Stress-Compressibility Characteristics of a Residual Soil from Gneiss

Relations Contrainte-Compressibilité d'un Sol Résiduel de Gneiss

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SYNOPSIS

The Paper describes field and laboratory investigations on strength-deformation characteristics of a residual soil from gneiss. The modulus of deformation in field was determined from 30cm diameter plate loading tests in horizontal and vertical directions every metre in a 9m deep pit. 30cm diameter screw plate loading tests were also executed. The behaviour in laboratory was determined from simple compression tests moulded in different directions as well as from oedometer tests in which the influence of the variation of pressure increment ratio ($\Delta p/p$=0,25, 1 and 3), and the duration of each pressure increment (1 hour and 24 hours) was examined. The results are analysed in the light of Janbu theory of compressibility. An evaluation of the coefficient of lateral pressure at rest, K_o, was made using electric strain gauges installed on a stiff oedometer ring.

INTRODUCTION

Observations of settlements of foundations in residual soil derived from gneiss show that the field values are significantly less than those derived on the basis of classical consolidation theory with conventional oedometer test parameters. The reasons for this discrepancy are many, and not yet quantitatively understood. A typically young residual soil, which preserves the rock structure (saprolite), shows a large variation in the degree of weathering and consequent soil types even in very closely spaced samples. It therefore becomes important to obtain relevant parameters from field tests or from laboratory tests on large sized samples. The other main problem relates to the partly saturated state of these soils which renders the application of the Terzaghi theory questionable. The results of field and laboratory investigations presented in this Paper have been analysed with special reference to Janbu theory of compressibility (Janbu 1963, 1967, 1969), since his concept of tangent modulus of deformation, M, simply represents the resistance of volumetric change caused by a change of stress and is not restricted to saturated soils.

i.e. $M = d\sigma/\sigma\varepsilon$

where σ and ε denote stress and strain in the same direction.

Janbu (1967) proposed the following formulation: (1-a)

$$M = m\sigma'_a(\sigma'/\sigma'_a) \quad ; \sigma'_a = 1 \text{ atm};$$

where σ', m and a denote vertical stress, the modulus number, and stress exponent respectively.

Analagous to the concept of the resistance to volumetric change, defined by M, Janbu (1969) introduced the concept of resistance with respect to time; i.e. $R = dt/d\varepsilon$. Hence the time resistance, R, is dependant on the state of confinement, the stress level and the duration of load application. Reference should be made to Janbu (loc. cit.) for detailed discussion of these parameters.

THE SOIL

The samples of residual soil were obtained from a hand-dug pit, 1.5m x 1.5 x 8m deep, in which field plate loading tests were also carried out. The parent rock is a plagioclase-quartzitic-biotitic-gneiss of the Pre-Cambrian Series. The saprolite, reddish brown in colour with yellow and grey mottles showed presence of decomposed feldspar, biotite and garnet, with a significant presence of concretions and a sill of quartz, (at 7m depth).

Microscopic and X-ray analysis revealed the following composition by volume: fragments of quartz and mica linked with iron oxides: 50%; quartz: 35%; garnet: 5%; caolinitic fragments covered with iron oxide: 10%. Table I lists the physical characteristics of the soil.

Table I: Physical Characteristics

Depth. (m)	clay %<2μ	silt %	sand %	LL %	PL %	e %	S %
1	14	31	55	47	34	0.96	71
2	16	34	50	44	32	1.19	62
3	12	38	50	50	40	1.03	66
4	13	47	40	53	40	1.14	68
5	10	37	53	48	41	1.06	67
6	7	54	39	65	NP	1.04	64
7	4	16	80	45	NP	0.80	61
8	12	50	38	65	NP	1.22	73

FIELD INVESTIGATIONS

The field investigations consisted of the following:

a) 30cm diameter plate loading tests in horizontal and vertical directions, at every metre depth of a 8m deep hand-dug pit.

b) 30cm diameter screw-plate loading tests.

A 10 ton hydraulic jack was used for loading, up to 8-10 kg/5 q.cm, with a spherical seating provided at the loading plate to avoid eccentricity. The irregularities on the loading soil surface were eliminated with the use of thin layer of chalk plaster. The reaction for the horizontal test was provided by a 30cm dia. plate resting against the opposite wall of the pit. A heavy-duty helicoidal spring was incorporated between the two plates to assure uniform pressure (Rocha, 1975).

The screw plate tests were also carried out every metre depth. The test plate was screwed two revolutions in a 30 cm diam. hand auger bore-hole.

LABORATORY INVESTIGATIONS

All laboratory investigations were carried out on block samples extracted from the pit, and consisted of the following:

a) Unconfined compression tests on samples moulded with axis at 0º, 30º, 45º, 60º and 90º to the vertical direction. To avoid end-friction and facilitate moulding, wide lubricated end platten technique was adopted which permitted testing of 2.5" dia. x 2.5" high samples. A shearing rate of 0,3%/min. was used.

b) The oedometer tests were carried out in rings of three sizes: 10,8cm Ø x 2cm; 10,8cm x 3cm and 6,4cm Ø x 1,96cm. Pressure increment ratios of 0,25, 1,0 and 3,0 were used with duration of each application of load of 1 hour and 24 hours. Silicon grease was used to minimise side friction. Both submerged and unsubmerged samples were tested in machines working on compressed air, in which the load increment could be applied instantly without shock effect.

c) An attempt to estimate the values of K_o was made by testing unsubmerged samples from 2, 4, 6 and 8m depths in a stiff oedometer ring (10.44 cm dia., 3 cm high and 3mm thick) instrumented with resistance strain gauges and coupled to a temperature compensating ring, in an arrangement similar to that used for stiff London Clay by Som (1968), (Fig. 1). Pressure increment ratios, $\Delta p/p$ = 0,25, 1,0 and 3,0 were used. Each loading was maintained for one hour since earlier results had shown this duration to be satisfactory.

RESULTS

Only an extremely brief presentation of results can be made here. Fig. 2 presents the modulus of elasticity from the vertical and horizontal plate loading tests and the screw plate tests, calculated from elastic theory at a pressure of 3 kg/cm^2, and adopting μ = 0,3. In the first 5m depth the elasticity with respect to vertical deformation is 1.3 to 5.3 times larger than the value with respect to horizontal deformations. A similar

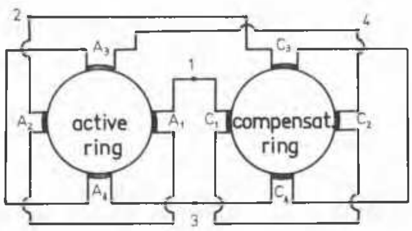

Fig. 1: Instrumented oedometer ring

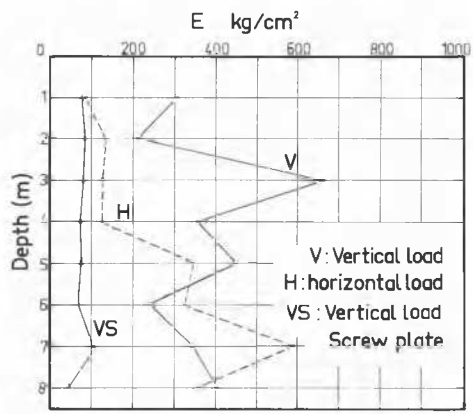

Fig. 2: E values from field tests.

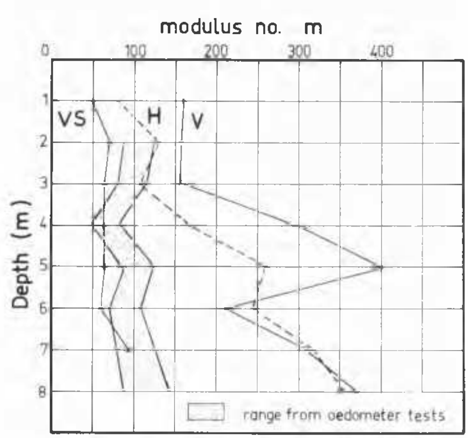

Fig. 3: Modulus no.m vs. depth.

trend is shown for Janbu's modulus number m in Fig. 3. A notable and unexpected feature is the low values of E and m from the screw plate tests. This is believed to be due mainly to bedding errors caused by disturbance of soil in the augering operation. The field tests showed an immediate settlement of 70 - 80% of the final settlement for stress increments and shows a compressibility behaviour similar to those of overconsolidated stiff clays (Som, 1968). Fig. 4 shows the secant modulus of elasticity, Es, (corresponding to half of the failure stress) from unconfined compression tests. Es for vertical samples was found to vary between 0.7 and 2.1 of Es for horizontal samples with a mean ratio of 1.4. Anisotropy of this order of magnitude is not an important factor in the reduction of surface settlements, (Barden,1963).

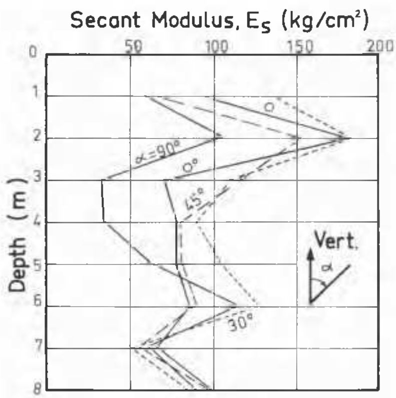

Fig. 4: Variation of secant modulus with depth.

For the purpose of comparison of results of oedometer tests, it was found convenient to plot these in a Ti/Ui graph.

$$T_i = 100\sqrt{ti}/\sqrt{te} \; ; \; (0 \leq Ti \leq 100)$$

where t_e denotes duration of each application of load, and t_i denotes any time during each application of load analysed.

$$U_i = 100 \,.\, \Delta\varepsilon i/\Delta\varepsilon e \;\; (0 \leq U_i \leq 100)$$

where $\Delta\varepsilon e$ denotes total variation of strain in each application of load, and $\Delta\varepsilon i$ denotes variation of strain at time ti.

For the same value of pressure increments ratio, $\Delta p/p$, it was observed that Ui/Ti curves were virtually identical irrespective of conditions of submergence, height of the sample and the duration of each loading, similar to results obtained for partly saturated clays by Yoshimi and Osterberg (1963). Increasing $\Delta p/p$ resulted in larger deformations.
The time resistance R as defined by Janbu (1969) may be obtained from Ui/Ti curves as

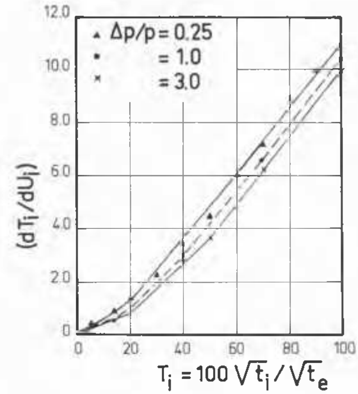

Fig. 5: (dTi/dUi) vs. Ti.

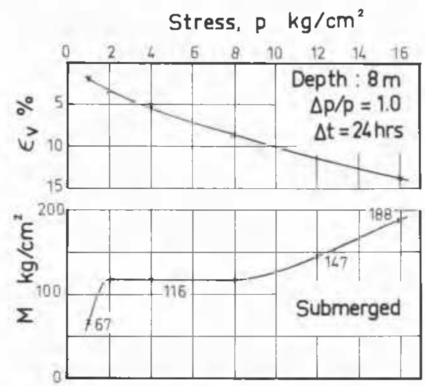

Fig. 6: Typical p-ε_v and p-M plot.

follows:

$$R = dti/d\varepsilon i = d(Ti^2 te/100^2)/d(\Delta\varepsilon e Ui/100) =$$
$$= (2teTidTi)/(100\Delta\varepsilon edUi) = (te/(50\Delta\varepsilon))(dTi/dUi)Ti$$

or $R' = (50\Delta\varepsilon/te)R = (dTi/dUi)Ti$

The relationship between (dTi/dUi) and Ti for different values of $\Delta p/p$ is shown in Fig. 5. An increasing resistance to deformation with decreasing $\Delta p/p$ is evident, as well as the presence of small time resistance, R, soon after loading thus implying large "immediate" deformations.

Fig. 6 shows a typical derivation of tangent modulus M curve from a stress-deformation plot. Three distinct features of the M/p plot may be noted:

a) small values of M at low stresses, showing the effect of disturbance of structure due to preparation and placing of sample in the oedo-

meter ring.
b) a stretch in the usual engineering stress range (up to 4 to 8 kg/cm^2) with constant modulus thus showing a similarity with behaviour of rocks ("elastic behaviour", Janbu 1967).
c) stress at large stress in which the modulus increases due to continuous decrease of volume under zero lateral deformation.

The values of modulus number m from all oedometer tests lie in the range of 50-140, similar to those from screw plate tests, but considerably less than values from vertical load test in the pit, (Fig. 3). The limitation of testing small sized laboratory samples is obvious.

Fig. 7 shows a typical curve of measured horizontal (radial) stress with increase in vertical stress. An initial stress of at least 0.5 kg/sq.cm. was required before the lateral stress measurement system could be activated.

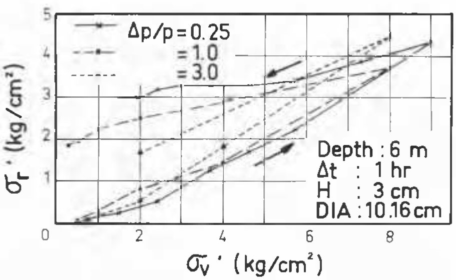

Fig. 7: radial stress vs. vertical stress

The parameter Ko, in its usual sense, is defined as ratio of effective horizontal to vertical stress (assuming principal stresses), (Bishop, 1958). Andrawes and El-Sohby (1973) expressed Ko as an incremental ratio, $\Delta\sigma_h'$ / $/\Delta\sigma_v'$, with horizontal lateral restraint (shown as $\bar{K}o$ in this Paper).

This latter general definition is considered to be of fundamental importance for residual soils, since it permits the soil to be subjected to any previous stress history, sufficing only that further stress changes take place under zero lateral strain. Table II shows the values of $\bar{K}o$.

Table II

Depth	Δp/p			$\bar{K}o$
m	0.25	1	3	av.
8	0.59	0.55	0.53	0.56
6	0.51	0.50	0.60	0.54
4	0.40	0.30	0.30	0.33
2	0.53	0.54	0.58	0.55

Explanation is not available for the particularly low values from the 4m depth. Excluding these values, an average of $\bar{K}o$=0.55 is obtained. The values of measured vertical stress should be decreased by 4-5% of applied load for the friction effect, thus incresing average $\bar{K}o$ to 0.58. Azvedo (1972) obtained a friction loss of 6% for a similar residual soil from gneiss in teflon lined oedometer ring of height/radius, ratio of 0.8. Jaky's equation, Ko= 1-sinϕ apparently underestimates Ko. Consolidated undrained tests with pore water pressure measurements for the 6m depth showed ϕ = 38.5º, indicating Ko= 0.38 in comparison with the experimental corrected value of 0.57.

CONCLUSIONS

1. The results donot show a significant and consistent directional anisotropy of E values.
2. The variation of tangent modulus, M, with stress shows the compressibility behaviour to be similar to rocks. The laboratory modulus number m varying between 50 and 140 is not affected by pressure increment ratio and duration of loading.
3. The time-deformation behaviour is influencedby the pressure increment ratio.
4. The coefficient of earth pressure at rest is approximately 0.6, and is probably underestimated by Jaky's equation.

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