Indian Geotechnical Journal, 28 (2), 1998

Technical Note

Estimation of Overconsolidation Ratio of Saturated Uncemented Clays from Simple Parameters

M. Chetia* and P.K. Bora†

Introduction

Soils primarily being particulate media, the stresses to which they have been subjected to, the environment in which they are formed and the time that has elapsed in the geological time scale, at different stages in the formation of sedimentary deposits, have all been recognized as potential factors that impart their effects to the natural deposits encountered. It is very well known that the equilibrium state of the in-situ deposits is influenced by stress, time and environment. They are neither mutually exclusive processes nor the effects are cumulative in nature. This investigation primarily considers the effects of stress history only and their characterization and assessment in terms of their over consolidation ratios. The soil samples were taken from natural deposits which were marked by the absence of saline environment and low sensitivity (degree of sensitivity below 4) – factors which are indicative of uncemented soils.

Previous Work on OCR

Widespread influence of OCR on soil properties has attracted considerable research on this topic. Table 1 gives a brief chronological list of some of these studies.

* Lecturer, Department of Civil Engineering, Assam Engineering College, Gauhati – 13, Assam, India.

† Retd. Professor & Head, Department of of Civil Engineering, Assam Engineering College, Gauhati – 13, Assam, India.

Table 1

S.No	Reference	Parameter studied in association with OCR
1	Samsioe (1953)	Coeflicient of earth pressure at rest (K_0)
2	Zeevaert (1953)	Coefficient of earth pressure at rest (K_0)
3	Ladd (1954)	Soil modulus (E)
4	Skempton (1954)	Pore pressure parameter at failure (A_f)
5	Kjellman and Jakobson (1955)	Coefficient of earth pressure at rest (K_0)
6	Henkel (1959)	Pore pressure parameter at failure (A_f)
7	Skempton (1961)	Coefficient of earth pressure at rest (K_0)
8	Lambe (1963)	Pore pressure parameter at failure (A_f)
9	Neyer (1963)	Coefficient of earth pressure at rest (K_0)
10	Hendron (1963)	Coefficient of earth pressure at rest (K_0)
11	Brooker and Ireland (1965)	Coefficient of earth pressure at rest (K_0) and plasticity index (PI)
12	Terzaghi and Peck (1967)	Undrained shear strength.
13	Lambe and Whitman (1969)	Soil modulus (E) and Poisson's ratio (μ)
14	D'Appolonia, Poulos and Ladd (1971)	Soil modulus (E) and Poisson's ratio (μ).
15	Sadd (1971)	Effective cohesion (c')
16	Bjerrum (1972)	Plasticity index (PI) and secondary compression.
17	Leonards (1976)	Width of the foundation (B) and the thickness of clay layer (Ht).
18	Meyerhof (1976)	Coefficient of earth pressure at rest (K_0)
19	Mesri, Erich and Choi (1978)	Swelling index (C_s)
20	Sheriff and Ishibashi (1981)	Coefficient of earth pressure at rest (K_0) and two soil parameters (α and λ)
21	Lee et al. (1983)	Coefficient of earth pressure at rest (K_0)
22	Sully, Campanella and Robertson (1988)	Pore pressure difference (PPD)

Phenomenological Approach

As stated before, the major objective of this study is to establish an empirical relationship between the OCR of the soils and the ratio e/e_L with the help of laboratory experiments. The reason behind the selection of this procedure is the examined relationship between e/e_L ratio and various soil parameters (Nagaraj et al., 1990). Moreover, there is the possibility that the natural water content considered in relation to the liquid limit and/or plastic limit may give some idea of the degree of over consolidation (Lambe and Whitman, 1969).

For determination of OCR, natural moisture content (W_n), bulk density (γ), specific gravity (G), liquid limit (W_L), plastic limit (W_p), unconfined compressive strength (q_u), in-situ voids ratio (e), voids ratio at liquid limit (e_L) respective Indian standard codes were used (Chetia, 1995).

Prediction of soil behaviour from simple parameters has been of abiding interest in soil mechanics. Recent past has witnessed a growing emphasis in this direction as success in this effort for an exceedingly complex material like soil is extremely rewarding. It is in this context that soil behaviour is often examined from micro-mechanistic considerations with efforts directed at establishing links between micro and macro parameters. Such a phenomenological approach has the advantage of striking a compromise between the difficult-to-achieve purely analytical approach and the highly locale-specific purely empirical approach divorced from deeper physical moorings. One feature usually incorporated in such a phenomenological approach is a unique datum or bench mark state of a material with relation to which its behaviour in all other states can be generalised. Reference to the inferential parameter liquid limit as such a datum state for fine-grained soils has been made by many (Terzaghi, 1926; Skempton, 1953; Seed et al., 1964; Nagaraj and Jayadeva, 1981; Warkentin, 1961; Russel and Mickle, 1970; Wroth and Wood, 1978; Whyte, 1982).

Nagaraj et al. (1990) have aptly summarised the unique conditions pertaining to the liquid limit state of soil and have used voids ratio (e_L) at liquid limit as a normalising parameter in the generalized state parameter (e/e_L). Nagaraj and Murthy (1986) has proposed the following generalised relationship to predict preconsolidation pressure of overconsolidated saturated uncemented soils:

$$e/e_L = 1.122 - 0.188 \log \sigma_c - 0.0463 \log \sigma \quad (1)$$

where e/e_L = the generalized soil state parameter.

σ_c = the preconsolidation pressure in kPa.

σ = the effective overburden pressure in kPa.

The relationship between swelling and liquid limit of clays has been studied by several researchers (Lambe and Whitman, 1969; McDowell, 1956; Vijayvergiya and Ghazzaly, 1973). In brief and in very general terms, the swelling of the soil increases with the increase of its liquid limit. Of course the amount of swelling will be dependent on the magnitude difference between the past and present effective overburden pressures. In other words when a soil stratum in a deposit increases in voids ratio due to reduction in effective overburden pressure the new voids ratio will be a function of the liquid limit of the soil and its past and present effective overburden pressures. Therefore the ratio e/e_L will tend to be a constant for a fixed overburden pressure and over consolidation ratio.

This conclusion can also be derived analytically from Eqn. (1) as follows:

Since $$e/e_L = 1.122 - 0.188 \log \sigma_c - 0.0463 \log \sigma$$

Therefore $$e/e_L = \log 10^{1.122} - \log \sigma_c^{0.188} - \log \sigma^{0.0463}$$

$$= \log \frac{10^{1.122}}{\sigma_c^{0.188}\, \sigma^{0.0463}}$$

$$= \log \frac{10^{1.122}}{(\mathrm{OCR})^{0.188}\, \sigma^{0.188}\, \sigma^{0.0463}}$$

or $$10^{e/e_L} = \frac{10^{1.122}}{(\mathrm{OCR})^{0.188}\, \sigma^{0.0463}}$$

or

$$\mathrm{OCR} = \left[\frac{10^{(1.122 - e/e_L)}}{\sigma^{0.2343}}\right]^{1/0.188} \qquad (2)$$

From Eqn. (2) it is clear that the OCR is not a unique function of e/e_L ratio, it also depends on σ. From literature survey too it is found that there is no exclusive relationship between OCR and e/e_L ratio.

Using Eqn. (2), OCR versus e/e_L plots are drawn keeping σ value constant for each plot. Fig. 1 shows this relationship for different values of σ. In natural scale plot Fig. 1(a) the relationship shows a curvilinear trend

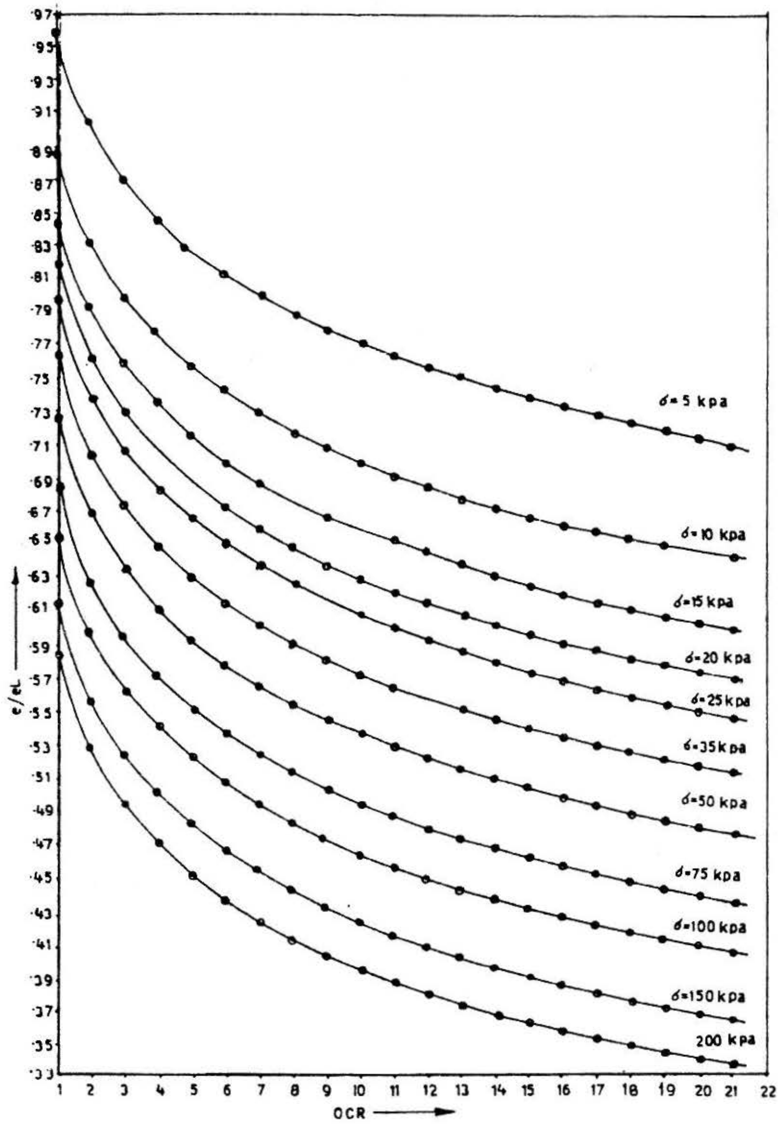

FIGURE 1 (a) : OCR vs. e/e_L Plots for Different Values of σ Computed from Equation 1

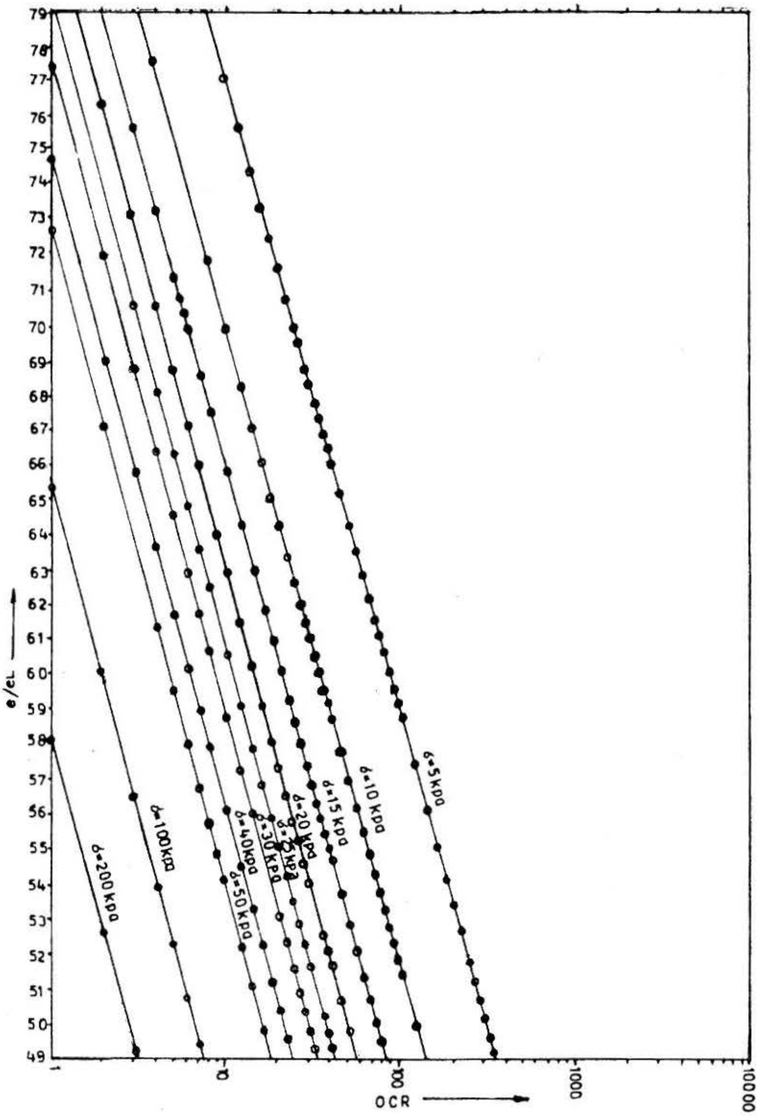

FIGURE 1 (b) : OCR vs. e/e_L Plots for Different Values of σ Computed from Equation 1 and in the semi-log plot Fig. 1(b) the same relationship becomes linear. From the semi-log plot the following linear relationship is established:

$$e/e_L = -0.187962 \log(OCR) + C \quad (3)$$

where $C = e/e_L$ where OCR is unity.

It is observed that the value of C changes as the value of overburden pressure σ changes. Fig. 2 shows the relationship between σ and C (i.e. e/e_L at OCR = 1). From Fig. 2(b) the following equation was obtained:

$$C = 1.122 - 0.234 \log \sigma \quad (4)$$

This is incidentally the equation that was proposed to define and predict the compressibility of normally consolidated saturated uncemented soils (Nagaraj and Murthy, 1986).

Figures 1 and 2 provide a basis for estimation of OCR values. From Fig. 2(b), C value corresponding to a particular σ value can be obtained, and from this with the help of Fig. 1(b), OCR value can be estimated for a particular value of e/e_L.

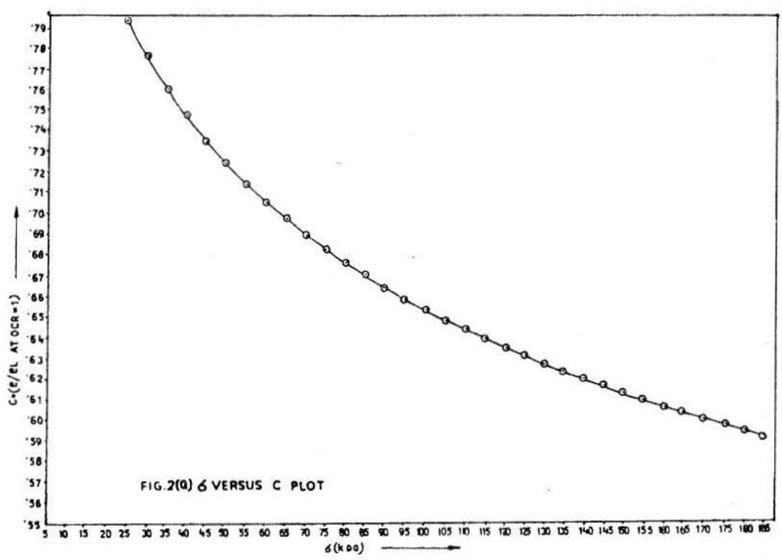

FIGURE 2 (a) : σ vs. C Plot

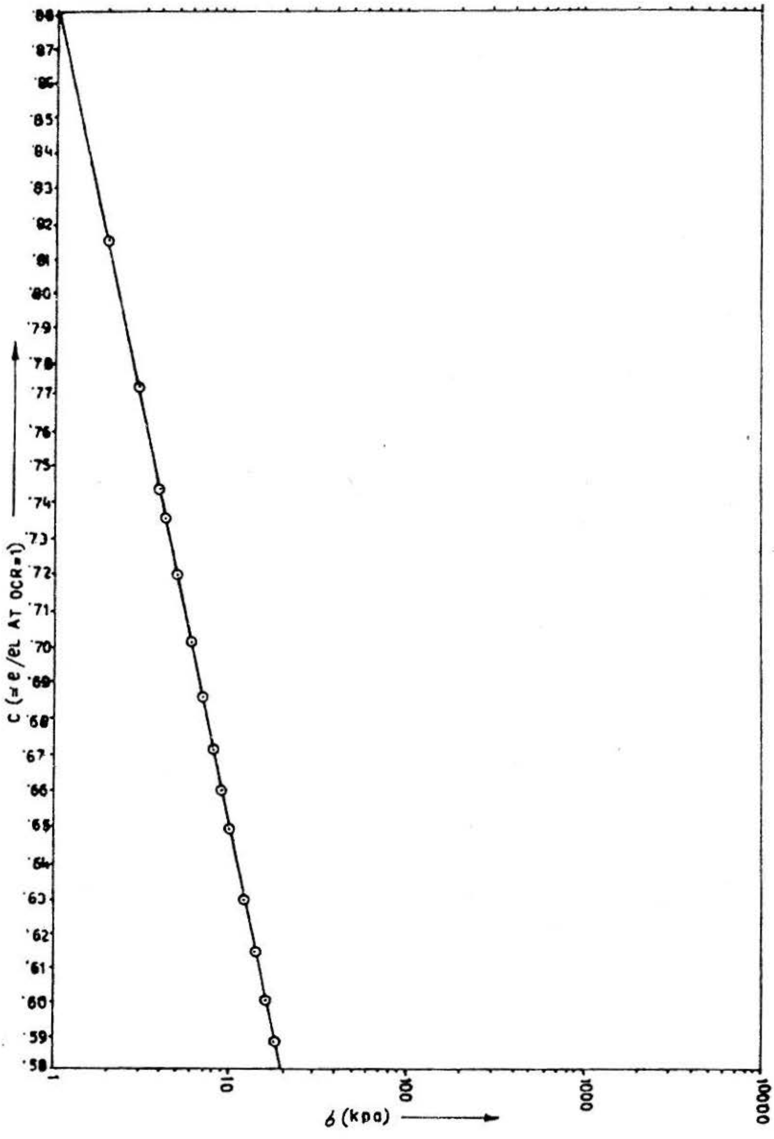

FIGURE 2 (b) : σ vs. C Plot

The scope of this paper was restricted to saturated uncemented soils. However, it is apparent that such relationships can be extended to cemented saturated and uncemented partly saturated soils too.

Effectiveness of Eqn.(2) in predicting OCR values has been examined

TABLE 2
Comparison of OCR Values with those Predicted (Data from Wroth, 1979)

S.No. (1)	Wn % (2)	WL % (3)	s (kPa) (4)	OCR (S)	OCR predicted from Eqn. 2 (6)	Remarks (7)
1	22	27	22	22.6	–	Normally consolidated
2	22	48	49	27.8	26.5	
3	23	49	59	14.6	18.4	
4	22	49	61	21.1	22.6	
5	22	49	71	17.5	18.7	
6	23	46	79	9.6	8.8	
7	24	50	88	10.8	8.8	
8	23	45	94	6.2	6.2	
9	25	52	100	7.65	8.3	
10	21	46	105	10.7	10.5	
11	26	49	106	3.8	4.2	
12	29	51	114	3.3	2.4	
13	28	51	120	3.2	2.9	
14	30	53	121	2.7	2.3	
15	22	41	130	4.75	3.0	
16	20	38	143	4.1	3.1	
17	21	41	155	3.1	3.3	
18	22	41	170	1.75	2.2	
19	15	36	180	3.00	8.7	

with the help of published literature. Wroth (1979) proposed a method based on critical state concept for prediction of preconsolidation pressure. For the present work Wroth's data have been computed for presentation in OCR form and Eqn.(2) has been used to compute the same OCR values for comparison. Table 2 shows the results. It is apparent that there is good agreement between the two sets of OCR values.

An experimental investigation programme was also undertaken for verification of effectiveness of Eqn.(2). The following is a brief description of it.

Experimental Investigations

Samples of soil, both disturbed and undisturbed, were obtained from boreholes made at various places of Guwahati from depths varying from 0.93m to 8.00m (Courtesy: DILIGENT GROUP). The soil layers from which the samples were taken were in all cases overlain by multiple layers of varying properties and barring one case all samples were taken from below the water table.

The bore holes made to collect the soil samples were made by wash boring, auguring and pit excavation. After making the bore holes the samples were collected into the sampler by static penetration method. Thin-walled tube samplers were used to collect the soil samples. Samplers of 77mm outside diameter and 1.50mm thickness were used to collect the consolidation test samples to reduce sampling disturbance substantially. A thin-walled tube sampler of 35mm internal diameter and 1.50mm thickness was also used to collect the unconfined compression test samples. The area ratio for the first case was 7.94% and for the second it was 17.88%. These are well within the recommended limit of 20% suggested by Terzaghi and Peck (1967). The samples collected for consolidation test and unconfined compression test had undergone minimum disturbance.

Discussion

In this work altogether twelve soil samples were subjected to consolidation test. Unconfined compression test, liquid limit and plastic limit tests were also carried out for each soil sample. It has been found that the moisture contents of all soils tested in the course of this study were near their respective plastic limits. This indicates that the soils tested were generally in an over consolidated state.

Consolidation Curve

From the consolidation test readings the equilibrium voids ratio or final voids ratio at the end of each pressure increment was calculated by the 'height of solids' method. From this analysis the voids ratio (e) versus consolidation pressure ($\log \sigma$) relationship was established. This gave the laboratory compression curve from which the field compression curve was obtained as per IS:8009 (Part I) 1976 to account for the stress release and change in moisture and fabric effects. From the rebound curves it was observed that there was small expansion on unloading. This was an indication that the swelling was due to elastic rebound. Current concepts of causes and effects of swelling in clayey soils postulate the existence of forces of attraction A and the repulsion R between soil particles. With the non-swelling clays characterized by strong A and weak R forces, the clay fabric as determined by inter particle forces and manifested as shearing resistance at inter particle contacts have a great bearing on the consistency limits, shrinkage, compressibility and shear strength characteristics. The increase in effective stress on enhanced inter-particle attraction (brought about by decrease in dielectric constant of the pore medium) also leads to lower compressibility and higher drained strength of the non-swelling clay.

Shear Strength

The shearing resistance of saturated clays is a function of particle contacts or the viscosity of the pore fluid when there are no particle contacts. Allam and Sridharan (1984b) from their observation had inferred that edge-face linkages or mineral-mineral contacts are dominating factors in soils of considerable shear strength. This may be the reason for the large undrained shear strength values exhibited by some soils in this study.

Overconsolidation Ratios

Tables 3 and 4 present a comprehensive summary of the sample details, the tests to which these were subjected and the results obtained therefrom. The highest OCR (3.61) was found at site 5 and the lowest OCR (1.18) was at the site 1. At sites 4 and 7 the soils were not found to be overconsolidated. The soils of other six sites were found to be overconsolidated. Table 5 shows the values of OCR as found from experimental results and the corresponding values of OCR as obtained from Eqn.(2). It is apparent that the values of OCR obtained from experimental results are in reasonably good agreement with those computed from Eqn.(2). Hence it can be concluded that the OCR is not a unique function of e/e_L ratio of the soil but it depends on the existing effective overburden pressure of that soil.

From the results of tests on overconsolidated soil is observed that the values of e/e_L, OCR and σ are satisfying the following empirical equations:

$$e/e_L = -0.187962 \log(\text{OCR}) + (e/e_L)_{\text{at OCR}=1} \quad (5)$$

and

$$e/e_L = 1.122 - 0.234 \log\sigma \quad \text{at OCR} = 1 \quad (6)$$

which were obtained from the equation

$$e/e_L = 1.122 - 0.188 \log\sigma_c - 0.0463 \log\sigma \quad (1)$$

Table 3
A Comprehensive Summary of the Sample Details

Site No./ Depth (m) of sample collection	Site Name	Colour	Smell	Natural moisture content	Sp. Gravity	Unit weight		In-situ		Liquid limit	Plastic limit	Plasticity index	At liquid limit			Soil classification	Number of samples tested
						Bulk	Dry	Degree of Saturation	Void ratio				Degree of Saturation	Void ratio			
(i)	(ii)	(iii)	(iv)	(v) W_n %	(vi) G	(vii) γ kN/m^3	(viii) γ_d kN/m^3	(ix) S_r	(x) e	(xi) W_L %	(xii) W_p %	(xiii) PI %	(xiv) $(S_r)/w_L$	(xv) e_L	(xvi) e/e_L	(xvii)	(xviii)
1/6	Paltan Bazar	Grey	Nil	30.79	2.720	19.50	14.91	1.00	.8375	42	24.0	18.00	1.00	1.142	.7331	CI	1
2/4.5	Fancy Bazar	Grey	Nil	28.81	2.650	19.60	15.22	1.00	.7635	42	25.0	17.00	1.00	1.113	.6860	CI	2
3/5.18	Santipur	Grey	Nil	29.72	2.650	19.20	14.80	1.00	.7876	46	27.10	18.90	1.00	1.219	.6461	CI	1
4/8	North Guwahati	Redish	Nil	21.37	2.560	20.00	16.48	1.00	.5471	32.45	20.10	12.35	1.00	0.8307	.6586	CL	1
5/0.93	Panbazar	Deep Grey	Ml	24. 89	2.614	19.78	15.84	1.00	.6506	33	22.00	11.00	1.00	0.8626	.7542	CL	1
5/3	Panbazar	Grey	Nil	32.68	2.601	18.70	14.09	1.00	.8500	52	27.00	25.00	1.00	1.3525	.6285	CH	1
5/5	Panbazar	Yellowish	Nil	29.65	2.609	19.07	14.71	1.00	.7736	45	25.20	19.80	1.00	1.1741	.6589	CI	1
6/5	Panbazar	Light yellowish	Nil	32.09	2.550	18.53	14.00	1.00	.8183	49	26.00	23.00	1.00	1.2495	.6549	CI	1
7/6	Panbazar	Grey Brownish	Nil	27.07	2.570	19.30	15.89	1.00	.6957	37.82	20.10	17.72	1.00	0.9720	.7158	CI	1
8/5	Chatribari	Brownish and greyish	Nil	29.99	2.620	19.10	14.69	1.00	.7857	43.22	25.20	18.02	1.00	1.132	.6939	CI	2

Table 4
A Comprehensive Summary of the Sample Details

Site No	Depth of sample collection frofn GL m	Existing effective overburden pressure σ or p kPa	Preconsolidation pressure σ_c kPa	Overconsolidation ratio OCR	Liquidity Index I_L	e/e_L	Undrained cohesion C_u kPa	C_u/p	Number of samples tested
(i)	(ii)	(iii)	(iv)	(v)	(vi)	(vii)	(viii)	(ix)	(x)
1	6. 00 (Below WT)	38.05	45	1.18	0.38	0.7331	45.40	1.19	1 (Sample No. 1)
2	4.50 (Below WT)	45.23	(i) 70 (ii) 70	1.55 1.55	0.22	0.6880	127.50	2.82	2 (Sample No. 2 & 3)
3	5.18 (Below WT)	68.26	114	1.67	0.14	0.6461	30.60	0.45	1 (Sample No. 4)
4	8.00 (Below WT)	95.00	70	Not overconsolidated	0.10	0.6586	9.30	0.098	1 (Sample No. 5)
5	0.93 (Above WT)	18.37	56	3.05	0.26	0.7542	12.04	0.66	1 (Sample No. 6)

Table 4 contd.

Site No	Depth of sample collection frofn GL m	Existing effective overburden pressure σ or p kPa	Preconsolidation pressure σ_c kPa	Overconsolidation ratio OCR	Liquidity Index* I_L	e/e_L	Undrained cohesion C_u kPa	C_u/p	Number of samples tested
(i)	(ii)	(iii)	(iv)	(v)	(vi)	(vii)	(viii)	(ix)	(x)
5	3.00 (Below WT)	37.40	135	3.61	0.23	0.6285	82.40	2.23	1 (Sample No. 7)
5	5.00 (Below WT)	55.345	120	2.17	0.23	0.6589	98.89	1.79	1 (Sample No. 8)
6	5.00 (Below WT)	59.29		1.86	0.27	0.6549	25.28	0.43	1 (Sample No. 9)
7	6.00 (Below WT)	55.26	36	Not overconsolidated	0.39	0.7158	260.50	4.71	1 (Sample No. 10)
8	5.00 (Below WT)	53.65	(i) 68 (ii) 65	1.25 1.20	0.27	0.6939	696.50	12.84	1 (Samplee No. 11)

* According to Simons & Menzis (1977) O.C. clays I_L is between 0 and 0.60 and N.C. clays I_L is between 0.60 and 1.

Table 5
OCR Values as found from Experimental Results and the Corresponding OCR Values as obtained from Eqn. (2)

Sample Number	OCR	
	Obtained from Experimental results	Computed from Eqn. (2)
(i)	(ii)	(iii)
1	1.18	1.26
2	1.55	1.76
3	1.55	1.76
4	1.67	1.78
5	Not overconsolidated	Not overconsolidated
6	3.05	2.40
7	3.61	4.62
8	2.17	1.95
9	1.86	1.88
10	Not overconsolidated	Not overconsolidated
11	1.25	1.32
12	1.20	1.32

This study, by implication, has verified the effectiveness of Eqn.(1) as postulated by Nagaraj and Murthy (1986) for overconsolidated uncemented saturated soils. All soil samples in this study were saturated and the e vs. $\log \sigma$ curves in all cases showed uncemented nature of the soils.

Concluding Remarks

This study leads to following conclusions:

(1) The equations used to predict the overconsolidation ratio of saturated uncemented clays in this study were as follows:

$$e/e_L = -0.187962 \log(OCR) + (e/e_L)_{at\ OCR=1} \quad (5)$$

and

$$e/e_L = 1.122 - 0.234 \log \sigma \quad \text{at OCR} = 1 \quad (6)$$

These equations were derived from the equation proposed by Nagaraj and Murthy (1985, 1986)

$$e/e_L = 1.122 - 0.188 \log\sigma_c - 0.0463 \log\sigma \quad (1)$$

in which e/e_L = the generalized soil state parameter

OCR = the overconsolidation ratio

σ = the exiting effective overburden pressure in kPa

σ_c = the pre-consolidation pressure in kPa

e = in situ voids ratio and

e_L = voids ratio corresponding to liquid limit water content (= $W_L G$).

Equations (5) and (6) were verified with published data and also with the experimental data of the present research work. The empirical equations were found to be effective in assessment of OCR values.

(2) Assessment of OCR values from simple parameters as used in this work will prove to be an expedient in many cases where OCR values are needed for proper interpretation of test results or examination of the validity of use of established equations but where OCR determination through conventional consolidation tests may be intolerably expensive and time consuming. By implication, this work reinforces the phenomenological approach proposed by Nagaraj and Murthy (1985, 1986).

Acknowledgement

The work presented in this paper was carried at the Department of Civil Engg., Assam Engineering College, Assam, India.

M/s Diligent Group provided the soil samples for this study from various parts of Guwahati and the authors are grateful for this help.

Special thanks are due to Mr. Diganta Sarma, Assistant Engineer, Road Research Laboratory, Guwahati, for the cooperation extended by him at various stages of this work.

References

ALLAM, M.M. and SRIDHARAN, A. (1984b) : "The Shearing Resistance of Saturated Clays", *Geotechnique*, 34:1:119-122.

BJERRUM, L. (1972) : "Embankments on Soft Ground", *Proc. ASCE Specially Conf. on Performance of Earth and Earth Supported Structures*, 2:1-54.

BROOKER, E.W. and IRELAND, H.O. (1965) : "Earth Pressures at Rest related to Stress History", *Canadian Geotechnical Journal*, 2, Feb:1-15.

CHETIA, M. (1995) : "Estimation of Overconsolidation Ratio of Saturated Uncemented Clays from Simple Parameters", *ME Dissertation*, Civil Engg. Deptt., Assam Engineering College, Guwahati.

D'APPOLONIA, D.J., POULOS, H.G. and LADD, C.C. (1971) : "Initial Settlement of Structures on Clay", *J. Soil Mech. Found. Div., ASCE*, Vol.97, No.SM10.

HENDRON, A.J. Jr. (1963) : "The Behaviour of Sand in One Dimensional Compression", *Ph.D. Thesis*, University of Illinois.

HENKEL, D.J. (1959) : "The Relationships between the Strength, Pore Water Pressure and Volume Change Characteristics of Saturated Clays", *Geotechnique*, 9:119.

KJELLMAN, W. and JAKOBSON, B (1955) : "Some Relations Between Stress and Strain in Coarse Grained Cohesionless Materials", *Proc. Royal Swedish Geotech. Inst.*, No.9.

LADD, C.C. (1964b) : "Stress-Strain Modulus of Clay from Undrained Traixial Tests", *Proc. ASCE*, 90, No.SM3.

LAMBE, T.W. (1963) : "Pore Pressure in a Foundation Clay", *Trans. ASCE*, 128, Part 1, 865.

LAMBE, T.W. and WHITMAN, R.V. (1969) : *Soil Mechanics*, Wiley Eastern Ltd., New Delhi.

LEE, I.K., WHITE W. and INGLES, O.G. (1983) : *Geotechnical Engineering*, Pitman Books Ltd., London.

LEONARDS, G.A. (1976) : "Estimating Consolidation Settlements of Shallow Foundations on Overconsolidated Clays", *Sp. Rep. 163*, Transportation Research Board, USA, 13-16.

MCDOWELL, C. (1956) : "Inter-relationship of Load, Volume Change and Layer Thickness of Soils to the Behaviour of Engineering Structures", *Proc. Highway Research Board 35.*

MESRI, G., ULLRICH, C.R. and CHOI, Y.K. (1978) : "The Rate of Swelling of Overconsolidated Clays Subjected to Unloading", *Geotechnique*, 28:3.

MEYERHOF, G.G. (1976) : "Bearing Capacity and Settlement of Pile Foundation", *J. Geotech. Engg. Div.*, ASCE, 102:G13:137-227.

NAGARAJ, T.S. and JAYDEVA, M.S. (1981) : "Re-examination of One Point Methods of Liquid Limit Determination", *Geotechnique*, 31:3:413-425.

NAGARAJ, T.S. and MURTHY, B.R.S. (1986) : "Prediction of Compressibility of Overconsolidated Uncemented Soils", *J. Geotech. Engg.*, 112, No.4.

NAGARAJ, T.S., MURTHY, B.R.S. and VATSALA, A. (1990) : "Prediction of Soil Behaviour, Part 1 - Development of Generalized Approach", *Ind. Geotech. J.*, 20, No.4.

NAGARAJ, T.S., MURTHY, B.R.S. and VATSALA, A. (1990) : "Prediction of Soil Behaviour, Part 2 - Saturated Uncemented Soils", *Ind. Geotech. J.*, 20, No.1.

NEYER, J.C. (1963) : "Lateral Pressure in Preconsolidated Clay", *MS Thesis*, University of Washington.

NISHIDA, Y. (1956) : "A Brief Note on Compression Index of Soils", *J. SMF Div., ASCE*, 82(SM3), Proc. Paper 1.27.

RUSSEL, E.R. and MICKLE, J.L. (1970) : "Liquid Limit Values of Soil Moisture Tension", *Journal of SMFE Division, ASCE*, 96:967-987.

SAMSIOE, A.F. (1953) : "Report on the Investigation of the Compressibility of the Ground of the Hydro-Electric Power Plant Suir 3", *Proc. 3rd Int. Conf. SMFE*, 3, Zurich, 41-47.

SEED, H.B., WOODWARD, R.J. and LUNDGREN, R. (1964) : "Fundamental Aspects of Atterberg Limits", *Journal of SMFE Division*, 90:6:75-105.

SHERIF, M.A. and ISHIBASHI, I. (1981) : "Overconsolidation Effects on K_0 Values", *Proc. 10th Int. Conf. SMFE*, 1.

SKEMPTON, A.W. and HENKEL, D.J. (1953) : "The Post Glacial Clays of the Thomes Estuary at Tilbury and Shellhaven", *Proc. 3rd Int. Conf. SMFE*, Switzerland.

SKEMPTON, A.W. (1954) : "The Pore Pressure Coefficient A and B", *Geotechnique*, 4:143-147.

SRIDHARAN, A. (1991) : "Engineering Behaviour of Fine Grained Soils - A Fundamental Approach", *Ind. Geotech. J.*, 21:1.

SULLY, J.P., CAMPANELLA, R.G. and ROBERTSON, P.K. (1988) : "OCR of Clays from Penetration Pore Pressures", *J. Geotech. Engg.*, 114:No.2.

TERZAGHI, K. and PECK, R.B. (1967) : *Soil Mechanics in Engineering Practice*, John Wiley and Sons, New York.

VIJAYVERGIYA, V.N. and GHAZZALY, O.I. (1973) : "Prediction of Swelling Potential for Natural Clays", *Proc. 3rd Int. Conf. Expansive Soils*, Haifa, Israel.

WROTH, C.P. and WOOD, D.M. (1978) : "The Correlation of Index Properties with some Basic Engineering Properties of Soils", *Canadian Geotech. J.*, 15:2:137-145.

WROTH, C.P. (1979) : "Correlation of some Engineering Properties of Soils", *2nd Int. Conf. on Behaviour of Offshore Structures*, Imperial College, London, pp.121-132.

ZEEVART, L. (1953) : "Discussion", *Proc 3rd Int. Conf. SMFE*, 3.