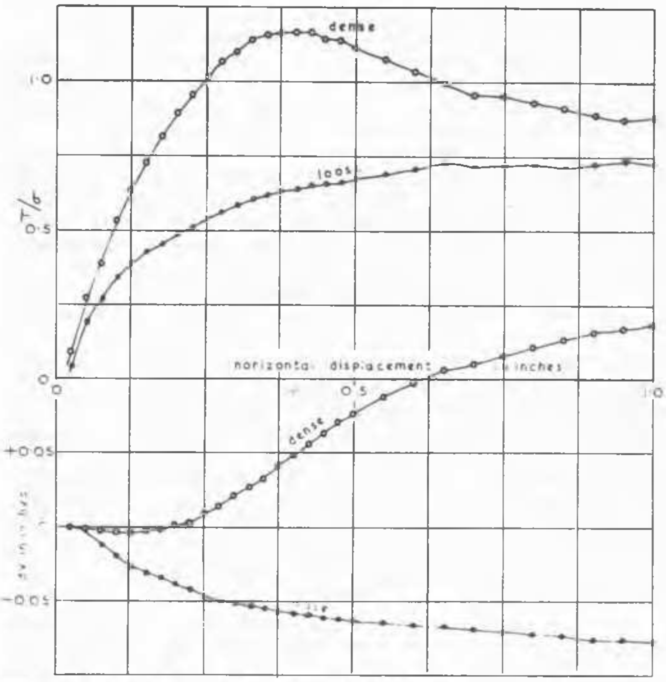
a) For the purposes of present design work the results shown in fig. 7 indicate that angles of friction of up to 45° may be obtained, even in not very well graded gravels at moderate porosities. The insitu porosity in this gravel from Walton could not be repeated in the laboratory by tamping the dry material.

Fig. 9 includes a similar curve for the well graded Heathrow gravel which gave angles



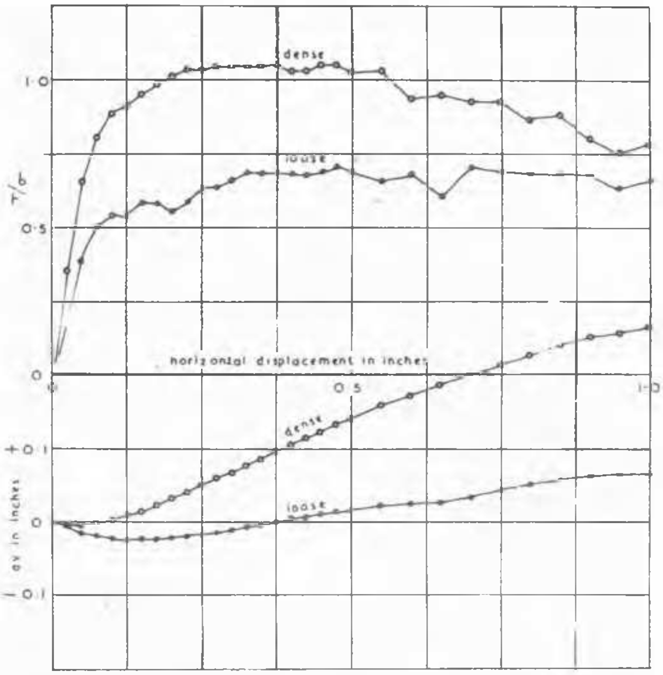
Shear tests on graded sand. Brasted.
Test no. 43. loose. n = 44.4% Ø = 32.3°
Test no. 44. dense. n = 33.3% Ø = 42.1°
σ = 2 tons / sq.ft.

FIG.3



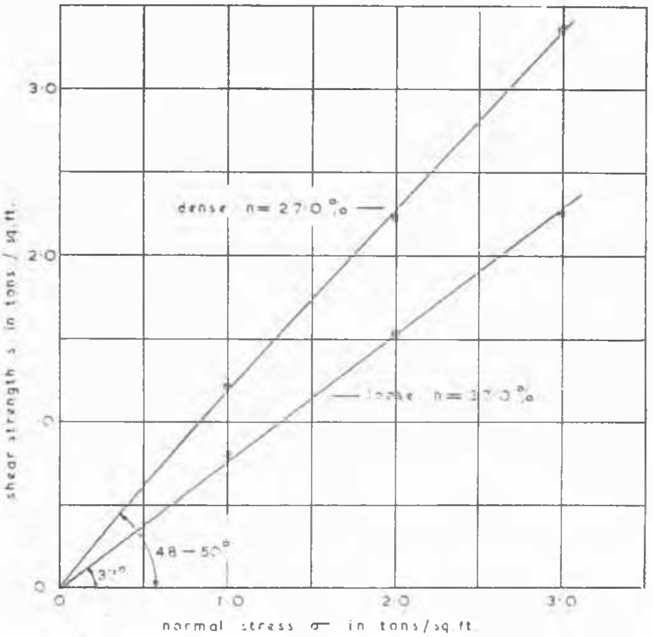
Shear tests on gravel. Heathrow.
Test no. 114. loose n = 36.8% Ø = 36.5°
Test no. 119. dense. n = 26.3% Ø = 49.5°
σ = 3 tons / sq.ft.

FIG.4



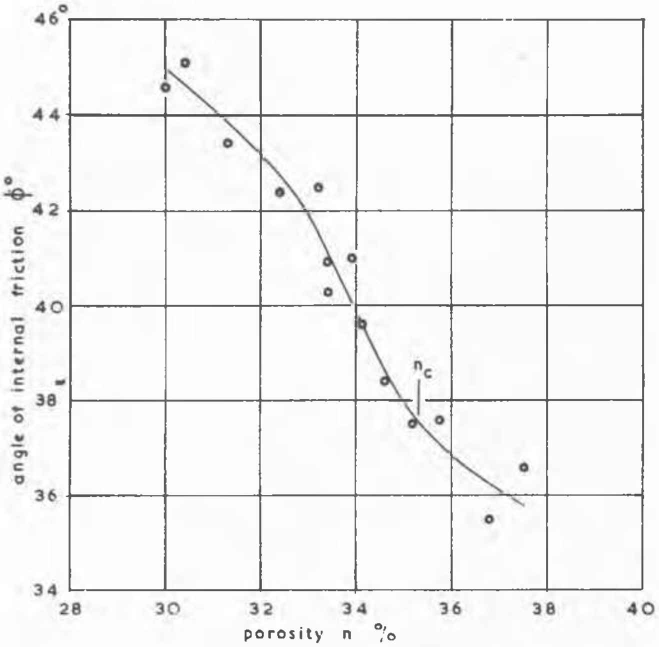
Shear tests on uniform gravel. Chesil Bank.
Test no. 96. loose. n = 42.0% Ø = 35.3°
Test no. 97. dense. n = 35.0% Ø = 46.5°
σ = 0.5 tons / sq.ft.

FIG.5



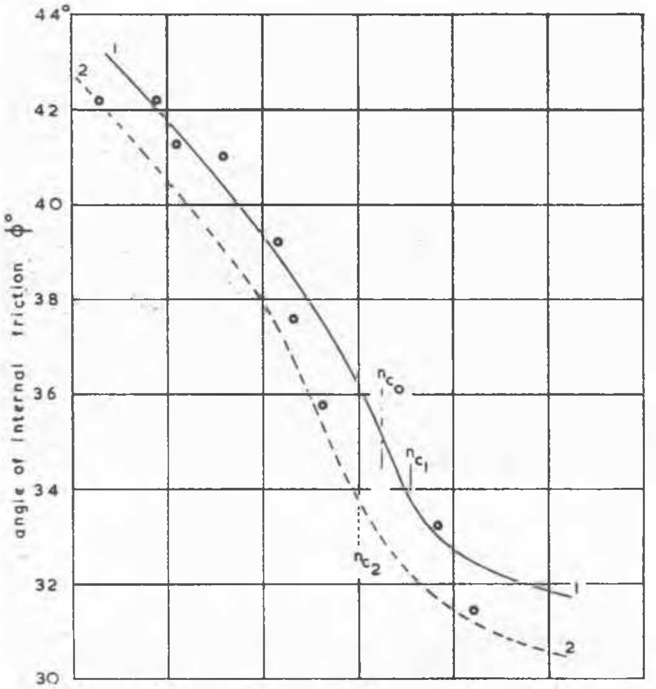
Relationship between shear strength and normal stress. Heathrow gravel.

FIG.6



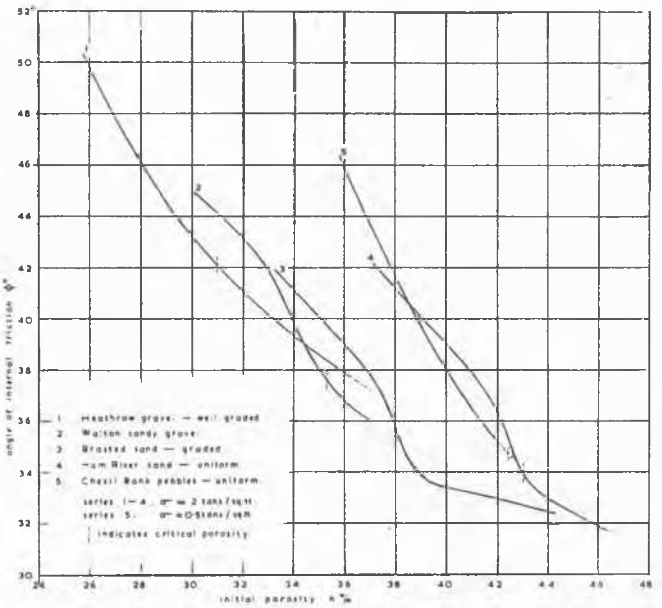
Variation of angle of internal friction with porosity. Walton gravel. σ= 2 tons / sq.ft.

FIG.7



Comparison of testing machines.
0..0. 6 cm. square. constant rate of strain.
1..1. 1 ft. square. constant rate of strain.
2..2. 6 cm. square. dead load.
Ham River sand.
σ= 2 tons / sq.ft.

FIG.8



Relationship of angle of internal friction to porosity for various gradings.

FIG.9

of friction of about 50° at porosities a little greater than those measured in the formation in which the material was being rolled.

Fig.4 shows typical test results for this material, and Fig. 6 gives the relationship between shear strength and normal stress for two different porosities. It is noteworthy that the high angle of internal friction x) does not decrease much within the range of stresses met with in earth dam design in this country.

b) 1. As pilot tests for a more detailed investigation the results given in Fig. 9 indicate that in the relation of angle of friction to porosity, graded and uniform materials separate out into two definite groups, between which a transition zone will presumably lie.

It is very striking that the tests on the Chesit Bank pebbles xa) give results very similar to those of the Ham River sand, which though about $^1/60$ in linear dimensions, has a similar grading curve. Tests results in Fig. 5 indicate that the shearing action (with the pebbles) is becoming jerky and that they probably represent an upper limit to the size of uniform material that can be tested accurately in this machine. In a graded material, however, the larger particles do not seem to control the behaviour, and it will be seen from Fig. 4 that the shearing action is remarkable smooth, and the expansion of the sample (dv) is smaller.

2. A comparison is also made in Fig. 8 with results obtained on the small (6 cm. square) boxes using both constant rate of stain (Golder 1942) and dead loading. The methods of compaction in this set of results are similar in the 3 cases.

It is seen from the figure that the results of the smaller box scatter on both sides of the average line for the large machine, although they tend to give a slightly lower value of Ø.

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x) defining Ø as = $\tan^{-1}\frac{\tau}{\sigma}$ at failure.

xa) these were tested at 0.5 ton/sq.ft. initially to avoid damage owing to the few contact points, but this will not materially affect the comparison.

There is, however, a much bigger difference between the two methods of testing in the same size box. This is also borne out by a few tests carried out at a very rapid constant rate of strain, which also gives lower values of ϕ. This indicates that the value of ϕ obtained does not depend to any great extent on the size of the machine (over a limiting size), but that discrepancies between measured values of ϕ and the results of field or model loading tests etc., may lie in the different types of failure induced.

3. The stress distribution within the sample and along the failure plane is not discussed in this paper, but preliminary examinations of the type of failure plane by Kotter's equation indicate that the deviation from the average of the normal stress on the failure plane is not serious.

VI. CONCLUSIONS.

A full range of cohesionless soils, ranging from sands to gravels and sandy gravels, has been tested in a shear box taking samples 1 foot square and 6 inches thick. For sands it has been found that the results obtained in the standard 6 cm. shear boxes are in good agreement with those obtained in this large box. It should be noted however that in the standard boxes the results appear to depend to some extend on the manner of applying the shear force.

It was found that materials with a uniform grain size, whether a medium sand or pebbles about 1 inch in diameter, gave similar results when plotted in the form of ϕ against porosity which were quite distinct from the relationships found for the graded materials.

The graded sands and sandy gravels lay more or less in the same general zone, but the lower porosities of the well graded sandy gravels resulted in higher angle of friction than the graded sand. The sandy gravels were found to have angles of friction as high as 50° at a dense packing.

VII. ACKNOWLEDGEMENTS.

For permission to publish details of the machine and of the Walton tests the Author is indepted to H.F. Cronin, M.I.C.E., Chief Engineer of the Metropolitan Water Board.

The Author is grateful to J.B. Miners, A.C.G.I. B.Sc. (Eng.) A.J. Smallman, A.C.G.I. B Sc. (Eng.) and M.A.A. Hafez B.Sc., for carrying out much of the testing programme referred to, and to Assistant Professor A.W. Skempton for his constant interest.

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II e 2

THE SHEARING RESISTANCE OF SOILS AS DETERMINED BY DIRECT SHEAR TESTS AT A CONSTANT RATE OF STRAIN

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State Rivers and Water Supply Commission, Victoria, Australia.

a) Three samples of soil, compacted at or near the optimum density, have been tested in direct shear in accordance with the latest developments of the procedures described elsewhere 1). Tests were made at three rates of strain: namely, 0.22, 0.055, and 0.007 inch per hour

b) Specimens were tested under normal stresses between 570 and 27.000 lb. per sq. ft., and after the determination of the maximum shearing resistance the cohesion, if any, was obtained by a new method 1).

c) The results were checked by tests wherein specimens were preconsolidated at a particular normal stress before testing at reduced normal stresses, also for conformity with certain geometrical requirements.

d) The same soils were also tested by means of the triaxial compression test in the Research Laboratory of the Melbourne and Metropolitan Board of Works in accordance with procedures described by D. F. Glynn 2).

1. The particle size distribution curves of the three soils are shown in Figure 1, and the classifications 3), Atterberg Limits, and compaction data are given in Table I.
2. The results of direct shear tests at various rates of strain are shown in Figures 2, 3, and 4; and the test curves, including the determination of the cohesion, are shown in Figures 5, 6 and 7. Data for tests on sample No. 2 are given in Table II. In order to check the values obtained by this new method for obtaining the cohesion of each specimen under a particular normal stress, several tests were made after preconsolidation and unloading to a lower normal stress.

It will be seen that the results by the two methods of testing are in close agreement in Figures 2(a) and 4(a). In Figure 3(a) agreement is also obtained when the particle displacement correction is made, and indicates one advantage of the additional information obtained by the new method. The curves of Figures 3(a) and 6(a) show the effect of a partial breakdown of the soil particles during