

Volume IV Number 1 June 1973



GEOTECHNICAL ENGINEERING

Journal of
SOUTHEAST ASIAN SOCIETY OF SOIL ENGINEERING

Sponsored by
ASIAN INSTITUTE OF TECHNOLOGY

CONTENTS

Papers :	Page
Stability Charts for Inhomogeneous Soil Conditions	
P.K. KRUGMANN and R.J. KRIZEK	1
Traffic Induced Vibrations at Wat Po, Bangkok	
J.D. NELSON and SUVIT VIRANUVUT	15
Field Compressibility of Soft Sensitive Normally Consolidated Clays	
YUDHBIR	31
The Relationship between Undrained Strength and Plasticity Index	
A. SRIDHARAN and S. NARASIMHA RAO	41
Discussion of Papers	55
International Society News	61
Southeast Asian Society News	63
Conference News	64
News of Publications	68
Asian Information Center for Geotechnical Engineering—AGE	71
Book Reviews	74
Notes on Contributions to This Journal	75
Southeast Asian Society of Soil Engineering	76

STABILITY CHARTS FOR INHOMOGENEOUS SOIL CONDITIONS

PETER K. KRUGMANN* and RAYMOND J. KRIZEK†

SYNOPSIS

Based on a $\phi = 0$ analysis with its associated restrictions, a direct search optimization procedure is used to develop a series of charts for evaluating the stability of a system in which a soft soil layer of infinite lateral extent is underlain by a firm substratum and overlain by a sloping embankment.

INTRODUCTION

In principle, the behavior of a given slope can be predicted by analyzing the distribution of displacements or stresses within it. At present, however, there is generally insufficient knowledge of the *in situ* stresses and the stress-deformation-time properties of soils to make this approach practicable. Therefore, certain assumptions are necessary to allow an analytical treatment of the stability problem, and the methods most frequently used are based on the principle of limit equilibrium. According to such methods, a free body is considered to be detached from the slope along an assumed failure surface, and the shear force necessary to balance the external forces, including the weight of the free body, is computed and compared to the available shear strength of the soil. In general, this problem is statically indeterminate, and the various methods of analysis arise, in part, from the differences in the assumptions made to overcome this situation.

When an embankment is constructed rapidly, no drainage and, hence, no dissipation of pore water pressures occur, and a saturated clay behaves as a purely cohesive material with respect to the applied stresses at failure. Under these conditions, an unconsolidated-undrained type of analysis, which is known as a " $\phi = 0$ analysis", is applicable, and the stability problem becomes statically determinate. TAYLOR (1948) used this approach, the limitations of which are outlined by SKEMPTON (1948), to solve the slope stability problem for the case where the embankment and the subsoil of finite thickness have the same shear strength. Included herein is an extension of this early work; following a brief description of the mathematical considerations on which these results are based, a series of stability charts are

* Project Engineer, Soil Testing Services Inc., Northbrook, Illinois, U.S.A.

† Professor of Civil Engineering, Northwestern University, Evanston, Illinois.

Discussion on this paper is open until 1 May 1974.

KRUGMANN AND KRIZEK

given for the case where the undrained shear strengths of the embankment and the subsoil differ.

DETERMINATION OF THE SAFETY FACTOR FOR ONE FAILURE SURFACE

The cross-section considered is shown in Fig. 1, where the embankment and subsoil have the shear strengths c_e and c_s respectively, and γ is the unit weight of the embankment soil. The assumed failure surface is a circular arc which is specified by its radius, R , and the coordinates of its center, (X_C, Y_C) . The factor of safety against sliding along this arc is defined as the ratio of the resisting and driving moments about the center of the circle; this definition is equivalent to the ratio of the available and mobilized shear strengths along the arc.

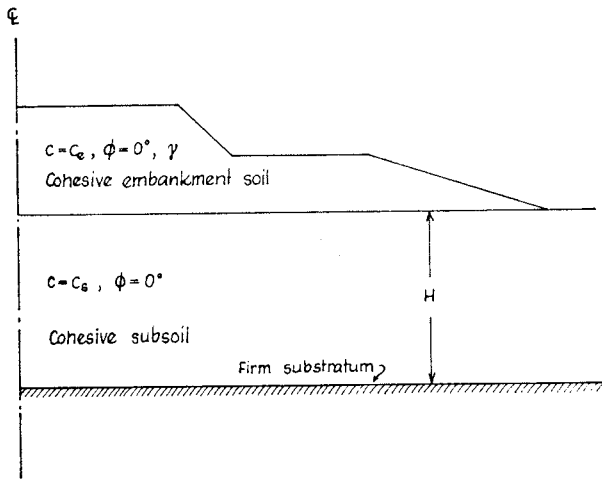


Fig. 1. Typical cross-section of problem considered.

Driving Moments

When the unit weight of the subsoil is a function only of depth, and not of the horizontal distance from the center-line of the embankment, the driving moment, M_D , is caused solely by the applied load, and it can be computed as the sum of the driving moments of the triangles and segments, as indicated in Fig. 2. In the following derivations, it is convenient to define two one-dimensional arrays, X and Y , such that:

- X_1 = X coordinate of the point of intersection of the arc and the ground surface, whereby $X_1 = X_C$,
- Y_1 = Y coordinate corresponding to X_1 ($Y_1 = 0$ for indicated coordinate system),

STABILITY CHARTS

X_2 = X coordinate of the point of intersection of the arc and the embankment boundary,

Y_2 = Y coordinate corresponding to X_2 .

If the arc outcrops beyond the embankment, we have:

X_k = X coordinate of the embankment toe,

Y_k = Y coordinate corresponding to X_k ($Y_k = 0$ for indicated coordinate system), and (X_i, Y_i) with $i = 3, \dots, (k-1)$ are identical to those points which describe the embankment contour between (X_2, Y_2) and (X_k, Y_k) .

If the arc does not outcrop beyond the embankment, we have:

X_k = X coordinate of the point of intersection of the arc and the ground surface, whereby $X_k > X_c$,

Y_k = Y coordinate corresponding to X_k ($Y_k = 0$ for indicated coordinate system),

X_{k-1} = X coordinate of the point of intersection of the arc and the embankment contour,

Y_{k-1} = Y coordinate corresponding to X_{k-1} , and (X_i, Y_i) with $i = 3, \dots, (k-1)$ are identical to those points which describe the embankment contour between (X_2, Y_2) and (X_{k-1}, Y_{k-1}) .

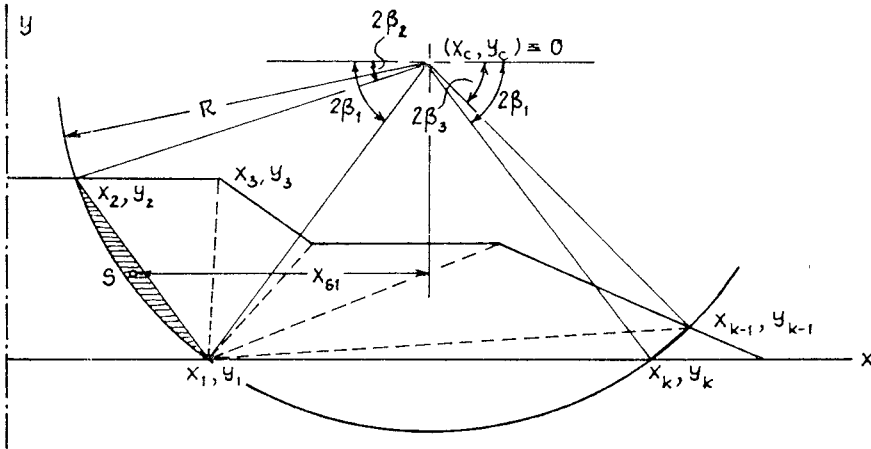


Fig. 2. Embankment cross-section with notation.

Reference to Fig. 2 shows that the part which contributes to the driving moment, M_D , may be subdivided into a number of triangles and the two hatched segments. The area of the triangle with corner points (X_1, Y_1) , (X_i, Y_i) and (X_{i+1}, Y_{i+1}) is given by:

$$A_{Ti} = \frac{1}{2} [X_1(Y_{i+1} - Y_1) + X_i(Y_1 - Y_{i+1}) + X_{i+1}(Y_i - Y_1)] \dots (1)$$

KRUGMANN AND KRIZEK

and the X coordinate of the center of gravity of this triangle is:

$$X_{Ti} = \frac{1}{3}[X_1 + X_i + X_{i+1}] \dots \dots \dots (2)$$

where i goes from 2 to $(k-1)$. The driving moments caused by the sum of the triangles is obtained by summing the products of A_{Ti} and X_{Ti} and multiplying by the unit weight, γ , of the embankment material; thus, since $Y_1 = 0$, we obtain:

$$M_{DT} = (\gamma/6) \sum_{i=2}^{k-1} \left\{ (3X_C - X_1 - X_i - X_{i+1}) [X_1(Y_{i+1} - Y_1) - X_i Y_{i+1} + X_{i+1} Y_i] \right\} \dots \dots \dots (3)$$

Additional moments are exerted by the hatched areas. The horizontal distances between the center of the arc and the centers of gravity of the hatched areas are given by:

$$X_{S1} = \overline{OS}_1 \cos(\beta_1 + \beta_2) = C_1^3 \cos(\beta_1 + \beta_2)/(12A_{S1}) \dots (4a)$$

and

$$X_{S2} = \overline{OS}_2 \cos(\beta_1 + \beta_3) = C_2^3 \cos(\beta_1 + \beta_3)/(12A_{S2}) \dots (4b)$$

where A_{S1} and A_{S2} are the hatched areas, and C_1 and C_2 represent the lengths of the chords between points (X_1, Y_1) and (X_2, Y_2) or (X_{k-1}, Y_{k-1}) and (X_k, Y_k) , respectively. The angles β_1, β_2 , and β_3 are given by:

$$\beta_1 = \frac{1}{2} \text{arc sin } (Y_C/R) \dots \dots \dots (5a)$$

$$\beta_2 = \frac{1}{2} \text{arc sin } [(Y_C - Y_2)/R] \dots \dots \dots (5b)$$

$$\beta_3 = \frac{1}{2} \text{arc sin } [(Y_C - Y_{k-1})/R] \dots \dots \dots (5c)$$

Hence, the moments due to the hatched areas become:

$$M_{DS} = (\gamma/12) \left\{ [(X_1 - X_2)^2 + (Y_1 - Y_2)^2]^{3/2} \cos(\beta_1 + \beta_2) - [(X_{k-1} - X_k)^2 + (Y_{k-1} - Y_k)^2]^{3/2} \cos(\beta_1 + \beta_3) \right\} \dots (6)$$

and the total driving moment, M_D , is obtained by adding Eqs. 3 and 6:

$$M_D = M_{DT} + M_{DS} \dots \dots \dots (7)$$

Resisting Moments and Factor of Safety

Resisting moments develop along the portions of the arc which pass through the embankment and the subsoil. In the case of a purely cohesive embankment material, M_{R1} and M_{R2} are given by :

STABILITY CHARTS

$$M_{R1} = 2c_e R^2 [(\beta_1 - \beta_2) + (\beta_1 - \beta_3)] \dots \dots \dots (8)$$

and

$$M_{R2} = R^2 (\pi - 4\beta_1) c_s \dots \dots \dots (9)$$

Combination of Eqs. 7, 8 and 9 yields the following expression for the factor of safety:

$$FS = (M_{R1} + M_{R2})/M_D \dots \dots \dots (10)$$

DIRECT SEARCH OPTIMIZATION PROCEDURE

The problem to be solved next is how to find that arc along which the ratio between the available and the mobilized shear strength (that is, the factor of safety) is a minimum. The classical approach would be to differentiate the expression for the factor of safety with respect to the three variables X_C , Y_C and R , and then substitute these values back into the original expression. Unfortunately, this approach becomes rather complicated for the problem treated herein, and another minimization technique is employed. Of the several possible procedures described by KATZ et al (1966), the “direct search procedure”, originally developed by HOOKE & JEEVES (1962), has been chosen. The term “direct search” is used

“...to describe sequential examination of trial solutions involving comparison of each trial solution with the “best” obtained up to that time together with a strategy for determining, as a function of earlier results, what the next trial solution will be.”

By varying the radii of trial arcs which have the same center, a minimum factor of safety corresponding to this center can be found. If the same procedure is repeated for different centers, and if the corresponding minimum safety factor is assigned to each point, a set of contour lines of equal values of the factor of safety can be plotted, as shown in Fig. 3. With reference to this figure, the direct search procedure embraces two tactical maneuvers —

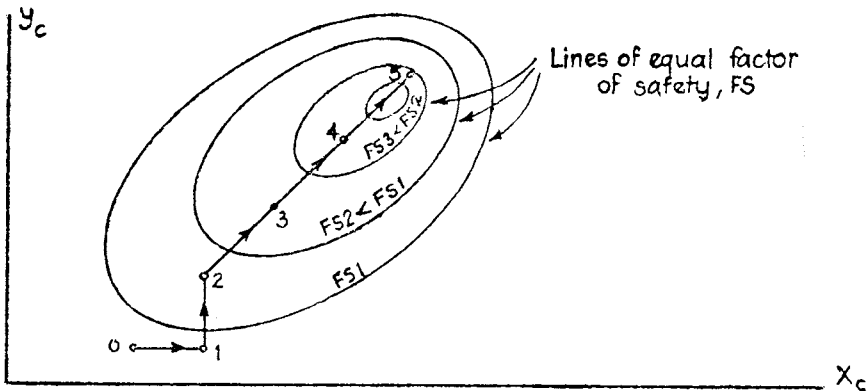


Fig. 3. Illustration of direct procedure.

KRUGMANN AND KRIZEK

“exploratory moves” and “pattern moves” — and these can be described as follows. Starting from the arbitrary base point 0 , an “exploratory move” is made by varying first X_C and then Y_C . If this move is successful, a “pattern

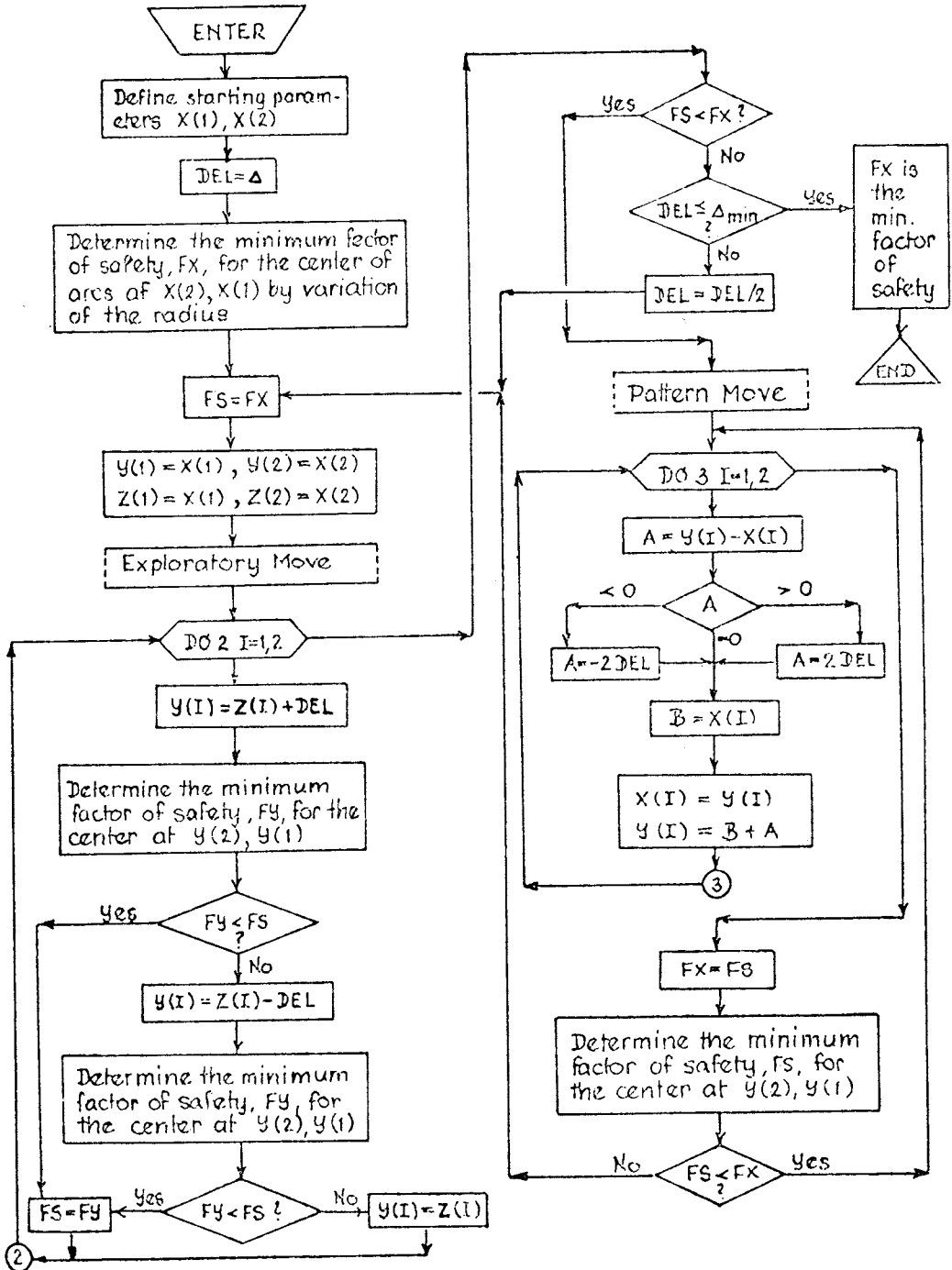


Fig. 4. Flow diagram of direct search procedure.

STABILITY CHARTS

move" directly from point 2 to point 3 is performed. For the example depicted, a number of pattern moves would follow, until point 5 was reached; then, an exploratory move would again be necessary.

Although an arbitrary number of parameters can be varied in this approach, it was found most advantageous to vary only X_C and Y_C in the above described manner and to compute the factors of safety for a fixed number, n , of radii for each center (X_C , Y_C). The programmed sequence of exploratory and pattern moves differs from the scheme developed by HOOKE & JEEVES (1962), and it is therefore presented in Fig. 4 in the form of a flow diagram. The time required to apply the search procedure can be decreased by imposing a few restrictions on the magnitude of possible radii; these are:

- (a) the slip circle must penetrate the subsoil,
- (b) the slip circle cannot penetrate the firm substratum at depth H , and
- (c) part of the slip circle must pass through the embankment.

STABILITY CHARTS

To allow an estimate of the factor of safety against a sliding failure in simple cases, solutions have been obtained for the conditions depicted in Fig. 5. The analyses are based on the assumptions that (a) the embankment of height E is homogeneous with an undrained shear strength c_e , and a unit weight γ , (b) the subsoil is homogeneous and has an undrained strength c_s , (c) the failure surface is circular, and (d) the maximum penetration depth of the failure surface is equal to the thickness of the soft soil layer. The results of these calculations are given in Figs. 6, 7, and 8, where the stability number, N_s , is plotted versus the quotient of the thickness of the soft layer and the

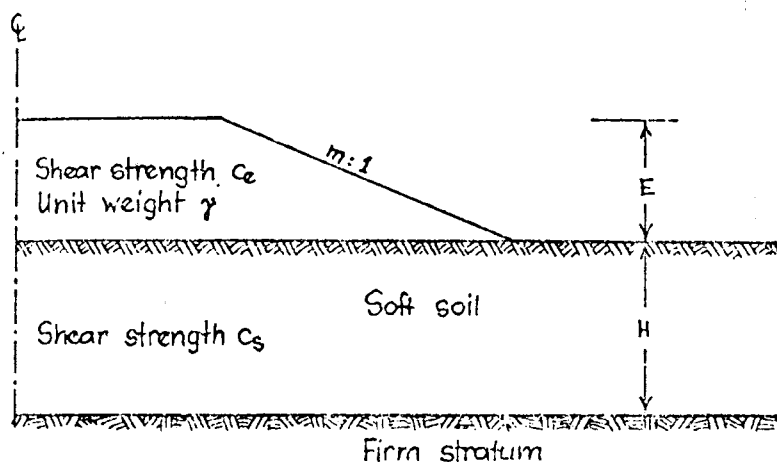


Fig. 5. Notation used in stability charts.

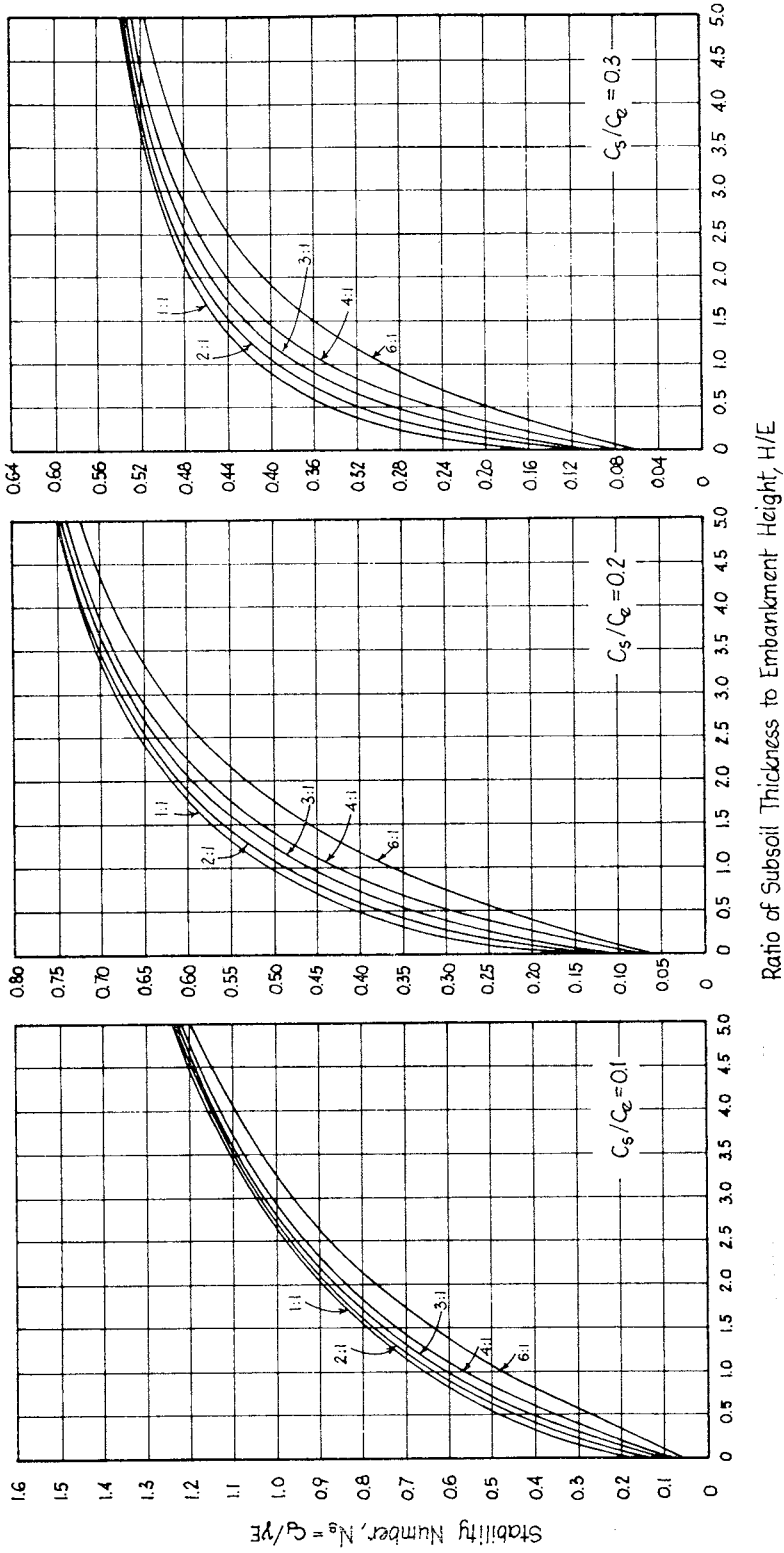


Fig. 6. Stability charts for strength ratios of 0.1, 0.2 and 0.3.

STABILITY CHARTS

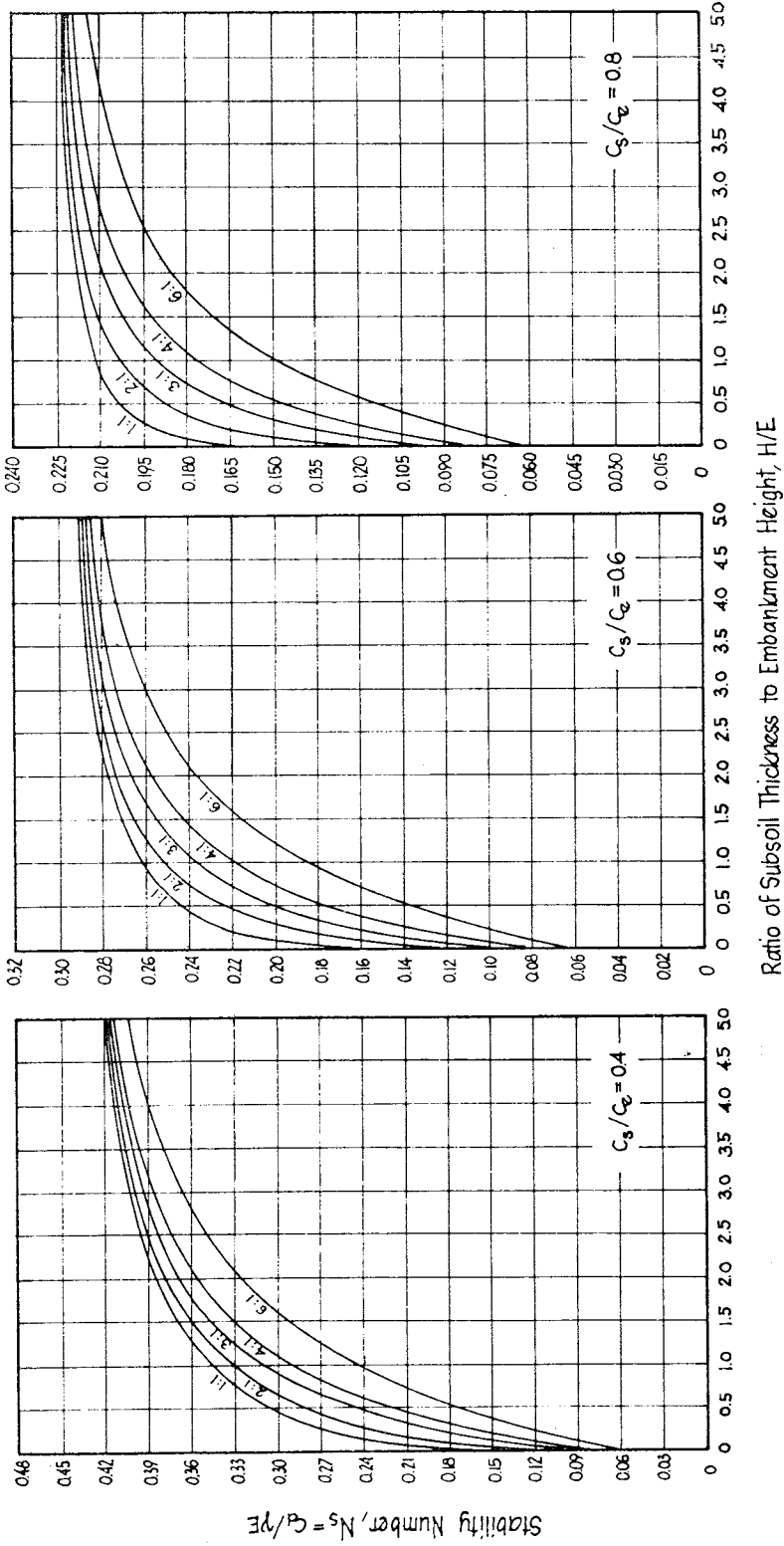


Fig. 7. Stability charts for strength ratios of 0.4, 0.6 and 0.8.

KRUGMANN AND KRIZEK

thickness of the soft soil layer; the results of these computations are depicted in Fig. 9 for $c_e = 1000 \text{ lb/ft}^2$, $\gamma = 125 \text{ lb/ft}^3$, and slopes of 6:1, 4:1 and 2:1. It can be noted that almost all curves are very steep for small values of H . This phenomenon suggests that a sliding surface other than the circular arc might result in smaller maximum possible embankment heights. To get some indication as to when the assumption of a circular sliding surface might be expected to give an unreliable value for the maximum embankment height, a wedge analysis was made. When the section of the embankment bounded by the center-line and the surface of the subsoil is considered as a free body, the only external horizontal forces are the at-rest earth pressures at the center-line, and these must be balanced by the shearing resistance along the base. Based on the assumption of a uniform strength distribution, the resulting equilibrium condition can be solved for the embankment height to give:

$$E = \frac{mc_s}{\gamma K_0} \left(1 + \sqrt{\frac{2\gamma K_0 W}{m^2 c_s} + 1} \right) \dots \dots \dots (12)$$

where K_0 is the at-rest earth pressure coefficient, and W is the half-width of the embankment crest.

The values of H for which the wedge method yields lower possible embankment heights than the assumption of a circular slip surface are indicated by the dashed curves in Fig. 9 for $K_0 = 0.75$. The safety factor associated with a wedge analysis may be defined as the ratio of the required strength of the subsoil to the available strength of the subsoil. For the example problem discussed above (with the additional information that the half-width, W , of the embankment crest is 37.5 ft and K_0 is assumed to be 0.75), the required subsoil strength can be determined from Eq. 12 to be 160 lb/ft^2 , whereas the available subsoil strength is 200 lb/ft^2 ; therefore, the safety factor, FS, is $200/160 = 1.25$.

CONCLUSIONS

Within the scope of this study and its attendant limitations regarding the applicability of a $\phi = 0$ analysis in a given situation and the associated choice of a test procedure for determining the required strength parameters, the following conclusions can be advanced:

1. The direct search optimization technique was found to be very effective for determining minimum factors of safety for the stability of an embankment resting on a weaker subsoil, and results calculated for the limiting case where the embankment and subsoil are homogeneous were shown to be in excellent agreement with previous results reported by TAYLOR (1948).

STABILITY CHARTS

2. Although the stability of a non-homogeneous soil system increases as the soil strengths and the slope angle increase and as the thickness of the weaker subsoil decreases, the beneficial effect of flattening the slope diminishes as the thickness of the subsoil increases. In other words, a smaller inclination of the slope or the use of berms give best results for small to moderate subsoil thicknesses.
3. There is a rapid increase in the stability number and an associated rapid decrease in the maximum possible embankment height for small thickness of the subsoil. This is attributed to the fact that only a small portion of the circular arc passes through the soft subsoil, thereby yielding an unreliably high factor of safety, and it is recommended that a failure mechanism other than a circular arc be assumed in cases where the subsoil layer is thin.

ACKNOWLEDGEMENTS

This work, which comprises part of a more comprehensive research effort (Project IHR - 602) dealing with "Placement Rates for Highway Embankments", was performed in cooperation with the State of Illinois, Department of Transportation, and the U.S. Department of Transportation, Federal Highway Administration. Grateful acknowledgement is given to the Project Advisory Committee, which consisted of Gordon R. Benson, Ralph L. Duncan, John Ebers, Arley G. Franklin, Jorj O. Osterberg and Ernest C. Sell, for their cooperation and guidance throughout the course of this study. In addition, the help and advice of John E. Burke, C.S. Monnier, Robert C. Mulvey and Donald R. Schwartz is sincerely appreciated. However, the opinions, findings, and conclusions expressed in this paper are those of the authors and not necessarily those of any of the above persons or agencies.

REFERENCES

- HOOKE, R. and JEEVES, T.A. (1962), Direct Search Solution of Numerical and Statistical Problems, *Journal Association for Computational Machines*, Vol. 8, No. 2, pp. 212-229.
- KATZ, D.L., CARNAHAN, B., DOUTY, R.T., HAUSER, N., McMAHON, E.L., ZIMMERMAN, J.R. and SEIDER, W.D. (1966), *Computers in Engineering Design Education; Vol. 1, Summary Report*, University of Michigan, Ann Arbor, Michigan.
- SKEMPTON, A.W. (1948), The $\phi = 0$ Analysis of Stability and Its Theoretical Basis, *Proc. 2nd Int. Conf. Soil Mech. Found. Eng.*, Vol. 1, pp. 71-78.
- TAYLOR, D.W. (1948), *Research on Consolidation of Clays*, Serial 82, Dept. Civil Eng., M.I.T., Camb., Mass.

TRAFFIC INDUCED VIBRATIONS AT WAT PO, BANGKOK

JOHN D. NELSON* and SUVIT VIRANUVUT⁺

SYNOPSIS

The Buddha image at Wat Po, Bangkok has begun to exhibit some cracking within the past 4 or 5 years. Because the primary consolidation should have been completed about 100 years ago, it was believed that the distress was caused by vibrations induced by heavy traffic on nearby streets. Measurements of the vibrations at various locations showed that the vibrations currently produced by the traffic were not of sufficient magnitude to cause structural damage. However, it is shown that the vibrations in the past could have been much greater than at present because of the poor condition of the roadway until recently. It is concluded that the structural damage was caused by either one or both of (i) deterioration of the ancient foundation structure, or (ii) settlements caused by a weakening of the soil structure due to the traffic induced vibrations.

INTRODUCTION

Some signs of distress have begun to appear in some of the old buildings and monuments in Bangkok even though the consolidation settlements should have ceased many years ago. In most instances, however, there are busy roads or intersections nearby and, because in the past decade the size and density of traffic has increased considerably, it has been postulated that vibrations from the traffic could be responsible for the distress.

One particular structure that has begun to exhibit some cracks is the Reclining Buddha image at Wat Po in Bangkok. This structure was constructed about 150 years ago and in just the past four or five years the cracks have begun to be noticed. Preliminary investigations by the Department of Public Works of Thailand indicated that vibrations caused by trucks and buses on roads adjacent to the Wat could have been responsible for those cracks. Consequently, an investigation was initiated to observe the nature of the traffic-induced vibrations with the intention of determining whether they were of a magnitude sufficient to cause structural damage. Also, the characteristics of the vibrations were determined for input into the design of any vibration screening barriers that may be desired.

*Associate Professor, +Laboratory Supervisor, Asian Institute of Technology, Bangkok, Thailand.

Discussion on this paper is open until 1 May 1974.

NELSON AND VIRANUVUT
EXPERIMENTAL PROGRAM

Site Description

The Reclining Buddha image is situated in the northwest corner of Wat Po near the intersection of Thai Wang Road and Maha Rat Road. The general location plan of the site is shown in Fig. 1. The base is 140 ft long, 16 ft wide and about 5 ft high. The Buddha image is about 6 ft high at the feet and rises to a height of about 30 ft at the head. It is assumed that the construction is of brick and mortar and covered by plaster. Almost all of the cracks are on or near the head, with the greatest number being near a point where the head rests on one hand. These cracks can be seen in Figs. 2 and 3. One crack also exists on the heel of one foot as shown in Fig. 4.

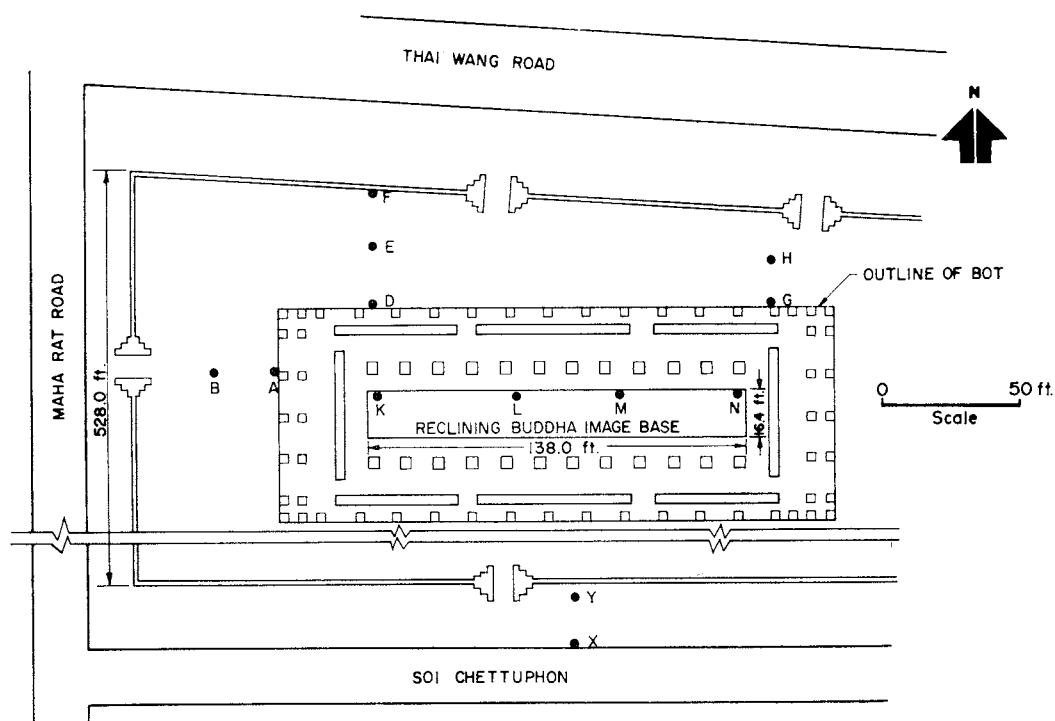
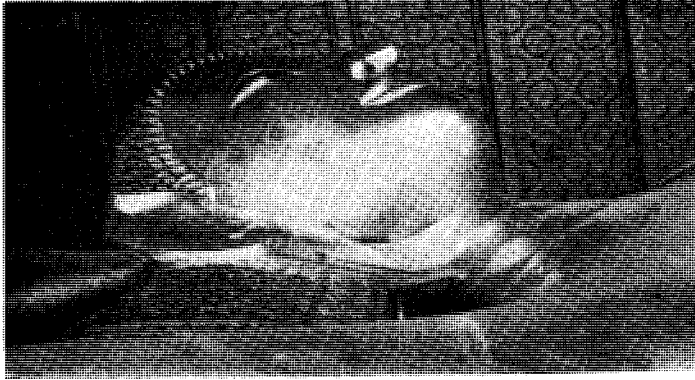


Fig. 1. Location plan of the investigation site.

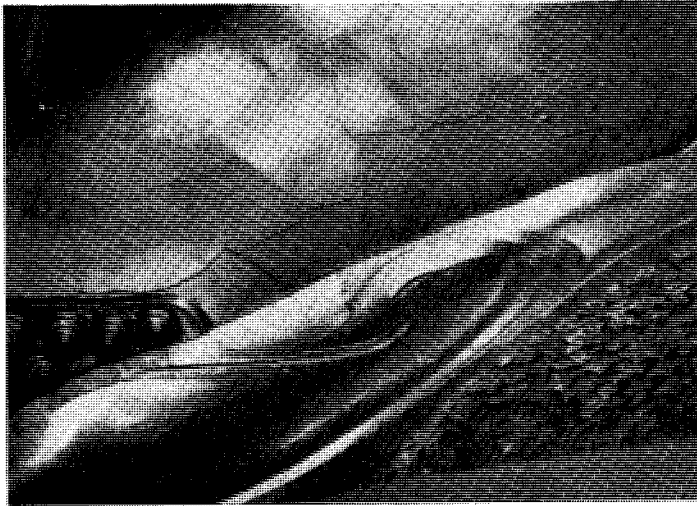
The relative elevations of various points on the base of the Buddha image and the floor of the Bot⁽¹⁾ are shown in Fig. 5. If it is assumed that the base and floor were originally level, it can be seen that relative settlements of almost 1 ft exist. Most of the settlement has occurred at the east end of the Buddha

⁽¹⁾ The Bot is the temple structure sheltering the Buddha Image.

TRAFFIC VIBRATIONS



(a) Cracks on side of face.



(b) Closeup of cracks.

Fig. 2. Head of Buddha image showing cracks.

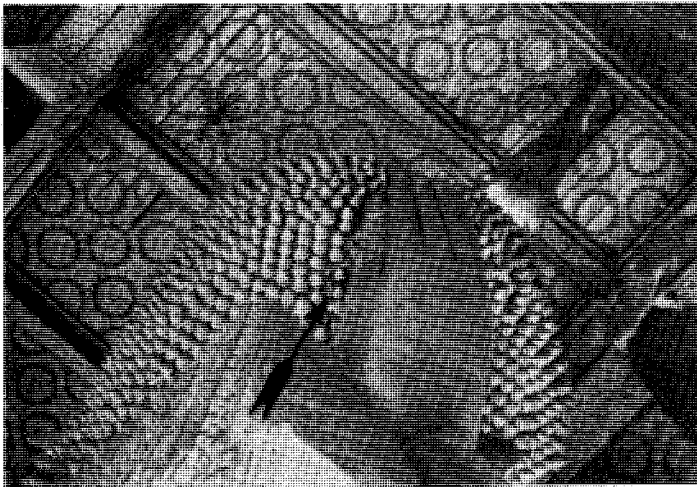


Fig. 3. Crack on back of head.

image (i.e. the end by the head). It should be noted that both the Bot floor and the base have experienced considerable settlement at that end.

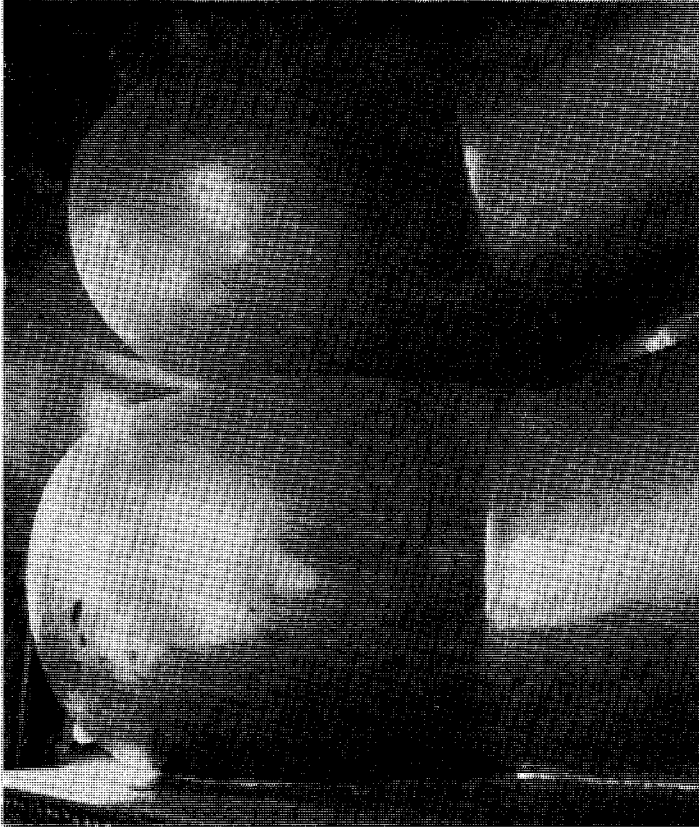
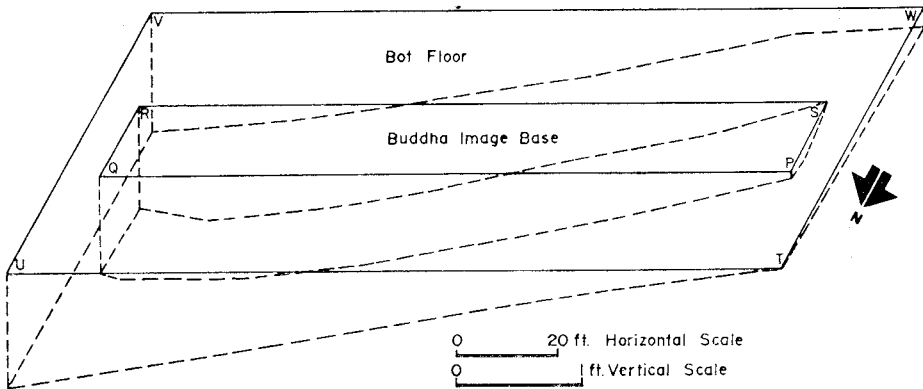


Fig. 4. Crack on heel of foot.



Note: Vertical dotted lines indicate distances from the horizontal

Fig. 5. Relative elevations of Buddha image base and Bot floor.

TRAFFIC VIBRATIONS

Soil Profile

No soil samples were taken at Wat Po. However, a 1 in. diameter pipe was driven into the ground to determine the depth and nature of the fill, and the general soil profile shown in Fig. 6 was then constructed on the basis of borings done nearby at Tha Chang Bridge (CIRIDON, 1972) and Memorial Bridge (BRAND, 1971). Basically, the profile consists of about 4 ft of sand and brick fill underlain by soft clay to a depth of about 60 ft. Below that is stiff clay. Some sand seams and fissures probably exist in the stiff clay and possibly in the soft clay as well.

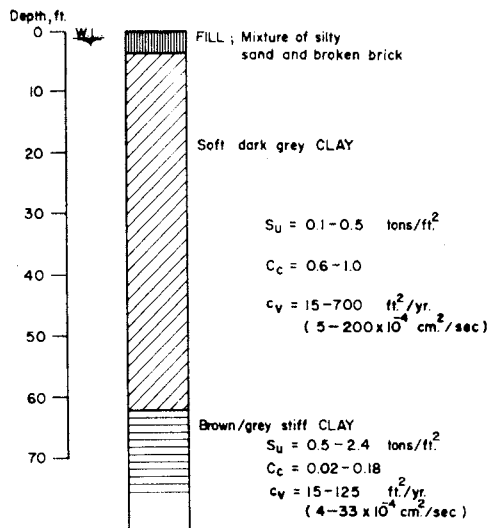


Fig. 6. General soil profile.

Vibration Measurements

The locations of the points at which vibration measurements were made are shown in Fig. 1. The pickup points consisted of 2 in. square steel plates 3/8 in. thick which were cemented to the concrete pavement outside the Bot and on the base of the Reclining Buddha image. Velocity transducers could be screwed onto the pickup plates. The use of fixed steel plates at the pickup points permitted vibration measurements to be made at identical locations each time. Surface wave velocities and coefficients of energy absorption were determined by simultaneously measuring the times of arrival and amplitudes of the stress waves at two points.

Velocity transducers were used to measure the vertical component of velocity of the ground surface at various locations. The signals from the velocity transducers were displayed on an oscilloscope and recorded on a Polaroid camera mounted on the oscilloscope. In order to filter out some high frequency electrical noise a simple R-C filter was used. The filter

NELSON AND VIRANUVUT

attenuated the noise by about 99% but had a negligible effect on the signal produced by the traffic vibrations. A schematic diagram of the instrumentation is shown in Fig. 7, and Fig. 8 shows the equipment set up at one point. Some typical records obtained at various locations are shown in Fig. 9.

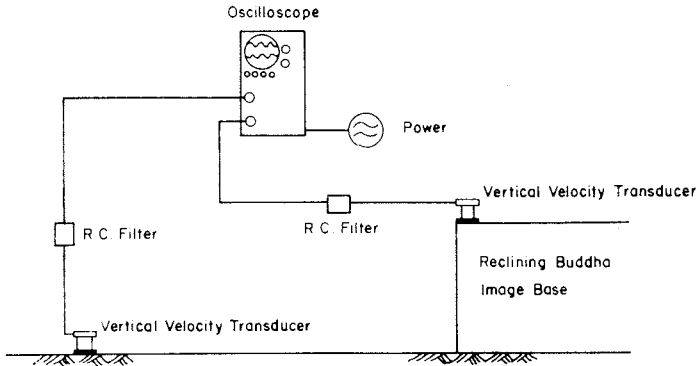


Fig. 7. Schematic layout of instrumentation.



Fig. 8. Instrumentation for vibration measurements.

The period and amplitude of vibrations could be determined directly from the Polaroid record. The circular frequency and displacement and the acceleration amplitudes were calculated from the relationships for harmonic motion, i.e.

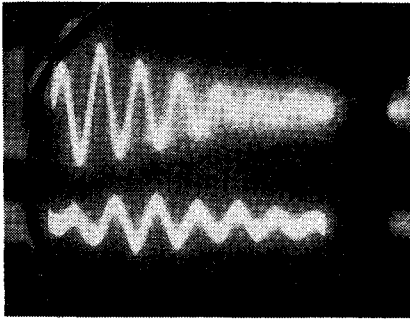
$$\omega = 2\pi f = \frac{2\pi}{T} \dots \dots \dots (1)$$

$$A = \frac{v}{\omega} \dots \dots \dots (2)$$

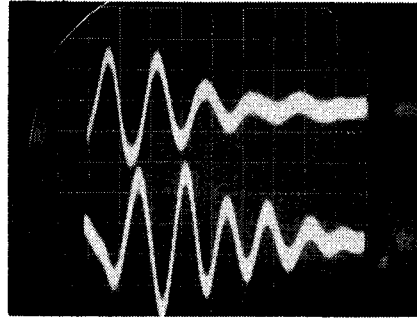
$$a = \omega v \dots \dots \dots (3)$$

where f is the frequency (Hz), ω is the circular frequency (rad./sec.), A is the displacement amplitude (in.), v is the particle velocity amplitude (in./sec.), and a is the particle acceleration amplitude (in./sec.²).

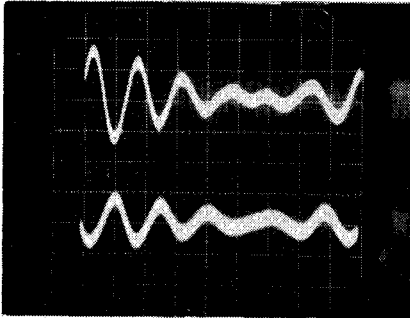
TRAFFIC VIBRATIONS



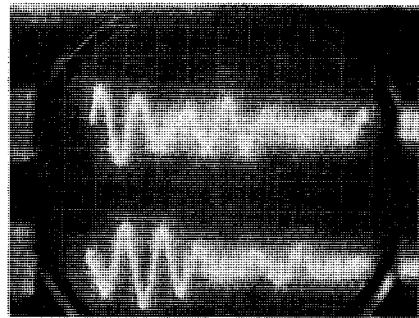
a) Test No. 1
Upper Trace - Location B
Lower Trace - Location A



b) Test No. 7
Upper Trace - Location E
Lower Trace - Location D



c) Test No. 12
Upper Trace - Location G
Lower Trace - Location N



d) Test No. 27
Upper Trace - Location X
Lower Trace - Location Y

Fig. 9. Typical vibration records.

The wave velocity, c , of the surface waves was computed from the differences in times of arrival of the waves at two different points. The wave length, λ , could be determined from the equation:

$$\lambda = c/f \quad \dots \dots \dots (4)$$

The following relationship between amplitude and distance introduced by BARKAN (1962) was used to compute the coefficient of energy absorption:

$$A_r = A_o \sqrt{\frac{r_o}{r}} \exp [-\alpha (r - r_o)] \quad \dots \dots \dots (5)$$

where A_r is the amplitude at a distance r from the vibration source, A_o is the amplitude at a distance r_o from the vibration source, and α is the coefficient of energy absorption.

NELSON AND VIRANUVUT

Table 1. Summary of vibration measurements

Test No.	Sta.	T, m-sec	f, Hz	ω , rad./sec.	A, in. $\times 10^{-5}$	v, in./sec.	a, in./sec. ²	c, ft/sec.	λ , ft	Remarks
1	A	70	14	88	2.2	.0019	.17			
1	B	65	15	94	5.7	.0053	.50			
2	A	75	13	81.7	5.5	.0045	.37			
3	A	75	13	81.7	6.7	.0055	.45			Big bus on Thai Wang Road ; fast
3	K	75	13	81.7	2.4	.0020	.16			
4	D	70	14	88	15.9	.014	1.24			Big bus on Thai Wang Road ; fast
4	E	70	14	88	9.1	.0080	.70			
5	D	70	14	88	5.0	.0044	.39	578	41	
5	E	70	14	88	5.7	.0050	.45			
6	D	80	12.5	78.5	12.3	.0097	.77	422	36	Big bus on both roads; fast
6	E	97	10	62.8	33.0	.0204	1.28			
7	D	80	12.5	78.5	23.6	.019	1.45	439	35	
7	E	80	12.5	78.5	19.1	.015	1.17			
8	E	75	13	81.7	8.9	.0073	.59	732	56	Big bus; slow, bumping
8	F	75	13	81.7	6.5	.0053	.43			
9	E	80	12.5	78.5	11.2	.0088	.69	488	37	
10	F	70	14	88	4.3	.0038	.33			
11	G	80	12.5	78.5	7.9	.0062	.48	792	63	Big bus on Thai Wang Road
11	N	80	12.5	78.5	1.8	.0014	.11			
12	G	75	13	81.7	14.1	.012	.94	851	65	
12	N	75	13	81.7	3.2	.0026	.21			
13	N	85	12	75.4	4.1	.0031	.24			Big bus on Thai Wang Road
14	G	75	13	81.7	10.6	.0087	.71	851	65	Big bus on Thai Wang Road ; fast
14	N	75	13	81.7	2.9	.0024	.20			
15	G	67	15	94.2	8.6	.0081	.76			Big bus on Thai Wang Road ; empty, fast
16	G	62	16	100.5	5.9	.0060	.60	535	35	Big bus on Thai Wang Road
16	H	65	15	94.2	7.9	.0075	.70			
17	G	70	14	88	3.7	.0033	.28			
17	H	67	15	94.2	7.3	.0069	.65			
18	G	70	14	88	8.3	.0073	.64	535	37	
18	H	67	15	94.2	11.4	.0107	1.01			
19	H	80	12.5	78.5	13.5	.0106	.83			10-wheel truck ; unloaded, slow
20	H	75	13	81.7	13.9	.0113	.92			Big bus ; empty, fast
21	K	75	13	81.7	5.9	.0049	.40			Big bus, both roads ; fast
22	K	75	13	81.7	5.9	.0049	.40			Big bus on Thai Wang Road ; fast
22	L	75	13	81.7	2.0	.0016	.13			
23	M	85	12	75.4	2.9	.0022	.17			Big bus on Thai Wang Road ; fast
23	N	80	12.5	78.5	3.4	.0027	.21			
24	M	75	13	81.7	2.4	.0020	.16			Big bus on Thai Wang Road
24	N	75	13	81.7	3.7	.0030	.25			
25	M	80	12.5	78.5	3.2	.0026	.20			
25	N	80	12.5	78.5	5.6	.0044	.35			
26	X	71	14	88	23.0	.0203	1.8	355	25	Big bus ; full, very slow
26	Y	70	14	88	16.5	.0145	1.3			
27	X	65	15	94.2	14.6	.0138	1.3	413	28	Big bus ; full, very slow
27	Y	65	15	94.2	15.2	.0143	1.4			
28	X	70	14	88	34.6	.0304	2.7	316	23	Small bus ; full, slow
28	Y	75	13	81.7	18.7	.0153	1.3			

TRAFFIC VIBRATIONS

RESULTS

Vibrations at Various Locations

The frequencies and amplitudes of the vertical components of ground motion produced by vehicles at the various locations are summarized in Table 1. The amplitudes presented in Table 1 are half of the maximum peak-to-peak amplitudes (Fig. 9). Also included in Table 1 are surface wave velocities and wave lengths determined from some of the tests. It can be seen that most frequencies varied from 12 to 15 Hz and amplitudes varied from about 2 to 30×10^{-3} in./sec. These values agree with typical values given by SUTHERLAND (1950) and STEFFENS (1952), as presented in RICHART et al (1970). The Rayleigh wave velocities are in the range of about 500 ± 75 ft/sec. between stations D, E and F and about 300 to 400 ft/sec. at stations X and Y. Between stations G and N the velocities were considerably higher, but it is believed that this reflected the higher velocities in the concrete floor of the Bot. These values are in general agreement with values obtained by PENG (1972).

Comparison of Measured Vibrations with Failure Criteria

The response spectra of the measured amplitudes of vibration are shown in Fig. 10. Instead of plotting each individual point, the bounds within which all the data points fell are shown. It can be seen that points K, L, M and N on the base of the Buddha image experienced the lowest amplitude vibrations. Although there was considerable overlap in the data at these four points, there appeared to be a tendency for the amplitudes at point K to be slightly higher than those at point N.

Points A, B, D, E, G and H outside the Bot exhibited higher amplitude vibrations. This was to be expected because of their closer proximity to the traffic. At these points, there was considerable overlap also, but there appeared to be a tendency for the amplitudes at points along Thai Wang Road to be slightly greater than at points A and B. This, however, may be the result of the vehicles on Maha Rat Road having a lower velocity than those on Thai Wang Road because of traffic and pedestrian congestion.

Two shaded areas are marked on Fig. 10 indicating areas in which damage or destruction to walls would occur for buildings subjected to steady state vibrations (RICHART et al, 1970). The U.S. Bureau of Mines criterion marked in Fig. 10 indicates the tolerable limits for buildings subjected to vibrations caused by blasting (RICHART et al, 1970). Below the Bureau of Mines criterion are designated two zones marked *minor defects* and *little or no defects* (DAWANCE & SEGUIN, 1965). These criteria are based on the

NELSON AND VIRANUVUT

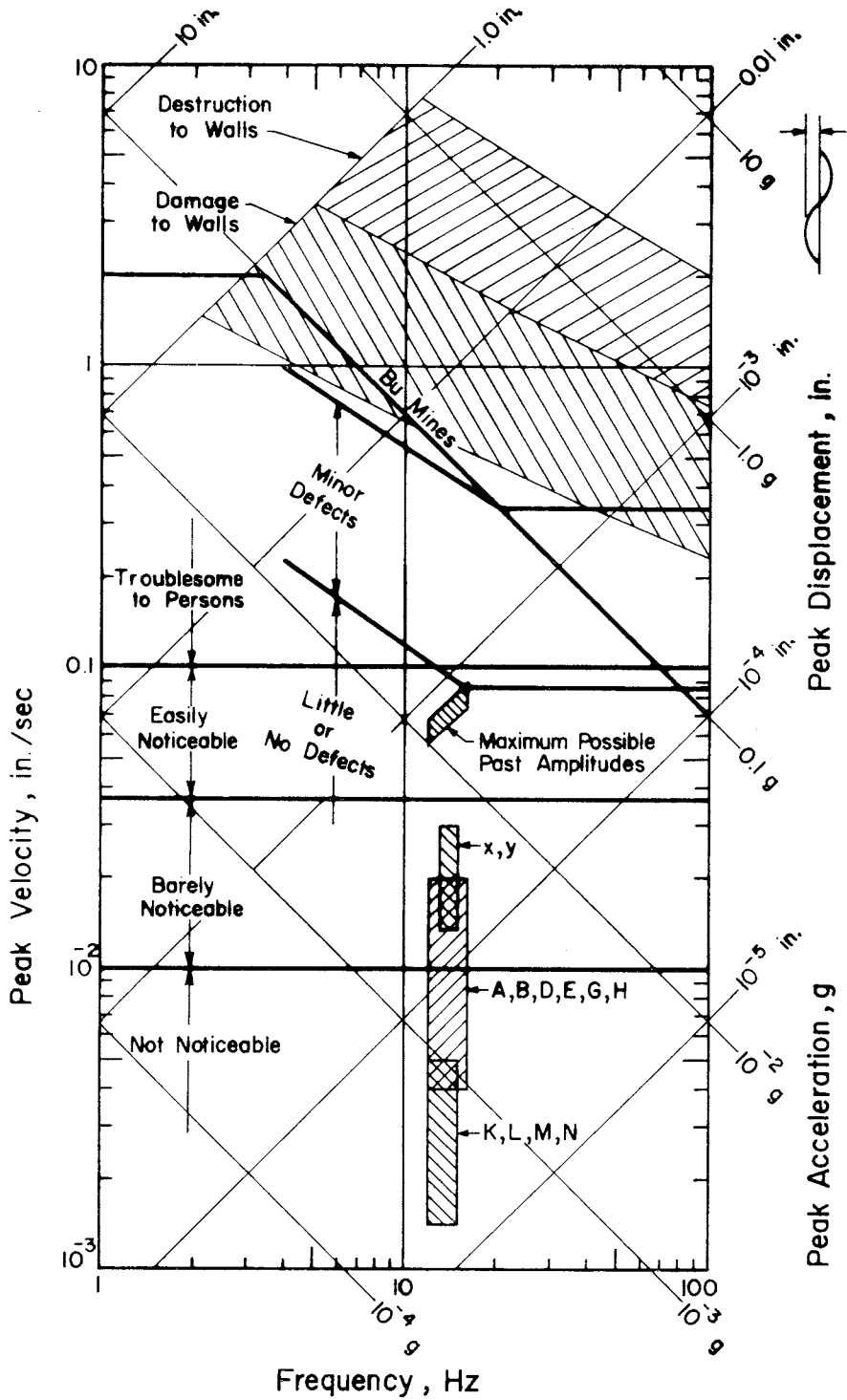


Fig. 10. Response spectra for vibration limits.

TRAFFIC VIBRATIONS

intensities of the modified Mercalli intensity scale for earthquakes. Each of these three criteria are based on somewhat different means of loading and, although none of them included traffic, they encompassed a range from transient loads to steady state vibrations. The threshold for damage to structures agrees fairly well for all three criteria; it is expected, therefore, that they will apply to traffic loading as well.

It can be seen from Fig. 10 that all vibration amplitudes were in the zone of *not noticeable* or in the lower range of *barely noticeable to persons*. They were well below the threshold for even minor defects to structures.

At the time the vibrations were measured (April and May, 1973) Thai Wang and Maha Rat Roads had fairly new and smooth asphalt surfaces in relatively good condition. Formerly, these roads had bumpy concrete surfaces. Because SUTHERLAND (1950) has shown that the primary factor influencing the amplitudes of induced vibrations is the size of the bumps on the road surface, it was desired to simulate the former conditions in order to ascertain whether or not vibrations produced under those conditions could have been of a magnitude that would have caused distress. Consequently, measurements were made near Soi Chettuphon that had a concrete road surface in similar condition to the former ones of Thai Wang and Maha Rat Roads. The amplitudes measured at points X and Y (Fig. 1) are indicated on Fig. 10.

The heavy vehicles along Soi Chettuphon, however, differed from those along the other roads in that they consisted only of fully loaded tourist buses travelling very slowly because of cars parked along the narrow roadway. An attempt was made, therefore, to correct the amplitudes to correspond to what would be expected to have been caused by a greater number of heavier vehicles travelling at higher speeds. SUTHERLAND (1950) presented data indicating a linear relationship between amplitude of displacement and vehicle velocity for different ramp heights in experiments on streets constructed on soft clays. The slope of this relationship was about 1.6×10^{-5} in./m.p.h. and, on the assumption that there is the same form of relationship for the data at points X and Y and that the velocity of the tourist buses was 2 m.p.h. (a lower bound), the maximum amplitude at point X would be increased from 3.5×10^{-4} in. to about 10×10^{-4} in. if the vehicle were travelling at 40 m.p.h.

SUTHERLAND (1950) also showed that the amplitude was approximately proportional to the weight of the vehicle traversing the bump. Therefore, if it is assumed that the vehicles had been very heavily loaded trucks of about twice the weight of the buses, the maximum amplitudes would have been twice that of the buses, or 2×10^{-3} in.

NELSON AND VIRANUVUT

Finally, the amplitudes presented in Table 1 were those caused by a single vehicle. On the assumption of two-way traffic, the maximum number of vehicles that could be travelling simultaneously on Thai Wang and Maha Rat Roads in the vicinity of the intersections, particularly at a speed of 40 m.p.h., would be about ten. If the vibrations caused by all of the vehicles were in phase and arrived at the structure at the same time (which is improbable), the amplitude of displacement would be ten times that of a single vehicle, or 2×10^{-2} in. This represents the vibration that would occur near the curb. Some wave amplitude attenuation will have occurred, however, because of spatial dispersion and energy absorption due to internal damping (Eq. 5). The amplitude-distance relationship for pairs of data points observed simultaneously is shown in Fig. 11. The values of α shown in Table 2 were computed from each pair of data points. An appropriate value of α in the area around

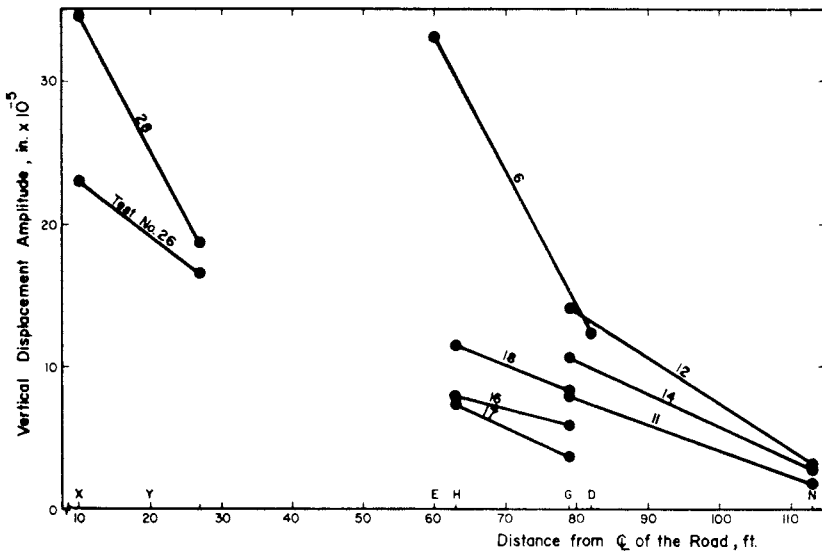


Fig. 11. Vertical displacement amplitude at various distances from the road.

Table 2. Coefficients of energy absorption

Test No.	Station	r_o , ft	A_o , in. $\times 10^{-5}$	r , ft	A_r , in. $\times 10^{-5}$	α , ft $^{-1}$
6	D, E	60	33.0	82	12.3	.038
11	G, N	79	7.9	113	1.8	.038
12	G, N	79	14.1	113	3.2	.038
14	G, N	79	10.6	113	2.9	.033
16	G, H	63	7.9	79	5.9	.011
17	G, H	63	7.3	79	3.7	.035
18	G, H	63	11.4	79	8.3	.013
26	X, Y	10	23.0	27	16.5	.004
28	X, Y	10	34.6	27	18.7	.021

TRAFFIC VIBRATIONS

points D and G is somewhat greater than 0.030 ft^{-1} . This value is in agreement with a value of 0.037 ft^{-1} given by BARKAN (1962) for saturated brown clays with some sand and silt.

The corrected amplitudes of vibration computed above (2×10^{-2} in.) occurred about 10 ft from the source. From Eq. 5, the amplitude at the wall of the Bot, approximately 80 ft away, would then be:

$$\begin{aligned} A &= 2 \times 10^{-2} \sqrt{\frac{10}{80}} \exp [-0.03 (80 - 10)] \\ &= 8.7 \times 10^{-4} \text{ in} \end{aligned} \quad (6)$$

Similarly, the "corrected" lower limit of amplitudes at points X and Y would be 7×10^{-4} in.

It should be noted that in arriving at these "corrected values" all assumptions made were on the extremely unconservative side. Furthermore, the amplitude of vibration near the source is of such a magnitude that severe damage would have occurred to the wall surrounding the Wat. It therefore represents an absolute upper bound that could have occurred.

The maximum possible past amplitudes of displacement are plotted in Fig. 10 and can be seen to be in the range of *little or no defects*. It is evident, therefore, that neither the vibrations induced in the structures now nor those that may have existed in the past are of sufficient magnitude to have caused any structural damage to the Bot or the Reclining Buddha image.

Vibration Induced Settlements

It has been seen that the vibration amplitudes were not of sufficient magnitude to cause damage in the Buddha image. However, there did exist vibrations of significant amplitude in the soil and those vibrations could have induced additional settlement of the structure. It was shown in Fig. 5 that large differential settlements have occurred. The 'spire' on the head of the Buddha image is connected to the wall of the Bot and, therefore, even small relative settlements between the wall and the Buddha image would cause some cracking. Consequently, it is probable that the cracking has occurred as a result of differential settlement between the Buddha image and the Bot.

Most of the consolidation settlement should have occurred in the first 50 years of the structure's existence and should have almost stopped by recent years (estimated from the soil data presented in Fig. 6). One possible reason for the recent resumption of settlement is deterioration of the foundation structure. If the foundation consists of a timber mat or timber piles (which is not known), it is quite possible that some decay of the wood could have

occurred at or above the ground water level, thus causing an increase in settlement.

Another possibility is that the vibrations from the traffic could have weakened the soil structure thereby causing a decrease in the void ratio. Some evidence does exist to indicate that the settlement was induced by traffic vibrations. CROCKETT (1965) studied very long-term vibration soil settlement on a series of over 40 ancient buildings, one of which was 1,300 years old. He observed that these buildings invariably leaned more toward the road carrying the traffic than away from it. Also the settlements were proportional to the estimated number of vibration cycles, their maximum amplitude, number of vehicles passing and the number of centuries they had been passing. Whereas it can be seen from Fig. 5 that the base of the Buddha image does not appear to be leaning toward the road, the large differential settlement away from Maha Rat Road is undoubtedly the result of the greater load on that end of the base. Inspection of the exterior of the Bot revealed that the exterior columns were out of line and those on the north side of the Bot were considerably more out of line than those on the south side. This somewhat more pronounced distress on the north side of the Bot is in agreement with the observations of Crockett.

Very little information is available on vibration induced settlements in clays, particularly Bangkok Clay. SATYAVANIJA & NELSON (1971) showed that vibratory loading could cause a significant decrease in the shear strength of Bangkok Clay. Although the results reported by them apply to shear strength, they should be indicative of the effects of vibration on consolidation as well because, in both cases, the influence arises through a weakening of the soil structure.

The threshold acceleration of a clay is defined as *that amplitude of vibration for which greater amplitudes cause an appreciable decrease in soil strength* (SATYAVANIJA & NELSON, 1971). For the Soft Bangkok Clay, the measured threshold acceleration was somewhat less than 0.1 g. The maximum possible past amplitudes at Wat Po are slightly less than this threshold acceleration reported for the Soft Bangkok Clay. However, the soils tested by Satyavanija & Nelson were taken from the AIT campus about 25 miles north of Wat Po. The soil at Wat Po may be somewhat softer and slightly more sensitive since it is a more recent deposit than that at the AIT site. Also, some disturbance due to sampling is always unavoidable. Furthermore, prolonged periods of vibration could lower the threshold acceleration. It is quite possible, therefore, that the threshold acceleration could have been below the amplitudes that existed in the past. It is feasible that it could have been as low as the maximum accelerations measured at points X and Y.

TRAFFIC VIBRATIONS

Whether it would have been as low as the accelerations measured at points A through H could only be determined through testing.

CONCLUSIONS AND RECOMMENDATIONS

The measured amplitudes of vibration are presented in Table 1. By comparison with various failure theories, it is seen that the present amplitudes as well as the maximum amplitudes that could have existed in the past are of too small a magnitude to have caused any structural damage. From measurements of relative elevations at various points, it is apparent that the cracks in the Buddha image are the result of differential settlement. Consolidation settlement should have been completed many years ago (except for secondary consolidation), and it is most likely, therefore, that the more recent settlement is the result of one or a combination of both of:

- (i) Deterioration of the foundation structure.
- (ii) Decrease in the void ratio of the soil as a result of a weakening of the soil structure caused by vibrations from the traffic on Maha Rat and Thai Wang Roads.

To ascertain the degree to which deterioration of the foundation may be contributing to the settlement, it is recommended that observation of the actual foundation structure be made. The type of foundation, its position relative to the ground water level and its condition should be determined.

It is likely that some settlement has been induced by vibration from the traffic, and some effort should be made to minimize the amplitude of this vibration. RICHART et al (1970) provide guidelines for the size of a trench barrier that would be needed to reduce the amplitude of vibration. The parameter governing the position and depth of the trench is the wave length of the Rayleigh waves causing the vibrations. The wave lengths computed from the observed data are shown in Table 1, from which it can be seen that a reasonable wave length would be about 35 ft. Thus, for active isolation, the trench would need to be within a distance of 35 ft from the traffic and its depth would need to be at least 21 ft if an open trench were to be used. A trench of that depth, however, would certainly require some support to keep it open. A bentonite slurry would suffice but it is doubtful if it would be satisfactory over a long period of time. Other trench filler materials have been found to be effective (PENG, 1972) but deeper trenches are required if they are used. Also, the suitability of the various filler materials after having been subjected to long-term vibrations is not known. Passive isolation, in which an open trench would be placed near the Bot, would require a trench

NELSON AND VIRANUVUT

approximately 375 ft long and 47 ft deep. For both active and passive conditions, the width of the trench would have little or no influence. The sizes of the trenches required for active or passive isolation of the Reclining Buddha are quite large and, even if they are constructed, a reduction in amplitude by only a factor of four could be assured. A reduction of more than one order of magnitude is highly unlikely. Consequently, it was not recommended that screening of the Rayleigh wave by trenches be attempted.

It was seen that the condition of the roadway surface had a primary influence on the vibration amplitudes. This was also concluded by SUTHERLAND (1950). It is believed, therefore, that if the roadway surfaces on Thai Wang and Maha Rat Roads are maintained in a good condition, such as exists at the present time, the vibrations induced in the Wat will be of sufficiently small magnitude.

It is recommended that observations be made of the settlements of the Buddha image and the Bot floor as functions of time. If vibration induced settlements appear to be continuing, a program of consolidation testing under vibratory loading conditions is recommended.

REFERENCES

- BARKAN, D.D. (1962), *Dynamics of Bases and Foundations*, McGraw Hill, New York.
- BRAND, E.W. (1971), Subsurface Investigation at the Site of Memorial Bridge Bangkok, *Report to Public and Municipal Works Dept.*, Asian Institute of Technology, Bangkok.
- CIRIDON, W.A. (1972), Performance of Sand Drains at Tha Chang Bridge, *M.Eng. Thesis No. 376*, Asian Institute of Technology, Bangkok.
- CROCKETT, J.H.A. (1965), Some Practical Aspects of Vibration in Civil Engineering, *Proc. Symp. on Vibration in Civil Engineering*, London, pp. 253-271.
- DAWANCE, G. and SEGUIN, M. (1965), Effects of Vibrations on Buildings, *Proc. Symp. on Vibration in Civil Engineering*, London, pp. 31 - 33.
- PENG, S.M. (1972), Propagation and Screening of Rayleigh Waves in Clay, *M.Eng. Thesis No. 386*, Asian Institute of Technology, Bangkok.
- RICHART, F.E., HALL, J.R. and WOODS, R.D. (1970), *Vibration of Soils and Foundations*, Prentice-Hall, New Jersey.
- SATYAVANIJA, P. and NELSON, J.D. (1971), Shear Strength of Clays Subjected to Vibratory Loading, *Proc. 4th Asian Regional Conf. Soil Mech. Found. Eng.*, Vol. 1, pp. 215 - 220.
- STEFFENS, R.J. (1952), The Assessment of Vibration Intensity and Its Application to the Study of Building Vibrations, *National Building Studies Special Report No. 19*, Building Research Station, London.
- SUTHERLAND, H.B. (1950), A Study of the Vibrations Produced in Structures by Heavy Vehicles, *Proc. 30th Annual Meeting Highway Research Board*, pp. 406 - 419.

FIELD COMPRESSIBILITY OF SOFT SENSITIVE NORMALLY CONSOLIDATED CLAYS

YUDHBIR*

SYNOPSIS

An effort is made to predict field compressibility of soft, sensitive, normally consolidated clays by utilizing the Atterberg limits, natural water content and sensitivity values of the material. A relationship between the ratio of remoulded compression index to field compression index, liquidity index and sensitivity is developed. Values of compressibility estimated on the basis of the developed relationship show very good agreement with actual values of field compressibility for three soft, sensitive, normally consolidated clays.

INTRODUCTION

In soft, sensitive, normally consolidated clays, disturbance resulting from the sampling techniques usually employed may influence considerably the values of shear strength and compressibility as obtained in laboratory tests. This problem is still more severe in the case of quick clays. To obtain really good undisturbed samples is a very expensive operation in these clays. Even after good undisturbed samples have been obtained, special techniques are needed in transferring the soil from the sampling tube to the testing apparatus. Evaluation of representative values of field compressibility, therefore, requires better testing equipment and techniques than are currently available. In this paper, a proposed relationship between the ratio of remoulded to field compression indices, sensitivity and liquidity index of the material is given which may be used to estimate the order of magnitude of field compressibility. This is, of course, an approximate estimate of field compressibility and cannot replace a detailed field and laboratory investigation where more accurate data are needed. It will, however, be noted that the suggested relationship involves soil parameters which are known to be inter-related.

BASIC ASSUMPTIONS

The following assumptions are made in deriving the proposed relationship:

(1) A plot of e or wG vs. $\log s_u$ is linear, as shown in Fig. 1, and is parallel to a plot of e or wG vs. $\log \bar{\sigma}_v$, where e is the void ratio, w is the water content, G is the specific gravity, s_u is the undrained shear strength and $\bar{\sigma}_v$ is the effective vertical stress at the completion of consolidation and before

*Assistant Professor of Civil Engineering, Indian Institute of Technology, Kanpur, India.
Discussion on this paper is open until 1 May 1974.

shearing. This assumption is commonly made in soil mechanics and is supported by much accepted experimental evidence.

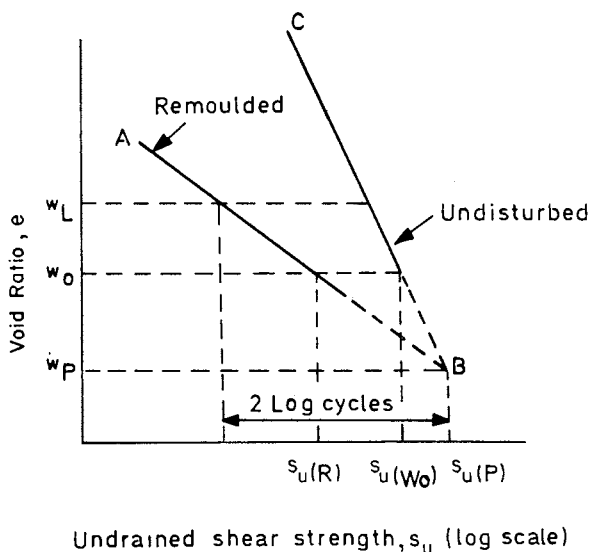


Fig. 1. Idealized relationship between void ratio and undrained shear strength.

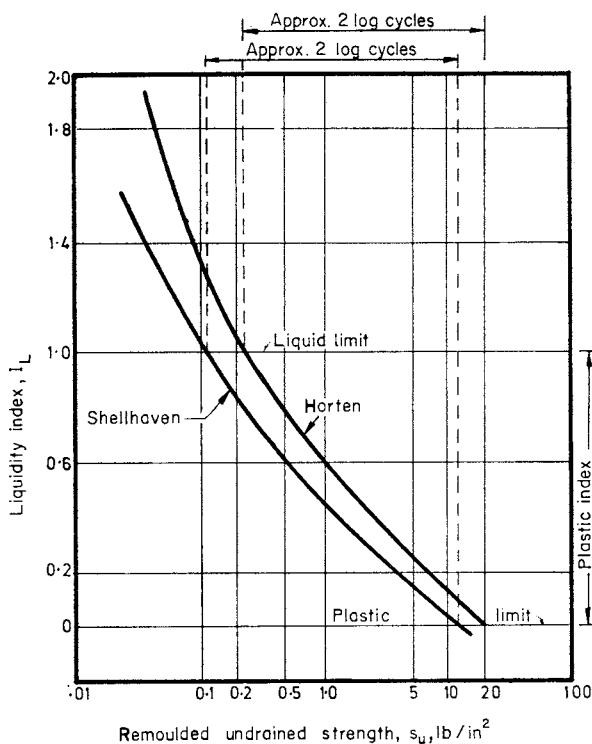


Fig. 2. Relationship between liquidity index and remoulded undrained shear strength (after SKEMPTON & NORTHEY, 1952).

(2) In the range of water content between the liquid and plastic limits, the variation of remoulded undrained shear strength is two log cycles, as shown

COMPRESSIBILITY OF CLAYS

in Fig. 2. This assumption is supported by the results of tests published by SKEMPTON & NORTHEY (1952) and reproduced in Fig. 2. SCHOFIELD & WROTH (1968) also used this assumption.

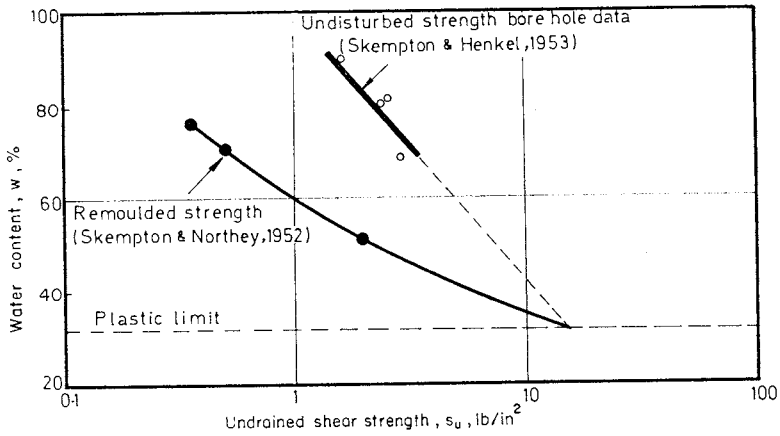


Fig. 3. Relationship between water content and undrained shear strength for Shellhaven clay.

(3). The sensitivity of soft, sensitive, normally consolidated clays is equal to unity at water contents equal to the plastic limit. Thus, lines of e or wG vs. $\log s_u$ for the undisturbed and remoulded states are assumed to meet at a point at a water content equal to the plastic limit, as shown in Fig. 1. While this assumption is obviously a simplification of the actual behaviour of soft, sensitive, normally consolidated clays, there are indications that it is not far from reality. Data taken from SKEMPTON & NORTHEY (1952) and SKEMPTON & HENKEL (1953) for Shellhaven clay are plotted in Fig. 3, and data taken

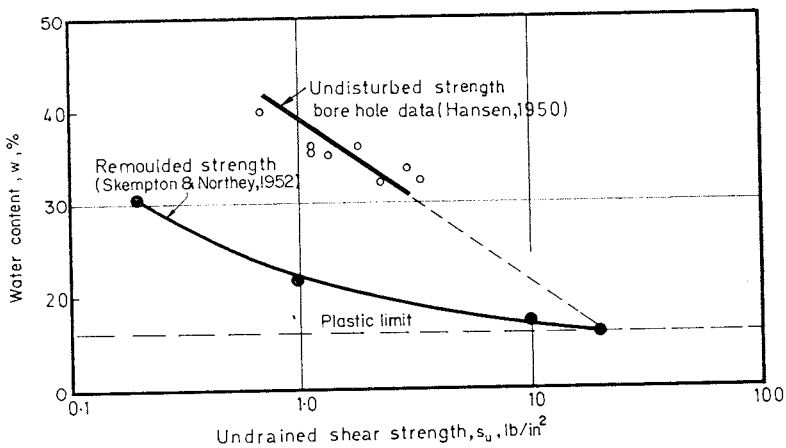


Fig. 4. Relationship between water content and undrained shear strength for Horten clay.

from SKEMPTON & NORTHEY (1952) and HANSEN (1950) for Horten clay are plotted in Fig. 4. The Shellhaven clay has a sensitivity of 7 to 8, while the Horten clay exhibits a sensitivity of 17. The *in situ* water contents of these clays were reported to be much higher than the plastic limit and, hence, no data are available for undisturbed strengths near the plastic limit. However, extrapolation of the curves of e vs. $\log s_u$ for the undisturbed state for these two clays tends to support the assumption. Similar trends can be observed from Fig. 5 for Drammen clay, with an average sensitivity of 8 to 10 (SIMONS, 1960; BJERRUM, 1967).

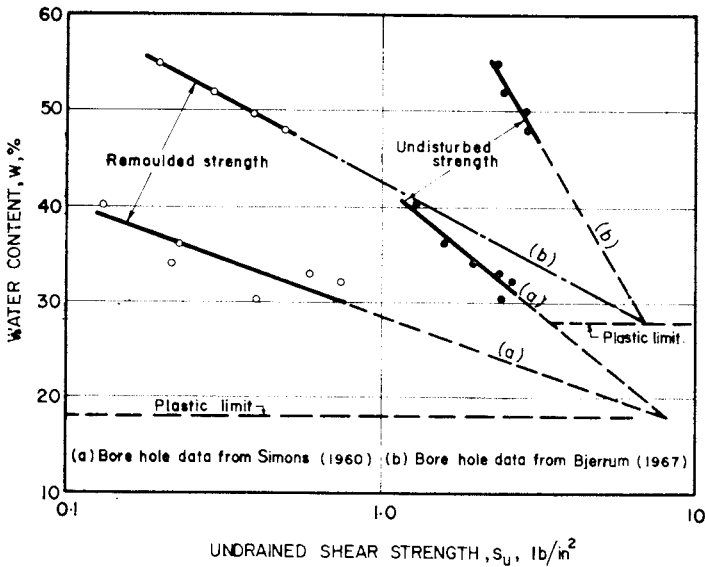


Fig. 5. Relationship between water content and undrained shear strength for Drammen clay.

SKEMPTON & NORTHEY (1952) found that the thixotropic strength regain with time after remoulding of a saturated clay was essentially zero when remoulded at water contents near the plastic limit. Thus, sensitivity due to thixotropic strength regain is unity at the plastic limit. Furthermore, the water content, w_i , at the point of intersection of the curves of w vs. $\log \bar{\sigma}_v$ for the undisturbed and remoulded soil computed by $w_i = 0.42 w_o$, as proposed by SCHMERTMANN (1955), is found to be equal to $0.8 w_p$ to w_p for most of the sensitive clays. Thus, the assumption that lines of e or wG vs. $\log s_u$ for the undisturbed and remoulded states meet at a point at a water content equal to the plastic limit is a reasonable approximation of the actual behaviour of soft, sensitive, normally consolidated clays.

COMPRESSIBILITY OF CLAYS

DERIVATION OF PROPOSED RELATIONSHIP

The slope of line BC in Fig. 1 may be written as :

$$e_o - e_p = (w_o - w_p) G = C_2 [\log s_{u(p)} - \log s_{u(w_o)}] \dots (1)$$

where w_o is the *in situ* water content, w_p is the water content at the plastic limit, G is the specific gravity of the soil grains, C_2 is the slope of the e vs. $\log s_u$ curve (which is equal to the compression index of the soil in the undisturbed state), $s_{u(p)}$ is the undrained strength of the soil at the plastic limit, and $s_{u(w_o)}$ is the undrained strength of the soil at the *in situ* water content in the undisturbed state. This gives:

$$\log s_{u(p)} = \frac{(w_o - w_p) G}{C_2} + \log s_{u(w_o)} \dots (2)$$

The slope of the line AB in Fig. 1 may be written as:

$$e_o - e_p = (w_o - w_p) G = C_1 [\log s_{u(p)} - \log s_{u(r)}] \dots (3)$$

where C_1 is the slope of the e vs. $\log s_u$ curve (which is equal to the compression index of the soil in the remoulded state), and $s_{u(r)}$ is the undrained strength of the soil at the *in situ* water content in the remoulded state.

From Eqs. 2 and 3:

$$\begin{aligned} (w_o - w_p)G &= C_1 \left[\frac{(w_o - w_p)G}{C_2} + \log s_{u(w_o)} - \log s_{u(r)} \right] \\ &= \frac{C_1}{C_2} \left[(w_o - w_p)G \right] + C_1 \log \frac{s_{u(w_o)}}{s_{u(r)}} \end{aligned}$$

$$\text{or:} \quad (w_o - w_p)G \left(1 - \frac{C_1}{C_2} \right) = C_1 \log \frac{s_{u(w_o)}}{s_{u(r)}} \dots (4)$$

Now, sensitivity is given by:

$$S_t = \frac{s_{u(w_o)}}{s_{u(r)}}$$

Thus, Eq. 4 becomes:

$$(w_o - w_p)G \cdot \left(1 - \frac{C_1}{C_2} \right) = C_1 \log S_t \dots (5)$$

The assumption No. 2 listed above (and depicted in Fig. 2) may be expressed as:

$$e_1 - e_p = (w_1 - w_p)G = C_1 \log 100$$

where w_1 is the water content at the liquid limit of the soil (see also SCHOFIELD & WROTH, 1968). From which we get:

$$C_1 = \frac{(w_1 - w_p)G}{2} \dots (6)$$

YUDHBIR

and, when substituted in the term on the righthand side of Eq. 5, gives:

$$(w_o - w_p)G \left(1 - \frac{C_1}{C_2}\right) = \frac{(w_1 - w_p)}{2} \log S_t$$

or:

$$\frac{(w_o - w_p)}{(w_1 - w_p)} \cdot 2 \cdot \left(1 - \frac{C_1}{C_2}\right) = \log S_t$$

$$\therefore 2 I_1 \left(1 - \frac{C_1}{C_2}\right) = \log S_t \quad \dots \dots (7)$$

where I_1 is the liquidity index of the soil. This equation implies that liquidity index is linearly related to $\log S_t$. Available data for various soft, sensitive, normally consolidated clays are shown in Fig. 6, together with the linear relationship between I_1 and $\log S_t$ proposed by BJERRUM (1954) for the Scandanavian clays. The deviation from this line of the actual data for Scandanavian clays may be attributed to different values of the ratio of their remoulded and *in situ* compressibilities, C_1/C_2 .

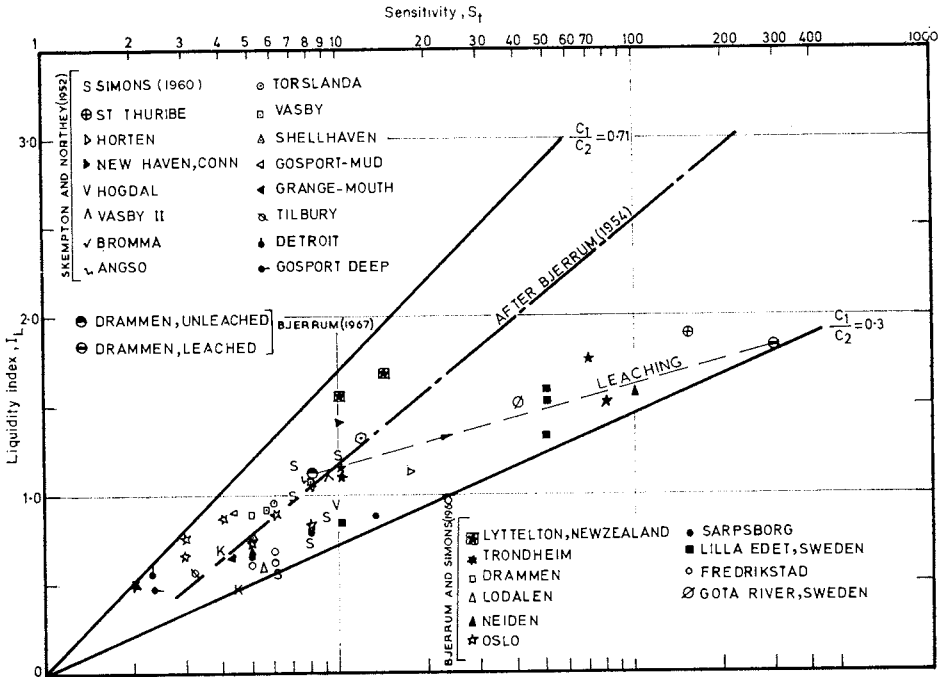


Fig. 6. Relationship between liquidity index and sensitivity.

From the available data (as shown in Fig. 6), the upper limit of the value of C_1/C_2 is 0.7, and the lower limit is 0.3. The upper limit is representative of insensitive to slightly sensitive clays ($S_t < 4$) and, incidently, it is close to the value of C_1/C_2 suggested by TERZAGHI & PECK (1967), namely :

COMPRESSIBILITY OF CLAYS

$$\frac{C_1}{C_2} = \frac{0.007 (w_1 - 10)}{0.009 (w_1 - 10)}$$

The lower limit of C_1/C_2 is representative of very sensitive to quick clays. Another important observation to be made from Fig. 6 is the position occupied by Drammen clay before and after leaching. It is quite clear that, during the leaching process, the clay cuts across lines representing different values of C_1/C_2 towards the lower limit. This is quite consistent with the well-known fact that, during leaching, the soil structure becomes more unstable, which results in increased compressibility. The extreme results of leaching are quick clays, which are known to possess significantly greater field compressibility than the original deposits.

DISCUSSION

In addition to the fact that Eq. 7 defines a linear relationship between I_1 and $\log S'_t$, it will be noted that the parameters of undrained shear strength, compressibility and water content, are fundamentally inter-related. It will be further noted from the proposed relationship that, if the representative average values of I_1 , C_1 and S'_t are known for a given deposit, the value of field compression index, C_2 , and hence field compressibility $C_2/(1 - e_o)$, can be easily evaluated. A knowledge of the *in situ* water content and Atterberg limits for a soil will enable values of I_1 and C_1 to be computed. The value of C_1 may be computed from either of the two relationships:

$$C_1 = .007 (w_1 - 10) \quad \dots \dots \dots (8)$$

or $C_1 = \frac{(w_1 - w_p)G}{2}$

The second relationship is based on assumption No. 2 listed earlier and has already been derived as Eq. 6.

It is interesting to observe (Fig. 7) that the values of C_1 obtained from these two relationships agree very well with actual values of compression index for soils tabulated by SKEMPTON (1944), particularly for soils of low plasticity (i.e. sensitive soils). This close agreement further indicates the validity of assumption No. 2 employed in the derivation of the proposed relationship, which may also be used to construct field compression curves or to correct laboratory consolidation curves for disturbance during sampling. The following procedure is suggested.

1. Extend the straight-line portion of the e or w vs. $\log \bar{\sigma}_v$ curve obtained from consolidation tests on undisturbed specimens to the void ratio or water content corresponding to the plastic limit. Where no undisturbed

YUDHBIR

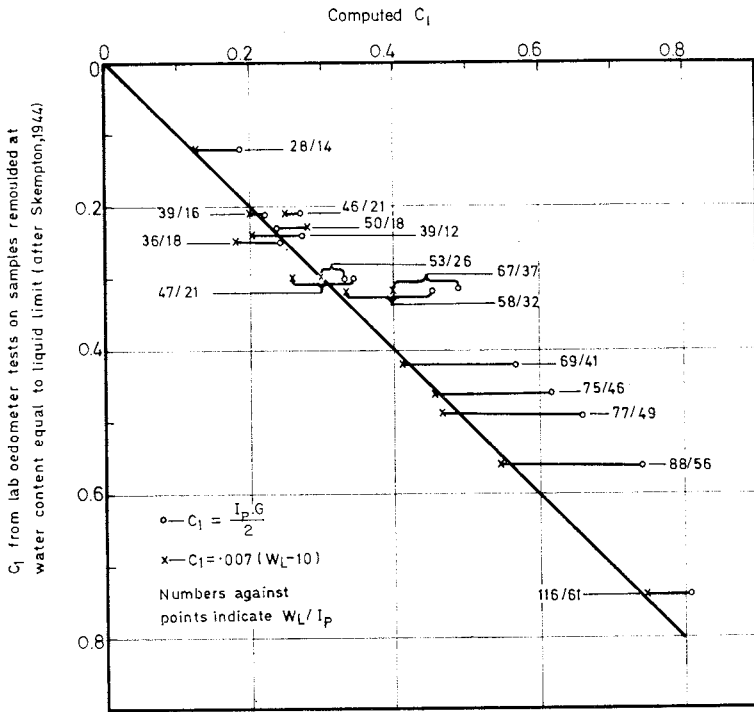


Fig. 7. Comparison of computed and measured compressibilities of remoulded clays.

samples are available, tests on samples remoulded at the natural water content may either be carried up to the pressure needed to reach a water content equal to the plastic limit, or the w vs. $\log \bar{\sigma}_v$ curve may be extended to this water content.

2. Compute the average value of I_1 for the sample.
3. From the average value of S_t from field vane tests, the value of the ratio C_1/C_2 can be computed from Eq. 7.
4. The value of C_1 may be computed from Eqs. 8 or 9, or from tests on remoulded specimens at the *in situ* water content.
5. Compute the value of C_2 from the determined values of C_1/C_2 and C_1 .
6. From the point representing the void ratio and pressure at the plastic limit as determined in step 1 above, draw a line of slope C_2 to meet the horizontal line representing the *in situ* void ratio of the specimen. The value of pressure at the point of intersection, $\bar{\sigma}_{vc}$, may be equal to or greater than the *in situ* overburden pressure, $\bar{\sigma}_{vo}$, depending on the engineering geologic history of the sediment (BJERRUM, 1967). The bilinear curve thus obtained represents a more realistic field compression curve as compared to the one obtained from consolidation tests on not-so-good undisturbed specimens.

COMPRESSIBILITY OF CLAYS

In order to check the validity of the proposed relationship (Eq. 7), it is essential to compare the predicted and measured values of field compressibility, $C_2/(1 + e_o)$, for these materials, and this will now be done.

Shellhaven Clay (SKEMPTON & HENKEL, 1953)

Shellhaven clay is a post glacial, estuarine, organic, soft, sensitive material. For a specimen of this clay, the *in situ* properties of the clay were given by SKEMPTON & HENKEL (1953) as $w_o = 69.3\%$, $w_1 = 72\%$, $w_p = 27\%$ and $S_t = 7.6$. With $G = 2.68$, Eq. 7 gives the value of field compressibility as 0.29. This predicted value compares well with the corresponding value of 0.30 obtained from laboratory consolidation tests.

Drammen Clay (BJERRUM, 1967)

Drammen clay is a post glacial, soft, sensitive, normally consolidated deposit; a detailed discussion of the engineering geology of this deposit is given by BJERRUM (1967). The average properties of the clay are $w_o = 55\%$, $w_1 = 55\%$, $w_p = 25\%$ and $S_t = 10$. With $G = 2.75$, the value of field compressibility is found from Eq. 7 to be equal to 0.25. This agrees well with the range of values of field compressibility of 0.18 to 0.30 as given by Bjerrum.

Soft Bangkok Clay (EIDE, 1968)

Soft Bangkok clay is an organic, sensitive, normally consolidated clay of high initial water content and compressibility. Representative properties from bore hole profiles are (depth of 7 m) $w_o = 80\%$, $w_1 = 105\%$, $w_p = 42\%$ and $S_t = 6.5$. With $G = 2.7$, Eq. 7 gives values of $C_2/(1 + e_o)$ of 0.65, which compares well with an average value of 0.7 reported by EIDE (1968).

CONCLUSIONS

A linear relationship between liquidity index and sensitivity has been derived as:

$$2I_1 \left(1 - \frac{C_1}{C_2} \right) = \log S_t$$

This relationship can be used to predict the order of magnitude of field compressibility from simple soil properties. A procedure has been outlined to obtain field compression curves for clays in which sample disturbance is likely to alter significantly the results of consolidation tests.

YUDHBIR

Comparisons have been made with some published experimental values of compressibility for some soft, sensitive, normally consolidated clays, and it is hoped that more such comparisons will be made in the future for other materials. It cannot be overemphasized that the proposed relationship provides only an order of magnitude of field compressibility and cannot replace careful and precise laboratory and field investigations to evaluate the appropriate field compressibilities of these sediments.

REFERENCES

- BJERRUM, L. (1954), Geotechnical Properties of Norwegian Marine Clays, *Géotechnique*, Vol. 4, pp. 49 - 69.
- BJERRUM, L. (1967), Engineering Geology of Norwegian Normally Consolidated Marine Clays as Related to Settlement of Buildings (7th Rankine Lecture), *Géotechnique*, Vol. 18, pp. 83 - 118.
- EIDE, O. (1968), Geotechnical Problems with Soft Bangkok Clay on the Nakhon Sawan Highway Project, *Norwegian Geotech. Inst. Pub. No. 78*, pp. 1 - 9.
- HANSEN, J.B. (1950), Vane Tests on a Norwegian Quick-Clay, *Géotechnique*, Vol. 2, pp. 58 - 63.
- SCHMERTMAN, J.H. (1955), The Undisturbed Consolidation Behavior of Clay, *Trans. A.S.C.E.*, Vol. 120, pp. 1201 - 1227.
- SCHOFIELD, A.N. and WROTH, C.P. (1968), *Critical State Soil Mechanics*, McGraw-Hill, London.
- SIMONS, N.E. (1960), Comprehensive Investigations of the Shear Strength of an Undisturbed Drammen Clay, *Proc. Research Conf. Shear Strength of Cohesive Soils*, A.S.C.E., Boulder, Colorado, pp. 727 - 745.
- SKEMPTON, A.W. (1944), Notes on the Compressibility of Clays, *Quart. J. Geol. Soc.*, London, Part C, pp. 119 - 135.
- SKEMPTON, A.W. and HENKEL, D.J. (1953), The Post Glacial Clays of the Thames Estuary at Tilbury and Shellhaven, *Proc. 3rd Int. Conf. Soil Mech. Found. Eng.*, Zurich, Vol. 1, pp. 302 - 312.
- SKEMPTON, A.W. and NORTHEY, R.D. (1952), The Sensitivity of Clays, *Géotechnique*, Vol. 3, pp. 30 - 53.
- TERZAGHI, K. and PECK, R.B. (1967), *Soil Mechanics in Engineering Practice*, Wiley, New York.

THE RELATIONSHIP BETWEEN UNDRAINED STRENGTH AND PLASTICITY INDEX

A. SRIDHARAN* and S. NARASIMHA RAO⁺

SYNOPSIS

The well-known linear correlation between linearly increasing s_u/p ratio and plasticity index in normally consolidated, undisturbed clays is critically examined. The relationship between s_u/p and I_p is studied with the aid of the data available from published literature. The results show marked deviations from the correlation proposed by SKEMPTON (1954). An attempt is made to explain the variation of s_u/p with I_p by means of theoretical considerations which relate s_u/p to A_f , K_o and $\bar{\phi}$. Results of isotropically consolidated undrained triaxial compression tests on seven remoulded clays with I_p values from 20 to 495 % are reported. These results, along with other published data, show that s_u/p tends to decrease as I_p increases. A similar trend is obtained from theoretical analysis. The results also bring out the significant influence of A_f on the value of s_u/p . Since many factors such as soil structure, stress level and type of testing govern the undrained strength behaviour of normally consolidated soils, any attempt to relate s_u/p with I_p should give a band rather than a unique relationship.

INTRODUCTION

The work of SKEMPTON & HENKEL (1953) and SKEMPTON (1954) shows that, in the case of normally consolidated undisturbed clays, the ratio of undrained shear strength, s_u , to effective overburden pressure, p , can be correlated closely with plasticity index, I_p . The suggested correlation shows that s_u/p ⁽¹⁾ increases linearly with I_p , as given by the equation :

$$s_u/p = 0.11 + 0.0037 I_p \dots\dots\dots (1)$$

Several workers in the field of soil mechanics have supported this correlation, and a brief review of the relevant literature is presented in the subsequent section. Few investigators, however, have brought out its limitations. In this paper, the correlation between s_u/p and I_p is re-examined in the light of the available published data, experimental results obtained by the authors and certain theoretical considerations.

* Assistant Professor, ⁺Senior Research Fellow, Department of Civil and Hydraulic Engineering, Indian Institute of Science, Bangalore, India.
Discussion on this paper is open until 1 May 1974.

(1) Even though the ratio of undrained shear strength to effective overburden pressure is widely denoted by c/p , the authors prefer to refer this ratio as s_u/p in order that c should not be mistaken for cohesion intercept.

BRIEF REVIEW OF PAST WORK

Apart from SKEMPTON & HENKEL (1953) and SKEMPTON (1954), there have been several studies of the correlation between s_u and I_p . BJERRUM (1954) presented extensive data for Norwegian marine clays to support Skempton's correlation. BISHOP & HENKEL (1962) supported this correlation and even pointed out that "...any test result not conforming with it should be re-examined in respect both of laboratory and sampling techniques". LEONARDS (1962) also referred to Skempton's line (Eq. 1) and observed that there appeared to be a close correlation between s_u/p and I_p . LUMB & HOLT (1968) compared their values with Skempton's line and suggested some modification for the values obtained for Hong Kong marine clay.

WU (1958) presented data the overall trend of which showed an increasing s_u/p ratio with increasing plasticity index. However, the data obtained appear to fall into two distinct alignment groups. The first group, consisting of clays of low sensitivity, was slightly above but very close to Skempton's line. The second group fell far below the first and was composed entirely of sensitive clays. The data presented by OSTERMAN (1960) for two types of clay showed different trends; the data for marine clays showed the same trend as Skempton's line, whereas for 'special clays' s_u/p decreased as I_p increased.

BJERRUM & SIMONS (1960) calculated the values of s_u/p using (i) the theoretical relationship connecting s_u/p with the effective angle of shearing resistance, $\bar{\phi}$, the pore pressure parameter A_f and the coefficient of earth pressure at rest, K_o , and (ii) the experimental correlation between I_p and $\bar{\phi}$. They compared these calculated values with those obtained from the field. Even though these comparisons showed quite opposite trends, they concluded that, for soils with an I_p greater than 30%, the field values were in agreement with those computed. BJERRUM (1961), using his correlation between s_u/p and I_p , and assuming the value of Skempton's pore pressure parameter at failure, A_f , in Eq. 4 (see below) to be 2.0, derived a relationship between $\bar{\phi}$ and I_p , and compared this with experimental results. The comparison showed that the angles of shearing resistance computed for low plasticity clays were very much smaller than those found from laboratory tests, the difference increasing with decreasing I_p . It was pointed out that either Eq. 4 is erroneous or low values of $\bar{\phi}$ result from progressive failure.

The data presented by METCALF & TOWNSEND (1960) generally do not follow closely the relationship suggested by Skempton. However, the data replotted with the I_p values corresponding to individual clay layers, rather than the average value of various clay layers, appeared to give good agreement with the correlation. GRACE & HENRY (1957) presented data which did not

UNDRAINED STRENGTH AND PLASTICITY

agree with Skempton's correlation. However, their subsequent discussion showed that there was a difference of about 100% between the shear strength results obtained by means of field vane shear and those measured by the unconsolidated undrained triaxial or unconfined compression tests.

In documented discussions, KENNEY (1959, 1960) suggested that the ratio s_u/p is strongly dependent upon the geological history of the soil, whereas I_p is dependent upon the results of two empirical tests (liquid limit and plastic limit) performed on completely remoulded soil. He derived two expressions for s_u/p , one for triaxial and the other for vane shear tests, in terms of $\bar{\phi}$ and A_f . Assuming a relationship with the data obtained experimentally between $\bar{\phi}$ and I_p , he then computed the values of s_u/p for various values of A_f on the basis of the relationship between $\bar{\phi}$ and I_p obtained experimentally. However, the field data superimposed on these computed curves did not suggest any correlation between s_u/p and I_p .

GOLDER & SPENCE (1960) suggested that s_u/p and $\bar{\phi}$ were not functions of plasticity index but rather of the structure of the soil and possibly, therefore, of the liquidity index. MILLIGAN et al (1962) expressed doubt as to whether any correlation between s_u/p and I_p exists for normally consolidated clay deposits. COX (1970) presented values of s_u/p for some clays of South East Asia and, in general, these did not fall on Skempton's line. BROWN (1970) pointed out that s_u/p ratios may have no meaning where the strengths were measured using field and laboratory vane tests, unconfined tests and unconsolidated undrained tests. He observed that the clays may in fact be normally consolidated in the geological sense but that the strength is the strength of the soil structure which has been created and modified by geological, geochemical and time-dependent factors.

A study of the literature cited above shows that the published data can be grouped into three broad categories, viz:

- (i) Those which agree with the correlation.
- (ii) Those which do not agree well with the correlation, but the authors who obtained these results believed in the existence of such a correlation and attempted to explain the deviations of their results.
- (iii) Those which do not agree with the correlation at all.

In this paper, most of the available published results as well as the authors' own results are used to re-examine the correlation between s_u/p and I_p .

THEORETICAL CONSIDERATIONS

Many factors, such as type of clay mineral, water content, nature of pore water, soil structure, stress history, stress path, rate of loading, method of

testing and temperature, govern the undrained strength parameters of a clay. Hence, any attempt to assign a unique relationship between undrained shear strength at failure and I_p is far from justified. The effect of such factors is to give a band rather than a unique line in any relationship or correlation. However, the theoretical relationship between s_u/p , $\bar{\phi}$, K_o and A_f can be derived from the geometry of the effective stress circle at failure for normally consolidated clays (SKEMPTON & BISHOP, 1954; BJERRUM & SIMONS, 1960; LEONARDS, 1962) as :

$$s_u/p = \frac{\sin \bar{\phi} [K_o + A_f(1 - K_o)]}{1 + (2A_f - 1) \sin \bar{\phi}} \dots \dots \dots (2)$$

No unique values can be assigned to $\bar{\phi}$, K_o and A_f for any particular soil, because these parameters are influenced by the factors mentioned above. For consolidation under conditions of no lateral yield, as occurs in the field, experimental evidence (BJERRUM & SIMONS, 1960) suggests that, as a first approximation :

$$K_o = 1 - \sin \bar{\phi} \dots \dots \dots (3)$$

and substitution of this into Eq. 2 gives :

$$\frac{s_u}{p} = \frac{(1 - \sin \bar{\phi} + A_f \sin \bar{\phi}) \sin \bar{\phi}}{1 + (2A_f - 1) \sin \bar{\phi}} \dots \dots \dots (4)$$

Equations 2 and 4 can be applied to undrained triaxial tests conducted on soils consolidated under K_o conditions. For the special case of soils consolidated isotropically, where $K_o = 1$, Eq. 2 reduces to :

$$\frac{s_u}{p} = \frac{\sin \bar{\phi}}{1 + (2A_f - 1) \sin \bar{\phi}} \dots \dots \dots (5)$$

In addition to Eqs. 4 and 5, KENNEY (1959) derived the following equation to suit the conditions prevailing in vane shear tests :

$$\frac{s_u}{p} = \frac{(1 - \sin \bar{\phi}) \sin \bar{\phi}}{1 + (2A_f - 1) \sin \bar{\phi}} \dots \dots \dots (6)$$

From Eqs. 4, 5 and 6, it is clear that the ratio s_u/p is very much dependent upon the values of $\bar{\phi}$ and A_f . Except for a few clays, there appears to be a good correlation between $\bar{\phi}$ and I_p (KENNEY, 1959; BJERRUM & SIMONS, 1960). As both $\bar{\phi}$ and I_p are primarily influenced by mineralogical composition, the correlation between $\bar{\phi}$ and I_p seems to be valid. However, such correlations between $\bar{\phi}$ and I_p cannot be the same for both undisturbed and remoulded soils. Although a unique correlation cannot exist between $\bar{\phi}$ and

UNDRAINED STRENGTH AND PLASTICITY

I_p , because of the influence of factors like soil structure, stress history, stress path and rate of loading on $\bar{\phi}$, it is reasonable to expect a narrow band of correlations to exist.

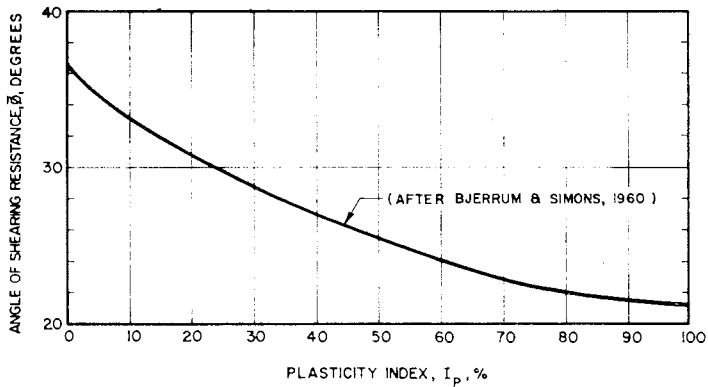


Fig. 1. Relationship between $\bar{\phi}$ and plasticity index for undisturbed clays.

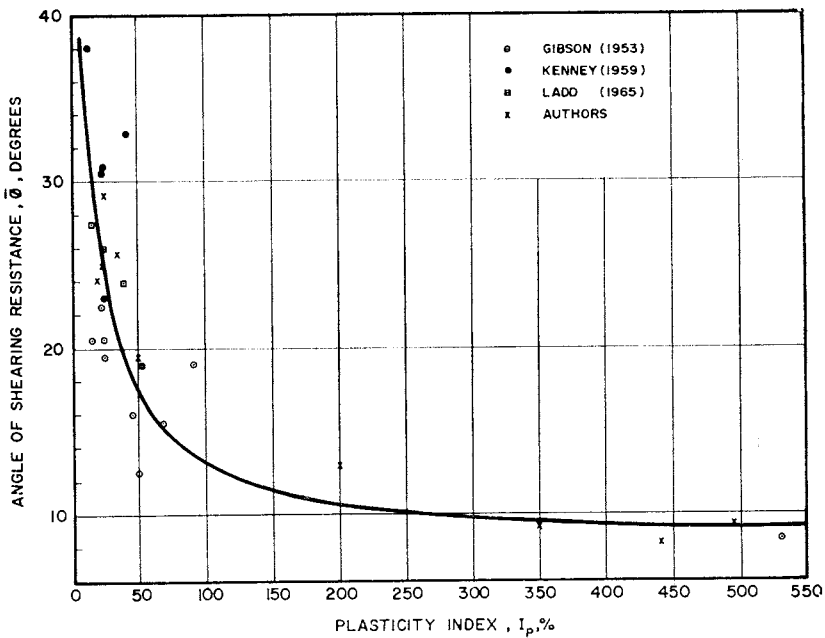


Fig. 2. Relationship between $\bar{\phi}$ and plasticity index for remoulded clays.

Figure 1 shows the relationship between $\bar{\phi}$ and I_p given by BJERRUM & SIMONS (1960) for undisturbed clays. Figure 2 shows a similar relationship obtained for remoulded clays using the authors' results (see below) together with previously published data. Although there is some scatter, there is a definite trend in Fig. 2 to indicate that $\bar{\phi}$ decreases as I_p increases.

SRIDHARAN AND RAO

On the basis of the relationship between $\bar{\phi}$ and I_p as given in Figs. 1 and 2, values of s_u/p for various values of I_p can be computed for given values of A_f using one of the Eqs. 4, 5 or 6 depending on the test conditions. These relationships can then be compared with the experimental data from both field and laboratory tests, provided the probable or actual value of A_f is known. LEONARDS (1962) reported values of A_f ranging from 0.7 to 1.3 for normally consolidated soils. KENNEY (1959) reported a value as low as 0.26, and LO (1962) as high as 2.7 for normally consolidated soils. Therefore, values of A_f ranging from 0.25 to 3.0 are used in this paper. In order to study the variation of s_u/p with I_p , the authors compiled the available published data on undisturbed clays, and conducted laboratory experiments on remoulded soils.

RESULTS AND DISCUSSION

Undisturbed Clays

Figures 3, 4 and 5 show values of s_u/p plotted against plasticity index for various undisturbed soils tested under different conditions. These test results were either reproduced from existing publications or they were deduced from published data. On the same figures are shown the theoretical curves of s_u/p vs I_p for A_f values of 0.25 to 3.0; these were calculated from the relationship between $\bar{\phi}$ and I_p (Fig. 1). Equation 1 (Skempton's correlation) is also plotted for reference.

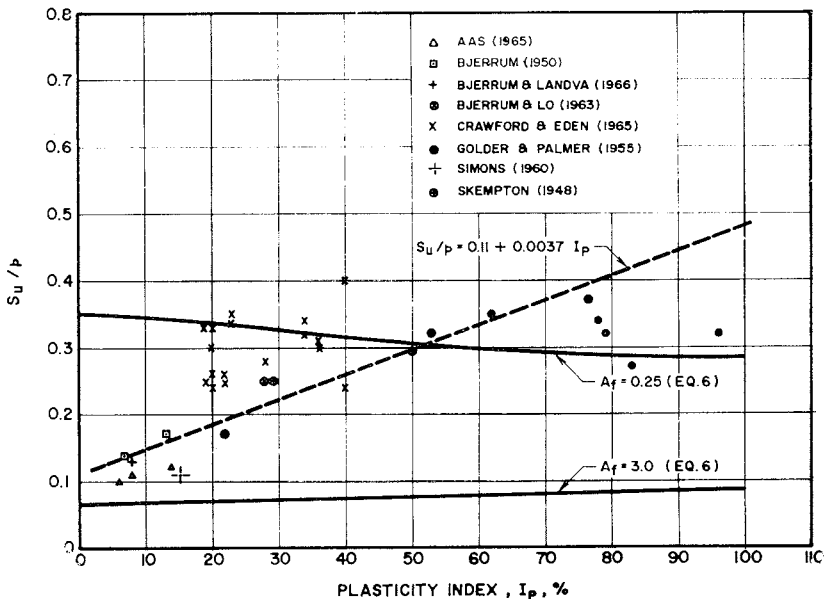


Fig. 3. Relationship between s_u/p and plasticity index for field vane tests.

UNDRAINED STRENGTH AND PLASTICITY

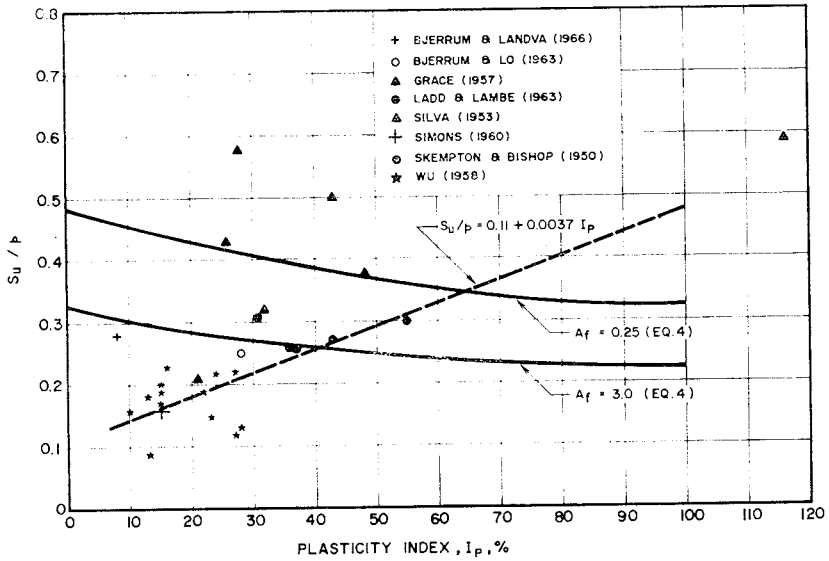


Fig. 4. Relationship between s_u/p and plasticity index for unconsolidated undrained triaxial tests on undisturbed samples.

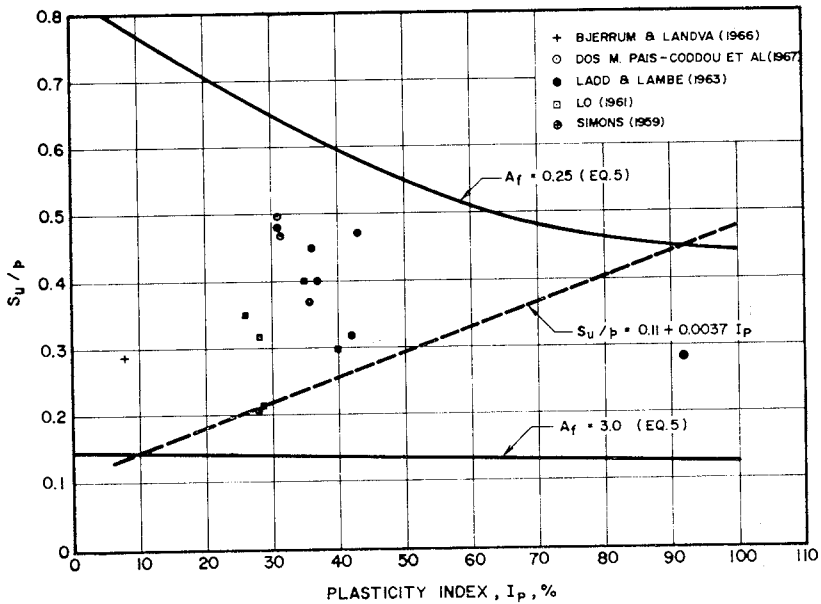


Fig. 5. Relationship between s_u^c/p and plasticity index for isotropically reconsolidated undrained triaxial tests on undisturbed samples.

In Fig. 3, which refers to field vane shear tests, it is clear that most of the results do not follow the correlation given by Eq. 1. It is also clear from the

figure that the value of A_f significantly shifts the theoretical s_u/p vs I_p relationship, and that the experimental results are within the range of A_f values usually encountered in natural soils. Since these results were obtained from tests conducted *in situ*, there is little possibility either for changes in structure due to disturbance or changes in effective stress. Consequently, no field value could be expected to lie below the curve computed for $A_f = 3.0$, and this can be seen to be so in Fig. 3.

Figure 4 shows the results of undrained triaxial compression tests conducted on soil samples consolidated in the field under no lateral yield (K_0) conditions. Once again, the results do not agree with Eq. 1, nor do they follow the theoretically computed curves. A large number of points lie below the curve computed for $A_f = 3.0$; this may be primarily due to a reduced effective stress level in the laboratory. Further, any deviation in the value of $\bar{\phi}$ from the assumed relationship between $\bar{\phi}$ and I_p (Fig. 1) will cause significant differences between the actual and computed values of s_u/p , especially at low values of I_p .

Figure 5 shows the results obtained from isotropically consolidated undrained triaxial compression tests on undisturbed samples. All the results plotted are well within the range covered by the computed curves. There is a likelihood that these results follow the A_f contours. It is also seen that these results do not agree with the line representing the relationship given by Eq. 1.

Figures 3, 4 and 5 show that Eq. 1 does not describe the relationship between s_u/p and I_p for many clays. A more detailed analysis of the existing data was not possible because the measured values of A_f were generally not available. Because of this, the authors attempted to study the applicability of Eq. 5 to remoulded soils by conducting a series of laboratory tests under controlled conditions.

Remoulded Clays

It is true that the results of tests on remoulded clays cannot be used directly for the solution of practical problems involving undisturbed clays. Nevertheless, many relationships concerning the physical properties of clays were first determined by means of tests on remoulded clays, and they were found to apply with minor modifications and limitations to undisturbed clays (HVORSLEV, 1960). There are several advantages in using remoulded soils for laboratory experiments. They have better uniformity and a controlled stress history, and many other variables which govern the strength behaviour can be either eliminated or properly accounted for. In this study, tests were

UNDRAINED STRENGTH AND PLASTICITY

conducted on seven clays, basically of the montmorillonite and kaolinite groups. They were chosen primarily because they represent the extreme types of clay covering a wide range of I_p : any natural clay is likely to have a behaviour between these two. Undrained triaxial tests with pore pressure measurement were carried out on isotropically consolidated specimens to measure s_u and A_f . Back pressure was applied to achieve specimen saturation. Extreme care was taken to see that end friction was minimized and that equilization of pore pressure was effected.

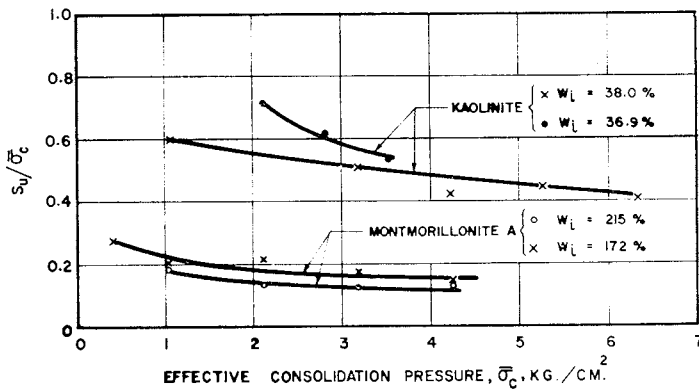


Fig. 6. Relationship between $s_u/\bar{\sigma}_c$ and consolidation pressure for remoulded clays.

Figure 6 shows some typical test results, which clearly bring out the dependency of $s_u/\bar{\sigma}_c$ (1) on the level of the consolidation pressure, $\bar{\sigma}_c$, and also on the initial moulding water content. $s_u/\bar{\sigma}_c$ decreases as $\bar{\sigma}_c$ increases. At any constant value of $\bar{\sigma}_c$, $s_u/\bar{\sigma}_c$ increases with decrease in the initial moulding water content. This corroborates the data of HOUSTON & MITCHELL (1969) whose plots of s_u/p vs p for different water contents also showed that s_u/p increases with decreasing water content at constant effective stress.

Figure 7 shows the relationship between $s_u/\bar{\sigma}_c$ and A_f obtained for remoulded specimens of kaolinite ($I_p = 20\%$), montmorillonite A ($I_p = 495\%$) and montmorillonite B ($I_p = 200\%$). The results clearly show that $s_u/\bar{\sigma}_c$ values are dependent on A_f as well as on the soil type. $s_u/\bar{\sigma}_c$ decreases with increase in A_f , and A_f increases with decrease in plasticity index at constant values of $s_u/\bar{\sigma}_c$. At a constant value of A_f , $s_u/\bar{\sigma}_c$ decreases with increase in I_p . HOUSTON & MITCHELL (1969) also obtained decreasing values of $s_u/\bar{\sigma}_c$ with increasing A_f . It is quite possible that this behaviour observed for remoulded soils may hold good for undisturbed soils also. It could be

(1) For remoulded soils, the effective overburden pressure, p , is replaced by the consolidation pressure, $\bar{\sigma}_c$.

SRIDHARAN AND RAO

reasoned that the good agreement between some of the test results reported in the published literature and Eq. 1 (Skempton's correlation) may be due to the values of A_f for those particular soils.

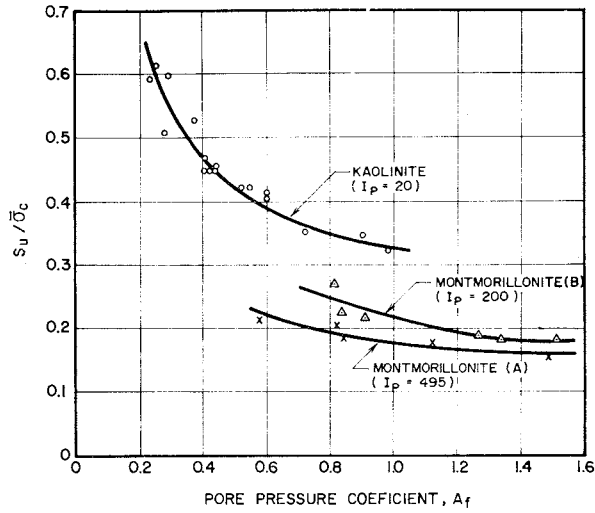


Fig. 7. Relationship between $s_u / \bar{\sigma}_c$ and pore pressure parameter A_f for three remoulded clays.

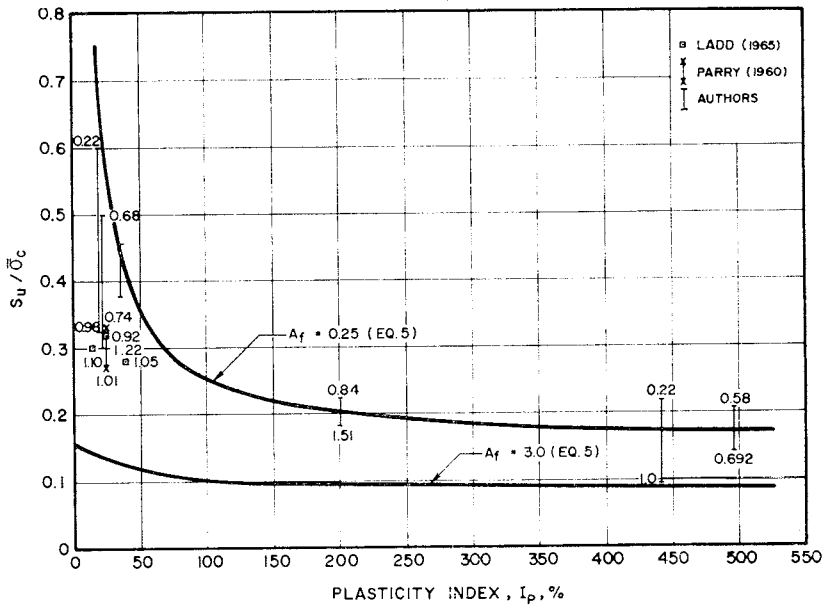


Fig. 8. Relationship between $s_u / \bar{\sigma}_c$ and plasticity index for isotropically consolidated undrained triaxial tests on remoulded clays.

Figure 8 shows the plot of $s_u / \bar{\sigma}_c$ vs I_p for the remoulded soils with their A_f values shown against them. Since $s_u / \bar{\sigma}_c$ is dependent on $\bar{\sigma}_c$, A_f and the

UNDRAINED STRENGTH AND PLASTICITY

initial moulding water content for the same plasticity index, a range of $s_u/\bar{\sigma}_c$ values are obtained, and these are shown by a vertical line covering the complete range of $s_u/\bar{\sigma}_c$ values. Also shown in the figure are the $s_u/\bar{\sigma}_c$ vs I_p relationships computed using Eq. 5, and the relationship between $\bar{\phi}$ and I_p (Fig. 2) for the two extreme A_f values of 0.25 and 3.0. It is clear that the trend of the experimental results follows the computed curves. Since this figure pertains to data obtained from tests on remoulded clays, no comparison can be made with Skempton's line (Eq. 1).

From the foregoing discussion it is clear that several factors affect the relationship between $s_u/\bar{\sigma}_c$ and I_p . Primary among them is the value of A_f . The magnitude of A_f is dependent upon the soil type, soil structure and stress level. Generally, the correlation given by SKEMPTON (1954) does not hold. It seems to be just a coincidence that some of the experimental results support Eq. 1, and this may possibly be due to the fact that variations in the value of A_f for some clays are such as to cause this correlation to hold.

SUMMARY AND CONCLUSIONS

On the basis of the authors' experimental data and the data which are available from the published literature, it is concluded that no linear increase in s_u/p occurs with increase in plasticity index as suggested by SKEMPTON (1954). Instead, both the theoretical analysis and the experimental results tend to show that s_u/p decreases as I_p increases.

Several factors affect the ratio s_u/p and, hence, any attempt to arrive at a unique relationship between s_u/p and I_p is not justified. The many factors which influence s_u/p are such as to result in a band rather than a single line for the relationship between s_u/p and I_p . The significant influence of the pore pressure parameter A_f on $s_u/\bar{\sigma}_c$ is evident from this study.

ACKNOWLEDGEMENTS

The authors are thankful to Dr. T.S. Nagaraj, Assistant Professor and Dr. B.V. Ranganatham, Professor, Department of Civil and Hydraulic Engineering, Indian Institute of Science, Bangalore for their keen interest in this investigation. They are indebted to Mr. G.V. Rao, Senior Research Fellow, for his help and criticism during this study. The fellowships offered by the University Grants Commission and Council of Scientific and Industrial Research to the junior author are gratefully acknowledged. The authors are thankful to the authorities of the Indian Institute of Science for providing the necessary facilities.

REFERENCES

- AAS, G. (1965), A Study of the Effect of Vane Shape and Rate of Strain on the Measured Values of in situ Shear Strength of Clays, *Proc. 6th. Int. Conf. Soil Mech. Found. Eng.*, Montreal, Vol. 1, pp. 141 - 145.
- BISHOP, A.W. and HENKEL, D.J. (1962), *The Measurement of Soil Properties in the Triaxial Test*, Edward Arnold, London, 2nd Edition.
- BJERRUM, L. (1954), Geotechnical Properties of Norwegian Marine Clays, *Géotechnique*, Vol. 4, pp. 49 - 69.
- BJERRUM, L. (1961), The Effective Shear Strength Parameters of Sensitive Clays, *Proc. 5th Int. Conf. Soil Mech. Found. Eng.*, Paris, Vol. 1, pp. 23 - 28.
- BJERRUM, L. and LANDVA, A. (1966), Direct Simple Shear Tests on a Norwegian Quick Clay, *Géotechnique*, Vol. 16, pp. 1 - 20.
- BJERRUM, L. and LO, K.Y. (1963), The Effect of Aging on the Shear Strength Properties of a Normally Consolidated Clay, *Géotechnique*, Vol. 13, pp. 147 - 157.
- BJERRUM, L. and SIMONS, N.E. (1960), Comparison of Shear Strength Characteristics of Normally Consolidated Clays, *Proc. Research Conf. on Shear Strength of Cohesive Soils*, A.S.C.E., Boulder, Colorado, pp. 711 - 726.
- BROWN, J.D. (1970), *Discussion on "Some Observations on the Undrained Shearing Strength Used to Analyze a Failure"*, *Canadian Geotech Jour.*, Vol. 7, pp. 343 - 344.
- COX, J.B. (1970), Shear Strength Characteristics of the Recent Marine Clays in South East Asia, *Jour. S.E. Asian Soc. Soil Eng.*, Vol. 1, pp. 1-28.
- CRAWFORD, C.B. and EDEN, W.J. (1965), A comparison of Laboratory Results with in-situ Properties of Leda Clay, *Proc. 6th Int. Conf. Soil Mech. Found. Eng.*, Montreal, Vol. 1, pp. 31-35.
- DOS M PAIS-CUDDOU, I.C., DESAI, M.D. and KHILNANI, K.S. (1967), Investigation of Marine Clays by Vane Test, *Proc. 3rd Asian Regional Conf. Soil Mech. Found. Eng.*, Haifa, Vol. 1, pp. 231 - 234.
- GIBSON, R.E. (1953), Experimental Determination of the True Cohesion and True Angle of Internal Friction in Clays, *Proc. 3rd Int. Conf. Soil Mech. Found. Eng.*, Zurich, Vol. 1, pp. 126 - 130.
- GOLDER, H.Q. and PALMER, D.J. (1955), Investigations of a Bank Failure at Scrapsgate, Isle of Sheppey, Kent, *Géotechnique*, Vol. 5, pp. 55 - 73.
- GOLDER, H.Q. and SPENCE, R.A. (1960), *Author's reply on "Engineering Properties of the Marine Clay at Port Mann, B.C."*, *Proc. 14th Canadian Soil Mech. Conf.* Ottawa, p. 153.
- GRACE, H. and HENRY, J.K.M. (1957), *Discussion on "The Planning and Design of the New Hong Kong Airport"*, *Proc. Inst. Civ. Engrs.*, London, Vol. 7, pp. 305 - 325.
- HOUSTON, W.N. and MITCHELL, J.K. (1969), Property Interrelationships in Sensitive Clays, *Jour. S.M.F. Div., A.S.C.E.*, Vol. 95, pp. 1037 - 1062.
- HVORSLEV, M.J. (1960), Physical Components of the Shear Strength of Saturated Clays, *Proc. Research Conf. on Shear Strength of Cohesive Soils*, A.S.C.E., Boulder, Colorado, pp. 169 - 273.
- KENNEY, T.C. (1959), *Discussion on "Geotechnical Properties of Glacial Lake Clays"*, *Jour. Amer. Soc. Civil Eng.* Vol. 85, No. SM3, pp. 67 - 79.
- KENNEY, T.C. (1960), *Discussion on "A Preliminary Study of the Geotechnical Properties of Varved Clays as Reported in Canadian Engineering Case Records"*, by Metcalf and Townsend, *Proc. 14th Canadian Soil Mech. Conf.*, Ottawa, pp. 242 - 251.
- LADD, C.C. (1965), Stress-Strain Behavior of Anisotropically Consolidated Clays during Undrained Shear, *Proc. 6th Int. Conf. Soil Mech. Found. Eng.*, Montreal, Vol. 1, pp. 282 - 286.
- LADD, C.C. and LAMBE, T.W. (1963), The Strength of 'Undisturbed' Clay Determined from Undrained Tests, *Proc. Symp. Laboratory Shear Testing of Soils*, A.S.T.M., S.T.P. No. 361, pp. 342 - 371.

UNDRAINED STRENGTH AND PLASTICITY

- LEONARDS, G.A. (1962) (Editor), *Foundation Engineering*, McGraw-Hill, New York.
- LO, K.Y. (1962), Shear Strength Properties of a Sample of Volcanic Material of the Valley of Mexico, *Géotechnique*, Vol. 12, pp. 303 - 316.
- LUMB, P. and HOLT, J.K. (1968), The Undrained Strength of a Soft Marine Clay from Hong Kong, *Géotechnique*, Vol. 18, pp. 25 - 36.
- METCALF, J.B. and TOWNSEND, J.B. (1960), A Preliminary Study of the Geotechnical Properties of Varved Clays as Reported in Canadian Engineering Case Records, *Proc. 14th Canadian Soil Mech. Conf.*, Ottawa, pp. 203 - 225.
- MILLIGAN, V., SODERMAN, L.G. and RUTKA, A. (1962), Experience with Canadian Varved Clays, *Jour. S.M.F. Div. A.S.C.E.*, Vol. 88, No. SM4, pp. 31 - 67.
- OSTERMAN, J. (1960), Notes on Shearing Resistance of Soft Clays, *Acta Polytechnica Scandinavica*.
- PARRY, R.H.G. (1960), Triaxial Compression and Extension Tests on Remoulded Saturated Clay, *Géotechnique*, Vol. 10, pp. 166 - 180.
- SILVA, F.P. (1953), Shearing Resistance of a Soft Clay Deposit Near Rio de Janeiro, *Géotechnique*, Vol. 3, pp. 300-305.
- SIMONS, N. (1959), Laboratory Tests on Highly Plastic Clay From Seven Sisters, Canada, *Norwegian Geotech. Inst. Pub. No. 31*, pp. 25 - 27.
- SKEMPTON, A.W. (1948), Vane Tests in the Alluvial Plain of the River Forth Near Grangemouth, *Géotechnique*, Vol. 1, pp. 111 - 124.
- SKEMPTON, A.W. (1954), *Discussion on "The Structure of Inorganic Soil"*, *Proc. A.S.C.E.*, Vol. 80, pp. 19 - 22, (Separate No. 478).
- SKEMPTON, A.W. and BISHOP, A.W. (1950), The Measurement of Shear Strength of Soils, *Géotechnique*, Vol. 2, pp. 90 - 116.
- SKEMPTON, A.W. and BISHOP, A.W. (1954), "Soils" (Ch. X) in *Building Materials—Their Elasticity and Inelasticity*, (M. Reiner Ed.), North Holland Pub. Co., Amsterdam.
- SKEMPTON, A.W. and HENKEL, D.J. (1953), The Post Glacial Clays of the Thames Estuary at Tilbury and Shellhaven. *Proc. 3rd. Int. Conf. Soil Mech. Found. Eng. Zurich*, Vol. 1, pp. 302-308.
- SKEMPTON, A.W. (1957). Discussion, *Proc. Inst. Civ. Engrs.* London, Vol. 7, pp. 305.
- WU, T.H. (1958), Geotechnical Properties of Glacial Lake Clays, *Jour. S.M.F. Div. A.S.C.E.*, Vol. 84, No. SM3, pp. 1732-1—1732-34.

DISCUSSION OF PAPERS

MECHANISTIC INTERPRETATION OF COMPRESSION CHARACTERISTICS OF A SOFT MARINE CLAY†

ZA-CHIEH MOH, EDWARD W. BRAND and ALFONSO S. TEVES

Philip Keene*

The authors are to be congratulated on their intelligent investigation of consolidation aspects of an important type of clay. Their mechanistic interpretation appears to be especially noteworthy and is the sort of investigation which will eventually enable us to answer some frustrating questions on clay behavior.

It is noted that, under *Secondary Compression*, the authors state that R_s , the secondary deformation per log cycle of time, is the coefficient of secondary compression. The writer believes that the most frequent definition of coefficient of secondary compression in North America (e.g. ALDRICH, 1964; CRAWFORD & SUTHERLAND, 1971; JOHNSON, 1970; KAPP, 1966; KEENE, 1964; KEENE & ZAWODNIAK, 1968; U.S. NAVY, 1961) is the unit deformation per log cycle of time and is shown by the symbol C_α , defined as:

$$C_\alpha = \frac{\Delta e}{1 + e} / \Delta \log t = \frac{\Delta H}{H} / \Delta \log t$$

where H is the thickness of the stratum. A recent paper by MESRI (1973) lists various definitions of the coefficient of secondary compression; Mesri uses the symbol ϵ for this coefficient.

It is hoped that by the time that further investigations in the laboratory and in the field have greatly increased our knowledge of the complexities of secondary compression, a uniform definition of the coefficient of secondary compression will be agreed upon. Such a definition should involve the thickness of the sample or stratum, since the thickness strongly influences the amount of secondary deformation. The definition should state whether the thickness is the original thickness of the compressible soil or its thickness prior to or during the load increment being studied. For most clays, the difference is small but, for peaty soils and very soft clays, the difference may be large.

† Published in Vol. 3, No. 1, 1972, pp. 21-40.

* Consulting Engineer; formerly Head, Soils and Foundations Division, Connecticut Dept. of Transportation, U.S.A.

DISCUSSION

Also, as suggested earlier by the writer (KEENE, 1964), consideration should be given to furnishing another coefficient or index number, defined as the ratio of secondary compression per log cycle of time to primary compression. This would be especially useful on stage construction projects or later improvement projects, where a large original load has caused substantial primary and secondary settlements, and some time thereafter an additional small load is added. It would be unrealistic to assume that the latter would cause secondary settlement of the same magnitude as that caused by the original load.

Of interest are the secondary compression data from the three curves in Fig. 6 of the paper. These show a secondary compression of about 0.025 cm/log cycle of time, which makes $C_{\alpha} = 0.013$ (approximately). The latter is in the range of values reported by the writer and, chiefly for organic silt-clay, by MESRI (1973). Since the secondary compression varies greatly with the consolidation pressure (Figs. 9 and 10), the "agreement" cited above must be viewed with this in mind.

Authors' Reply

The authors thank Mr. Keene for his contribution on the subject of second-

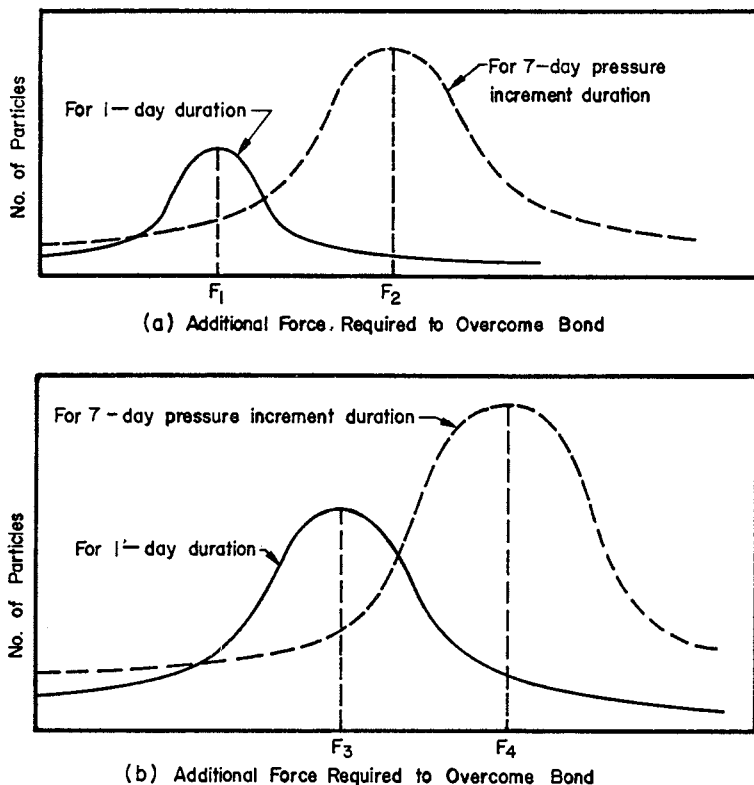


Fig. 12. Assumed normal frequency distribution of bond forces.

DISCUSSION

dary compression. They must agree that a universally accepted definition of coefficient of secondary compression is desirable, and that the most rational definition is strain per log cycle of time, as suggested by Mr. Keene. The specimens used to obtain the data presented in Figs. 9 and 10 of the paper were 0.75 in. thick and the values of C_{α} , as defined by Mr. Keene, are between about 0.01 and 0.02. The authors also support the usefulness of an index specifying the ratio of secondary to primary compression.

Unfortunately, a serious drafting error occurred to Fig. 12 which was not noticed at the proof stage. It seems appropriate to reproduce the corrected figure here.

REFERENCES

- ALDRICH, H.P. (1964), Precompression for Support of Shallow Foundations, *Jour. S.M.F. Div., A.S.C.E.*, Vol. 90, No. SM5, pp. 471-486.
- CRAWFORD, C.B. and SUTHERLAND, J.G. (1971), The Empress Hotel, Victoria, B.C.: Sixty-five Years of Foundation Settlements, *Canadian Geotechnical Jour.*, Vol. 8, pp. 77-93.
- JOHNSON, S.J. (1970), Precompression for Improving Foundation Soils, *Jour. S.M.F. Div., A.S.C.E.*, Vol. 96, No. SM1, pp. 111-144.
- KAPP, M.S. (1966), Construction on Marshland Deposits, *Highway Research Record No. 133*, pp. 4-15.
- KEENE, P. (1964), Discussion of Horn & Lambe, "Settlements of Buildings on the M.I.T. Campus", *Jour. S.M.F. Div., A.S.C.E.*, Vol. 90, SM5, pp. 232-244.
- KEENE, P. and ZAWODNIAK, C.D. (1968), Embankment Construction on Peat Utilizing Hydraulic Fill, *Proc. 3rd. Int. Peat Congress*, Quebec, pp. 45-50.
- MESRI, G. (1973), Coefficient of Secondary Compression, *Jour. S.M.F. Div., A.S.C.E.*, Vol. 99, pp. 123-137.
- U.S. NAVY (1961), *Design Manual, Navdocks DM-7*, Sec. 3, pp. 11-15.

MEASUREMENT OF TENSILE STRENGTH OF COMPACTED SOIL†

B. SATYANARAYANA and K. SATYANARAYANA RAO

V. Thanikachalam*

The writer congratulates the authors on their comparative study of the measurement of tensile strength of compacted soils. The authors have used beam tests, direct tension tests, cube diagonal split tests, prism split tests, briquette tests and cube split tests. From their study, they have recommended the use of the cube diagonal split test over all the other tests without

† *Technical Note* published in Vol. 3, No. 1, 1972, pp. 61-66.

* Senior Research Fellow (CSIR), Hydraulics and Water Resources Department, College of Engineering, Madras, India.

DISCUSSION

considering its limitations. In fact, the use of the direct tensile test is preferable, which may be brought out from analyses of the assumptions made in the various tests and their states of failure.

The methods of determining tensile strength may be grouped into the following categories:

1. Flexural tests (beam tests).
2. Indirect tensile tests (splitting tests).
3. Direct tensile tests.

In the case of flexural tests, elastic behaviour of the beam is assumed. This results in an overestimate of the tensile strength. Also, it has been shown experimentally that the tensile strength depends on the dimensions of the beam, the larger the beam the lower the tensile strength (WRIGHT, 1955).

Indirect tensile tests rely on the development of tension across the diameter of a cylinder or diagonal of a cube by the application of compressive stress. The tensile stress at failure is calculated on the false assumption that the specimen remains elastic up to failure. The stress system from biaxial compression immediately underneath the packer gives compression-tension in the centre section with the magnitude and sign of the third principal stress parallel to the longitudinal axis being unknown. The measured strength is therefore unlikely to be a direct function of the tensile strength but may be a complex function of the strength and size (HANNANT, 1972). Further, the split-tensile test does not provide a test loading condition to resemble that in the field, nor does it permit determination of tensile strain during loading (WANG & HUSTON, 1971).

The direct tensile test has the advantage that it is the only tensile test in which the tensile stress at failure is known with accuracy. Also, it does not rely on the assumptions of elasticity or plasticity for the calculation of the tensile stress at failure. Further, the application of quick-setting epoxy resins for fixing the specimens to the moulds eliminates the thixotropic effects that may be introduced as a result of Araldite application.

The relation between tensile strength of a compacted soil and the moulding water content is governed by the percentage of clay. If the clay content increases, the tensile strength also increases with increase in the moulding water content. But if the clay content decreases, the tensile strength decreases with increase in the moulding water content (MOORE et al, 1971).

The main disadvantage of the direct tensile test, as discussed in the paper, is that failure takes place only at the gripping ends which casts doubt on the results. The results presented in the paper also showed that tensile strengths from cube diagonal split tests were lower than those from direct tensile

DISCUSSION

tests. Hence, the authors recommended only the use of cube diagonal split tests in view of the type and zones of failure and the reproducibility of the results.

H.Y. Fang*

The authors have presented interesting and comprehensive test data on the comparison of various methods including the split tensile, beam, direct tensile, and briquette tests for measuring the tensile strength of compacted soil. It may be of interest to note that, in addition to the above mentioned methods, there is available a new tensile test known as the double-punch test (FANG & CHEN, 1971, 1972). The double-punch test has proved to be a simple and reliable test. The test results have been compared with the split tensile test using various materials including soil, concrete (CHEN, 1970, CHEN & TRUMBAUER, 1972), mortar, bitumen and cement treated bases, and rock (DISMUKE et al, 1972). Good agreement between both tensile strength results is observed. The double-punch test may be briefly described as follows: by the use of two steel discs (punches) centered on both top and bottom surfaces of a cylindrical soil specimen, a vertical load is applied on the discs until the specimen reaches failure. The tensile strength of the specimen can be calculated from the maximum load by the formula:

$$\sigma_t = \frac{P}{\pi (KbH - a^2)} \dots \dots \dots (1)$$

in which σ_t is the tensile strength, P is the maximum load, b is the radius of the specimen, H is the height of the specimen, a is the radius of the disc and K is a constant. Recommended values of K are:

	Soil	Stabilized Materials
Proctor mold, 4 × 4.6 in.	1.0	1.2
CBR mold, 6 × 7 in.	0.8	1.0

It has been found that height/diameter ratios of specimens of 0.8 to 1.2 and ratios of the diameter of the specimen to the diameter of the disc of 0.2 to 0.3 are suitable for the test. The rate of strain used for the double-punch test is the ASTM loading rate for the unconfined compression test.

The double-punch test can be conveniently performed in conjunction with routine CBR and compaction tests. The other test methods measure the tensile strength across a predetermined failure plane, whereas the double-

* Associated Professor of Civil Engineering and Director of the Geotechnical Engineering Division, Lehigh University, U.S.A.

DISCUSSION

punch test always causes failure on the weakest plane, which results in the measurement of the true tensile strength.

Authors' Reply

The authors very much appreciate Mr. Thanikachalam's keen interest in their paper. The advantages of the cube diagonal split test over the flexural test and the direct tensile test were discussed in detail in the paper, sufficient emphasis having been given to the types and zones of the failure planes. The assumption that the soil specimen remains elastic up to failure is not false and has been shown experimentally to be perfectly true (SATYANARAYANA & JAYARAM, 1972). It is a very well established fact that the tensile strength is a function of the unconfined compressive strength but is much lower (RAWAT, 1968).

The authors appreciate Dr. Fang's contribution. He described the double-punch test developed recently and points out that this can be conveniently performed in conjunction with routine CBR and compaction tests. The authors will extend their studies on tensile strength by using the double-punch test.

REFERENCES

- CHEN, W.F. (1970), Double-Punch Test for Tensile Strength of Concrete, *Jour. Am. Conc. Inst.*, Vol. 67, pp. 993 - 995.
- CHEN, W.F. and TRUMBAUER, B.E. (1972), Double-Punch Test and Tensile Strength of Concrete, *Jour. of Materials*, A.S.T.M., Vol. 7, pp. 148 - 154.
- DISMUKE, T.D., CHEN, W.F. and FANG, H.Y. (1972), Tensile Strength of Rock by the Double-Punch Method, *Jour. Int. Soc. Rock Mech.* (in press).
- FANG, H.Y. and CHEN, W.F. (1971), New Method for Determination of Tensile Strength of Soils, *Highway Research Record*, No. 354, pp. 62 - 68.
- FANG, H.Y. and CHEN, W.F. (1972), Further Study of Double-Punch Test for Tensile Strength of Soils, *Proc. Third Southeast Asian Conf. Soil Eng.*, Hong Kong (in press).
- HANNANT, D.J. (1972), The Tensile Strength of Concrete: A Review Paper, *The Structural Engineer*, No. 7, Vol. 50, pp. 253 - 258.
- MOORE, R.K., KENNEDY, T.W. and HOZUH, J.A. (1971), Tensile Properties for Design of Lime Treated Materials, *Highway Research Record No. 351*, pp. 78 - 88.
- RAWAT, P.C. (1968), *Tensile Strengths of Compacted Soils*, M.E. Thesis, Roorkee University, Roorkee, India.
- SATYANARAYANA, B. and JAYARAM, H.V. (1972), Tensile Stress Strain Characteristics of Compacted Soil, *Proc. 3rd Southeast Asian Conf. Soil Eng.*, Hong Kong, 1972 (in press).
- WANG, M.C. and HUSTON, M.T. (1971), Direct-Tensile Stress and Strain of a Cement Stabilized Soil, *Highway Research Record No. 379*.
- WRIGHT, P.J.F. (1952), The Effect of the Method of Test on the Flexural Strength of Concrete, *Magazine of Concrete Research*, No. 11, pp. 67 - 76.
- WRIGHT, P.J.F. (1955), Comments on an Indirect Tensile Test of Concrete Cylinders, *Magazine of Concrete Research*, No. 20, pp. 87 - 96.

INTERNATIONAL SOCIETY NEWS

Dr. Laurits Bjerrum Dies



Dr. Laurits Bjerrum, Director of the Norwegian Geotechnical Institute, died suddenly on 23 February at the age of 54. As he had done so often in the past, Dr. Bjerrum was in London to participate in the annual Rankine Lecture and to give a special lecture at Imperial College. His death represents a tragic loss to the entire international geotechnical community.

Dr. Bjerrum received a degree in Civil Engineering in 1941 from the Technical University of Denmark in Copenhagen. He was employed by a consulting engineering firm in Denmark from 1941 to 1947, and then joined the staff of the Swiss Federal Institute of Technology in Zurich, where he was awarded a Doctorate in Engineering in 1952. In 1951, he assumed leadership of the newly established Norwegian Geotechnical Institute which, under his able leadership and inspiration, attained undisputed international fame.

Dr. Bjerrum made an outstanding contribution to the field of Geotechnical engineering. His many publications are well-known. He was Vice-President for Europe of the International Society from 1961 to 1965 and President from 1965 to 1969. He delivered one of the first Terzaghi Lectures in 1966 as well as the seventh Rankine Lecture in London in 1967. In 1971, he received the Terzaghi Award of the American Society of Civil Engineers.

Laurits Bjerrum had friends throughout the world who will mourn his passing and be thankful for the privilege of having known him. Our thoughts go out to his wife and children in Oslo and to his friends and colleagues at the Norwegian Geotechnical Institute.

List of Members

The 1972 printed List of Members of the International Society is now available. The lists were edited and the proofs were checked by the Southeast Asian Society of Soil Engineering, the work being largely done by Mr. Peter

Lumb, assisted by Dr. John D. Nelson and Professor Za-Chieh Moh. More than half of the 9411 members are seen to be in Europe (4800); there are 43 member countries, with Western Germany having the largest membership (900) and Ireland (6) the smallest. This list is produced every four years and is timed to appear before the International Conference. In addition to names and addresses of members, the list includes the ISSMFE Constitution and By-Laws.

Copies for members are available free of charge from the Secretaries of their National Societies, or at US \$ 25.00 from the Secretary General ISSMFE, c/o Institution of Civil Engineers, Great George Street, London, SW1P 3AA, England.

SOUTHEAST ASIAN SOCIETY NEWS

The Fourth Conference on Soil Engineering

The Society's Fourth Conference on Soil Engineering is planned to take place in Kuala Lumpur from 7 to 10 April 1975. The Fourth Conference follows the first three held in Bangkok (1967), Singapore (1970) and Hong Kong (1972); all of these were a great success. This early notice has been given so that Society members and others may prepare their contributions and make plans to attend. The Society Conferences present an opportunity for those interested in soil engineering to spend a few days discussing problems of mutual interest. Participants have found past Conferences to be of great professional value, especially since the papers and discussions have tended to focus on problems of the region.

The Fourth Conference will be sponsored by the Southeast Asian Society of Soil Engineering, the Institution of Engineers, Malaysia, and the Asian Institute of Technology. The language of the Conference will be English. The Organizing Committee intends to invite an eminent guest lecturer in the field of soil engineering.

There will be no special theme for the Conference. Papers may be submitted on general topics of soil mechanics and engineering geology dealing with testing and site investigations, foundations, earth dams, slope stability, and roads and runways. Three copies of a preliminary title and brief summary of each paper are required to be submitted by 15 January 1974. Final manuscripts will be required by 15 July 1974.

Bulletin No. 1, which gives full details about the Conference and the procedure for the submission of papers, is now available from The Secretary, IV SEACSE, c/o Institution of Engineers, Malaysia, P.O. Box 223, Petaling Jaya, Selangor, Malaysia.

CONFERENCE NEWS

The Fourth Southeast Asian Conference on Soil Engineering will be held in Kuala Lumpur, Malaysia from 7 to 10 April 1975. It will be sponsored by the Southeast Asian Society of Soil Engineering, the Institution of Engineers Malaysia, and the Asian Institute of Technology. The language of the Conference will be English.

There will be no special theme for the Conference. Papers may be submitted on general topics of soil mechanics and engineering geology dealing with testing and site investigations, foundations, earth dams, slope stability, and roads and runways. Three copies of a preliminary title and brief summary of each paper are required to be submitted by 15 January 1974. Final manuscripts will be required by 15 July 1974.

Bulletin No. 1 is now available from The Secretary, IV SEACSE, c/o Institution of Engineers, Malaysia, P.O. Box 223, Petaling Jaya, Selangor, Malaysia.

A Conference on the Settlement of Structures will be held at Cambridge University from 2 to 4 April 1974. The proceedings will be conducted under the headings of (1) Granular materials (2) Normally consolidated and lightly over-consolidated cohesive materials, (3) Heavily over-consolidated cohesive materials (4) Rocks, and (5) Allowable and differential settlements, including damage to structures and soil-structure interaction.

For each session there will be a General Reporter who will prepare a state-of-the-art paper covering existing knowledge as well as the data presented in papers and technical notes. Review papers, papers and technical notes will be issued to participants about one month before the Conference.

Correspondence about the Conference should be addressed to The Secretary, Settlement of Structures Conference, The Institution of Civil Engineers, Great George Street, London SW1, England.

The Sixth European Conference on Soil Mechanics and Foundation Engineering will be held in Vienna in March 1976 on the general theme *Deep Foundation and Deep Excavations*. The technical sessions will be:

- (1) Deep excavations; stability of temporary and permanent slopes; dewatering problems, slurry walls, walls with batter piles; bracing, freezing techniques.
- (2) Deep foundations; tunnelling.
- (3) Deep foundations in open pits; pile foundations; caisson foundations.

All correspondence should be addressed to the Secretary, VI European Conference on S.M.F.E., A 1040 Wien, Technische Hochschule, Karlsplatz 13, Austria.

A Specialty Conference on Analysis and Design in Geotechnical Engineering will be held from 9 to 12 June 1974 at The University of Texas at Austin under the auspices of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers.

There will be five general sessions, four of which will feature a state-of-the-art speaker on analysis and design relating to embankments, foundations for light structures, foundations for heavy structures, and underground structures. The fifth general session will be an overview of the Conference and will include the participation of a panel.

A feature of this Specialty Conference will be a series of workshops, in which speakers will present methods of analysis and design in a variety of areas, probably making use of documented computer programs, with audience participation. The speakers will make available copies of documentation to workshop participants at the cost of reproduction.

Papers related to the theme of the conference are solicited, and those accepted will be published in the Proceedings of the Conference along with the addresses of the state-of-the-art speakers and workshop leaders. Finished copies of papers (typed on available model paper) must be submitted no later than 15 December 1973.

Those wishing to participate in the Conference should write now to The Secretariat, 1974 ASCE Conference, Taylor Hall 161, The University of Texas, Austin, Texas 78712, U.S.A.

A European Symposium on Penetration Testing is planned for 5 to 7 June 1974 in Stockholm, Sweden. The aim of the Symposium is to document the use of penetrometer testing in soil investigations, to outline areas where further research is desirable, to investigate the need for standardisation, and to provide guidelines for future developments.

The organizers have limited the number of participants to about 80 specialists comprising one to five delegates from each national society in Europe. National societies are invited to contribute state-of-the-art reports about the practice of penetration testing in their countries and, in addition, individual papers are invited. The Proceedings will be published in two volumes.

Information can be obtained from Dr. Bengt Broms, Swedish Geotechnical Institute, Banergatan 16, Stockholm, Sweden.

The **25th International Geological Congress** will be held in Sydney, Australia from 16 to 25 August 1976. Those who wish to receive a copy of the first circular when it is available are asked to write to The Secretary-General, 25th International Geological Congress, P.O. Box 1892, Canberra City, ACT 1601, Australia.

The **Third International Congress on Rock Mechanics** of the International Society for Rock Mechanics will be held in Denver, Colorado, U.S.A. from 1 to 7 September 1974. The object of this Congress is to ascertain on an international scale the advances that have been made in rock mechanics since the Second International Congress, and to indicate directions for future effort.

Attention will be centred on the major themes (1) Physical properties of intact rock and rock masses, (2) Tectonophysics, (3) Surface workings, (4) Underground openings, and (5) Fragmentation systems. At each Plenary Session, the major theme will be reviewed by a Congress General Reporter and a panel will respond to questions submitted prior to the Congress sessions. Selected individual papers will be presented and discussed at the discussion sessions. Brief questions from the floor will be permitted if time allows. The Congress technical sessions will be presented in English, French and German by simultaneous translation.

Bulletin No. 1 contains details of the Congress, including information on submission of papers. This can be obtained from Secretary, Third International Congress on Rock Mechanics, 2101 Constitution Avenue NW, Washington, D.C. 20418, U.S.A.

The **Third International Conference on Expansive Soils** will be held at the Israel Institute of Technology, Haifa, Israel during the week of 30 July to 4 August 1973. This is the third conference of this type, following the Second International Research and Engineering Conference on Expansive Clay Soils which was held at Texas A & M University in August 1969.

The primary aim of the Conference is to discuss the latest developments in research, experience, and prediction of expansive clay behaviour in relation to engineering problems. Papers have been accepted on the themes (1) Expansive soil properties and behaviour (laboratory, field, expansion-shrinkage prediction), (2) Structural performance (buildings, pavements, etc.), and (3) Interaction of soil and engineering structures. Correspondence regarding the Conference, should be addressed to Secretary, Third International Conference on Expansive Soils, Soil Engineering Building, Israel Institute of Technology, Technion City, Haifa, Israel.

A **Regional Conference on Tall Buildings** will be held in Bangkok, Thailand from 23 to 25 January 1974 under the sponsorship of the Asian Institute of Technology and under the auspices of the Joint Committee on Planning and Design of Tall Buildings of the American Society of Civil Engineers and the International Association for Bridge and Structural Engineering.

A large number of paper have been accepted on the topics (1) Structural systems, methods of analysis and design structural stability, (2) Foundation design, (3) Wind loads, (4) Earthquake loading and response, (5) Construction methods and management, (6) Environmental and service systems, (7) Structural safety and quality control, and (8) Fire protection and fire resistance. The papers accepted for publication will be printed in the Conference proceedings for distribution before the time of the Conference.

Information is available from Secretary, Regional Conference on Tall Buildings, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

An **International Symposium on Development of Ground Water Resources with Special Reference to the Asian Region** will be held in Madras, India from 26 to 29 November 1973. The technical sessions will cover (1) Hydrogeology of aquifers and methods of prospecting for water, (2) Hydrological processes affecting subsoil water tables, (3) Instrumentation and methods of observations, (4) Aquifer management and optimization, (5) Special problems in ground water, and (6) Planning and administration in ground water development.

All correspondence should be addressed to Secretary, International Symposium on Development of Ground Water Resources, College of Engineering, Madras-25, India.

A **Symposium on Engineering-Geological Problems Related to Soluble Rocks Collapses and Subsidence** will be held in Hannover, Germany from 10 to 13 September 1973. The themes will be (1) Geologic and geochemical conditions of occurrence of landfalls and natural settlements, (2) Mechanism of landfall and settlement, (3) Methods of location of underground cavities, (4) Case histories, and (5) Practical precautions during construction in landfall areas.

All correspondence should be addressed to Dr. M. Lager, Bundesanstalt für Bodenforschung, 3 Hannover Buchholz, Alfred-Bentz Haus, Postfach 34, Stille-Weg 2, Germany.

NEWS OF PUBLICATIONS

The volume of **Proceedings of the Second Southeast Asian Conference on Soil Engineering**, held in Singapore in June 1970, is now available at a price of U.S. \$ 18 with hard covers. Orders for this volume should be sent to the Geotechnical Engineering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand, Cheques should be made payable to "Asian Institute of Technology".

The two volumes of **Proceedings of the Fourth Asian Regional Conference on Soil Mechanics and Foundation Engineering**, Bangkok, July 1971 are available at U.S. \$ 30 from the Geotechnical Engineering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

Proceedings of the Symposium on Shallow Foundations, held in Bombay in November 1970, are available in two volumes for U.S. \$ 20 from M/S Sarita Prakashan, 175 Nauchandi Ground, Meerut, India.

Proceedings of the Symposium on Application of the Finite Element Method in Geotechnical Engineering, held in Vicksburg, Mississippi in May 1972, are now available. The Proceedings contain 36 papers including six state-of-the-art reports on (1) General Review and Theory (2) Dams, Excavations and Slopes, (3) Foundations and Pavement, (4) Seepage, Consolidation and Creep, (5) Earthquake Analysis and Dynamics, and (6) Earth-Structure Interaction. Payment of U.S. \$ 20 made payable to "Treasurer of the United States, Vicksburg, Miss." should be sent to Director, USAE Waterways Experiment Station, P.O. Box 631, Vicksburg, Miss. 39180, U.S.A.

Proceedings of the A.S.C.E. Symposium on Underground Rock Chambers, held at Phoenix, Arizona in January 1971, are available for U.S. \$ 14 from American Society of Civil Engineers, 345 East 47th Street, New York, N.Y. 10017, U.S.A.

Terzaghi Library Memories have recently been initiated by the Norwegian Geotechnical Institute. No. 1 of this series is now available free of charge. Anyone who wishes to receive the future editions should write to Mr. Finn Jorstad, Norwegian Geotechnical Institute, Forskningsvn 1, Oslo 3, Norway.

Under the sponsorship of the International Society, the German National Society of Soil Mechanics and Foundation Engineering are publishing their

Geotechnical Abstracts. These abstracts provide a regular worldwide literature information service in the fields of soil mechanics, foundation engineering, rock mechanics and engineering geology. The abstracts are published monthly, at an annual subscription rate of DM 240, by Deutsche Gesellschaft für Erd-und Grundbau, 35a Kronprinzenstrasse, 43 Essen, Germany.

Proceedings of the Symposium on Strength and Deformation Behaviour of Soils, held at Bangalore, India in February 1972, are available at a cost of U.S. \$ 20 (two volumes) from Indian Geotechnical Society, University College of Engineering, Bangalore 1, India.

Proceedings of the Roscoe Memorial Symposium, held at Cambridge University in March 1972 in honour of the late Professor K.H. Roscoe, are now available under the title *Stress-Strain Behaviour of Soils*. The volume of Proceedings, containing 36 papers, four general reports and discussion is available at a price of £12.80 from the publisher, G.T. Foulis & Co. Ltd., 50a Bell Street, Henley-on-Thames, Oxfordshire, England.

Proceedings of the First Canadian Conference on Earthquake Engineering (1971) are now available for distribution and may be purchased for U.S. \$ 15 through The Bookstore, University of British Columbia, Vancouver 8, B.C., Canada. Cheques or money orders made payable to "University of British Columbia" should accompany each order.

The three volumes of **Proceedings of the A.S.C.E. Specialty Conference on the Performance of Earth and Earth-Supported Structures**, held at Purdue University in June 1972, are now available at a cost of U.S. \$ 17.50 from American Society of Civil Engineers, 345 East 47th Street, New York, N.Y. 10017, U.S.A.

Two volumes of **Proceedings of the Specialty Session on Lateritic Soils**, held at the Seventh International Conference on Soil Mechanics and Foundation Engineering in Mexico City in 1969, are available for U.S. \$ 25 from the Geotechnical Engineering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

Proceedings of the 1972 North American Rapid Excavation and Tunnelling Conference are now available. The price of the hard-bound two volumes (1664 pages) is U.S. \$ 29. These are obtainable from A.I.M.E., 345 East 47th Street, New York, N.Y. 10017, U.S.A.

Proceedings have now been published of the **First International Conference**

on the Application of Statistics and Probability to Soil and Structural Engineering, which was held in Hong Kong in September 1971. This volume, which was edited by Mr. Peter Lumb, is priced at HK \$ 60 and is obtainable from Hong Kong University Press, 94 Bonham Road, Hong Kong.

Publication has just begun of the **Abstract Journal in Earthquake Engineering**. This is being produced by the U.S. National Information Service for Earthquake Engineering. The new journal is international in scope and provides broad, interdisciplinary coverage of the field of earthquake engineering. The abstracts are drawn from pertinent literature in the areas of engineering seismology, seismometry, dynamics of natural and man-made structures, earthquake resistant design and construction, earthquake damage, public policy, disaster prevention, emergency activities and indemnification. Each issue will cover a single calendar year beginning with 1971.

Volume 1 of this new journal is now available from the Earthquake Engineering Research Center, University of California, 1301 South 46th Street, Richmond, California 94804, U.S.A. Subscription rate is U.S. \$ 10.00 per year.

ASIAN INFORMATION CENTER FOR GEOTECHNICAL ENGINEERING — AGE

The *Asian Information Center for Geotechnical Engineering* (Asian Geotechnical Engineering for short, abbreviated to AGE) has been established within the library of the Asian Institute of Technology under the joint sponsorship of its Division of Geotechnical Engineering and the Library. The idea of establishing AGE was conceived at the meeting of representatives of national societies of soil mechanics and foundation engineering in the Asian region which was convened in Bangkok in July 1971. Through one of the resolutions of the meeting, AIT was requested to undertake the task of establishing and operating AGE for the benefit of engineers in Asia and those interested in the region.

The newly established AGE is an invaluable source of information for all those concerned with investigation, feasibility, design and construction for all types of civil engineering projects. In addition, it is indispensable to those concerned with teaching and research in any aspect of geotechnical engineering. It is aiming to serve as a clearing house in the Asian region for information on SOIL MECHANICS, FOUNDATION ENGINEERING, ROCK MECHANICS, ENGINEERING GEOLOGY, EARTHQUAKE ENGINEERING, and other related fields. In cooperation with national societies, universities, governmental agencies, research organizations, engineering and consulting firms, contractors, etc., both within and outside the region, AGE is collecting information on all phases of geotechnical engineering research and projects, including published and unpublished reports which are of relevance to Asian conditions. AGE is undertaking the responsibility of designing a computer based information storage and retrieval system for the effective handling of such information, and is providing both *Current Awareness Service* and *Selective Dissemination of Information Service* through its publication of journal abstracts and subject bibliographies. Dissemination of collected information takes place through photocopying and micro-filming.

The Head Librarian of AIT serves as Director of the Center, which operates as a special administrative unit of the AIT Library, and is guided by a Policy Advisory Committee and a Technical Committee. The Technical Committee meets at least once every two months to advise the Director of the Center on all important technical and operational matters. At the moment, Liaison Officers act as communication links between their country and AGE in Bangkok.

To ensure the effective functioning of AGE as an information center for geotechnical engineering in Asia, it is necessary that all organizations and

individuals who are engaged in any kind of geotechnical engineering work in Asia consider **AGE** as a central depository for their information and publications regardless of language. These publications and other information will be abstracted and analyzed, and will be publicized and disseminated through **AGE**'s various channels to the benefit of the region. Among the major data files which have been started at **AGE** are :

- (1) Data on all design, construction and research projects in geotechnical engineering of concern to the region.
- (2) Data on organizations engaged in any kind of geotechnical engineering work in the region.
- (3) Data on individuals who are engaged in any kind of geotechnical engineering work in the region.
- (4) Data on published papers and technical literature on geotechnical engineering work of concern to the region.

Great benefit can be gained by companies and research organizations treating **AGE** as a central depository and service agent to provide information when it is needed.

AGE has started to publish the following :

- (1) *Asian Geotechnical Engineering Abstracts*: a quarterly publication consisting of abstracts of available publications on geotechnical engineering relevant to Asia.
- (2) *Asian Geotechnical Engineering in Progress*: a semi-annual publication consisting of information on current design, construction and research projects in geotechnical engineering being undertaken in Asia.
- (3) *Asian Geotechnical Engineering Directory*: a bi-annual publication consisting of information on various organizations and individuals who are doing geotechnical engineering work in Asia or relevant to Asia.
- (4) *AGE Current Awareness Service*: published quarterly to inform readers of recent geotechnical engineering publications and contents of geotechnical engineering journals received at **AGE**.
- (5) *AGE Journal Holdings List*: published annually to facilitate the request of photocopies.
- (6) *AGE Bibliography Series*: either recurrent or demand bibliographies published as a result of general interest or demand.

In addition to its publications, **AGE** will provide the following three services:

- (1) *Reference Service*: for bibliographical questions.
- (2) *Referral Service*: for technical questions.
- (3) *Reproduction Service*: for photocopying or microfilming of required documents.

AGE is a non-profit making service organization. For the initial three years, it is being financially supported by a generous grant from the International Development Research Centre of Canada and by the Asian Institute of Technology. It will be necessary, however, for **AGE** to recover a very small portion of its operation costs from fees received on certain services. Much can be gained through membership which is available to Individuals and Institutions at nominal fees.

Anyone who wishes to have details about the Asian Information Center for Geotechnical Engineering should write to Dr. H.W. Lee, Director AGE, P.O. Box 2754, Bangkok, Thailand.

BOOK REVIEWS

The Penetrometer and Soil Exploration by G. Sanglerat, translated from French by G. Gendarme, Elsevier Publishing Co., Amsterdam, 1972, U.S. \$28.

In nearly 500 pages of text, Professor Sanglerat has collected together a vast amount of published data on the use and interpretation of penetrometers of all kinds. Prominence is given to static cone penetrometers of the Dutch type, but dynamic penetrometers are also fully dealt with.

While being something of an encyclopaedia on penetrometers, this book tends to read rather like a catalogue. The treatment is comprehensive in that absolutely all the available literature on the subject is reviewed. Unfortunately, however, each piece of published work is simply reviewed under such headings as *The Work of X* or *The Research of Y*, with little or no discussion. No serious attempt is made to compare the many isolated investigations so minutely described or to synthesise the published results. The reader is confronted with a source book which is certainly the most important work to be published on penetrometers, but it cannot be considered to be a text book. As a reference, it would be excellent except for the regrettable omission of an index.

E.W. Brand

Foundation Instrumentation by T.H. Hanna, Trans Tech. Publications, Aedermannsdorf, Switzerland, and Cleveland, Ohio, U.S.A., 1973, U.S. \$30 (student price \$20).

Professor Hanna has written the first ever comprehensive book on the subject of instrumentation in soil and foundation engineering. After a short introduction to the problems of design and construction in soil engineering, separate chapters deal with the measurement of load, pore pressure, earth pressure, and ground movements. There are chapters on the recording and processing of field data and on instrumentation of laboratory scale foundations. The remaining chapter describes some actual cases of the use of field instrumentation and presents the results obtained. Each section has been carefully planned and written with an abundance of good figures. The coverage is excellent, except perhaps for the omission of a chapter on instrumentation for dynamic measurement. An unusual but most useful feature is the inclusion of a list of instrument manufacturers.

This book will fill a gap in the existing range of literature on geotechnical engineering. It will be invaluable to practicing engineers and will be the choice for university courses concerned with this subject. There is no doubt that it will be accepted as the standard work of reference on field instrumentation.

E.W. Brand

NOTES ON CONTRIBUTIONS TO THIS JOURNAL

Contributions to **Geotechnical Engineering** are invited from anyone. Items submitted to the Editor will be published under one of the following headings.

Original Papers

Original papers should be submitted in accordance with the *Notes for the Guidance of Authors* given inside the back cover of this journal. The Editor undertakes to acknowledge all manuscripts immediately they are received and to arrange for early review of each paper by *two* reviewers. The earliest possible publication date of contributions will be aimed for. Each author will receive 25 free copies of his paper.

Technical Notes

Technical notes will be accepted for publication. These contributions should be presentations of technical information which might be useful to the practicing or research engineer but which are not sufficient in themselves to warrant a full paper. The format to be followed for technical notes is the same as that for papers but only *two* copies need be submitted and no *Synopsis* is required. The author will receive 25 free copies of his technical note.

Reprints

Consideration will be given to reprinting papers which have been published previously but which are unlikely to have come to the attention of Society members. Only papers of a high standard which would be of particular interest to SEASSE members will be considered.

Discussions

Discussion is invited on any of the papers published in this journal. The closing date for discussion is indicated at the foot of the first page of each paper. Discussions sent to the Editor may be in any form, but figures and references should comply with the general requirements for publications in this journal. *Two* copies are required.

News Items

As the official organ of the Southeast Asian Society of Soil Engineering, this journal will publish any news item of interest to the Society members. Items to be included in the next issue (December, 1973) should be sent so as to reach the Editor not later than 1 November, 1973.