SYNOPSIS. Granular fills often contain large particles which do not fit into conventional test apparatus. Triaxial tests are reported on such a fill, both whole, and after removing larger particles. The peak angle of shearing resistance ϕ_{max} reduces, together with the ultimate angle ϕ_{crit}, as the grading of the material is narrowed. The original soil was also subjected to a pilot test, in which a 2m x 2m concrete base was cast over the compacted fill, loaded with weights, and sheared by jacking. The comparison between these various determinations of ϕ is used to discuss the selection of a design value.

BACKGROUND

1. A wide range of granular materials recovered from excavation, quarrying or demolition may be identified as suitable for compaction as fill. Some of these will contain particles so large that they can not be accommodated in standard soil test apparatus. The original material is then usually "scalped": over-size particles are removed prior to testing the finer soil matrix. The resulting distortion of properties has been investigated.

2. Bolton (ref. 1) recalled that the peak secant angle of shearing in compression tests could be expressed as follows:

$$\phi_{max} = \phi_{crit} + \Delta\phi \tag{1}$$

where ϕ_{crit} is the component for shearing at constant volume which is taken to be a soil constant, and $\Delta\phi$ is the variable component due to dilatancy. $\Delta\phi$ was then shown to be proportional to a relative dilatancy index I_R which was the product of the initial relative density I_D and a newly defined relative crushability I_C, so that

$$\Delta\phi = A \cdot I_R \tag{2}$$

with A = 5 in plane strain, A = 3 in triaxial strain, and

$$I_R = I_D I_C \tag{3}$$

where

$$I_D = \frac{(e_{max} - e)}{(e_{max} - e_{min})} \quad (4)$$

$$I_C = \ln {}^{p_c}/_{p'} \quad (5)$$

The relative crushability I_C is the logarithm of the ratio of an aggregate crushing stress p_c (to be determined), and the mean effective stress p' at failure.

3. These precepts gave a linear increase of $\Delta\phi$ with relative density at constant p', and a linear decrease of $\Delta\phi$ with log p' at constant density. They fitted the data of 17 sands within about 1 or 2°, with different values of ϕ_{crit} observed in the range 30° to 37° and a single value p_c = 22 000 kPa. The value p_c was selected to give a good fit of triaxial test data to equation 2, and was inferred to be related to particle crushing, on the basis of breakage observed by sieving after tests.

THE PILOT SCALE SHEAR TEST

The trial fill

4. It was decided to select a material that would be suitable for use both in embankment construction and as back-fill to structures, according to the Department of Transport Specification for Highway Works (1986). The selected river gravel was obtained from a site near Crowthorne in Berkshire, convenient to the Transport Research Laboratory at which the pilot test was to take place. A few random samples were taken from the 60 tonnes of fill required for the test, and the averaged grading curve is marked (i) in Fig. 1. The well-graded fill had less than 5% of fines (< 63 μm) and the nominal maximum particle size was 45 mm. This grading was found to be consistent in every compacted layer.

5. The mean limiting densities of the fill, and its derivatives used for triaxial tests, are recorded in Table 1. Since some of the particles were larger than permitted in standard tests, a non-standard brass CBR mould 200 mm in diameter and 215 mm high was used for the dry density. The maximum density was tested by vibrating the dry soil under 14 kPa surcharge, following ASTM D4253a (1983) except that the duration was increased from 8 to 15 minutes. The minimum density was found using ASTM D4254b (1983), filling the larger mould by inverting a thin-walled tube containing the dry fill. Notwithstanding that the coarse gravel was inherently lighter, its removal resulted in successively smaller limiting densities, due to the narrowing of the range of particle sizes.

The compaction of the test fill

6. The test was carried out in a concrete-lined pit, 3.43 m x 6.25 m in plan and dug to 1.3 m depth where natural stiff clay was encountered. Angular aggregates were compacted in a thin layer of cement paste to form a rough and solid base for subsequent compaction of the fill.

	max. particle size d_{max} (mm)	G_s	$\rho_{d,max}$ Mg/m³	$\rho_{d,min}$ Mg/m³
(i) material used in pilot test	45 *	2.62	2.276	1.866
(ii) fill tested in 9" triaxial	38	2.62	2.230	1.820
(iii) fill tested in 4" triaxial	20	2.65	2.049	1.655
(iv) fill tested in 4" triaxial	12	2.65	1.981	1.649

* particles of up to 75 mm are sparsely distributed in the fill

Table 1 Properties of the pilot test fill and its derivatives

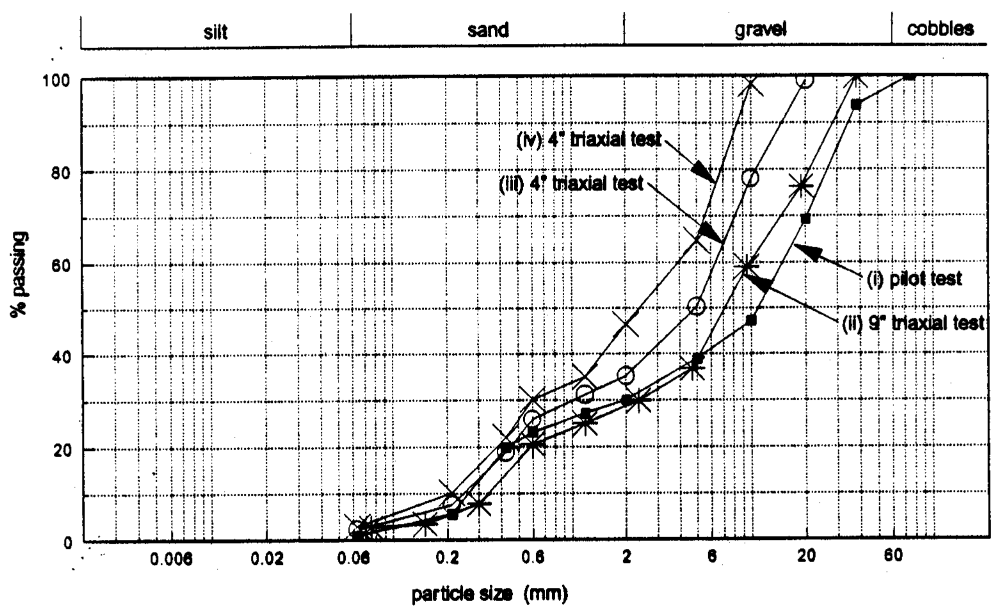

Fig. 1 Grading curves of the pilot test fill and its derivatives

7. The compaction procedure was according to the DTp Specification for Highway Works (Part 2, Series 600, 1986). A Wacker DPU vibrating plate compacter, weighing 605 kg and providing a static loading of approximately 15 kPa (to be doubled due to inertia effects), was used to compact each layer of soil to 125 mm in 6 passes. Fig. 2 shows the data of compaction tests carried out in CBR moulds to BS 1377, for which it was necessary to remove particles larger than 38 mm. Maximum compaction was achieved at about 6% moisture content. The moisture content of the fill as placed varied from 5 to 9%. The compacted dry density of each layer was determined by a Troxler 3411B nuclear moisture-density gauge at 4 to 6

locations in each layer, calibrated by conventional sand replacement tests at different densities. It became apparent that some nuclear gauge tests had been affected by the presence of very large particles. When obviously excessive values were discarded, the average dry density of the whole fill was estimated to be 2.23 Mg/m^3, which indicated a relative density I_D = 90.6%. In all, 7 total pressure cells, 20 markers, and 2 piezometers were installed in the fill, as shown in Fig. 3. Further details are given in Lee (ref. 2).

The reinforced concrete pad

8. Shearing in the fill was induced by the horizontal displacement of a lightly reinforced concrete pad cast on top of the fill surface. A low slump of 1 inch was specified to avoid bleed into the fill. Care was taken to prepare the fill surface, using first a stiff brush and then a vacuum cleaner to remove smooth patches of fines left after compaction, without disturbing the body of the fill. A steel plate was cast into the end face of the pad, for jacking. Fig. 4 shows the test arrangement.

9. A vertical stress of 75.4 kPa was provided by a weight of 302 kN – the pad itself, with 48 concrete blocks stacked evenly over it. This magnitude of stress normal to a slip plane was thought typical of earthworks and structures. The horizontal force was provided by a 500 kN hydraulic jack supported on a pair of load cells, acting only 105 mm above the soil surface to minimise the complication of an induced moment. The movement of the pad was picked up by four 50 mm LVDTs and two 300 mm displacement potentiometers. LVDTs and dial gauges were also used to locate the anticipated passive zone of heave.

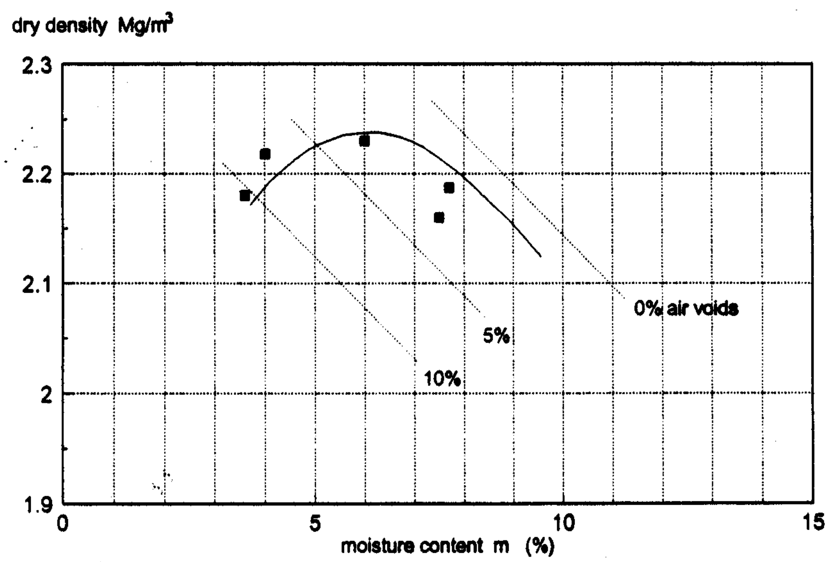

Fig. 2 Compaction tests using CBR mould on pilot material with particles larger than 38mm scalped off

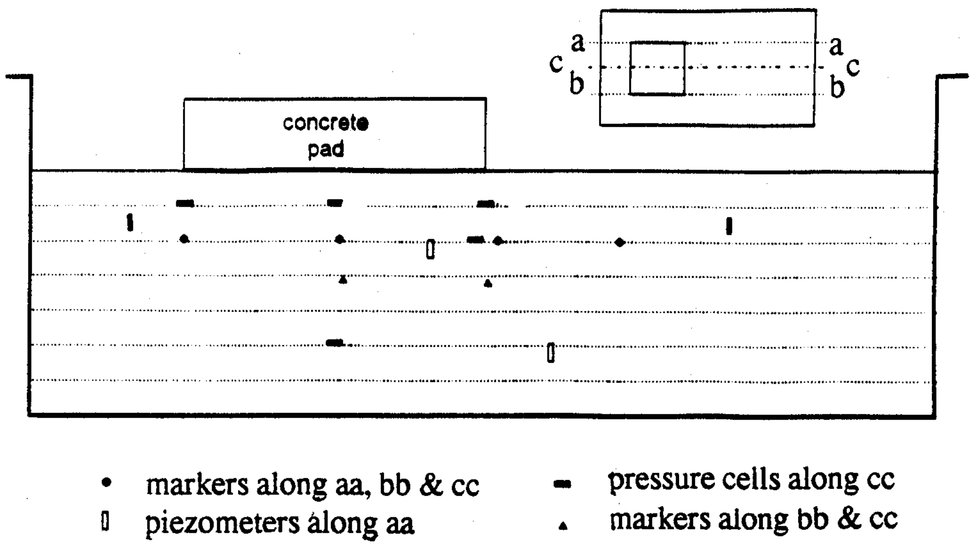

Fig.3 Positions of buried instruments

Test procedure

10. The test was conducted with the fill fully saturated to eliminate suction. Water was introduced from the base 24 hours prior to testing. Within 0.9 mm of shear displacement it became obvious that the pump for the jack was inadequate. The test was suspended and restarted with a more powerful unit. The shear force had, in that cycle, risen to 62% of the peak value recorded subsequently; its influence on the magnitude of that peak has been disregarded. The shear test proper lasted 90 minutes, the jack being advanced at 1 mm/min for the first 35 mm, increasing gradually thereafter to 5 mm/min up to 138 mm of horizontal displacement. During the drive, the block yawed by up to ± 3° in plan view, but only after achieving peak shear force; this was also disregarded. After the test, the concrete blocks were removed and the reinforced concrete pad was lifted for inspection. The concrete/soil boundary was found to be consistently rough and free from evidence of sliding. The final locations of pressure cells and markers were then measured.

Results

11. Fig. 5 shows the data of (a) shear force, (b) pad heave, (c) soil heave in front of pad – all with respect to the horizontal displacement of the pad. The peak shear strength of 313 kN gives an interface shear stress of 78.2 kPa and a mobilised angle of friction on the base of $\delta = \tan^{-1}(78.2/75.4) = 46.1°$. It is mobilised at a shear displacement of 5 mm. At that stage, the rate of heave of the pad was at its maximum, though the four corners were lifting at different rates. Evidence of soil heave in front of the pad does not show until the displacement is 10 mm, by which time δ has reduced to 43.9°. The rate of softening is a maximum at this point, and two thirds of the drop from peak to

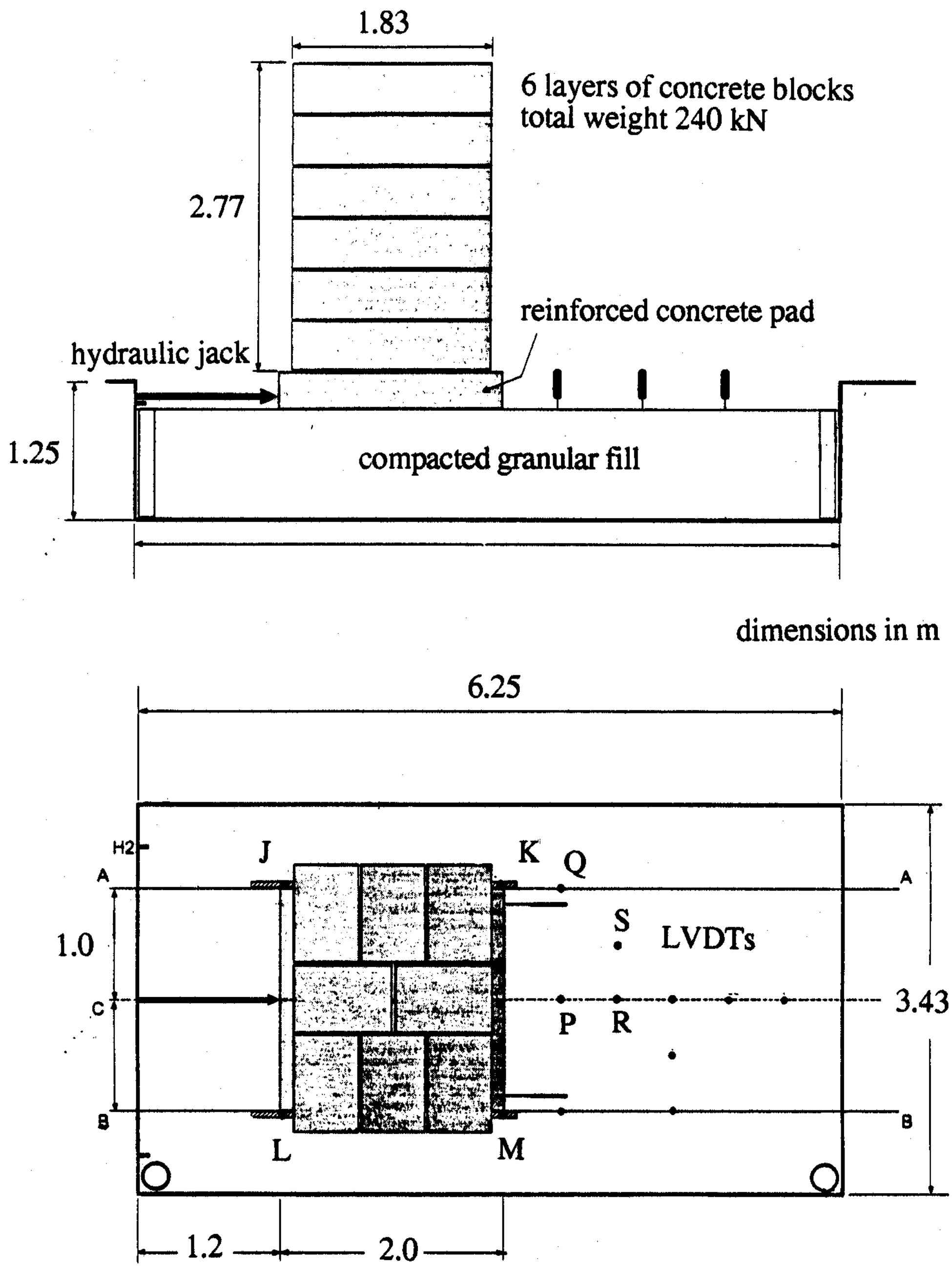

Fig. 4 Arrangement of the pilot test

residual strength has occured after 20mm of displacement. δ then drops to its ultimate value of 33.7° after a total shear displacement of only 50 mm, at which stage the mean rate of heave of the pad had reduced to approximately zero.

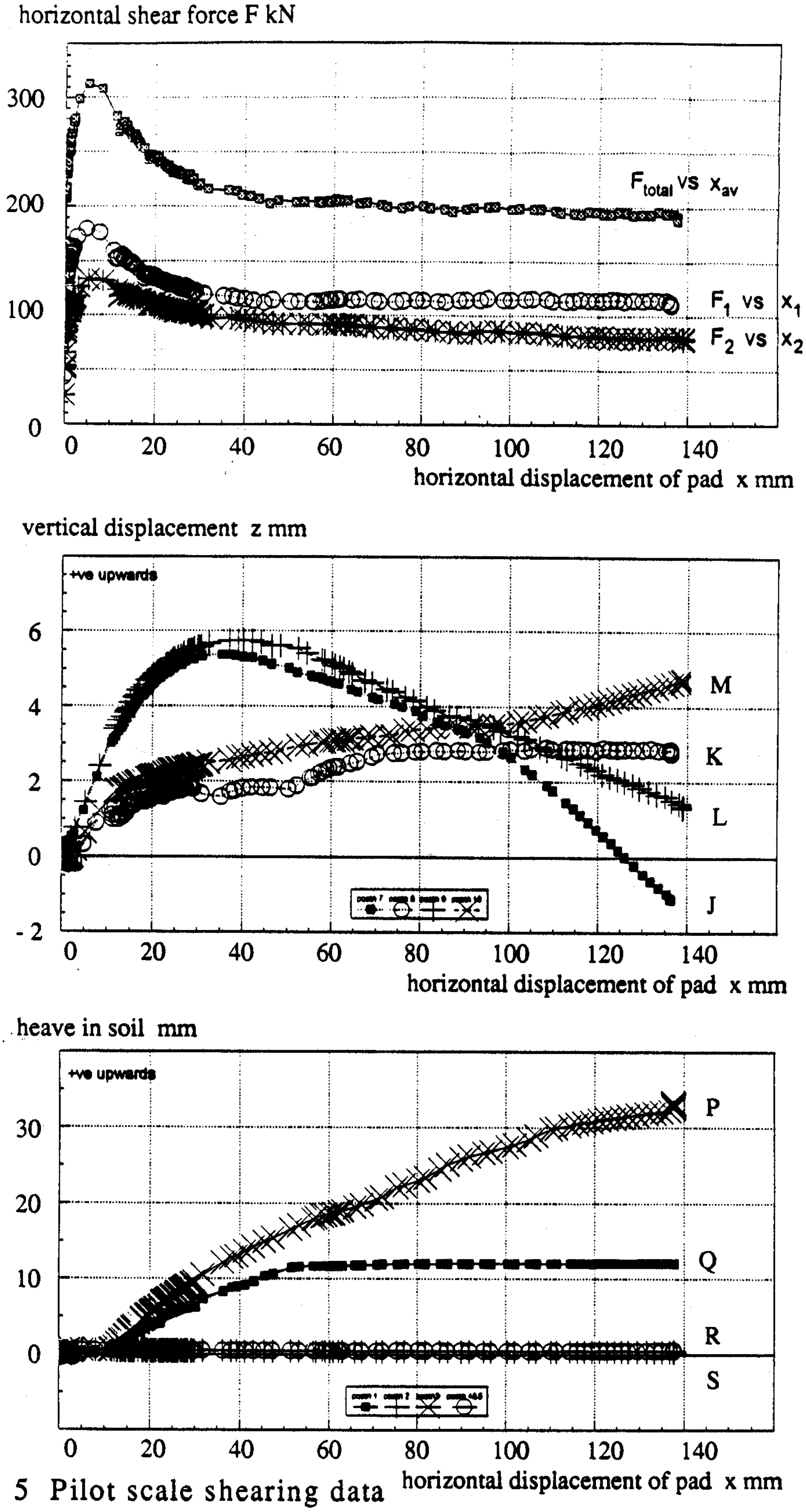

Fig. 5 Pilot scale shearing data

12. The pneumatic pressure cells were monitored throughout, but showed evidence of severe local arching effects when the shearing began. No confidence could be placed on their results.

<u>Post-shear investigation and analysis</u>

13. The location of the outcrop of the rupture surface, the magnitude of final surface heave, and the movements of the buried markers, are shown in Fig. 6. A hypothetical zone of shear, consistent with all the evidence, is shaded in. The plastic mechanism is indicative of a bearing capacity failure under a strongly inclined load, as was anticipated. It was clear that the trench was deep enough not to have influenced the results.

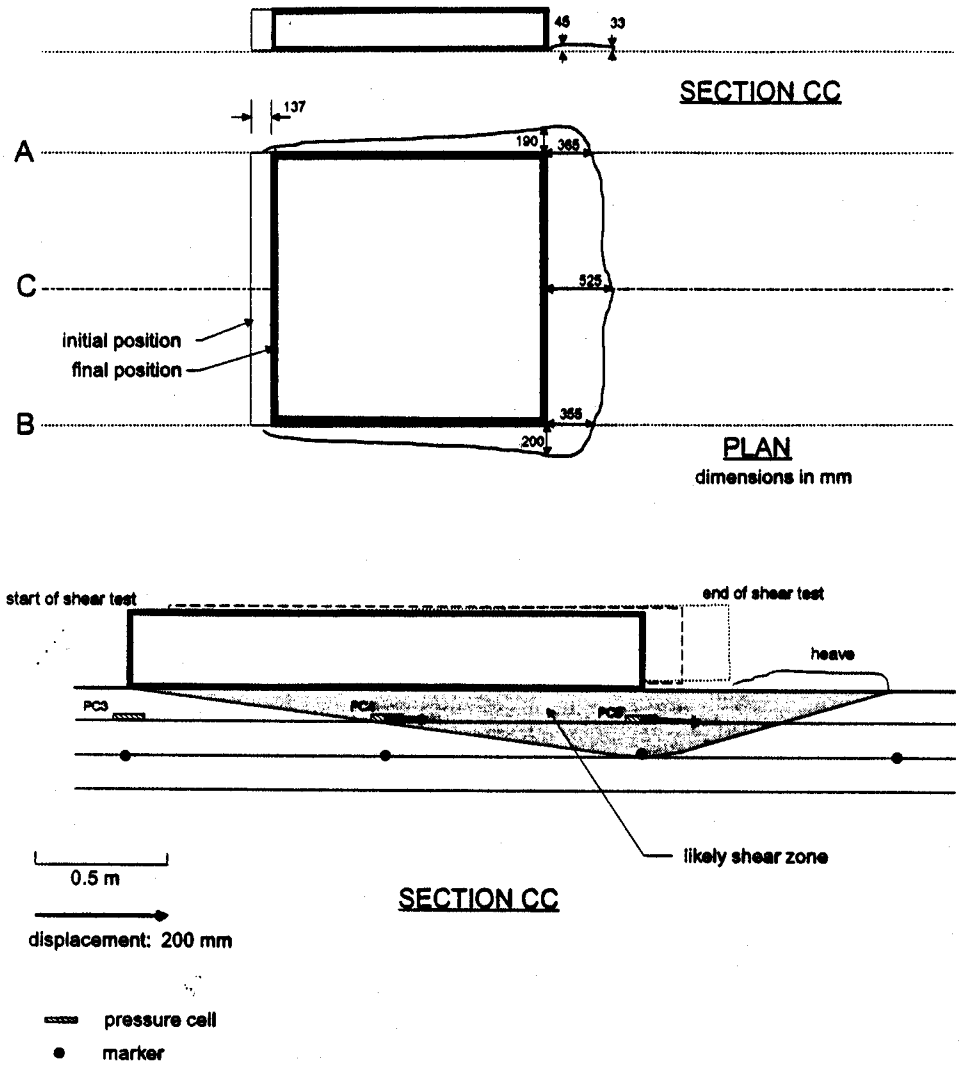

Fig. 6 Approximate shear zone found after the test

14. Sokolovskii's method of characteristics (ref.3) was used to analyse the limiting plastic equilibrium of a base under an inclined load, bearing on heavy soil (the buoyant unit weight was used) with constant angle ϕ. Fig. 7 shows two solutions for ϕ = 47.5° and 50°. The characteristic lines of maximum stress obliquity are inclined, and the angle of shearing ϕ mobilised upon them is now seen to be distinct from the angle of friction δ = 46.1° mobilised on a horizontal plane beneath the pad, at peak strength. As can be seen, the geometry of the plastic zone is very sensitive to the value of ϕ used in the analysis, and 48° was chosen as the appropriate back-analysis. Side effects have been ignored: there is no accepted way of accounting for them.

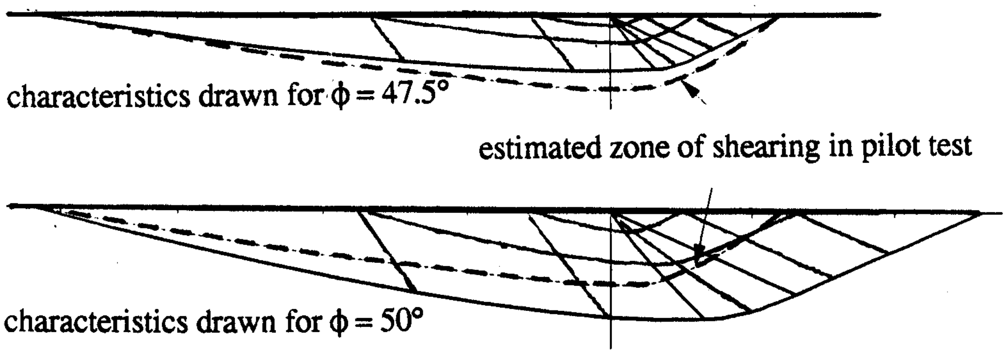

Fig. 7 Shear zones by method of characteristics

15. Sokolovskii's analysis is based on equilibrium, not kinematics, and a perfect match between failure geometries must not be expected. It must be recalled, of course, that evidence of soil heave in the passive zone came *after* peak shear force was developed. This suggests that failure was progressive, and means that no analysis assuming uniform soil properties can be expected to match the evidence exactly, even if it accounted consistently for friction and dilation. In the circumstamces, the error of assuming that the base was a plane of maximum stress obliquity is small. The back-analysis of a pad shear test such as this seems at least as secure as the conventional back-analysis of a direct shear test, which it very strongly resembles.

TRIAXIAL TESTS

16. Triaxial tests were conducted at Cambridge on saturated samples of 100 mm (4 inch) diameter, and one test was carried out at BRE on a sample of 230 mm (9 inch) diameter. The larger particles in the test fill were scalped out to achieve a diameter ratio no smaller than 6, and to test the effect on triaxial ϕ of reducing the spread of grading in this way. Fig. 1 compared the three gradings which were used in the tests with that of the fill in the pilot test on the pad.

17. Fig. 8 shows the results of the tests conducted on samples in the range of relative densities 82% to 92%, and with an effective minor stress of 47 kPa. It will be seen that the 230

mm test with particles scalped at 38 mm offers a much higher peak ϕ_{max} value, and at a very much smaller strain, than the 100 mm tests scalped at 20 mm or 12.5 mm. The increase in initial stiffness can be attributed to the method of compaction. Soil was compacted in 11 layers with a Kango hammer in the 230 mm test, whereas it was vibrated to achieve the test density in the 100 mm configuration. A similar pre-loading effect was observed when compaction was used as a check in further 100 mm tests.

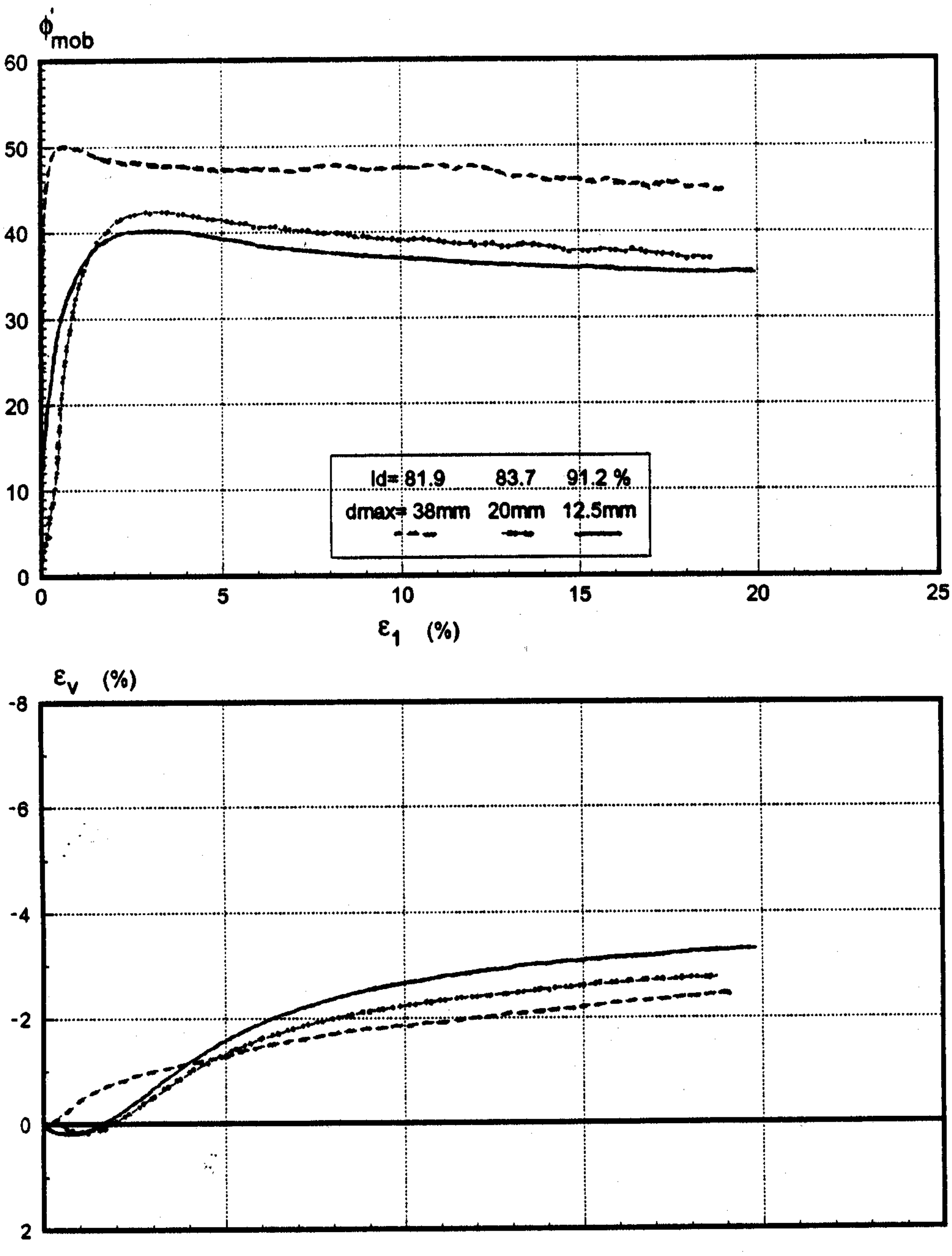

Fig. 8 Triaxial tests on pilot fill with the largest 5, 20 & 40% of the particles removed

18. Fig. 9 compares all the data of ϕ_{max} versus I_D, at a minor effective stress of 47 kPa and a normal effective stress on slip planes of about 75 kPa. The inclusion of large particles is seen to enhance the angles of shearing of all the samples, irrespective of density. It has been shown (ref. 4) that large particles can act as soil reinforcement, due to the tendency of smaller particles to flow around them. The large particles tend to develop compressive stress in the direction of principal compressive strain in the surrounding fine soil matrix, and tensile stress in the direction of principal tensile strain in the matrix. Comparing the large particle with the fine soil it replaces, therefore, it tends both to steal deviatoric stress and to enhance the effective confining pressure of the surrounding matrix. Fig. 9 tends to support the hypothesis that the reinforcement effect of large particles is equally effective at a critical state as at peak strength, though the evidence here could not be regarded as conclusive.

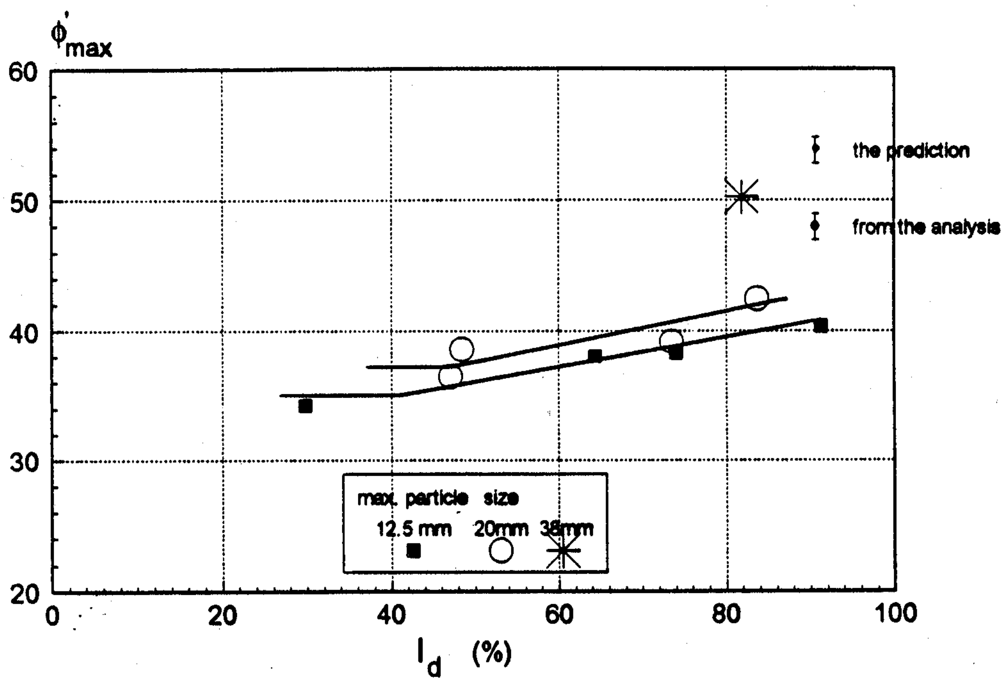

Fig. 9 ϕ versus relative density: analysis of pilot shear test compared with triaxial tests

CONCLUSIONS

19. An angle of shearing resistance can be measured on a test fill by casting a concrete pad on a prepared surface, loading the pad with weights, and jacking it horizontally. The behaviour is brittle, showing evidence of some progressive failure. Peak strength of the test fill reported here was developed at 5 mm displacement, and full softening had occured after 50 mm.

20. The pad shear test can be regarded as an analogue to the laboratory direct shear test and is directly relevant to the

maximum and ultimate sliding resistance of in situ concrete bases. No great error is made by interpreting the point of peak strength by assuming horizontal shear within the soil. However, a thorough investigation revealed that the mechanism conforms to that of a bearing failure with a severely inclined load. Taking into account the combination of vertical bearing and horizontal shearing, the surface mobilising greatest angle of shearing was seen to be slightly dished. This results in an estimate of ϕ_{max} of 48°. A slightly larger error might be involved in assuming that the ultimate mobilised angle of friction δ on the base was a good estimate of ϕ_{crit} since δ fell to 33.7° which is at least 4° smaller than the lowest likely estimate of ϕ_{crit} for the pilot fill.

21. Triaxial tests gave conservative strengths in the well-graded fill when large particles were scalped prior to testing. The largest test configuration, and the minimum scalping of the test fill, gave a triaxial ϕ_{max} of 50° in a single test. After using equations (1) to (5) to correct for density differences and plane strain, the peak angle of shearing resistance in the pad shear test should apparently have been 58° in the absence of progressive failure. The pilot shear back-analysis agrees more closely with the uncorrected triaxial value. Weakness on bedding planes in the pilot test fill can not be discounted.

22. It seems likely that some progressive failure did occur in the pilot test. Certainly, the shear strength softened very rapidly post-peak. If a value of ϕ had to be chosen for design, to be used without partial factors covering uncertainty, the critical state value must be used. If this were obtained from the ultimate strength of loose samples of scalped fill, the reinforcing effect of large particles would be lost and a conservative value of ϕ_{crit} would be registered. If designers chose to mobilise ϕ_{crit}, the strains in properly compacted granular fill would be small, less than 0.2% according to the triaxial tests.

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