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MECHANICAL PROPERTIES OF RECONSTITUTED SOFT LONDON CLAY*

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ABSTRACT : Undrained triaxial compression and extension tests, and one-dimensional incremental loading oedometer tests were carried out to evaluate the stress-strain-strength, stiffness, compressibility and permeability properties of K_0-normally consolidated reconstituted soft London clay. The behaviour of the London clay was found to be markedly different in compression and extension. The effective angle of internal friction, ϕ' was significantly higher in extension than in compression. The clay also showed significant stiffness anisotropy. Initial tangent modulus, E_i, undrained shear modulus, G_u and secant stiffnesses at various strain levels were considerably higher in extension than in compression. Pore pressure changes, however, were significantly smaller in extension than in compression. The values of compression index, C_c and swelling index, C_s are 0.55 and 0.15, respectively. The coefficient of consolidation, c_v reduces quite markedly with increasing vertical effective stress up to the magnitude of preconsolidation pressure. Values of c_v obtained using log t method are less than those obtained using $\sqrt{t}$ method for all the stress ranges. Approximately linear e-log k relationship has been obtained for soft London clay. Permeability change index, C_k for this clay was found to be 0.54. The ratio of C_c/C_k for this soft clay is 1.02.

KEYWORDS: Triaxial compression and extension, clay, undrained strength, deformation, stiffness, pore pressure

INTRODUCTION

Undrained triaxial compression and extension tests, and one-dimensional incremental loading oedometer tests were carried out on reconstituted samples of soft London clay. The principal objective of this work is to examine the basic mechanical properties of London clay in order to evaluate the soil parameters required for geotechnical analyses and designs. This paper presents the undrained stress-strain-strength, compressibility and permeability characteristics of soft normally consolidated reconstituted London clay. Similar work to assess the mechanical behaviour of one-dimensionally consolidated reconstituted soft clays has been carried out in the past (Parry and Nadarajah, 1973; Hight et. al., 1985; Allman and Atkinson, 1992).

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PREPARATION OF SAMPLES

The London clay was obtained from Stag Hill site of the University of Surrey. The index properties of the clay are as follows:
specific gravity = 2.74; liquid limit, LL = 69; plastic limit, PL = 24; plasticity index, PI = 45; clay fraction = 54%; and activity = 0.83.

Large blocks of clay were mixed with water in a soil mixer to prepare a homogeneous slurry which had a water content of approximately 1.5 times the liquid limit. The slurry was consolidated in a hydraulic consolidation cell of 1000 mm dia. by 490 mm deep. A pressure of 100 kPa was used during K_0-consolidation. It took approximately 133 days for complete consolidation of the slurry.

Block samples were cut by hand from the consolidated clay using wires. The blocks were placed in plastic bags which were de-aired and sealed. The samples were stored in a temperature controlled laboratory at 20° C.

TESTING PROGRAMME

The test programme consisted of two types of tests which are as follows:

(1) The first type of tests carried out was conventional undrained triaxial compression and extension tests. The stress path is shown in Fig. 1. In these tests, samples were first brought back to their in-situ stresses (point C) from their initial set up stresses (point B) by applying an undrained stress path at constant radial stress. The samples were then consolidated under K_0-conditions (K_0 = 0.64) to vertical effective stress of 1.75 to 2 times the in-situ effective vertical stress, before being sheared in undrained compression (path DJ) and extension (path DK). Reconsolidation beyond in-situ stresses was done to eliminate the effects of sampling and sample preparation. A back pressure of 250 kPa was used during K_0-consolidation.

(2) The second type of tests was incremental loading one-dimensional consolidation tests.

APPARATUS AND PROCEDURES

Undrained triaxial compression and extension tests were carried out using 102 mm triaxial cell in a stepless compression machine. The cell had the facility of single drainage by providing high air entry porous ceramic disc embedded at the base pedestal. Samples were trimmed from the block samples, using piano wire, a soil lathe and a split mould, to a nominal dimension of 102 mm dia. by 203 mm high. Water contents and bulk densities of the samples were 45 ± 1 % and 1.76 ± 0.01 Mg/m^3, respectively

Axial forces were measured using an internal load cell with a resolution of 1 N. The cell and back pressures were measured adjacent to the triaxial cell using pressure transducers with a resolution of 0.25 kPa. A miniature pressure transducer (Hight, 1982) with a resolution of 0.25 kPa was used to measure mid-plane pore pressures.

Local deformations were measured using Hall effect devices (Clayton et. al., 1989). Two axial Hall effect devices having a gauge length of 70 mm and one radial calliper were used. The resolutions of these axial strain and radial strain devices were about 1 μm and 0.5 μm, respectively. Axial strains were also measured externally using a displacement transducer with a resolution of approximately 6 μm. All instrumentation was monitored using a microcomputer.

An automated stress path system (Khatrush, 1987; Siddique, 1990) was used to control stresses imposed on samples during K_0-consolidation and undrained shearing . The system was controlled by a microcomputer. Two automated pressure controllers controlled the air pressure applied to the cell and back pressure air/fluid Bellofram rolling diaphragm. A third automated pressure controller regulated the air supply to a double acting Bellofram rolling diaphragm air actuator that was connected to the internal load cell. Air pressure increments of 0.1 kPa was attainable which corresponds to about 0.07 kPa for the axial pressure on the sample.

During drained stress path (path CD in Fig. 1) vertical stress was increased at a rate of 0.7 kPa/hr and excess pore pressure (difference between mid-plane and base pore pressures) of about 4 to 8% of vertical effective stress at the end of K_0-consolidation (point D in Fig. 1) was generated. The stresses were held constant at point D until the excess pore pressure completely dissipated. During undrained shearing to failure (paths DJ and DK in Fig. 1) deviator stress was increased at a rate of 10 kPa/hr with constant cell pressure.

The incremental loading oedometer tests were carried out on 76.2 mm diameter by 19.1 mm thick samples. Drainage was provided through top and bottom of the samples. The tests were carried out in accordance with the procedure outlined in British Standards (BS 1377, 1975). A stress increment ratio of 1 (i.e., a load ratio of 2) was used. The vertical consolidation stresses applied in each test were 50 kPa, 100 kPa, 200 kPa, 400 kPa and 800 kPa. The samples were also allowed to swell under stresses of 400 kPa, 200 kPa, 100 kPa and 50 kPa. Duration of each loading step was approximately 24 hours. For each loading step deformation was recorded by a dial gauge at specified intervals of time.

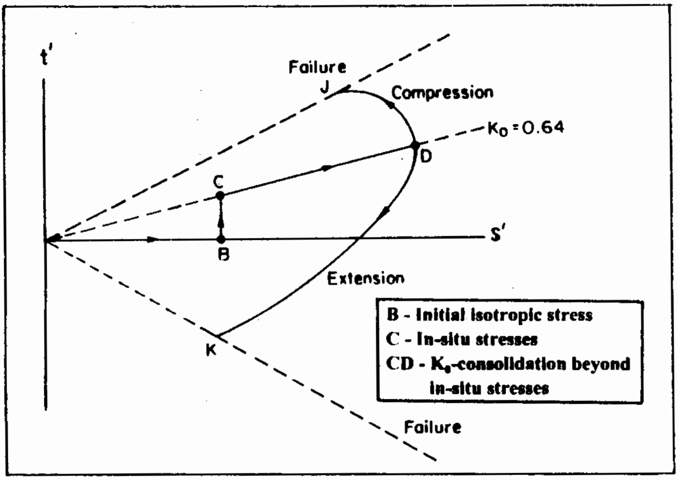

Fig 1. Stress paths applied to London clay samples in triaxial compression and extension tests

BEHAVIOUR IN TRIAXIAL COMPRESSION AND EXTENSION

Stress Paths

In Fig. 2, the observed stress paths in compression and extension are shown. It can be seen from Fig. 2 that the mean effective stress, p' decreases during both loading (i.e., q' increases) and unloading (i.e., q' decreases), i.e., shearing in compression and extension, respectively. Assuming failure occurs on the Critical State Line (CSL), the equations of the Critical State Lines shown in Fig. 2 can be given by the following equations:

$$q' = M_C \, p' \tag{1}$$

$$q' = M_e \, p' \tag{2}$$

where M_c and M_e are the critical state parameters in compression and extension, respectively. The values of M_c and M_e are 0.757 and 0.94, respectively, indicating that the Critical State Lines are not symmetrical about the p' axis. The effective angle of internal friction (or critical state friction angle), which can be conveniently calculated from the M values, are 19.6° and 33.9° in compression and extension, respectively. Since effective angle of internal friction in extension, ϕ_e' is significantly higher than the effective angle of internal friction in compression, ϕ_c', it appears that the relative ϕ' values in compression and extension are influenced by stress anisotropy in the soil. These results contrast with those reported by other research workers (Jardine, 1983; Gens, 1982; Hight et. al., 1987) who found ϕ_c' approximately equal to ϕ_e' for K_0-normally consolidated reconstituted samples of clay. However, ϕ_e'-values higher than ϕ_c'-values for K_0-normally consolidated clays were found in a number of investigations (Allman and Atkinson, 1992; Parry and Nadarajah, 1974; Ho, 1985; Atkinson et. al., 1987). Using a database derived from 100 different clays, Mayne and Holtz (1985) found that ϕ_e' was typically 20% to 50% greater than ϕ_c'. A comparison of the values of ϕ_c' and ϕ_e' reported by various researchers for different clays consolidated under K_0-condition is presented in Table 1.

Table 1. Effective Friction Angles in Compression and Extension for K_0-Normally Consolidated Clays

Soil Type	PI	K_0 Value	Effective Friction Angle, ϕ' (degree)		Reference
			Compression	Extension	
Marine silty clay	18 ± 5	0.5 to 0.53	29.2	31.7	Koutsoftas (1981)
Spestone kaolin	32	0.64	20.8	28.0	Parry and Nadrajah (1974)
Lower Cromer till	13	0.50	30.0	30.0	Gens (1982)
Speswhite kaolin	30	0.67	22.8	36.7	Ho (1985)
Speswhite kaolin	30	0.63	24.8	27.5	Ho (1985)
London clay	38	0.60	22.5	22.5	Jardine (1983)
Magnus clay	17	0.50	30.0	30.0	High et. al. (!987)
Speswhite kaolin	30	0.66	22.0	29.0	Atkinson et. al. (1987)
Bothkennar clay	38	0.50	34.0	37.0	Allman and Atkinson (1992)
London clay	45	0.64	19.6	33.9	Present investigation

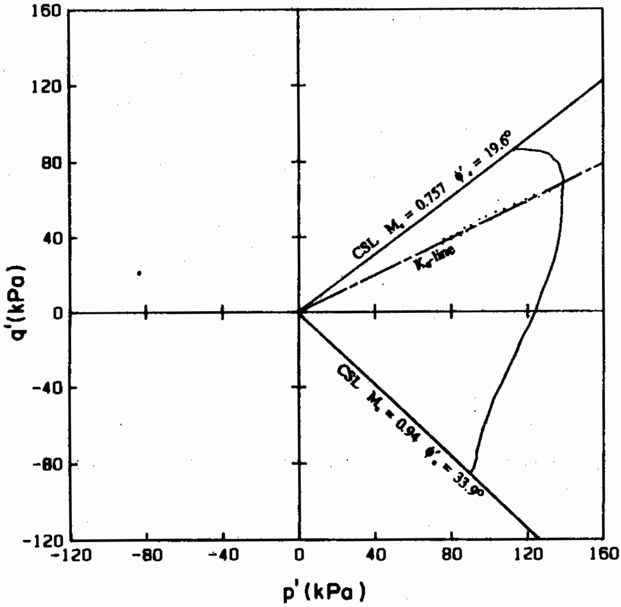

Fig 2. Effective stress paths in triaxial compression and extension tests

Stress-Strain-Strength and Stiffness Characteristics

Values of deviator stress (q) are plotted against axial strain (measured both locally and externally) for the compression and extension tests in Figs. 3(a) and 3(b), respectively. The following are the main observations:

(i) In triaxial compression, the peak undrained strength is mobilised at a small axial strain (ε_p = 1.5 %).

(ii) The peak and ultimate strength are almost equal.

(iii) In triaxial extension, the clay also appears to be non-brittle. Both peak and ultimate strength are mobilised at large axial strain.

(iv) The stress-strain relationships in both compression and extension are non-linear.

For this soft London clay, the triaxial compression and extension strength ratio (c_u/σ_{vc}') are found approximately equal. The undrained strength ratio is 0.23. The initial tangent modulus, E_i (determined from the slopes of the initial part of the stress-strain curves) and the undrained shear modulus, G_u (determined from the initial part of stress-local shear strain relationships) were, however, considerably higher in extension than in compression. E_i and G_u for triaxial compression are 9800 kN/m^2 and 4000 kN/m^2, respectively, while the respective values in triaxial extension are 14000 kN/m^2 and 5050 kN/m^2.

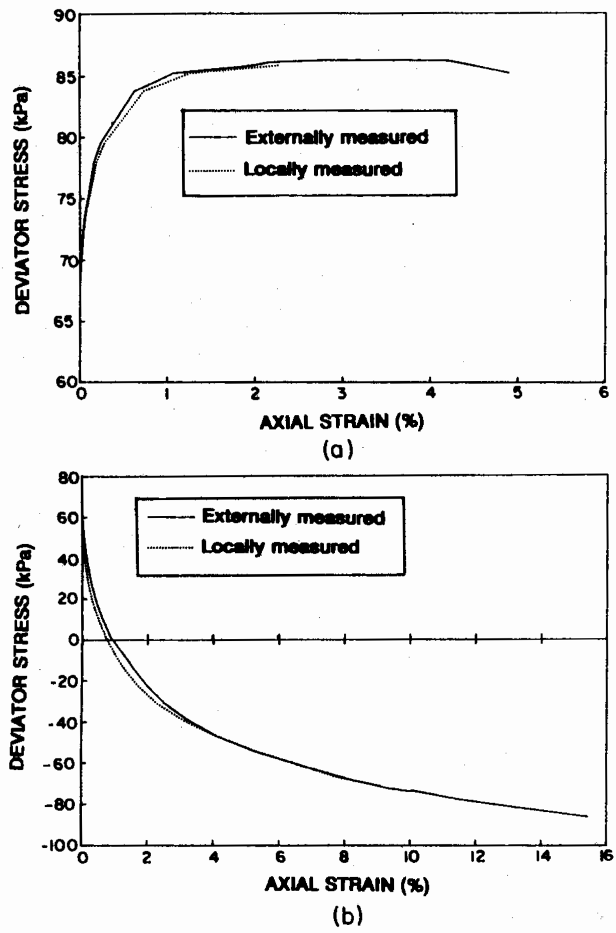

Fig 3. Deviator stress versus axial strain plots (a) Compression test (b) Extension test

Parry and Nadarajah (1974) and Gens (1982) also found higher stiffness in extension than in compression for K_0-normally consolidated samples of kaolin (PI = 32) and low plasticity Lower Cromer till (PI = 13), respectively. Koutsoftas (1981), however, observed higher stiffness in compression than in extension for a marine clay (PI = 18 ± 5).

The secant stiffness (E_u) determined over different strain levels are plotted against strains (on a logarithmic scale) for both the compression and extension tests and are shown in Figs. 4 and 5, respectively. It can be seen from these plots that secant stiffness decreases rapidly with increasing strain levels both in triaxial compression and extension.

Similar behaviour was reported by Hight et. al. (1987) for normally consolidated reconstituted clay and also for overconsolidated (OCR = 1.4 to 4) Lower Cromer till, Magnus clay and London clay. Smith et. al. (1992) also found similar behaviour for natural Bothkennar clay.

The small stress-strain characteristics were also evaluated in terms of indices $(E_u)_{0.01\%}/p_0'$ and L [L = $(E_u)_{0.1\%}/(E_u)_{0.01\%}$] as suggested by Jardine (1983). E_u is the secant stiffness and p_0' is the initial mean effective stress prior to shear. The first index provides a measure of the size of small strain region. The small strain zone may be considered as a region around a point in stress space, within which strains accompanying stress changes from that point are less than some small limiting value, e.g., 0.1% (Baldi et. al., 1988). The second index, L is an indicator of non-linearity in the stress-strain behaviour, the higher the value of L, the greater is the degree of linearity; L = 1 indicates a linear behaviour. The values of $(E_u)_{0.01\%}/p_0'$ and L were calculated for the extension and compression tests. In compression, the values of $(E_u)_{0.01\%}/p_0'$ and L are 108 and 0.476, respectively. The respective values in extension are 248 and 0.609. It is, therefore, evident that the size of the small strain zone (rather initial stiffness) is considerably larger in extension than that in compression. The L values, however, indicate that the stress-strain relationships are more non-linear in compression than in extension.

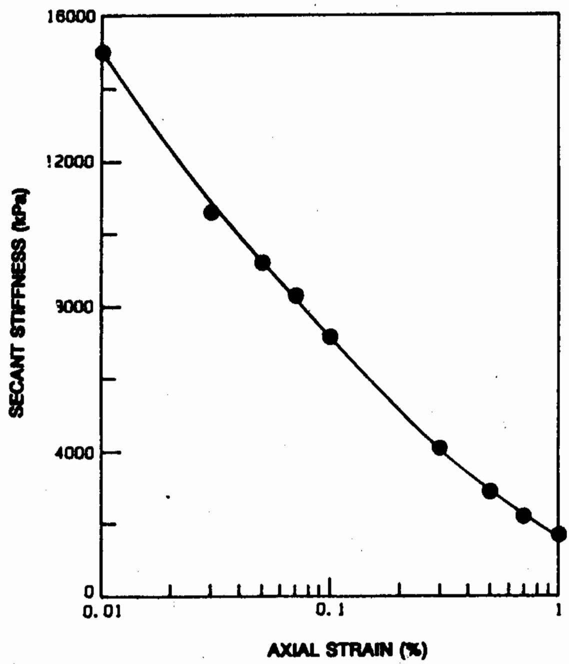

Fig 4. Secant stiffness - log.axial strain relationship in compression test

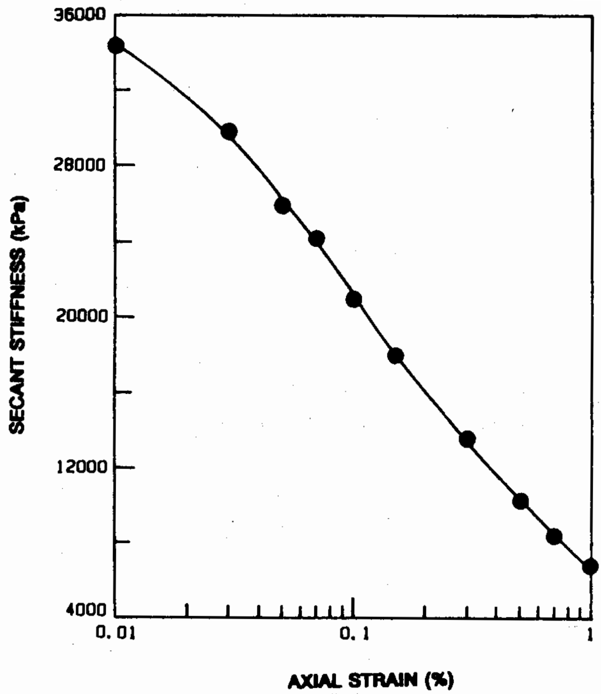

Fig 5. Secant stiffness - log. axial strain relationship in extension test

Jardine (1983) reported the values of $(E_u)_{0.01\%}/p_0'$ and L for K_0-normally consolidated reconstituted Magnus clay (PI = 17) and London clay (PI = 38) sheared in compression and extension. The mean effective stresses prior to shear, p_0', for the Magnus clay and London clay were approximately 266 kPa and 291 kPa, respectively. Values of $(E_u)_{0.01\%}/p_0'$ and L for these two clays are listed in Table 2 for comparison. It can be seen from Table 2 that for both clays stiffness index, $(E_u)_{0.01\%}/p_0'$ is considerably higher in extension than that in compression. Also for both the clays the degree of non-linearity is higher in compression than that in extension. Similar type of results have been obtained for the K_0-normally consolidated soft London clay (PI = 45, p_0' = 139 kN/m^2) from the present investigation.

Pore Pressure Response

In Figs. 6(a) and (b), the changes in pore pressure developed due to change in deviator stress only are plotted against external axial strain for the compression and extension tests, respectively. It can be seen from Fig. 6(a) that, during undrained shearing in compression, the pore pressure increases rapidly with the increase in deviator stress. For the extension test, however, it can be seen from Fig. 6(b) that the pore pressure change is very small as compared with that in the compression test.

The pore pressure parameter at peak deviator stress, A_p in compression test is 1.25. The pore pressure continues to increase after the peak deviator stress is reached and therefore results in higher values of pore pressure parameter A at failure, A_f. The value of A_f is 1.96. In extension test, at peak strength the pore pressure change is slightly positive and the value of A_p is only -0.003. Pore pressure response due to change in deviator stress is, therefore, much more significant when the sample is sheared in compression than that when the sample is sheared in extension.

Table 2. Comparison of Stiffness Index and Linearity Parameter for K_o-Normally Consolidated Clays

Clay Type	PI	Shearing Mode	$(E_u)_{0.01\%}/p_o'$	L	Reference
Magnus	17	Compression	831	0.185	Jardine (1983)
Magnus	17	Extension	1190	0.439	
London	38	Compression	482	0.319	
London	38	Extension	623	0.497	
London	45	Compression	108	0.476	Present investigation
London	45	Extension	248	0.609	

COMPRESSIBILITY AND EXPANSIBILITY PROPERTIES

The compressibility and expansibility characteristics of reconstituted soft London clay undergoing incremental loading in an oedometer are presented in Figs. 7 and 8. In Fig. 7, void ratios (e) at the end of each loading and unloading stages have been plotted against logarithm of vertical effective consolidation pressure while Fig. 8 shows the plotting of coefficient of volume compressibility, m_v and coefficient of volume increase, m_s as a function of logarithm of vertical effective consolidation pressure. Figs. 7 and 8 show the features similar to that expected for a normally consolidated clay. The values of compression index, C_c and swelling index, C_s, determined from the loading and unloading curves in Fig. 7, are 0.55 and 0.15, respectively. For loading up to 200 kPa, m_v increases from 0.47 m^2/MN to 0.85 m^2/MN; m_v then decreases up to 0.21 m^2/MN at a stress range of 400 to 800 kPa. During unloading from 800 kPa, m_s increases from 0.056 m^2/MN to 0.514 m^2/MN.

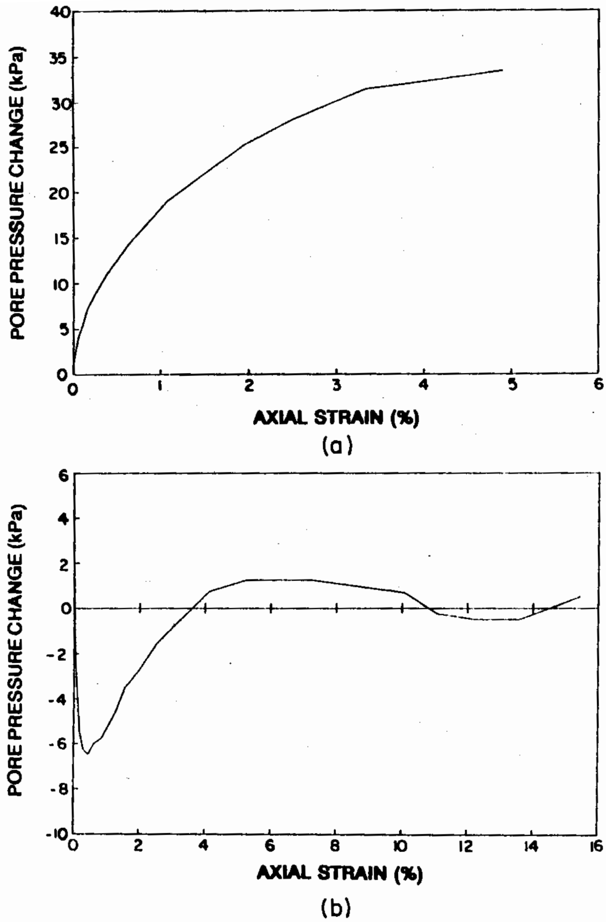

Fig 6. Pore pressure change versus axial strain plots
(a) Compression test (b) Extension test

Hopper (1988) also carried out standard incremental loading oedometer tests on reconstituted overconsolidated London clay [LL = 70, PI = 49,($\sigma'_v)_{max}$ = 100 kPa, OCR = 10] for three different stages of loading. The average C_c values for the stress ranges 100 to 200 kPa and 200 to 1000 kPa were 0.57 and 0.42, respectively. For loading up to 100 kPa, m_v increased from 0.33 m^2/MN to 0.8 m^2/MN; m_v then decreased up to 0.06 m^2/MN at a stress range of 800 to 1600 kPa. At a stress range of 400 to 800 kPa, m_v was 0.17 m^2/MN. Dial gauge readings for each loading stage were plotted as a function of both logarithm of elapsed time and square root of elapsed time.

The coefficients of consolidation, c_v have been calculated from these curves using Casagrande's curve fitting method (BS 1377, 1975) and Taylor's method (BS 1377, 1975). These methods are also called log-time (log t) and square-root-time ($\sqrt{t}$) method, respectively. Fig. 9 shows coefficient of consolidation, c_v as a function of vertical effective stress. It can be seen that c_v reduces quite rapidly on passing through the preconsolidation pressure (100 kPa). Beyond the preconsolidation pressure, i.e., in the normally consolidated range, change in c_v is insignificant. Similar observations have been reported for reconstituted low plasticity (PI = 17) Magnus clay (Jardine, 1983; Hight et. al., 1987). From Fig. 9, it is also evident that c_v-values obtained from the log t are less than those obtained from the $\sqrt{t}$ method for all the stress ranges. The difference is considerable at stresses less than the preconsolidation pressure. Beyond the preconsolidation pressure, however, the difference is small. The average value of c_v in the normally consolidated range (100 kPa to 800 kPa) obtained from log t method is 0.27 m^2/yr, while it is 0.3 m^2/yr when calculated using $\sqrt{t}$ method. Hopper (1988), however, reported an average value of 0.24 m^2/yr (at a stress range of 50 to 2000 kPa), calculated using $\sqrt{t}$ method, for London clay.

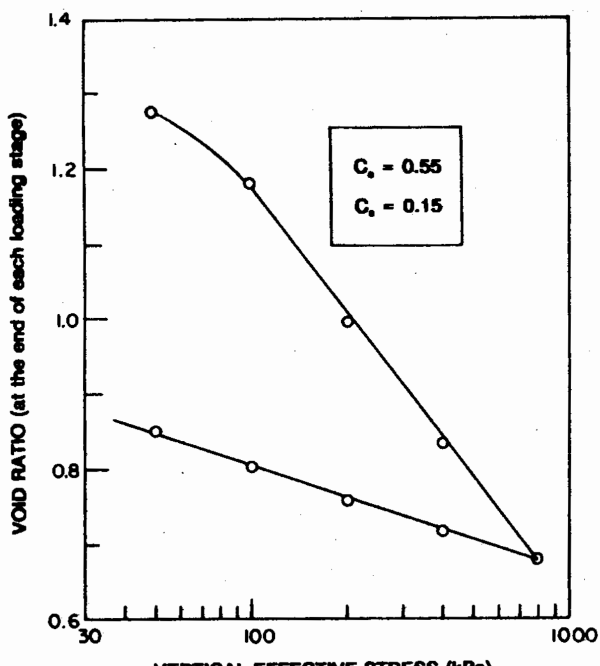

Fig 7. Void ratio versus log. vertical effective stress plot for normally consolidated London clay

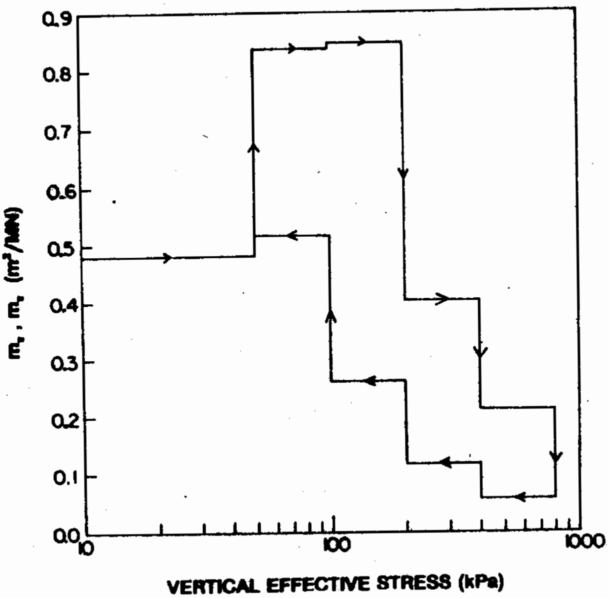

Fig 8. m_v / m_s versus log.vertical effective stress plot for normally consolidated London clay

PERMEABILITY PROPERTIES

Coefficients of permeability, k (= $c_v m_v \gamma_w$) were also computed for different stress levels of loading. In Figs. 10 and 11, the vertical permeability of the clay is shown in relation to changes in vertical effective consolidation stress and to void ratio at the end of each loading stage, respectively. It can be seen from Fig. 10 that permeabilities vary between 2 x 10^{-11} and 3 x 10^{-10} m/s. In the plots of void ratio against logarithm of permeability, shown in Fig. 11, the relationships are approximately linear over the whole stress ranges. The average slope of the relationships, which is termed the permeability change index (Tavenas et. al., 1983), C_k is 0.54 for this clay. Hight et. al. (1987) reported a value of 0.66 for reconstituted Magnus clay. The approximately linear relationship between void ratio and logarithm of permeability has been found to apply to other reconstituted clays (Hight et. al., 1987; Lambe and Whitman, 1969) and has been shown to extend to high void ratios in very soft sedimented clays (Been and Sills, 1981). The ratio of compression index to permeability change index, i.e., C_c/C_k for the soft London clay studied is 1.02. Berry and Wilkinson (1969) reported that for many soils C_c/C_k often lies within the limits of 0.5 and 2.0 while Mesri and Rokhshar (1974) observed that the experimental values of C_c/C_k were found to vary between 0.5 and 5.0.

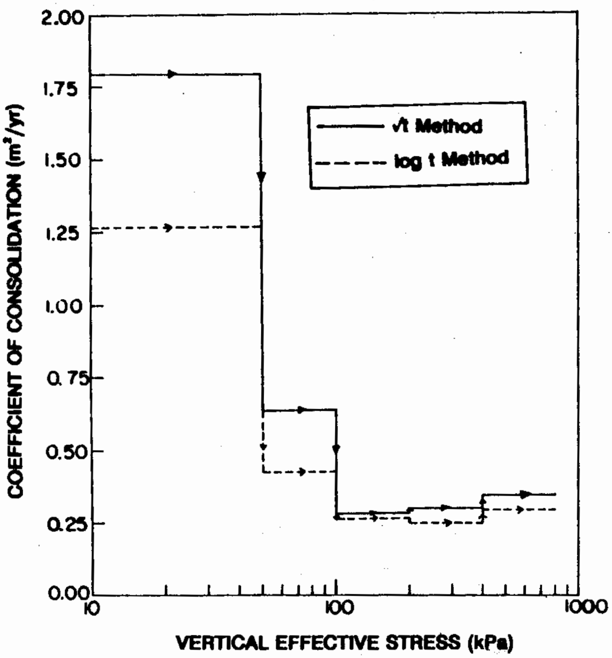

Fig 9. Coefficient of consolidation - log. vertical effective stress relation for normally consolidated London clay

CONCLUSIONS

Undrained triaxial compression and extension tests, and one-dimensional incremental loading oedometer tests have been carried out to evaluate the stress-strain-strength, stiffness, compressibility and permeability properties of normally consolidated reconstituted soft London clay.

The behaviour of the K_0-normally consolidated London clay was found to be markedly different in compression and extension. The effective angle of internal friction, ϕ' was significantly higher in extension than in compression. The undrained strength ratio, however, was approximately the same. The clay also showed significant stiffness anisotropy. E_i, G_u and secant stiffnesses at various strain levels, and the normalised stiffness parameter, $(E_u)_{0.01\%}/p_0'$, were considerably higher in extension than in compression. Pore pressure changes, however, were significantly smaller in extension than in compression. The clay also showed significant evidence of yielding.

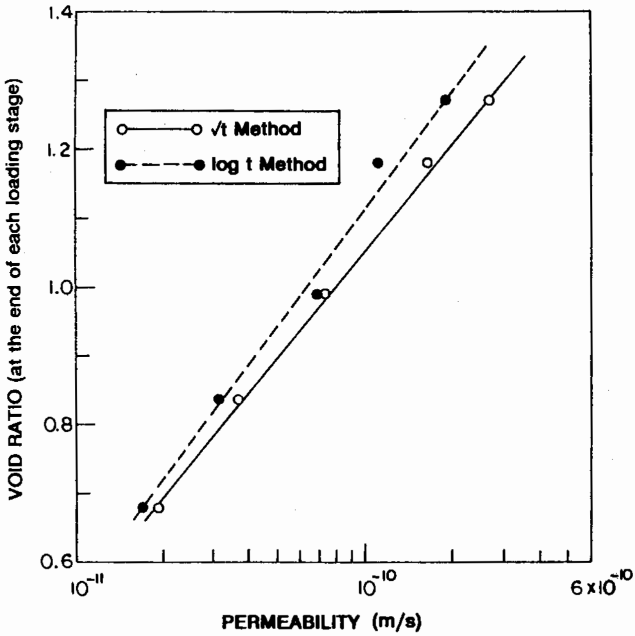

Fig 10. Coefficient of permeability - log. vertical effective stress relation for normally consolidated London clay

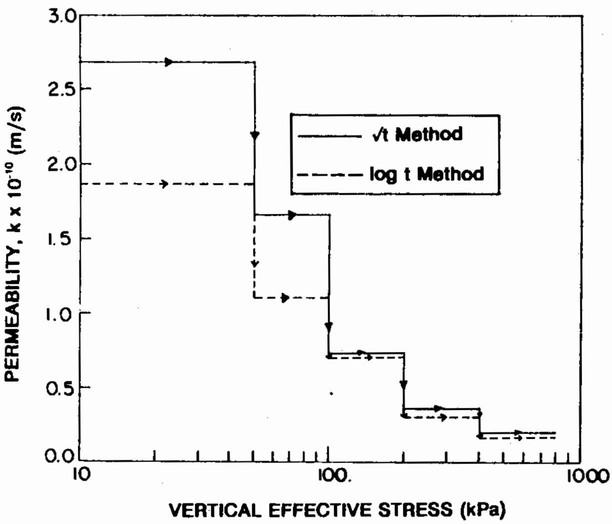

Fig 11. Void ratio versus log. coefficient of permeability plot for normally consolidated London clay

The values of compression index, C_c and swelling index, C_s are 0.55 and 0.15, respectively. The coefficient of consolidation, c_v reduces quite markedly with increasing vertical effective stress up to the magnitude of preconsolidation pressure. Beyond preconsolidation pressure changes in c_v are insignificant. Values of c_v obtained using log t method are less than those obtained using $\sqrt{t}$ method for all the stress ranges. The difference is significant at stresses less than preconsolidation pressure. Beyond preconsolidation pressure, however, the difference is small.

Approximately linear e-log k relationship has been obtained for soft London clay. Permeability change index, C_k for this clay was found to be 0.54. The ratio of C_c/C_k for this soft clay is 1.02 which lies within the limits reported by other researchers (Berry and Wilkinson, 1969; Mesri and Rokhshar, 1974).

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NOTATION

A	Skempton's pore pressure parameter
A_p	Skempton's pore pressure parameter A at peak deviatoric stress
CSL	critical state line
c_u	peak undrained shear strength
c_v	coefficient of consolidation
C_c	compression index
C_k	permeability change index
C_s	swelling index
e	void ratio
E_u	undrained secant stiffness
E_i	initial tangent modulus
G_u	undrained shear modulus
k	coefficient of permeability
L	linearity parameter
LL	liquid limit
M_c, M_e	critical state parameters in compression and extension, respectively
m_s	coefficient of volume increase
m_v	coefficient of volume compressibility
PL	plastic limit
PI	plasticity index
p'	mean effective stress $(\sigma_a' + 2\sigma_r')/3$
p_0'	mean effective stress prior to undrained shearing
q	deviatoric stress
q'	$\sigma_a' - \sigma_r'$
s'	$(\sigma_a' + \sigma_r')/2$
t'	$(\sigma_a' - \sigma_r')/2$
u	pore pressure
Δu	change in pore pressure
γ_w	unit weight of water
ε_p	axial strain at peak deviatoric stress
σ_a'	axial effective stress
σ_r'	radial effective stress
σ_{vc}'	vertical effective stress at the end of K_0-consolidation
ϕ'	effective angle of internal friction
ϕ_c'	effective angle of internal friction in compression
ϕ_e'	effective angle of internal friction in extension

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