Discussion

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The strain levels induced in soils in common geotechnical engineering practice cover a wide range, say $10^{-6} \sim 10^{-1}$. The non-linearity of the stress-strain relationship is indispensable when characterizing the stiffness of a soil. Results of triaxial tests using a specimen of 30cm in diameter and 60cm high were discussed herein with particular attention paid to the behaviour at very small strain levels. Two types of soils, Hime gravel(HG) and a cement-treated sandy soil (CSS) were used. HG is a natural gravel with D_{50} equal to 1.85mm and U_c equal to 1.33. The CSS was prepared by curing a mixture, in each cubic meter volume, of a cement(80kgf), a natural sand (1177kgf), a clay(110kgf) and water(520kgf). The strength and deformation properties of this material resemble those of natural soft rocks.

The specimens were first isotropically consolidated to an effective confining pressure, σ_c', then sheared, under a constant cell pressure, by applying the axial load in two manners;the deviatoric stress(q) was monotonically increased to failure(the monotonic tests) and q was cycled by an equal amount in compression and in extension (the cyclic tests). The axial displacements were measured using a local displacement transducer (LDT)(Tatsuoka,1988) mounted vertically on the lateral surface of the specimen so that the bedding error could be excluded from the measurements. By using this device,it was possible to measure the range of axial strain of from 10^{-6} to 10^{-2} for the size of the specimen used.

In the monotonic tests, cyclic loadings using small amplitudes of q were applied(Fig.1). From these cyclic loadings, a deformation modulus, E_{eq}' or G_{eq}', was obtained in relation to the stress level(q')(see a sketch in Fig.1(a)). In Figs.2 and 3, the secant moduli (E_{sec} or G_{sec}) obtained from the monotonic tests were compared with the stiffness observed in the cyclic tests (E_{eq} or G_{eq})(see insets in Fig.2). For the undrained tests(Fig.3), the shear moduli were calculated by assuming a poisson's ratio of 0.5.

The followings are to be noted:

(1) The stiffness measured using a conventional method(cf termed "external" in Fig.1(b)) gave much smaller values than those determined from LDT(cf termed "local"). The divergence was more significant when smaller specimens with heights of 10cm or 15cm were used.

(2) For the monotonic loading tests, the stress strain relationship was linear up to 2×10^{-5} for HG(Fig.2) and up to 9×10^{-5} for CSS(Fig.3) (see also insets in Fig.1).

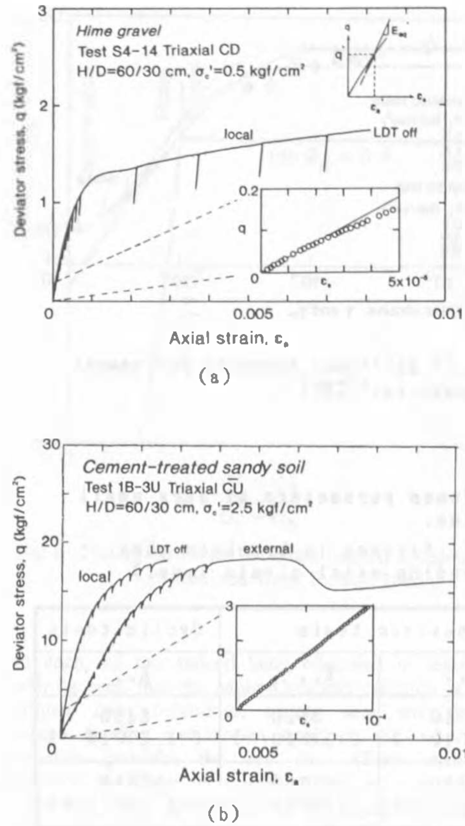

Fig.1 stress strain relationship observed.

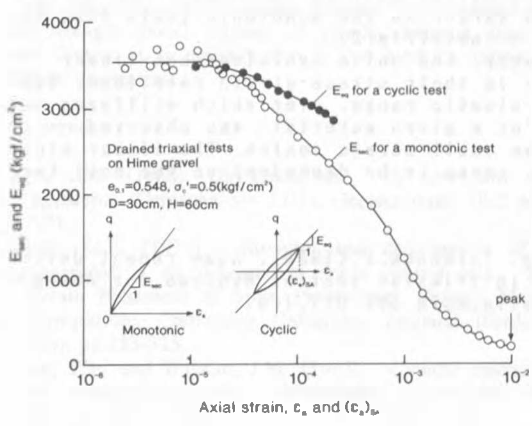

Fig.2 Variation of stiffness observed for Hime gravel(HG).

(3) In the linear region, the stiffness parameters at very small strains obtained both from monotonic and cyclic tests showed little difference(see Table 1) and this gave the maximum stiffness for the materials tested.

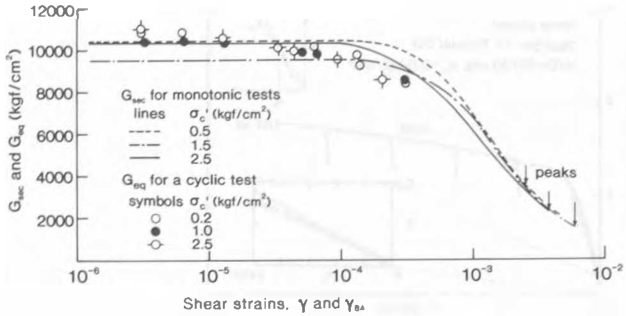

Fig.3 Variation of stiffness observed for cement treated sandy soil(CSS).

Table 1 Stiffness parameters at very small strains.

(unit:kgf/cm^2; figures in brackets show the corresponding axial strain level)

	Monotonic tests		Cyclic tests
soil	E_{sec}	$E_{eq}{}^1$	E_{eq}
HG	3540 (<2×10^{-5})	3520 (<2×10^{-5})	3480 (<1.2×10^{-5})
CSS	31110 (<5×10^{-5})	32760 (<7×10^{-5})	32580 (<2.1×10^{-6})

(4) Based on other tests on various cohesionless soils, the strain dependency of secant modulus was found larger in the monotonic tests than in the cyclic tests(Fig.2).

In summary, the soils exhibited non-linear behaviour in their stress-strain relations. But a linear-elastic range, over which stiffness was maximum for a given material, was observed within the small strain region. The linear elastic limit seems to be dependent on the soil type.

Reference: Tatsuoka,F.(1988). Some recent developments in triaxial testing systems for cohesionless soils,ASTM STP 977,7-67.

Discussion

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In the list of strength models normally called up to apply to granular materials, that of Parkin (1965) remains almost unknown, despite being the only criterion in existence based solely on particle mechanics, as opposed to various curve-fitting procedures. Elsewhere in this Conference, for example, Ramamurthy and Tokhi (Session 8) state that there were no theoretical relationships linking triaxial and plane strain strengths before 1980.

Of the models that Puccini et al. offer, one is quoted as having the advantage of "offering no sharp edges in stress space", as if this is a necessary or desirable condition. This idea is clearly shared by various other writers, such as Lomize and Kryzhanovsky (1967), despite the fact that there are some good theoretical reasons why sharp discontinuities might well occur, as indeed they do in the original Coulomb envelope. Of all the theories quoted, none identifies or makes reference to the associated mode of failure, and in consequence none hints at the condition of instability arising at the condition of triaxial extension, as reported by Proctor and Barden (1969) and Green (1971).

This whole subject is greatly complicated by the fact that it is virtually impossible to conduct reliable tests in the region between plane strain and triaxial extension (as stated by Bell, 1968), despite all the ingenuity and care that has been applied to the matter. This is clearly evident from the divergence between the many published results and in the often anomalous results (such as the strong dependence of ϕ' on b for loose sand, Lade and Duncan, 1973). In the Authors' case, wherein the tests relate to a new type of cube apparatus with six rigid orthogonal platens, the results are unlikely to provide any improvement on preceding ones because of the degree of interference with any potential failure mode. It is only because of these acute experimental problems that the Mohr-Coulomb theory is discarded prematurely. Significantly, however, extension test results are mostly in good agreement with plane strain results, with a fairly well defined plateau between (e.g. Proctor and Barden, 1969; Green, 1971; Lade and Duncan, 1973), which might also include the Authors' results, except for a strange occurrence at $\beta = 60°$.

The essence of the above-mentioned solution from particle mechanics (Parkin, 1965) is that it predicts failure characterised by three quite distinct failure mechanisms, operating over different stress regimes (Fig.1). Plane strain is associated with one of these transitions, for which

$$\sigma_2 = (\sigma_1 + \sigma_3)/3$$

an equation which is in as good agreement with test data as any of the empirical alternatives (as used by Ramamurthy and Tokhi). Instability occurs at the condition of extension, when two failure mechanisms become equally critical (viz. plane sliding and necking), with different strengths applying

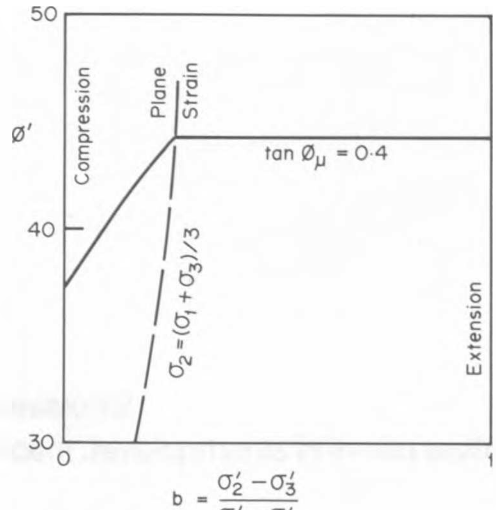

Fig.1 Failure envelope for close-packed spheres (derived from Parkin, 1965)

to each, as has indeed been observed in tests (above). It will be seen that the Mohr-Coulomb criterion is the dominant control over behaviour, except near triaxial compression where another mechanism operates and where the Tresca criterion provides the best fit. (Some criticism of this solution by Thornton (1981) relates to a second order error affecting the accuracy of the solution close to the compression axis and for large $\phi\mu$. This, however, does not bear on any of the essential principles above.)

In brief, this topic revolves around the failure mechanism. If it cannot be specified for the solution put forward, or if it cannot be seen in the apparatus used, or if it is interfered with, then the work becomes dubious. Conversely, unless the strength model consists of simple elements, like Mohr and Tresca, it is unlikely that it can ever be related to a failure mechanism.

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