ON THE PROPERTIES OF A SANDY GRAVEL

CONCERNANT LES CARACTERISTIQUES D'UN GRAVIER SABLEUX

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SYNOPSIS

The effective stress strength and stiffness properties of a sandy gravel, Thames Terrace Gravel, obtained from consolidated drained large shear box tests and stress path tests conducted in the triaxial apparatus are presented. The peak angles of friction (ϕ') measured in the shear box show a far greater dependence on the specimen density than observed in triaxial stress path tests, for which peak and ultimate angles coincide. The ultimate (critical state) angles in the shear box were, however, closely comparable to those measured in triaxial tests. The stiffness of the material measured using local strain transducers show a degree of non-linearity appreciably higher than that shown by typical clays. At the same strain, stress level and density, the shear stiffness of specimens measured in conventional drained and undrained tests are comparable. Stiffness values are observed to depend significantly on the previous direction of loading and on the sample density.

INTRODUCTION

Many building and civil engineering projects in central London involve underground excavations through Made Ground and Thames Terrace Gravel into underlying London Clay. The London Clay has been investigated extensively and its engineering behaviour is reasonably well understood. By comparison, the Thames Terrace Gravel, a sandy gravel, has received little attention and its engineering design parameters are usually based on empirical correlations with in situ test results in finer grained cohesionless material. The large shear box and stress path tests described in this Paper were carried out to obtain a better understanding of the strength and stiffness properties of this material.

ORIGIN AND CLASSIFICATION

Origin

The Thames Terrace Gravel was deposited during the Pleistocene epoch (about 2 million years ago) as a series of gravel sheets that formed as a result of heavy seasonal snow-melt run off. This run off formed a series of braided channels that meandered and interlinked across a wide flood plain. The material deposited is a well graded mixture of sand and gravel, occasionally with cobbles (where stream flow was intense) and with silt or silty sand layers (formed by slacker water). The positions of the water courses producing the Thames Terrace Gravel varied with time; these coupled with erosional downcutting of the River Thames led to the formation of a series of terraces. The material under consideration in this Paper was taken from Terrace 2 (Taplow Terrace).

A typical gravel deposit is light brown or light reddish brown fine to coarse sand and fine to coarse angular to subrounded gravel. The gravel content predominantly comprises flint pebbles, while quartz grains predominate in the finer sands. The deposit is generally between 5 and 10 m thick but can be up to 30 m thick where scour hollows completely penetrate the underlying London Clay. Within central London the surface of the Thames Terrace Gravel is generally 2 to 5 m below ground level.

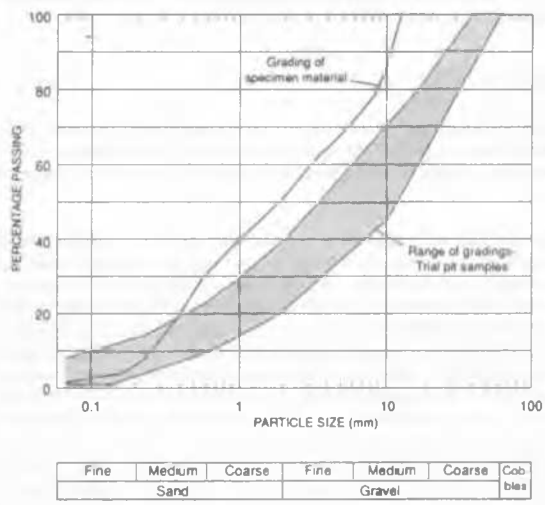

Figure 1 Grading curves

Classification

Bulk samples, obtained from trial pits, were graded using the BS1377 (1990) wet sieve test method, giving the range of results shown on Figure 1. The mean particle size, D_{50}, ranges from 3 to 10 mm. The uniformity coefficient, defined as D_{60}/D_{10} is generally in excess of 20 i.e. the material is well graded.

Based on correlations proposed by Youd (1973) between the void ratio limits and the degree of angularity of a cohesionless material, it is expected that the maximum void ratio (e_{max}) lies between 0.55 and 0.65 and the minimum void ratio (e_{min}) falls between 0.25 and 0.35.

Consistency

Standard Penetration Tests (SPTs) were carried out using the BS1377 (1990) test method with a 60° solid cone replacing the conventional split barrel sampler. N values (blows/300 mm) showed a wide range but generally lay between 10 and 40, indicating the material to be medium dense to dense at depths between 3 and 15 m below ground level (Thorburn, 1963).

Skempton (1986) concluded that for natural fine to medium dense sands $(N_1)_{60}/D_r^2 \approx 55$, where $(N_1)_{60}$ is the SPT blowcount corrected for depth and energy effects and D_r is the relative density. No equivalent correlation between $(N_1)_{60}$ and D_r has been proposed for gravel, but a tentative extrapolation based on the data presented by Skempton, suggests $(N_1)_{60}/D_r^2$ should lie between 60 and 80. This implies the mean in situ relative density of the Thames Terrace Gravel is about 60 ± 15%.

EFFECTIVE STRESS STRENGTH

The effective stress strength properties of the Terrace Gravel were determined from consolidated drained large shear box tests and stress path tests conducted in the triaxial apparatus. To reduce problems associated with scale and membrane penetration effects, all particles larger than 14 mm were first removed from the bulk samples retrieved in trial pits. The specimens for testing were then taken from a thoroughly mixed batch of all bulk samples.

The grading curve for the specimens tested is shown on Figure 1 and is seen to give a mean particle size (D_{50}) of 2 mm, somewhat lower than the mean D_{50} value of 5 mm for the in situ material. The maximum and minimum void ratios of the specimen material were 0.65 and 0.22 respectively. These were determined using the BS 1377 (1990) specified methods which in the case of a minimum void ratio determination involved placing the material in a 152 mm diameter CBR mould using a vibrating hammer, and for the maximum void ratio involved pluviation of the material.

Shear Box Tests

The effective stress strength properties of the Terrace Gravel in direct shear was assessed using a large (305 mm square) shear box. The compactive effort was varied while placing the samples in the apparatus to achieve a range of densities.

Twenty-two samples were sheared at a typical (drained) rate of 0.1 mm/min to a total relative displacement of 8 mm so that the peak strength could be measured, while five were sheared to a total relative displacement of 20 mm enabling both the peak and constant volume (critical state) angles to be determined (defined in this Paper as $\phi' = \tan^{-1}[\tau/\sigma'_n]$). It was not possible to discern useful data (e.g. angles of dilation) from the measured volumetric strains. The mean of the five tests indicated a constant volume friction angle (ϕ'_{cv}) of 38 ± 2°. The loosest samples (at an initial void ratio of 0.58) also mobilised peak and ultimate friction angles within this range. These data are consistent with ϕ'_{cv} values of between 36° and 41° recommended by Youd (1973) and Stroud (1988) for *well graded* quartz sands and gravels; both workers found the higher angles applied for materials with more angular particles.

The dependence of the measured peak friction angles (ϕ'_p) on the initial void ratio (e_i) is illustrated on Figure 2 which shows that ϕ'_p for dense samples ($e_i \approx 0.3$) is about 12° higher than ϕ'_{cv}, while the loosest samples ($e_i \approx 0.58$) give ϕ'_p angles that are comparable to ϕ'_{cv}. As shown on this figure, the trend and magnitudes of these data are closely comparable to results from large shear box tests on Thames Terrace Gravel from Heathrow, West London, reported by Bishop (1948) carried out at normal stresses of 200 ± 100 kPa.

The apparent scatter of the data on Figure 2 is reduced when the decrease in ϕ'_p with increase in normal effective stress (σ'_n) is recognised (because of

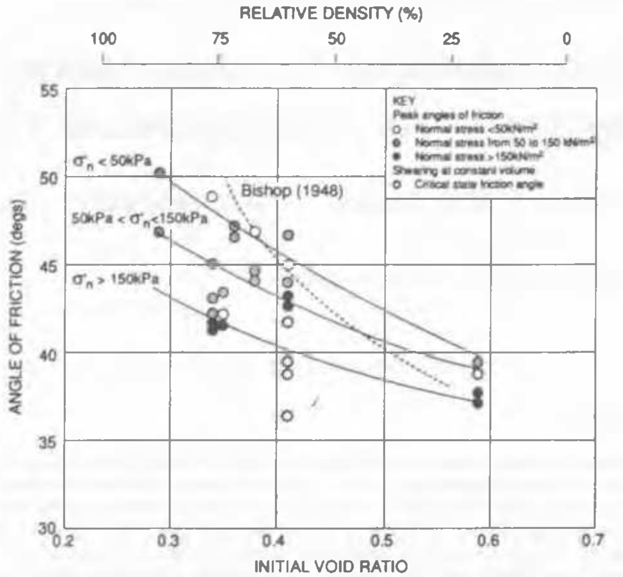

Figure 2 Angles of friction in shear box tests

reduced dilatancy). To highlight this effect, the data points plotted distinguish between tests carried out at normal stresses less than 50 kPa, between 50 and 150 kPa and above 150 kPa.

These trends agree with the published data on granular materials by Bolton (1986) and others.

Triaxial Tests

6 No. 100mm diameter specimens were prepared for triaxial tests by gently placing material into de-aired water using a split former. This procedure gave relative densities (D_r) of about 50%. Slight tamping and vibration were applied to achieve higher D_r values of 65% and 75%.

'Hall effect' strain gauges (Clayton and Khatrush, 1986) were mounted locally at the middle third of the specimens to allow accurate resolution of both the axial and radial strains (using two and three gauges respectively). It is anticipated that strains could be resolved to an accuracy of about 0.005% using these gauges. Specimens were also equipped with base and mid-height pore pressure probes.

After initial isotropic consolidation to a mean effective stress of 50 kPa, specimens were consolidated and swelled anisotropically along the paths A - B -C or A_1 -B_1 -C_1 shown on Figure 3. These paths attempted to simulate the depositional history of the in situ material and were such that, at points C/C_1, the ratio of axial effective stress (σ'_a) to horizontal effective stress (σ'_r) was ≈0.6 and the overconsolidation ratio was 1.4 ±0.1.

Three different types of stress paths were imposed on specimens after attaining points C/C_1. In tests 1 and 2, σ'_a was held constant while σ'_r was reduced; this stress path replicated that experienced by soil behind a retaining wall. Tests 3 and 4 were conventional drained triaxial tests in which σ'_r was constant and σ'_a was increased, while tests 5 and 6 were conventional undrained (constant volume) triaxial tests.

The stress paths followed in all tests are shown on Figure 3. These indicate:

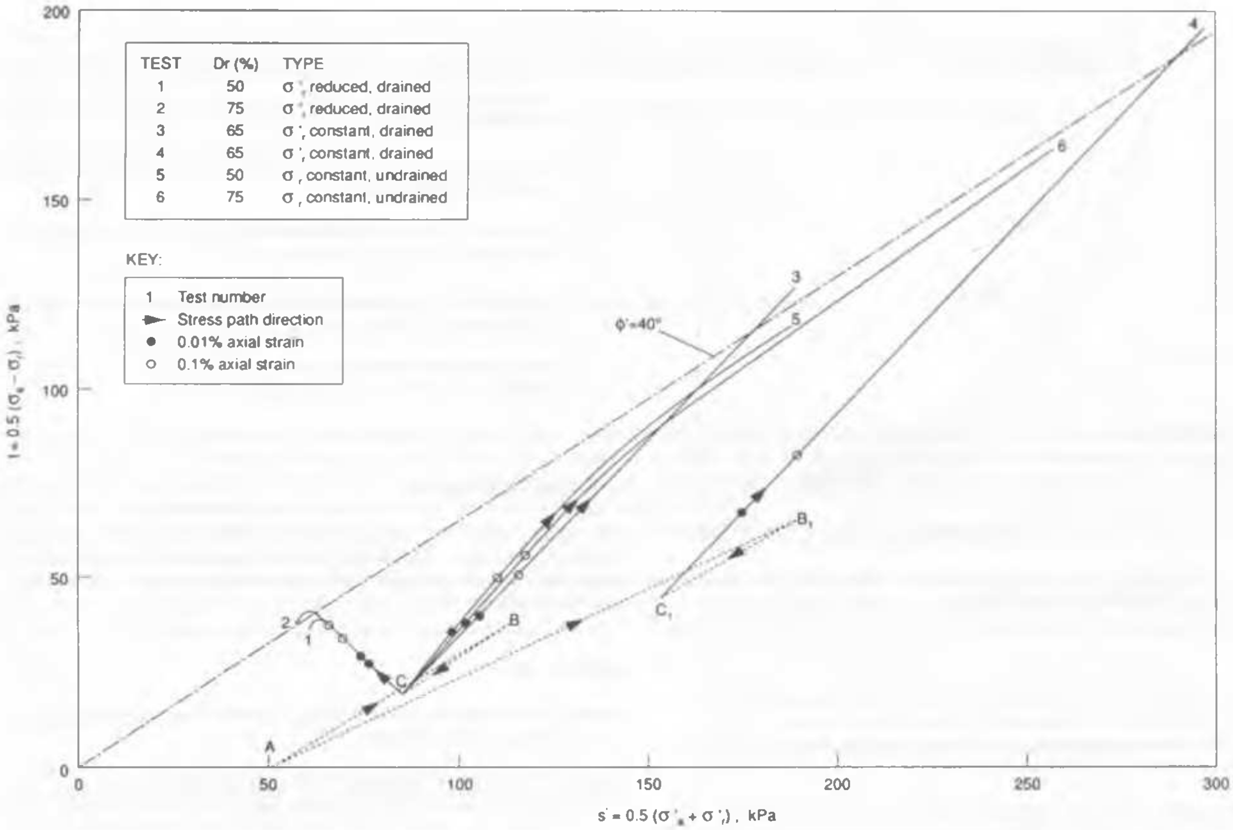

Figure 3 Triaxial stress paths

i) For the range of relative densities investigated (50 to 75%), all specimens reach peak deviator stress at a friction angle (ϕ') close to 40°; the drained stress paths (tests 1 to 4) indicate an increase in ϕ' from ≈39° at D_r = 50% to 42° at D_r = 75%, while ϕ' was ≈39° in both undrained tests. There was no discernible dependence of ϕ' in stress level.

ii) The specimens that followed stress paths increasing to the right (tests 3 to 6) failed in a ductile manner at axial strains of between 5 and 9%. Dilation (or reductions in pore pressure in undrained tests) generally commenced when the mobilised friction angle reached about 35°.

iii) A brittle failure occurred in tests 1 and 2 (which followed stress paths to the left); these failed at (smaller) axial strains of 2.0 and 0.7% respectively. As with the other tests, the onset of dilation coincided with a mobilised friction angle of 35°.

Figure 3 also plots the stress points at which local axial strains were 0.01% and 0.1% (assuming zero strain at C/C$_1$). The lower strain at failure seen in tests 1 and 2 is consistent with the fact that the strain contours are broadly at a similar radial distance from the initial stress point (C/C$_1$).

STIFFNESS

The variation of the secant shear stiffness (G) with the shear strain invariant, ε_s [$= 2/3\,(\varepsilon_1 - \varepsilon_3)$], measured during the triaxial stress paths from points C/C$_1$ are shown on Figure 4. These G values are normalised with respect to the initial mean effective stress, p'_0, (i.e. at points C/C$_1$ and where $p'_0 = 1/3[\sigma'_1 + 2\sigma'_3]$) and it has been assumed that changes in p' do not induce (significant) shear strains.

It is apparent that G is strongly dependent on strain level: G values at ε_s =0.01% are typically about five times those at ε_s =0.1%. This degree of non-linearity is significantly higher than that commonly observed in clays for which a corresponding ratio of stiffnesses at these strain levels is 3 ±0.5 (Jardine 1985).

Table 1 Shear Stiffness at ε_s =0.01%

Test	D_r	G(MPa)	p'_0(kPa)	G/p'_0	p'(kPa)	G/p'
1	50	39	75	520	66	590
2	75	49	75	650	63	775
3	65	110	75	1480	92	1205
4	65	175	138	1270	158	1100
5	50	105	75	1415	86	1235
6	75	135	75	1820	89	1535

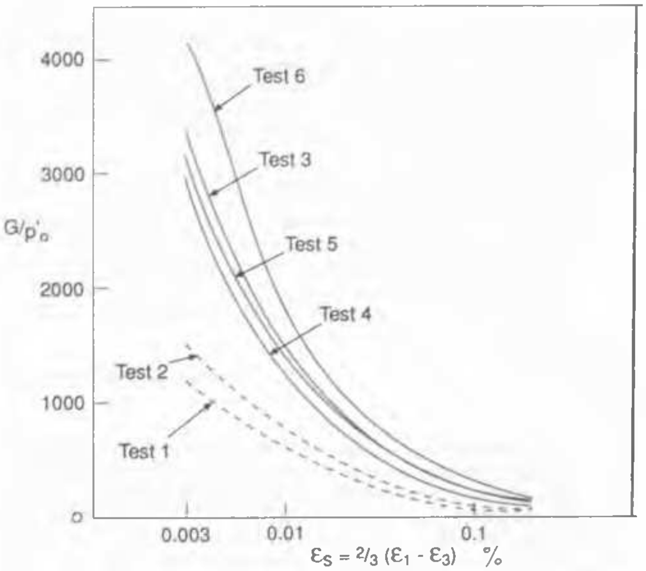

Figure 4 Normalised shear stiffness

The values of G at ε_s = 0.01% normalised by the p' values applying at that strain are listed in Table 1 in conjunction with the G/p'_0 ratio at ε_s =0.01%. These data add further insight into the stiffness characteristics of the gravel and allow the following observations to be made:

i) At the same strain, stress level and density, the stiffness of specimens tested in conventional drained and undrained triaxial tests are comparable.

ii) The stiffness depends significantly on the previous direction of loading: tests 1 and 2 which loaded the specimens at an angle of about 90° to the anisotropic swelling path (B-C; see Figure 3) gave normalised stiffness values which were only ≈50% of those of the other specimens (which were reloaded approximately in the reverse direction to path B -C). This dependence of stiffness on the previous loading direction has been observed frequently in many soils; e.g. see Arthur et al (1977).

iii) An increase in relative density from 50% to 75% gave an increase in stiffness of about 30%; a similar dependence on D_r has been measured by Seed and Idriss (1970) and others.

iv) Normalised stiffness values reduce moderately with stress level; the (limited) data in these tests suggest that G values at shear strains less than 0.1% vary in proportion to $(p')^{0.8}$. Further tests are required to investigate more fully the specific dependence of the shear stiffness of Thames Terrace Gravel on the stress level.

v) The apparent sharp increase in G with ε_s as ε_s reduces to below 0.005% was unexpected and may be because stress and strain measurements were at the limit of their resolution.

Bulk moduli (given as $\Delta p'/\Delta\varepsilon_v$, where ε_v is the volumetric strain), at relatively low strains, displayed a similar dependence on ε_v to that observed between G and ε_s.

CONCLUSIONS

The stiffness and strength measurements made on a sandy gravel revealed:

1. The test results were broadly in agreement with published correlations and data reported for finer grained well graded cohesionless materials.

2. The ultimate friction angle of the gravel in both shear box and triaxial tests was about 40°

3. The peak angles of friction in shear box tests showed a far greater dependence on the initial specimen density.

4. The stiffness of the gravel displayed a higher degree of non-linearity to that shown by typical clays.

5. Shear stiffnesses depended primarily on the previous direction of loading and on the specimen density.

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REFERENCES

Arthur, J.R.F.; Chua, K.S. and Dunstan, T. (1977) Induced anisotropy in a sand. Géotechnique, England, 27 (1), pp.13-30.

Bishop, A.W. (1948) A large shear box for testing sands and gravels. Proc. 2nd Int. Conf. Soil Mech. Found. Engg. Rotterdam, Volume 1, pp.207-211.

Bolton, M.D. (1986) The strength and dilatancy of sands. Géotechnique , England, 36 (1), pp.65-78.

British Standard BS1377 (1990) British Standard methods of test for soils for civil engineering purposes, British Standards Institution.

British Standard BS5930 (1981) Code of Practice for Site Investigations, British Standards Institution.

Clayton, C.R.I and Khatrush, S.A. (1986) A new device for measuring local axial strains on triaxial specimens. Geotechnique, England, 36 (4), pp. 593 - 597.

Jardine, R.J. (1985) Investigations of pile-soil behaviour with special reference to the foundations of offshore structures. PhD Thesis, University of London (Imperial College).

Seed, M.B. and Idriss, I.M. (1970) Soil moduli and damping factors for dynamic response analysis. EERC Report No. 70-10, Univ. of California, Berkeley.

Skempton, A.W. (1986) Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, aging and overconsolidation, Géotechnique, England, 36 (2), p.425-447.

Stroud, M.A. (1988) The standard penetration test - its application and the engineering properties of glacial materials. Proc. Conf. on Penetration Testing in the UK. Publ. Thomas Telford.

Thorburn, S. (1963) Tentative correction chart for the standard penetration test in non-cohesive soil. Civil Engg. Public Works Review, Vol. 58, pp.752-753.

Youd, T.D. (1973) Factors controlling maximum and minimum densities of sands, Proc. Symp. Eval. Dens., ASTM STP523, pp.98-112.