Increase of Undrained Strength of Normally Consolidated and Over-Consolidated Soil due to Dissipation of Pore Pressure in Two Stage Loading

by

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Introduction

THE height to which an embankment can be built on a weak sub-soil is primarily restricted by the low value of in situ strength of the sub-soil. Rapid construction on such sub-soil may not allow the excess pore pressures to dissipate to such an extent as to cause appreciable gain in strength of the sub-soil thus permitting greater heights to be built. If, however, a pause in construction is allowed after building the first stage of the embankment to near failure condition of the sub-soil, the developed pore pressure will have time to dissipate. The consequent gain in strength of the sub-soil may permit building the embankment to greater height, if necessary.

This principle of stage construction is essentially similar to that of soil stabilization by preloading, a technique often used to improve the strength of soft sub-soils, if they have to support heavy structures (Aldrich 1957, Lambe 1962, Raymond 1966). Although the underlying principle has been known for a very long time, few attempts have so far been made to study the gain in strength on a quantitative basis and the factors that influence the process.

Skempton and Bishop (1955) investigated the gain in factor of safety resulting from pore pressure dissipation in connection with the construction of the Chew Stoke Dam. Straight line relationships between the percentage dissipation of pore water pressure set up in the first stage of construction and the factor of safety for both the upstream and downstream slopes were obtained. Gangopadhyay, Das and Sen (1969 derived a theoretical expression for the increase in in situ undrained strength resulting from the dissipation of pore pressure during stage construction. It was observed that in a two stage construction the gain in strength was largely dependent on the values of Skempton's pore pressure parameters

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A_{f1} and A_{f2} (Skempton 1954) corresponding to the first and second stage of loading respectively.

The present investigation is an attempt to study in the laboratory this behaviour of artificially sedimented kaolinite. Normally consolidated and oveer-consolidated samples of commercially available kaolinite were prepared under *Ko* condition. The samples were then loaded undrained to near failure state of stress. The resulting pore water pressure was allowed to dissipate and the samples were loaded further to determine the increase in strength and to study the relationship between the pore pressure parameters A_{f1} and A_{f2} and their effect on the strength increase.

A Theoretical Expression for the Increase in In-situ Strength due to Full or Partial Pore Pressure Dissipation

let, $\overline{P}_o$ = Effective vertical stress on a soil element in the ground before construction.

K_o = Coefficient of earth pressure at rest.

u_n = Neutral pore pressure at the same depth.

P_1 & P_3 = Increase in vertical and lateral stresses on the soil element in the first stage of construction.

u_i = Initial pore pressure set up in the first stage of construction.

A_{f_1} = Skempton's pore pressure coefficient associated with the first stage of loading.

$\triangle\sigma_1$ & $\triangle\sigma_3$ = Increase in vertical and lateral stresses on the soil element in the second stage of loading.

u = Pore pressure at failure in the second stage of loading.

A_{f_2} = Skempton's pore pressure parameter associated with the second stage of loading.

Then, the in situ undrained shear strength is given by, (Skempton and Bishop 1954)

$$\underline{Cu_i}=\frac{C'\cos\phi' + \overline{P}_o \sin\phi' \,[K_o+A_{fi}\,(1-K_o)]}{1+(2\,A_{fi}-1)\sin\phi'} \qquad \ldots(1)$$

where C' and ϕ' are the effective shear strength parameters of the soil.

The new value of the undrained shear strength, when the pore pressure u_i is allowed to dissipate, fully or partially, is given by, (Gangopadhyay et al 1969)

$$Cu=\frac{C'\cos\phi' + [K_o\overline{P_o}+U.u_i-2Cu_i\,(A_{f1}-A_{f2})+\overline{P_o}\,(1-K_o)A_{f1}]\sin\phi'}{1+(2A_{f2}-1)\sin\phi'} \qquad \ldots(2)$$

where U=Degree of consolidation after the pause in construction of time t_p.

Scope of the Present Investigation

The experimental work carried out in the present investigation consists of a series of tests on simulated field elements of normally consolidated and over-consolidated kaolin, carried out to study the increase in in situ strength due to complete dissipation of pore water pressure. Samples were consolidated under K_o-condition to different vertical effective stresses to represent samples at different depth—and were subsequently loaded in stages to simulate stage loading in the field. Owing to instrumental limitations, the tests were conducted within the vertical effective consolidation stress range of 2.1, 5.25 kg/cm^2 and maximum over-consolidation ratio of 3. The results have been compared with the theoretical analysis of Gangopadhyay et al (1969).

Test Programme

Tests were carried out both on normally consolidated (NC) and over-consolidated (OC) samples. Specimens were initially consolidated (NC or OC) under K_o-condition to represent in situ elements before construction. Tests were then conducted according to the following programme :

Normally Consolidated Samples

NC—Series A

Samples were initially consolidated to nominal vertical effective stresses $\sigma_{1c}'=2.10$, 3.15, 4.20 and 5.25 kg/cm^2 and sheared undrained to determine the in situ strength of the simulated field elements.

NC—Series B

Samples were initially consolidated to the same nominal vertical effective stresses as in Series A, i.e., $\sigma_{1c}'=2.10$, 4.20 and 5.25 kg/cm^2. They were then loaded axially under undrained condition up to about 80 percent of the failure load determined from tests in NC—Series A. The resulting pore water pressures were measured. Thereafter the samples were allowed to consolidate and then sheared undrained to determine the gain in strength.

Over-Consolidated Samples

OC—Series A

Samples were initially normally consolidated, as in NC—Series A, to vertical effective stresses of 2.10, 3.15 and 4.20 kg/cm^2. The axial and lateral stresses were then decreased under drained condition maintaining zero lateral yield, to obtain K_o-over-consolidated specimens with OCR values of 1.5, 2 and 3. The specimens were then sheared undrained to determine the in situ strength of the simulated field elements.

OC—Series B

The over-consolidated samples obtained as in OC—Series *A* at OCR values of 1.5, 2 and 3 were loaded axially in undrained condition to about 80 percent of the failure load determined from tests in OC—Series A. The resulting pore water pressures were measured. The samples were then allowed to consolidate after which they were sheared undrained to determine the increased strength.

Test Procedure and Equipment

46.7 gm of oven-dried kaolin was mixed uniformly with 110 c.c. of distilled water to form a slurry. This slurry was poured into a 3.8 cm ($1\frac{1}{2}$ in.) diameter sedimentation tube in three equal instalments. Sedimentation was allowed overnight and the soil was then consolidated, in stages, to a final consolidation pressure of 0.89 kg/cm². After consolidation was complete, the samples were pushed out of the sedimentation tube and mounted in the triaxial cell. The amount of oven-dried kaolin and the final consolidation pressure of 0.89 kg/cm² were chosen from a number of pilot tests so as to obtain a consistency sufficient to facilitate easy handling and a sample length of 3 8 cm ($1\frac{1}{2}$ in.) that is suitable for testing under "free end" condition. A circular latex membrane of diameter 3.8 cm with four radial slits was placed inbetween the sample and the top and bottom caps which were coated with thin films of silicone grease to ensure "free end" condition. Testing under this condition enables one to eliminate end restraint on the samples and thereby ensure a deformation which is more uniform than is possible to abtain with conventional testing. Filter paper strips were fixed around the sample and connected to a porous stone fixed at the bottom cap as shown in Figure 1, thereby permitting radial drainage during consolidation. Another small porous stone was inserted centrally into the bottom cap to facilitate measurement of pore pressure. The drainage line and the pore pressure line were separated by pressfitting a plastic tubing 0.25 cm I.D.$\times$0.63 cm long by Araldite to the vertical hole in the bottom cap. The details of the arrangement are shown in Figure 1.

The sample and the triaxial cell were mounted on an anisotropic frame, the details of which are shown in Figure 2. This frame is a stress-controlled type unit, the axial load being applied by filling with water a perspex loading tank suspended from a hanger. The triaxial cell is centred below the hanger and when a deviator load is to be applied the tank which is precalibrated is filled with water up to the desired level.

This type of anisotropic frame permits gradual application of vertical load without imparting any shock to the sample. Deviator load can be applied in small increments by controlling the amount of inflow of water into the tank. The axial load on the soil specimen was measured by a transducer fitted to the bottom of the loading ram within the triaxial cell (Fifgure 2) thus eliminating the effect of piston friction from the measurement of the axial load. The transducer consists of a simply supported stainless steel eam, 0.5 cm thick and 6.25 cm long, the distance between the edge supports being 5 cm. The load on the soil sample causes the simply supported transducer beam to deflect and the resulting flexural strains are meastured by strain-gauges fixed to the top and bottom of the beam. A strain measuring bridge type RZ—003 manufactured by *ORION—ENG*, *Budapest*, *Hungary* was used to measure the strains. The transducer was pre-calibrated with dead weights and an essentially linear relationship between load and micro-strain reading was obtained (Figure 3)

For measurement of pore pressure and volume change the standard Bishop type null indicator and volume gauge were used.

Each sample after mounting was subjected to an all round cell pressure of 2.1 kg/cm² and a back pressure also of 2.1 kg/cm² and was left over night to dissolve any air that might have been present. Further testing was carried out from this condition.

Ko-Consolidation (NC)

In the absence of a lateral strain indicator, the following principle was used to obtain *Ko-NC* specimens. If strains are small, the axial

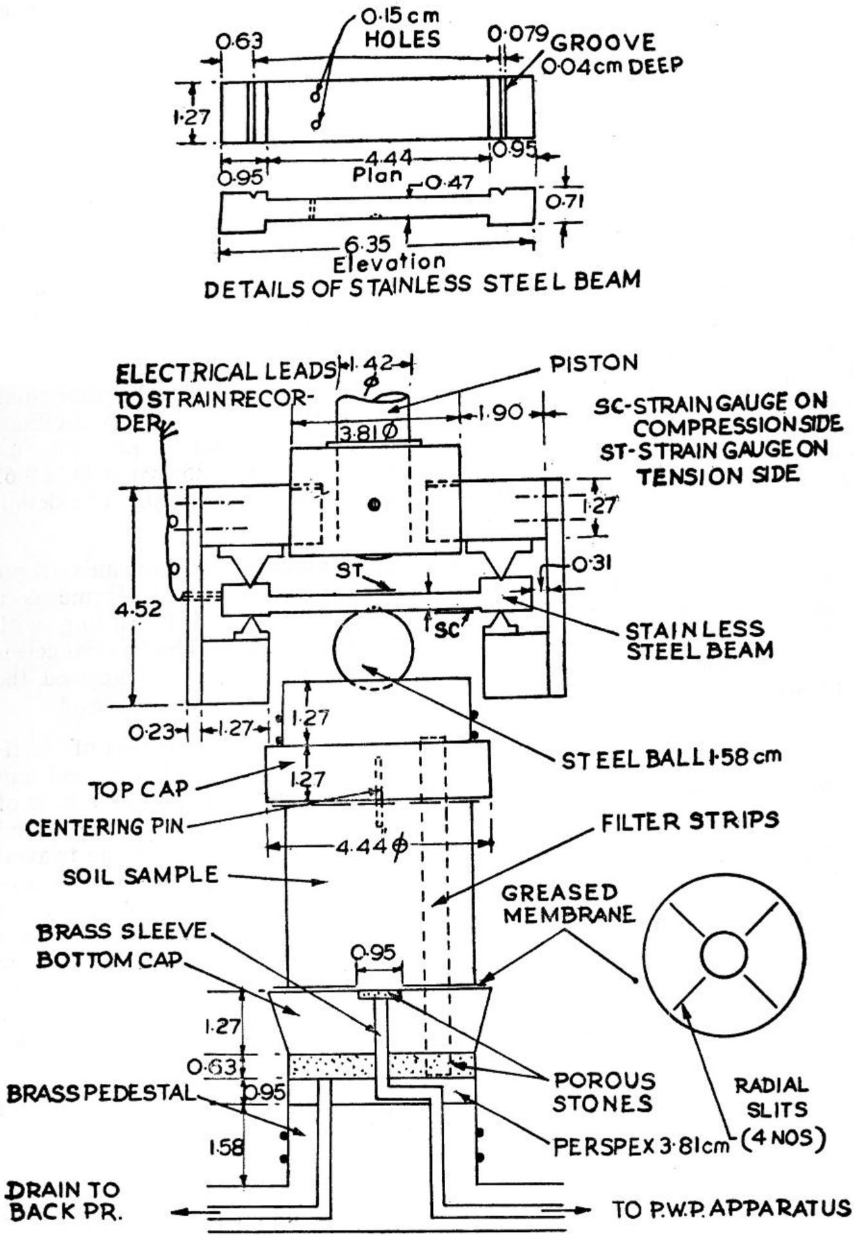

FIGURE 1 : Details of transducer, top and bottom caps.

strain $\in_1$ and the volumetric strain $\in_v$ must be equal for the condition of no lateral strain to be satisfied. The value of *Ko* is thus equal to the effective stress ratio σ_3'/σ_1' corresponding to the strain ratio $\in_1'\in_v=1.0$, *Ko* was, therefore, determined from a series of dissipation tests in which different stress ratios were applied on a number of identical specimens. The resulting axial and volumetric strains were measured after consolidation

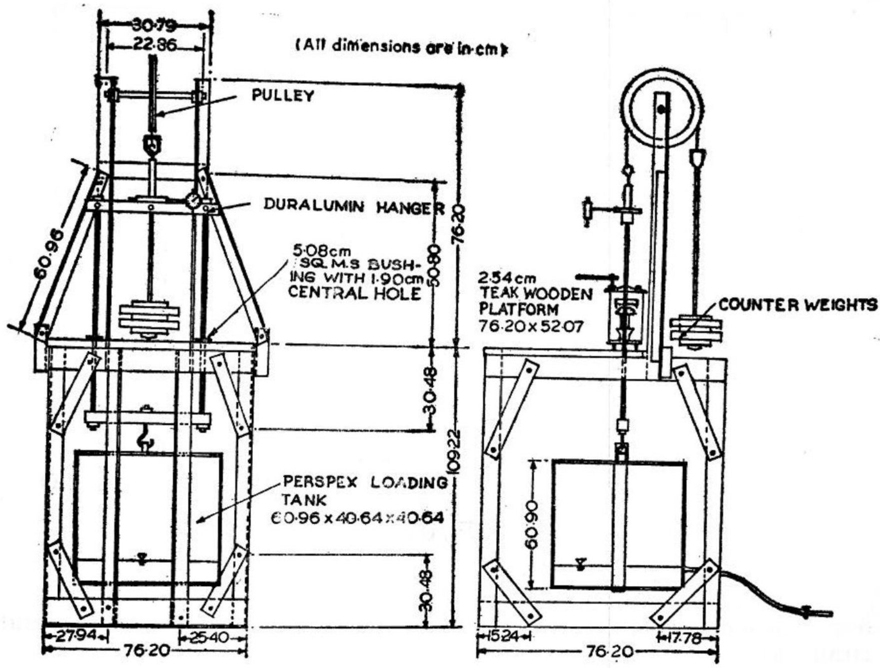

FIGURE 2 : Details of anisotropic frame.

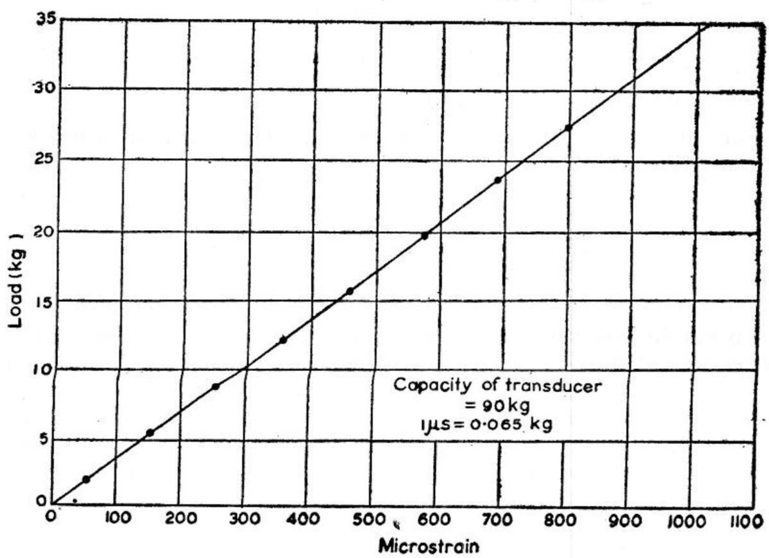

FIGURE 3 : Calibration of transducer.

under these stress ratios and the value of $Ko=0.46$ corresponding to $\in_1/\in_v=1.0$ was obtained by interpolation (Figure 4).

Ko-Consolidation (OC)

Since the value of *Ko* for over-consolidated soil is not unique, and is a function of the over-consolidation ratio as well as stress level, *Ko* for

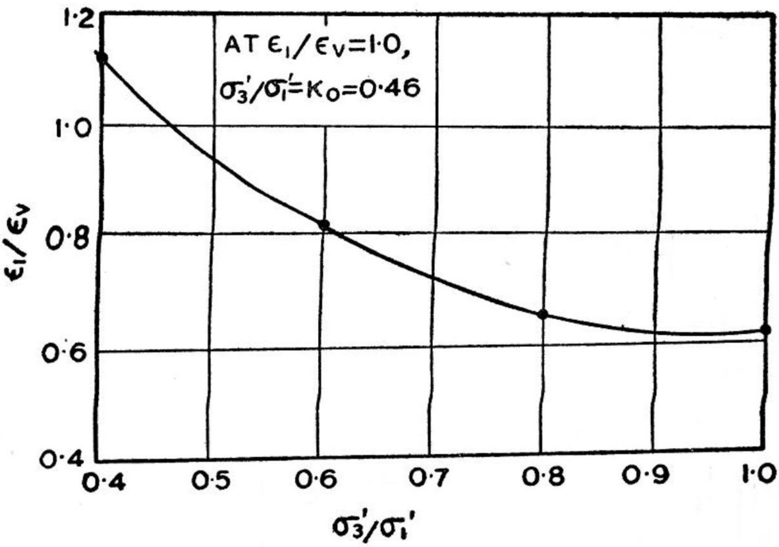

FIGURE 4 : ϵ_1/ϵ_v versus σ_3'/σ_1'.

over-consolidated samples were determined in the absence of a lateral strain indicator, from Alpon's (1967) empirical relationship :

$K_o^{OC} = K_o^{NC}(OCR)^{\lambda}$ where the parameter λ is related to the plasticity index of the soil by the relation—

$$I_p = -281 \log (1.85\,\lambda)$$

For this soil, plasticity index was 20 (see test results below) and therefore,

$$\lambda = 0.464.$$

$$\therefore \quad K_o^{OC} = K_o^{NC} \left[OCR \right]^{0.464} \qquad ...(3)$$

To obtain *Ko*-over-consolidated specimens samples were first normally consolidated under *Ko*-condition to vertical effective stresses σ_{1c}' = 2.10, 3.15 and 4.20 kg/cm². The vertical and lateral stresses were then released under drained condition according to the ratio indicated by Equation (3). The measured axial and volumetric strains indicated that *Ko*-condition was indeed maintained. This was further substantiated by occassionally checking the lateral dimension of the samples by cathetometer.

Test Results

Index Properties

The liquid limit and plastic limit of the kaolin used in the experimental programme were 57.7 and 37.7 percent respectively. The grain size distribution, determined by hydrometer analysis indicated presence of predominantly silt sized particles with only 9.6 percent clay fraction.

Normally Consolidated Samples

The results of dissipation tests carried out to determine the value of *Ko*, as described above, are shown in Figure 4. The stress ratio (σ'_3/σ'_1) corresponding to strain ratio $\in_1/\in_v=1.0$ is found to be 0.46 and in the main series of tests *Ko*-consolidation was carried out for each sample by applying this stress ratio. The strain ratios at the end of consolidation were checked and were found to range between 0.87 and 1.123 with an average of 0.99. Thus the condition of zero lateral strain can be said to have been effectively maintained during *Ko*-consolidation.

TESTS TO DETERMINE C′ AND ϕ'

Three isotropically consolidated undrained shear tests (*CIU*) were

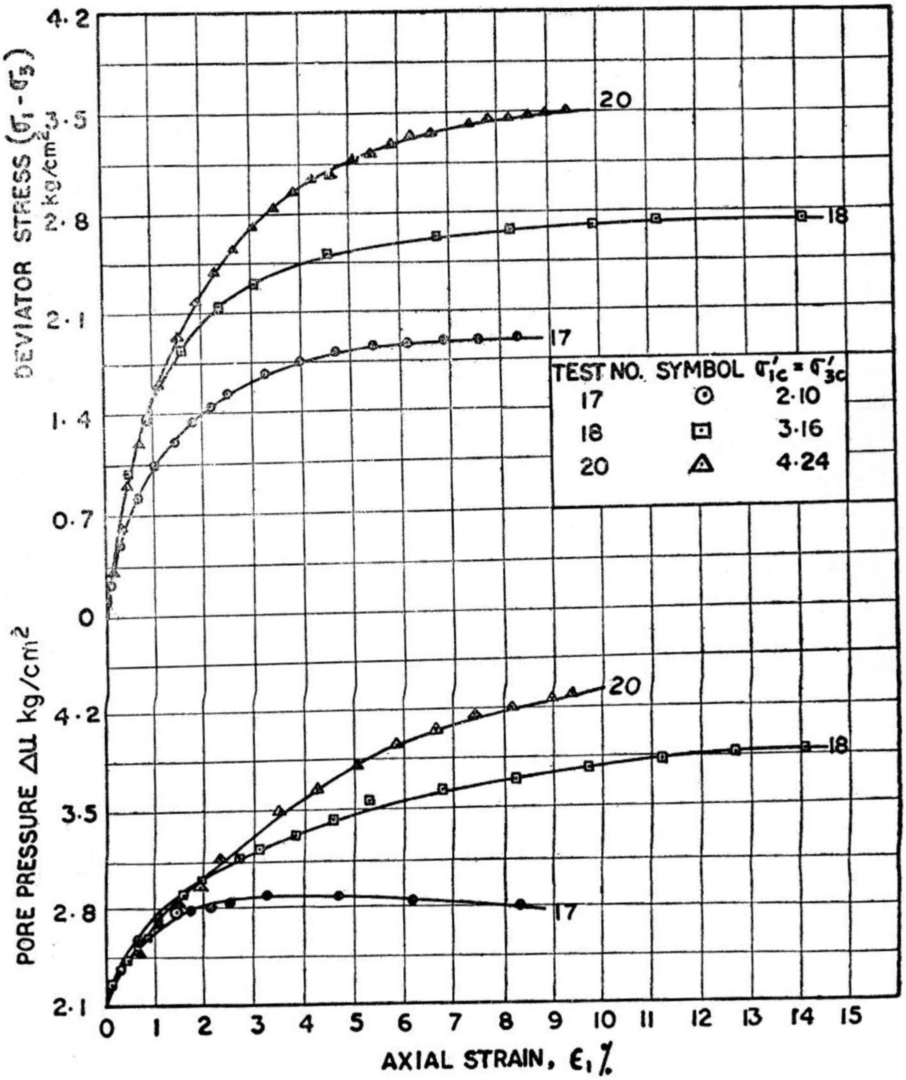

FIGURE 5 : Stress-strain curves for CIU tests.

run with effective consolidation pressures of 2.10, 3.16 and 4.24 kg/cm² to determine the C' and ϕ' of the soil. The stress versus strain and pore pressure versus strain curves for these tests are plotted in Figure 5. The corresponding Mohr Envelope gives the shear strength parameters $C'=0$ and $\phi'=30.2°$ while the pore pressure coefficients A_f at failure is found to be 0.64.

TESTS ON NC SAMPLES—SERIES A

Four samples initially consolidated under *Ko*-condition to $\sigma_{1c}'=5.23$, 4.13, 3.15 and 2.06 psi were sheared undrained by increasing the axial stress only, keeping the cell pressure constant. The resulting deviator stress and pore pressure versus strain curves are plotted in Figure 6. The shear strength parameters for these samples are found to be the same as for the *CIU* tests, i.e., $C'=0$ and $\phi'=30.2°$ and the pore pressure parameter at failure $A_f=0.85$.

TESTS OT NC SAMPLES—SERIES B

Three samples were initially *Ko*-consolidated to approximately the same vertical effective stress as in the NC-Series A tests, then loaded undrained by increasing the vertical stress only between 67 and 81 percent of the failure deviator stress as obtained in Series A tests. The excess pore pressures were allowed to dissipate and the samples were finally sheared undrained. The test results are plotted in Figure 7.

The shear strength parameter for the samples are again found to be $C'=0, \phi'=30.2°$. The pore pressure coefficient A_{f1} in the first stage of loading varied between 0.66 and 0.83 while A_{f2}, in the second stage loading, was between 0.85 and 1.05.

All the test results of NC-Series A and Series B are summarized in Tables I and II.

TABLE I

In Situ Strength of Normally Consolidated Samples: C_{ui}, NC—Series A Tests

$\sigma_{1c'}$	$\sigma_{3c'}$	$(\sigma_1 - \sigma_3)_f$	u_f	A_{f_1}	C_{ui} (obs)	C_{ui} (Calc. Eq. 1)
2.06	0.97	0.35	0.29	0.84	0.72	0.71
3.15	1.45	0.49	0.41	0.84	1.09	—
4.13	1.93	0.58	0.49	0.85	1.39	1.43
5.23	2.41	0.65	0.57	0.86	1.74	1.76

Stresses in kg/cm²

TABLE II

Increased Strength of Normally Consolidated Samples :
C_u, NC—Series B Tests

		1st Stage			2nd Stage				
σ_{1c}'	σ_{3c}'	$(\sigma_1 - \sigma_3)$	u_i	A_{f_1}	$(\sigma_1 - \sigma_3)$	u_2	A_{f_2}	C_u (obs)	C_u (Calc. Eq. 2)
2.06	0.97	0.24	0.20	0.83	0.13	0.11	0.84	0.74	0.80
4.14	1.93	0.47	0.31	0.66	0.30	0.26	0.88	1.48	1.66
5.14	2.41	0.47	0.35	0.74	0.33	0.35	1.05	1.76	1.89

Stresses in kg/cm²

Over-Consolidated Samples

KO-CONSOLIDATION

The procedure of *Ko*-consolidation to obtain *Ko*-over-consolidated specimens has already been described.

TESTS ON OC SAMPLES—SERIES A

In this series of tests the samples were initially normally consolidated under *Ko*-condition to effective vertical stress $\sigma_{1c}'=2.10$, 3.15 and 4.20 kg/cm² and then over-consolidated to nominal over-consolidation ratios of 1.5, 2 and 3. Three samples at each OCR were obtained. The samples were then sheared by increasing vertical stress only to determine the undrained strength C_{u_i} of the samples. Typical stress versus strain and pore pressure versus strain curves for the samples at OCR=3 are shown in Figure 8 and the results of all the tests are summarized in Table III.

TESTS ON OC SAMPLES—SERIES B

Specimens with identical stress history as those of OC—Series A were loaded undrained to 77—96 percent of the failure deviator stress obtained from Series 'A' tests. The pore pressures developed were allowed to dissipate and the samples were finally sheared undrained, again by increasing the deviator stress only. The test results for samples at OCR=3.0 are plotted in Figure 9 and the complete data are summarized in Table IV.

Analysis of Test Results and Discussions

Shear Strength Parameters

The Mohr Circles for the isotropically consolidated undrained tests, tests of NC—Series A and tests of NC—Series B are plotted in Figure 10. It is possible to draw a single envelope for all the tests, which gives $C'=0$ and $\phi'=30.2°$.

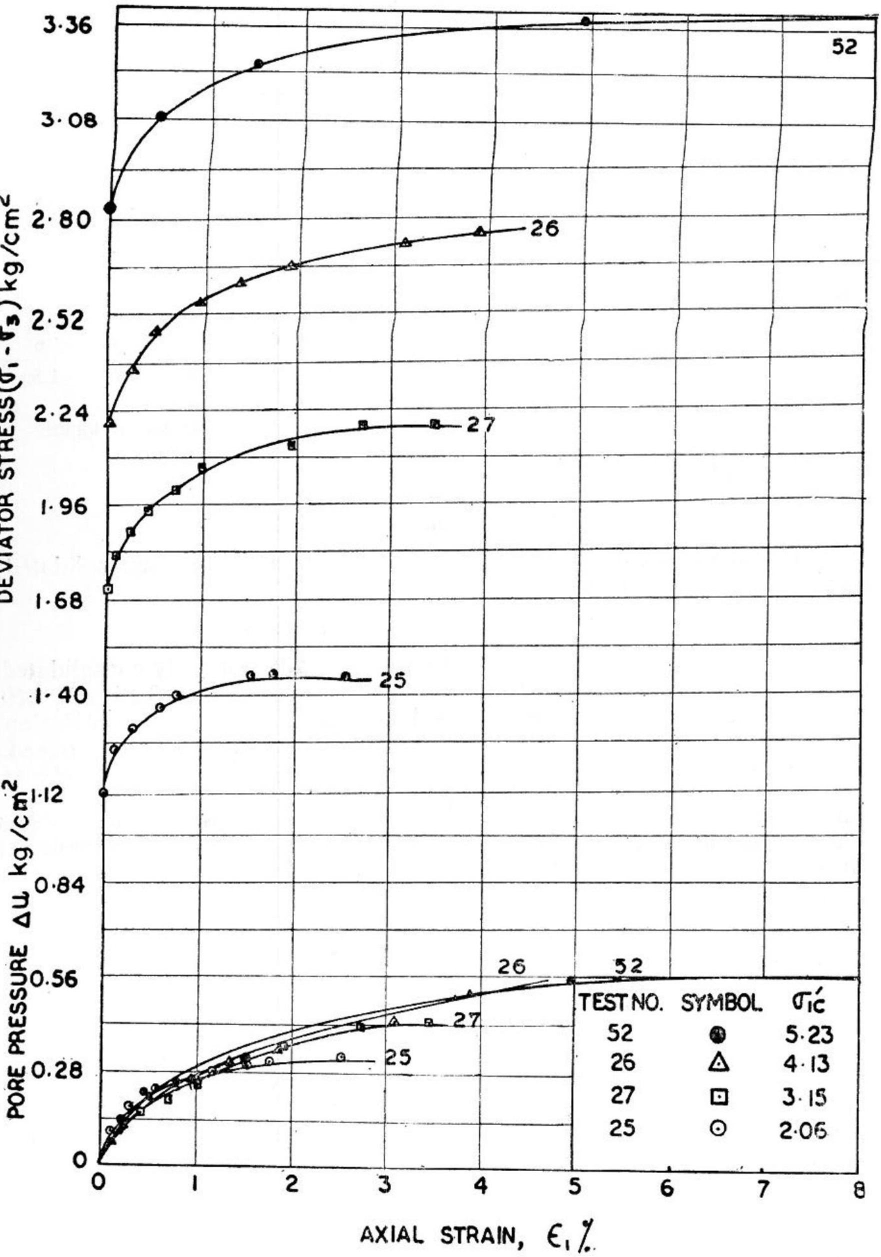

FIGURE 6 : Stress-strain curves for NC—Series A.

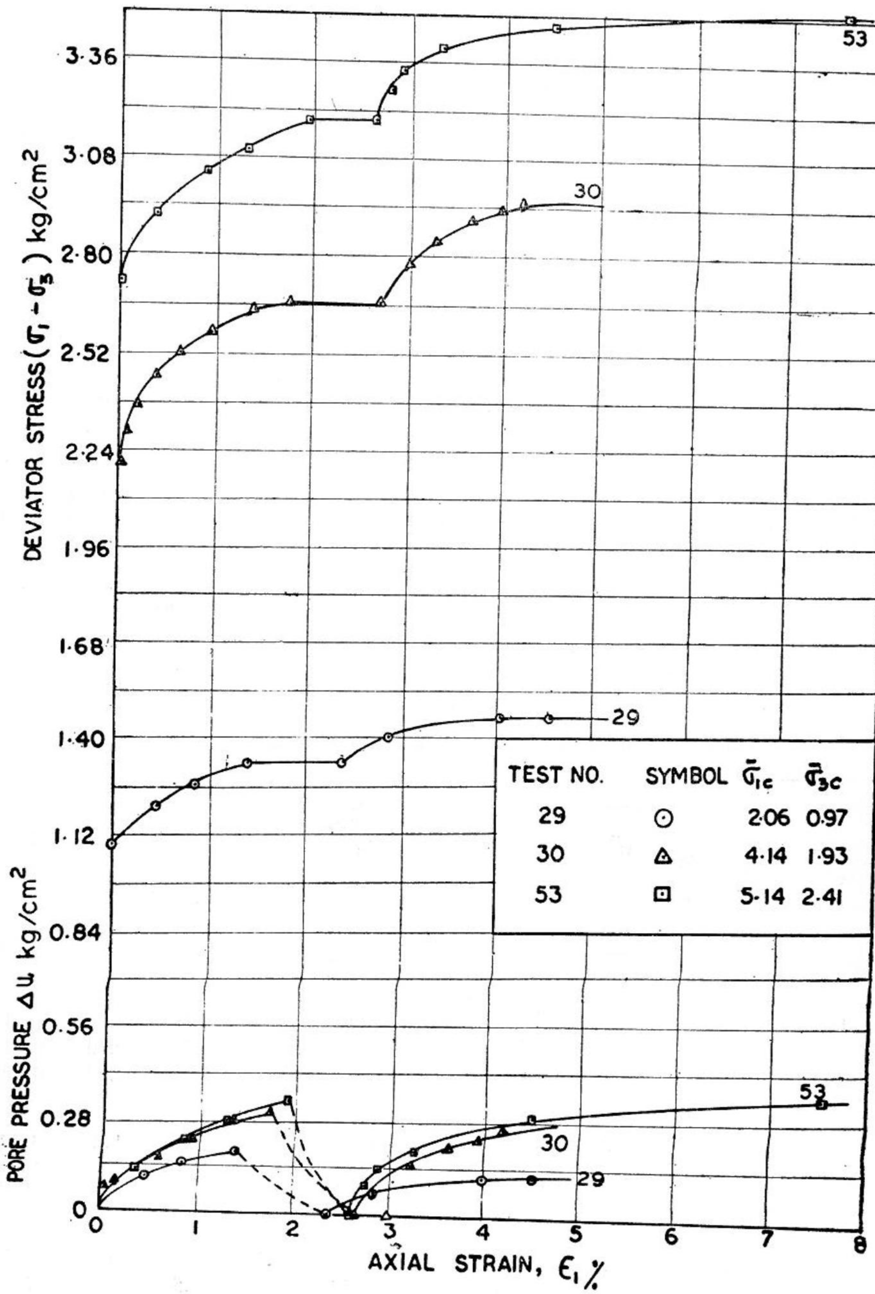

FIGURE 7: Stress-strain curves for NC—series B.

TABLE III

In Situ Strength of OC Samples : $C_{ui,}$ OC—Series A Tests.

σ_{1c}'	σ_{3c}'	OCR	$(\sigma_1 - \sigma_3)_f$	u_f	A_{f_1}	C_{ui} (Obs)	C_{ui} (Calc. Eq. 1)
1.45	1.05	3	1.66	0.16	0.10	1.03	0.81
1.07	0.80		1.30	0.17	0.13	0.78	0.74
0.70	0.54		0.081	0.13	0.16	0.49	0.48
2.12	1.31		1.55	0.19	0.12	1.18	1.21
1.46	1.00	2	1.38	0.20	0.15	0.92	0.89
1.03	0.67		0.84	0.20	0.24	0.60	0.66
2.72	1.55		1.43	0.33	0.23	1.30	1.32
2.03	1.16	1.5	1.00	0.21	0.21	0.93	1.02
1.40	0.78		0.66	0.14	0.21	0.64	0.68

Stresses are in kg/cm^2

TABLE IV

Increased Strength of OC Samples : OC—Series B Tests.

σ_{2c}'	σ_{3c}'	OCR	$(\sigma_1 - \sigma_3)$	u_i	A_{f_1}	$(\sigma_1 - \sigma_3)_f$	u_2	A_{f_2}	C_u (Obs)	C_u (Calc. Eq. 2)
1.46	1.05		1.31	0.27	0.21	0.60	0.63	0.10	1.13	1.14
0.98	0.81	3	1.08	0.16	0.15	0.33	0.91	0.27	0.80	0.82
0.71	0.54		0.69	0.13	0.19	0.25	0.91	0.36	0.57	0.51
2.10	1.31		1.19	0.15	0.13	0.51	0.11	0.22	1.24	1.22
1.58	1.00	2	1.13	0.16	0.14	0.52	0.15	0.30	1.08	0.99
1.03	0.67		0.67	0.15	0.22	0.31	0.10	0.32	0.67	0.75
2.81	1.55		1.17	0.23	0.20	0.37	0.17	0.47	1.40	1.43
2.03	1.16	1.5	0.72	0.20	0.28	0.42	0.21	0.50	1.00	1.06
1.37	0.78		0.64	0.19	0.30	0.18	0.18	1.00	0.70	0.78

Stresses are in kg/cm^2

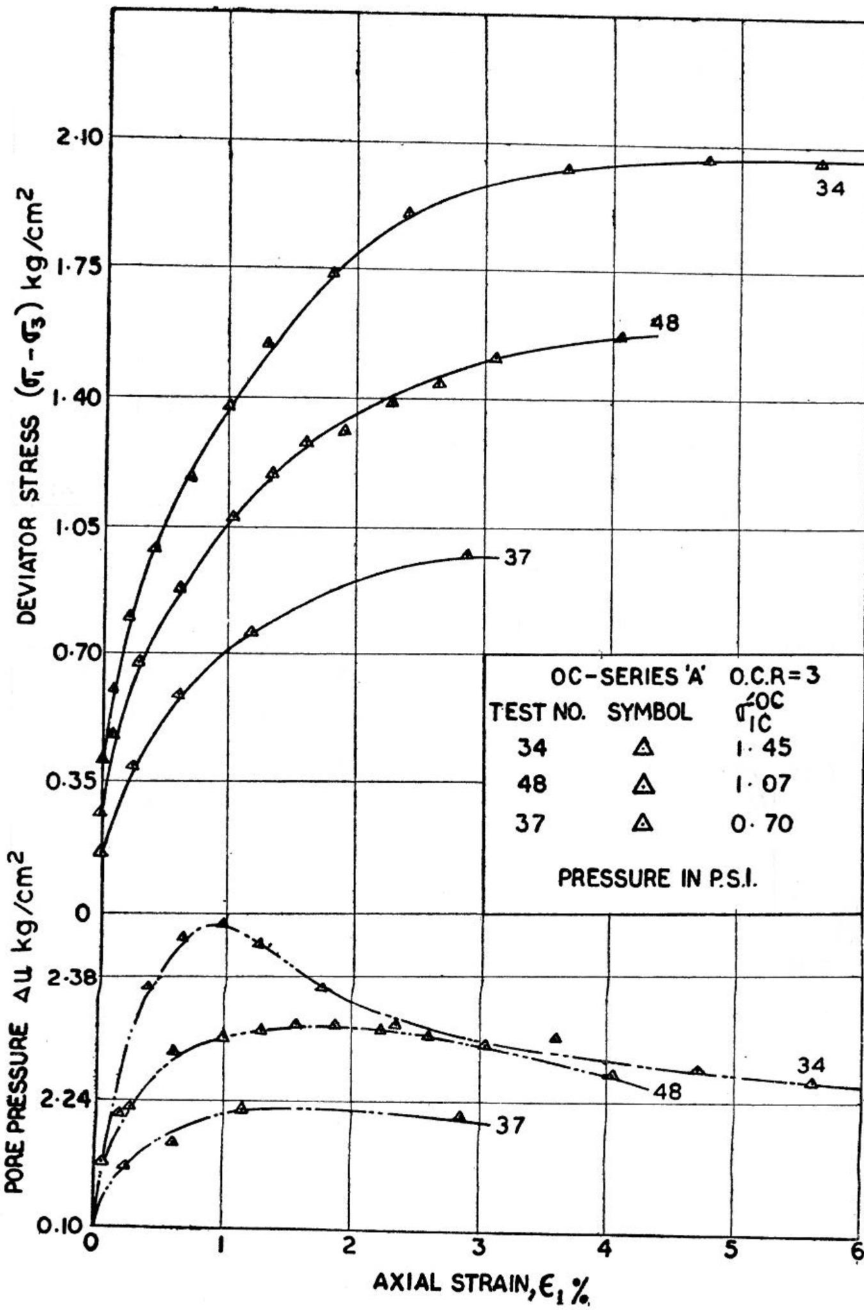

FIGURE 8 : Stress-strain curves for OC—Series A.

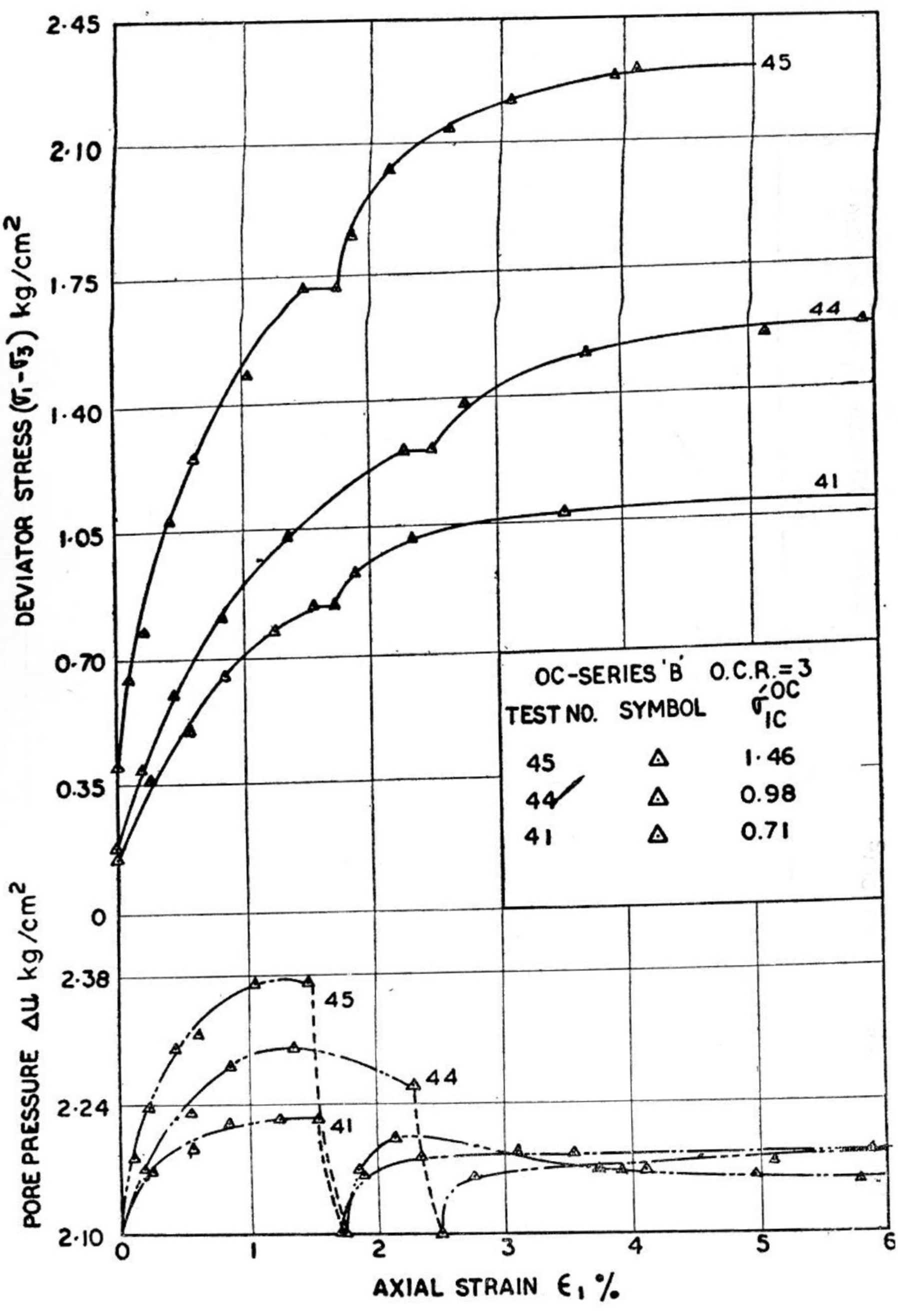

FIGURE 9 : Stress-strain curves for OC—Series B.

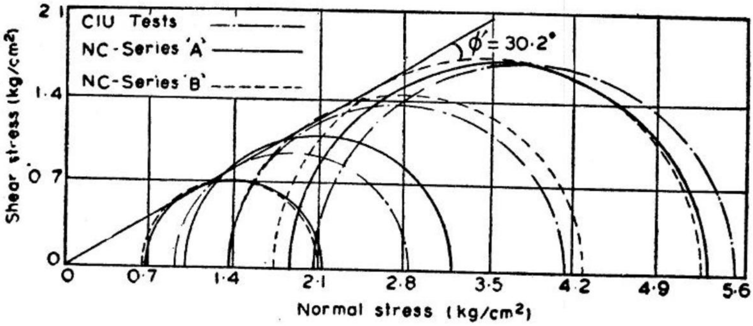

FIGURE 10 : Mohr's circles for NC samples.

The tests of OC—Series A similarly give $C' = o$ and $\phi' = 32°$, 28° and 30.5° corresponding to OCR=3.0, 2.0 and 1.5 respectively. The average value of $\phi' = 30.3°$ is close to the value of 30.2° obtained for the NC samples. This seems to indicate that the strength parameters of the kaolin tested are not dependent on the stress history of the soil.

The value of $C' = o$ even for the over-consolidated soil may be attributed to the fact that the kaolin used consisted primarily of silt sized particles and there was very little clay present.

Strength of In-Situ Samples

The shear strength C_{u_i} of the field elements tested in NC—Series A are found to increase, as expected with initial vertical effective stress and the observed values compare favourably with the values calculated from Equation (1). However, the *K*o value for the kaolin used being low (0.46), the field elements after *K*o-consolidation had already a considerable amount of deviator stress acting on them and further deviator stress required to take the samples to failure were, therefore, small. For example, the sample which was initially consolidated to $\sigma_{ic}' = 5.23$ kg/cm² and $\sigma'_{3c} = 2.41$ kg/cm² had an initial deviator stress of 2.82 kg/cm² and required a further deviator stress of only 0.65 kg/cm² prior to failure the total deviator stress at failure being 3.47 kg/cm² (Table I).

The in situ undrained strength C_{v_i} observed for the OC — Series A tests are summarized in Table III. Also included in the table are the values of C_{u_i}, calculated from Equation (1). As is evident the calculated and observed values agree fairly well, the difference being within an acceptable limit of 10 percent.

Increased In-Situ Undrained Strength C_u

In Tables III and IV are summarized the values of increased in situ strength C_u observed for the samples of NC—Series B and OC—Series B. Also included in the table are the values of C_u calculated from Equation (2). It can be seen that although there is good agreement between the calculated and observed values, the actual strength increases have not been great. This is due to the fact that the gain in strength is primarily a function of the increase of effective stress caused by the dissipation of pore pressure set-up in the first stage of loading, as indicated by the term U_i in Equation (2). In the tests conducted in the present investigation only the vertical effective stress was increased in the first stage of loading. The pore pressure developed was thus small. In a field problem, of course, there would be simultaneous increase of lateral stress during loading which would set up greater pore pressure. The gain in strength is, therefore, likely to be more significant.

Pore Pressure Coefficients A_{f_1} and A_{f_2}

Normally Consolidated Samples

The pore pressure coefficient at failure A_{f_1}, for the samples tested in NC—Series A was between 0.84 and 0.86 with an avergae value of 0.85 (Table V). The value of A_{f_1} for the first stage of loading in the NC—Series B tests was 0.74 while the value of A_{f_1} corresponding to the second stage of loading was 0.93 (Table V). The lower value of A_{f_1} for the Series 'B' test compared to the Series 'A' tests may be due to the fact that these samples were not actually taken to failure, the applied deviator stress being 67 — 81 percent of that required for failure.

It can be seen, however, that the values of A_{f_1} obtained from Series A and Series B and the value of A_{f_2} obtained from Series B are not significantly different, varying as they do, only between 0.74 and 0.93, the effect of stage loading does not, therefore, appear to have any major effect on the pore pressure coefficient and in predicting the gain in strength during second stage of loading from Equation (2), A_{f_1} and A_{f_2} can be taken to be approximately equal.

TABLE V

Pore Pressure Coefficients A_{f_1} and A_{f_2} : NC Samples.

$\bar{p}'_c$ (kg/cm²)	A_{f_1} (Ser. A)	Average A_{f_1}	A_{f_1} (Ser. B)	Average A_{f_1}	A_{f_2}	Average A_{f_2}
2.06	0.84		0.83		0.85	
4.13	0.85	0.85	0.66	0.74	0.88	0.93
5.23	0.86		0.74		1.05	

TABLE VI

Pore Pressure Coefficients A_{f_1} and A_{f_2} : OC Samples.

OCR	$lc'A$ (kg/cm^2)	A_{f_1} (Ser. A)	Average A_{f_1}	A_{f_1} (Ser. B)	Average A_{f_1}	A_{f_2}	Average A_{f_2}
3	1.46	0.10		0.21		0.10	
	0.98	0.13	0.13	0.15	0.18	0,27	0.24
	0.71	0.16		0.19		0.36	
	2.10	0.12		0.13		0.22	
2	1.58	0.15	0.17	0.14	0.16	0.30	0.28
	1.03	0,24		0.22		0.32	
	2.81	0.23		0.20		0.47	
1.5	2.03	0.21	0.22	0.28	0.26	0.50	0.66
	1.37	0.21		0.30		1.00	

OVER-CONSOLIDATED SAMPLES

The values of A_{f_1} as given in Table VI and as obtained from OC-Series A and OC-Series B tests, are not greatly dependent on the stress level at any over-consolidation ratio but increases with decreasing OCR. The values of A_{f_2}, however, are considerably greater than A_{f_1} for the corresponding OCR. This is only to be expected because the second stage loading causes the stress history of the OC samples to change in the positive direction, thereby reducing the OCR which should be associated with a corresponding increase of A_{f_2}. The effect of over-consolidation is less prominent for a slightly OC soil, e.g., OCR$=1.5$ and dissipation under first stage loading tends to wipe out the over-consolidation stress history to a considerable extent thus bringing the specimen nearer its normally consolidated stage. The value of A_{f_2} thus increases to 0.66.

That the pore pressure coefficients A_{f_1} and A_{f_2} are primarily function of the over-consolidation ratio can be clearly seen from Figure 12. In this figure the values of A_{f_1} have been plotted against the over-consolidation ratio corresponding to the beginning of the first stage of loading while the values of A_{f_2} have been plotted against the reduced values of OCR given by the effective vertical stress prior to the second stage of loading. It is evident that the relationship between OCR and A_{f_1} and A_{f_2} can be represented by a single curve and the values of A_{f_1} and A_{f_2} to be used with Equation (2) can be obtained for the appropriate OCR corresponding to the beginning of the first and second stage of loading.

Summary and Conclusions

The factors affecting the gain in strength of normally consolidated and over-consolidated kaolin during stage loading have been studied. The gain in strength is primarily dependent upon the pore pressure set-up in the first stage of loading. Gangopadhyay et al (1969) theoretical expression can reliably predict the gain in strength if appropriate values of A_{f_1} and A_{f_2} are known.

For normally consolidated clays the values of A_{f_1} and A_{f_2} will approximately be equal and a knowledge of A_{f_1} will be sufficient to predict the gain in strength by Equation (2).

For over-consolidated clays, however, A_{f_1} *will not generally be equal* to A_{f_2}. Tests should, therefore, be run to obtain a relationship between over-consolidation ratio and the parameters A_{f_1} and A_{f_2}. For the kaolin tested it appears that at low OCR A_{f_1} and A_{f_2} are much sensitive to OCR (Figure 11) while for OCR greater than 2, A_{f_1} and A_{f_2} are not greatly dependent on the latter.

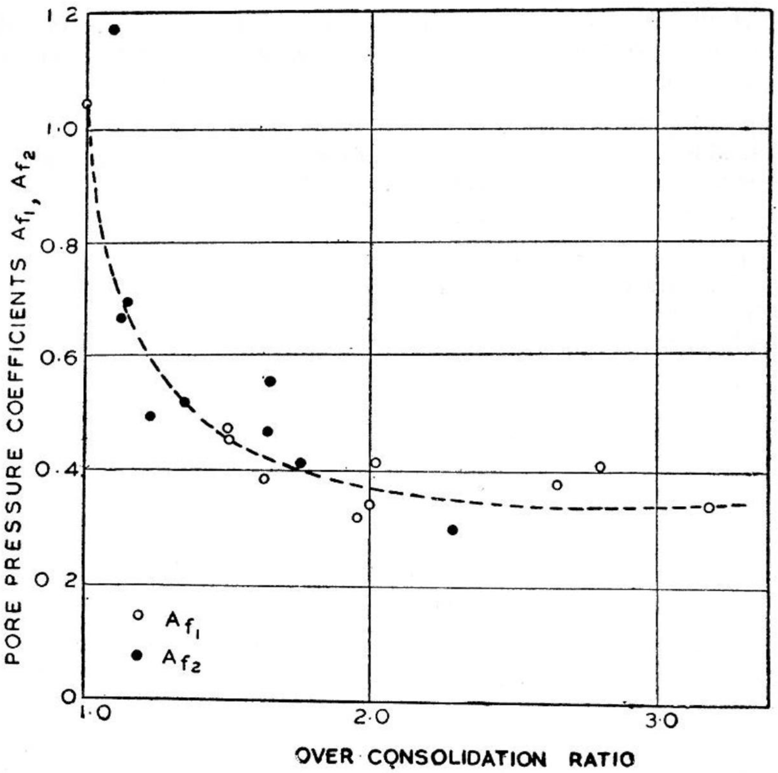

FIGURE 11 : Relationship between pore pressure coefficient retio and OCR.

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Notations

A_{f_1} = Pore pressure coefficient A after first stage undrained loading in Series 'B' and at failure in Series 'A'.

A_{f_2} = Pore pressure coefficient A at failure in Series 'B'.

C_{u_i} = In situ undrained shear strength.

C_u = Increased in situ undrained shear strength.

C' = Effective cohesion component.

$\bar{P}_o$ = Effective vertical stress on a soil element, at a depth, at rest condition.

K_o = Coefficient of earth pressure at rest.

u_n = Neutral pore pressure at the same depth.

P_1 & P_3 = Increase in vertical and horizontal stresses on the soil element due to preload.

u_i = Initial pore water pressure set-up due to the increase in vertical and lateral stresses, i.e., P_1 and P_3.

u_t = Residual excess pore water pressure after a pause of *time t_p under preload.*

σ_1 & σ_3 = Increase in vertical and horizontal total stresses on the soil element at failure.

u = Total pore pressure at failure.

ϕ' = Effective angle of shearing resistance.

U = Average degree of consolidation.

$\in_1$ = Axial strain.

I_p = Plasticity Index.

OCR = Over-consolidation ratio.

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