

*Journal of*

**SOUTHEAST ASIAN SOCIETY OF SOIL ENGINEERING**

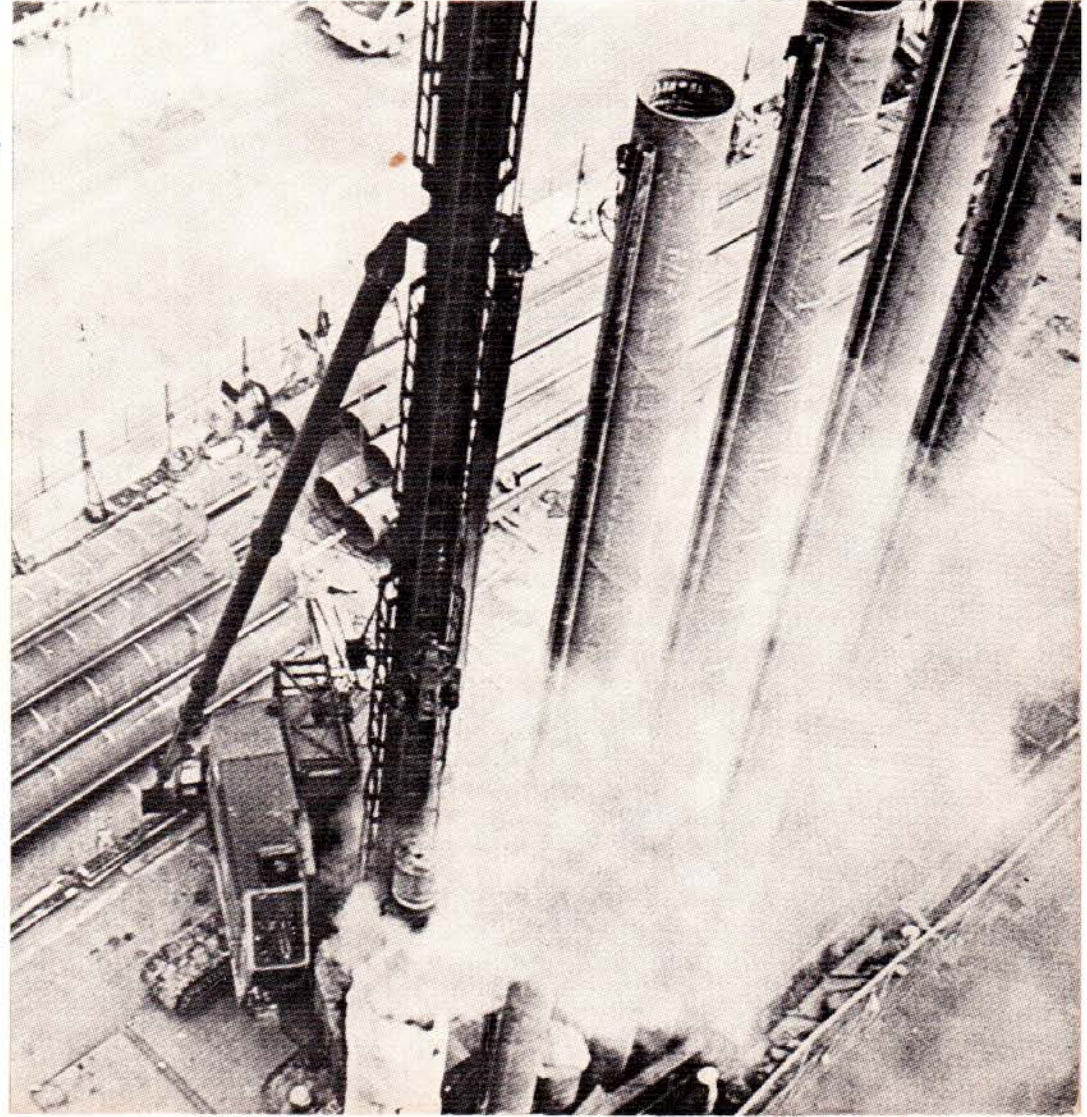
*Sponsored by*

**ASIAN INSTITUTE OF TECHNOLOGY**



Volume 10 Number 2

December 1979



# GEOTECHNICAL ENGINEERING

# GEOTECHNICAL ENGINEERING

VOLUME 10 NUMBER 2 DECEMBER 1979

| <b>Papers:</b>   | <b>Page</b> |
|--|-------------|
| <b>Preliminary Investigation of Saltwater Encroachment into the Nakhon<br/>Luang Acquifer, Bangkok</b><br>A. DAS GUPTA, A. ARBHABHIRAMA and B. AHMAD . . . . . | <b>141</b>  |
| <b>Analysis of Long-Term Compression of Peats</b><br>T.B. EDIL and ABDULMOHSIN W. DHOWIAN . . . . .  | <b>159</b>  |
| <b>State-of-the-Art Report on Settlements and Time Rates of Consolidation</b><br>T. AKAGI . . . . .  | <b>179</b>  |
| <b>Applications of Steel Pipe Piles in Japan</b><br>M. SAWAGUCHI . . . . .   | <b>199</b>  |
| <b>Review of Japanese Subsurface Investigation Techniques</b><br>H. MORI . . . . .   | <b>219</b>  |
| <b>The Influence of Fabric on Shrinkage Limit in Clay</b><br>S. NARASIMHA RAO . . . . .  | <b>243</b>  |
| <b>Technical Note:</b>   |             |
| <b>Dynamic Response of Footings in a Saturated Soil Medium</b><br>K.S. SANKARAN, N.R. KRISHNASWAMY, and P.G. BHASKARAN NAIR                                    | <b>253</b>  |
| <b>Book Reviews . . . . .</b>  | <b>259</b>  |
| <b>Conference News . . . . .</b>   | <b>268</b>  |
| <b>News of Southeast Asian Society of Soil Engineering . . . . .</b>   | <b>271</b>  |
| <b>SI Units and Symbols . . . . .</b>  | <b>280</b>  |
| <b>Afterword to Volume 10 . . . . .</b>  | <b>281</b>  |
| <b>Author Index: Volume 10, 1979 . . . . .</b>   | <b>282</b>  |
| <b>Subject Index: Volume 10, 1979 . . . . .</b>  | <b>283</b>  |
| <b>Contents of Volume 10 . . . . .</b>   | <b>i</b>    |



## **PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT INTO THE NAKHON LUANG AQUIFER, BANGKOK, THAILAND.**

A. DAS GUPTA\*, A. ARBHABHIRAMA\*\* AND B. AHMAD<sup>+</sup>

### **SYNOPSIS**

The Nakhon Luang Aquifer, one of eight aquifers underlying the Bangkok Metropolis, provides the major portion of groundwater supply for the area. At present (early 1979) the total groundwater pumpage amounts to 1.1 million cubic meters per day. Due to the heavy withdrawal of groundwater for public water supply as well as by private enterprises, a considerable extent of this aquifer has been contaminated by saltwater encroachment. The simplified analysis procedure using the solute transport model for predicting the movement of chloride in flowing groundwater, presented herein, provides an insight into the mechanism of saltwater encroachment. Field data available is not adequate to carry out a precise analysis. Apart from the movement of saltwater from the Gulf of Thailand, a connate water body lying in the western side of the Chao Phraya River acts as predominant source of contamination of freshwater in the aquifer. Furthermore, possibilities of vertical leakage of saline water from other aquifers and confining layers are inferred.

### **INTRODUCTION**

Bangkok Metropolis is one of the fast growing capitals of the world, with a population slightly over 4.8 million. Spreading over an area of about 750 square kilometers, the Metropolis consists of the cities of Bangkok and Thonburi. The City of Bangkok is situated about 40 kilometers from the sea along the left bank of the Chao Phraya River whilst Thonburi is located along the opposite bank. The Chao Phraya River traverses the flat deltaic plain of the same name (Fig. 1). The average ground elevation in Bangkok is about 1.5 meters above the mean sea level (MSL), while 150 kilometers to the north it is about 15 meters. The city is situated on very thick unconsolidated sediments which, according to PIANCHAROEN & CHUAMTHAISONG (1976), can be approximately divided into a number of confined aquifers, separated from each other by clay layers. These aquifers are named as follows:

- Bangkok Aquifer (50 m zone),
- Phra Pradaeng Aquifer (100 m zone),
- Nakhon Luang Aquifer (150 m zone),
- Nonthaburi Aquifer (200 m zone),

---

\*Assistant Research Professor, \*\*Professor, +Graduate Student, Water Resources Engineering, Asian Institute of Technology, Bangkok, Thailand.

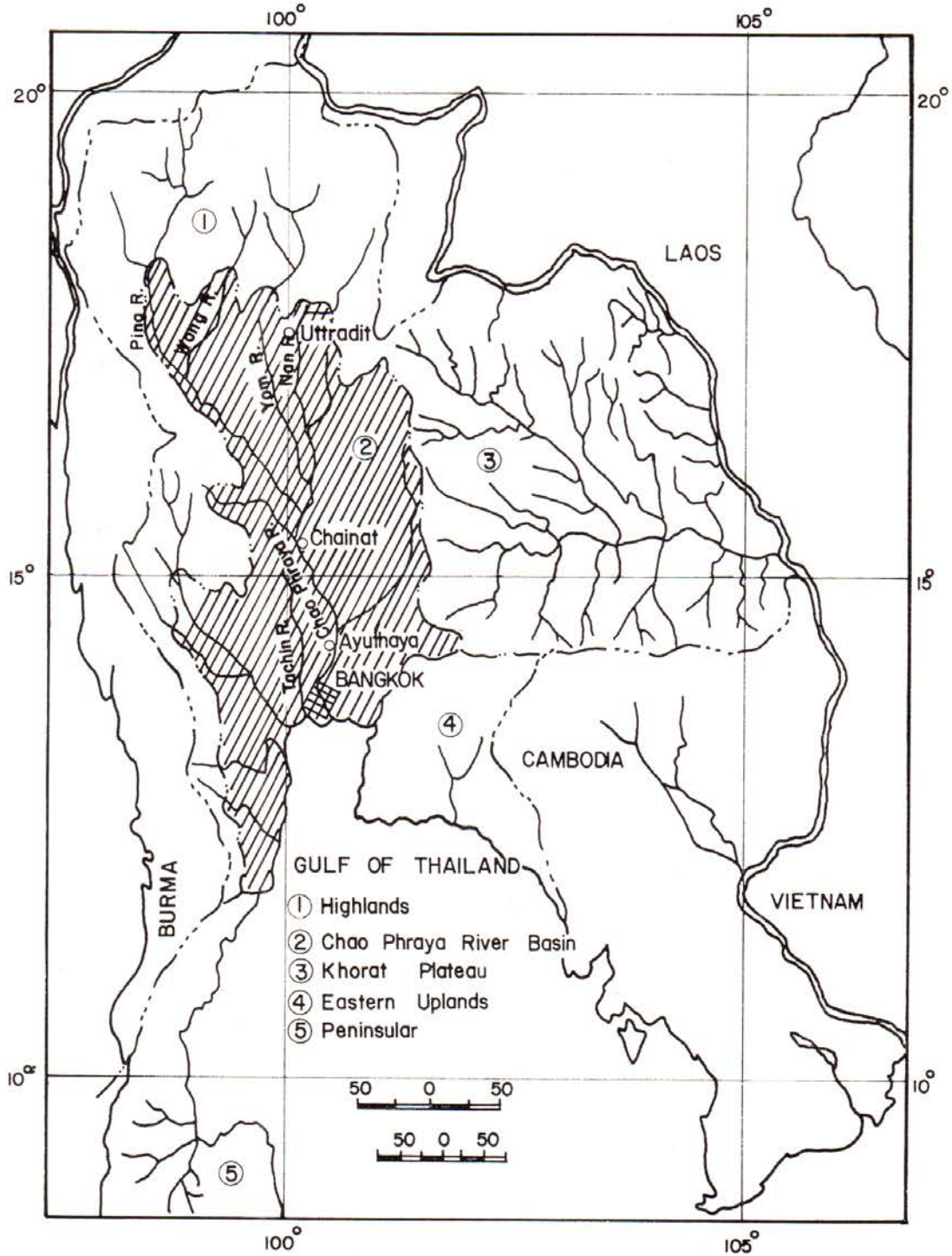


Fig. 1. Chao Phraya River Basin.

## *PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT*

Samkok Aquifer (250 m zone),  
Phya Thai Aquifer (360 m zone),  
Thonburi Aquifer (450 m zone), and  
Paknam Aquifer (550 m zone).

During the past few decades a rapid development has taken place in the industrial and agricultural sectors with a consequent increase in population in the Metropolis. The need of water for municipal and industrial purposes has, therefore, steadily grown. To cope with this need, groundwater resources have been extensively developed in both public and private sectors. Most of the production wells in Bangkok penetrate the Phra Pradaeng, Nakhon Luang and Nonthaburi Aquifers. Not many wells tap the deeper aquifers. Aquifers are extensive and widespread so that drilling for groundwater can be made without strong technological support. A sign of critical overpumping has been observed from the significant decline of piezometric head, approximately at the rate of 2 to 3 meters per year in these three aquifers. This has resulted in contamination of freshwater supply by saltwater encroachment and land subsidence. A large number of wells have been abandoned due to saltwater contamination and due to water level falling below the lowest possible pump setting.

Unfortunately, no planned scientific investigation has yet been carried out on the saltwater encroachment problem which is seriously threatening the freshwater supplies. This preliminary investigation may be taken as a first step in this respect and providing an understanding of the mechanism of saltwater encroachment in Nakhon Luang Aquifer. This aquifer is most extensively utilised and several wells withdrawing water from this aquifer have been abandoned due to saltwater contamination. It must be pointed out that no planned observation on hydraulic and water quality aspects of the aquifer was made before this study was undertaken. As such, the analysis is restricted to limited data available from the government agencies. Nevertheless, this simplified analysis will provide an insight into the mechanism of saltwater encroachment based on the hydrodynamic dispersion phenomena.

### **NAKHON LUANG AQUIFER AND ITS HYDROLOGICAL CHARACTERISTICS**

The third aquifer underneath Bangkok Metropolis, the Nakhon Luang Aquifer, is the major source of supply since groundwater resources have been exploited. Almost all the municipal wells penetrate this aquifer which is very productive and yields water of excellent quality except in the southern region, particularly in the Phra Pradaeng area, where saltwater is encountered because of overpumping.



*DAS GUPTA, ARBHABHIRAMA AND AHMAD*

The aquifer consists of permeable sand and gravel with clay lenses and leaky clay beds. In Bangkok area, the depth to the aquifer is about 110 to 140 meters, with an average thickness of 50 meters (PIANCHAROEN & CHUAMTHAISONG, 1976).

The work on the evaluation of hydraulic characteristics of the aquifer was initiated by CAMP, DRESSER & MCKEE and GERAGHTY & MILLER, INC. for the Metropolitan Water Works Authority (MWWA) in 1968. Later on, the Groundwater Division of the Department of Mineral Resources (DMR) conducted pumping tests at a few locations in the first three aquifers namely Bangkok, Phra Pradaeng and Nakhon Luang. Because of the impossibility of controlling pumping from nearby production wells during the tests, the results can be regarded as approximate only. The results on hydraulic characteristics for Nakhon Luang Aquifer deduced from these tests are summarised in Table 1.

**Table 1. Hydraulic properties of Nakhon Luang Aquifer.**

| Location                                   | Transmissibility<br>(m <sup>2</sup> /hr) | Permeability<br>(m/hr) | Storage<br>Coefficient                           |
|--|--|------------------------|--|
| Wat Phai Ngoen <sup>1</sup>                | 65                                       | 2.21                   | $1 \times 10^{-4}$                               |
| Lumpini Park <sup>1</sup>                  | 100                                      | 3.4                    | $2 \times 10^{-4}$                               |
| Pak Kret <sup>1</sup>                      | 100                                      | 2.55                   | —  |
| Dept. of Mineral<br>Resources <sup>1</sup> | 50                                       | 2.65                   | $2.6 \times 10^{-4}$                             |
| Bang Bun <sup>2</sup>                      | 130-155                                  | 3.45                   | $2.03 \times 10^{-3}$<br>to $3.4 \times 10^{-3}$ |
| Well 89-1 <sup>3</sup>                     | 67                                       | —                      | —  |

1-Piancharoen and Chumthaisong, 1976

2-Ramnarong, 1976

3-Geraghty and Miller, Inc., 1969

The aquifer is highly transmissive and productive and therefore, close spacing of wells is possible without much interference effect. The specific capacity for the aquifer ranges between 15 m<sup>3</sup>/hr/m to 40 m<sup>3</sup>/hr/m depending upon the construction and development of wells.

GROUNDWATER DEVELOPMENT IN BANGKOK

The exact year in which groundwater exploitation began is not known. It appears that the groundwater utilisation in the Metropolis must have started at

*PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT*

least 40 years ago when a few shallow wells were drilled for domestic supply. The first large scale utilisation of groundwater for public water supply began in the year 1954-1955 with the drilling of 10 deep wells by the Department of Public and Municipal Works. However, heavy utilisation started only after the year 1957. Governmental authorities drilled more than 150 production wells in the Metropolitan area between 1957 and 1960. The trend of adding new wells to the water supply system continued after the establishment of the MWWA in 1967. A total of about 272 wells have been drilled since 1964 out of which only 127 are in operation (METCALF & EDDY INC., 1977). The capacities of public supply wells vary from 30 to 400 cubic meters per hour, although initially higher rates were obtained. Private wells have also been installed in very large number without governmental control. After the promulgation of the Groundwater Act in 1978, the registered number of such wells was nearly 7,000, with pumping rates estimated to be varying from 2 to 400 cubic meters per hour. These supply such consumers as factories, office buildings, apartments, hotels, schools, hospitals and housing estates. Based on the records of the MWWA and the DMR, the present (early 1979) estimate of total groundwater production is 1.1 million cubic meters per day while it was 8,360 cubic meters per day in 1954.

Initially the piezometric level was very close to the ground surface since the first shallow well was said to be a flowing artesian well. With the advent of large scale pumping after 1954, the piezometric level started declining and several cones of depression developed in the areas of heavy pumpage. By 1958-1959 the deepest water level reached 12 meters below ground surface in the centre of Bangkok while that in the suburb was about 10 meters. The rate of decline was however, comparatively slow during this period. The decline rate has increased since 1966 when more and more wells were installed by private and government sectors. By 1968-1969, the deepest water level was 25 meters below ground surface in the centre of Bangkok and 12 meters in the suburbs. Since then, heavy utilisation of groundwater has started in suburb areas along the eastern bank of the Chao Phraya River while no significant increase took place in the central part of Bangkok. Accordingly, the decline of water level was only gradual in central Bangkok but much more rapid in the suburbs, especially in the southeastern, eastern and northeastern part of Bangkok. By early 1974, several deep cones of depression more than 30 meters below ground surface developed in the central and in the eastern part of Bangkok, namely in the Makkasan, Bangsue and Hua Mark areas. The groundwater exploitation in the eastern suburbs kept on increasing as the western and south western parts were being contaminated by saltwater. Presently (early 1979) it has been observed that in Nakhon Luang Aquifer,

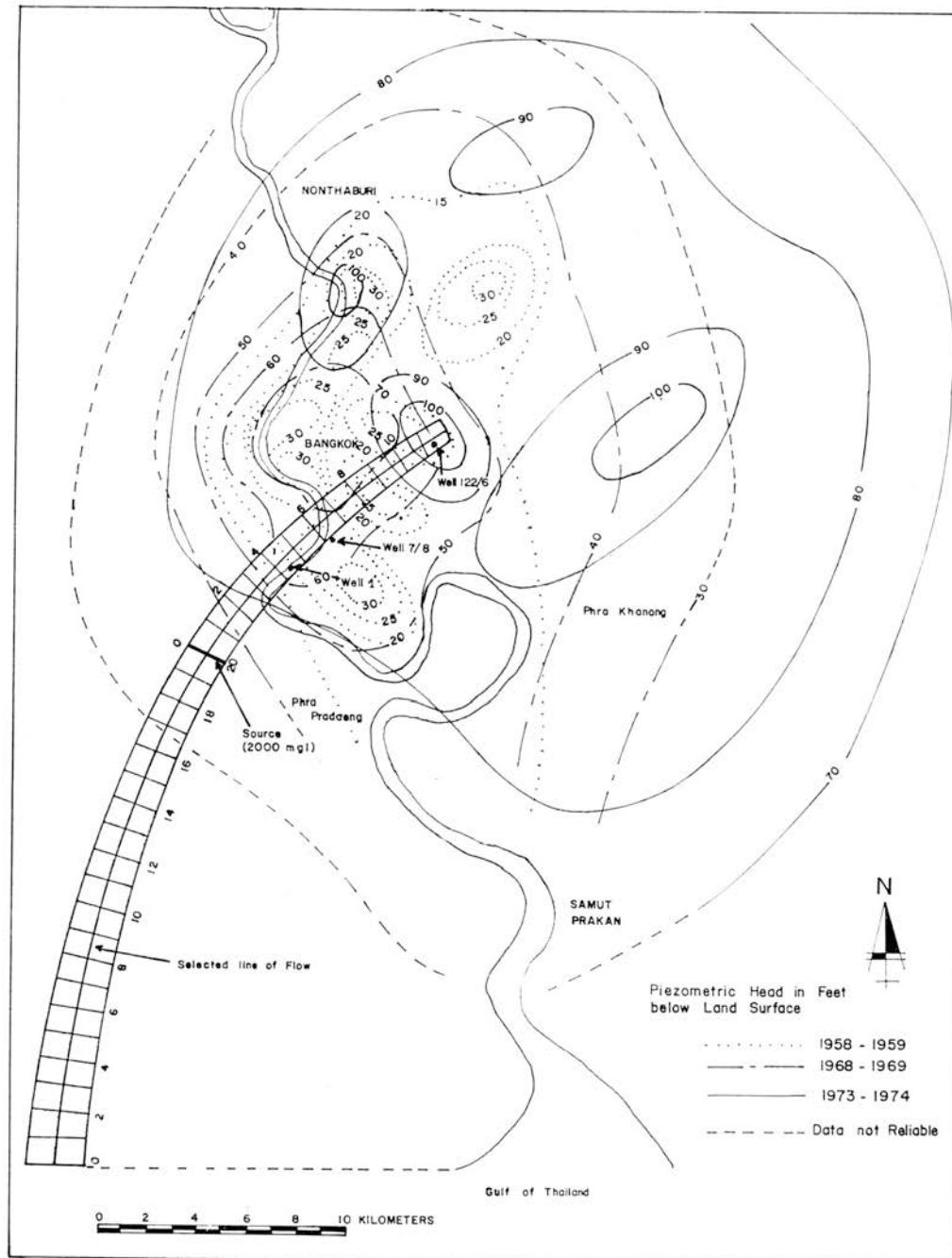


Fig. 2. Piezometric map for Nakhon Luang Aquifer.



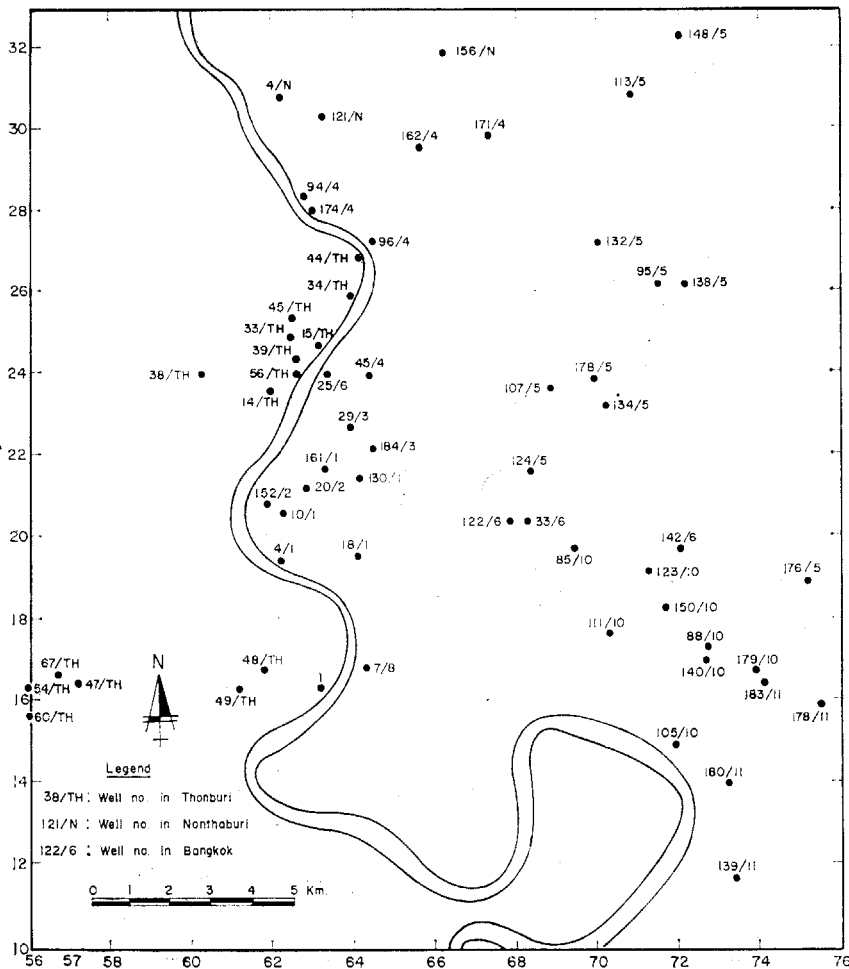
**PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT**

the piezometric level has gone down to 46 meters below ground surface and several deep cones of depression have developed in many parts of Bangkok, particularly in the areas where housing projects have grown up. A combined piezometric map of the Nakhon Luang Aquifer for the years 1958-1959, 1968-1969 and 1974 is shown in Figure 2.

**SALINITY DISTRIBUTION IN NAKHON LUANG AQUIFER**

The intensity of groundwater pumpage in the Bangkok area and the resulting horizontal movement of water in the aquifer has caused encroachment of saltwater into some of the main production areas.

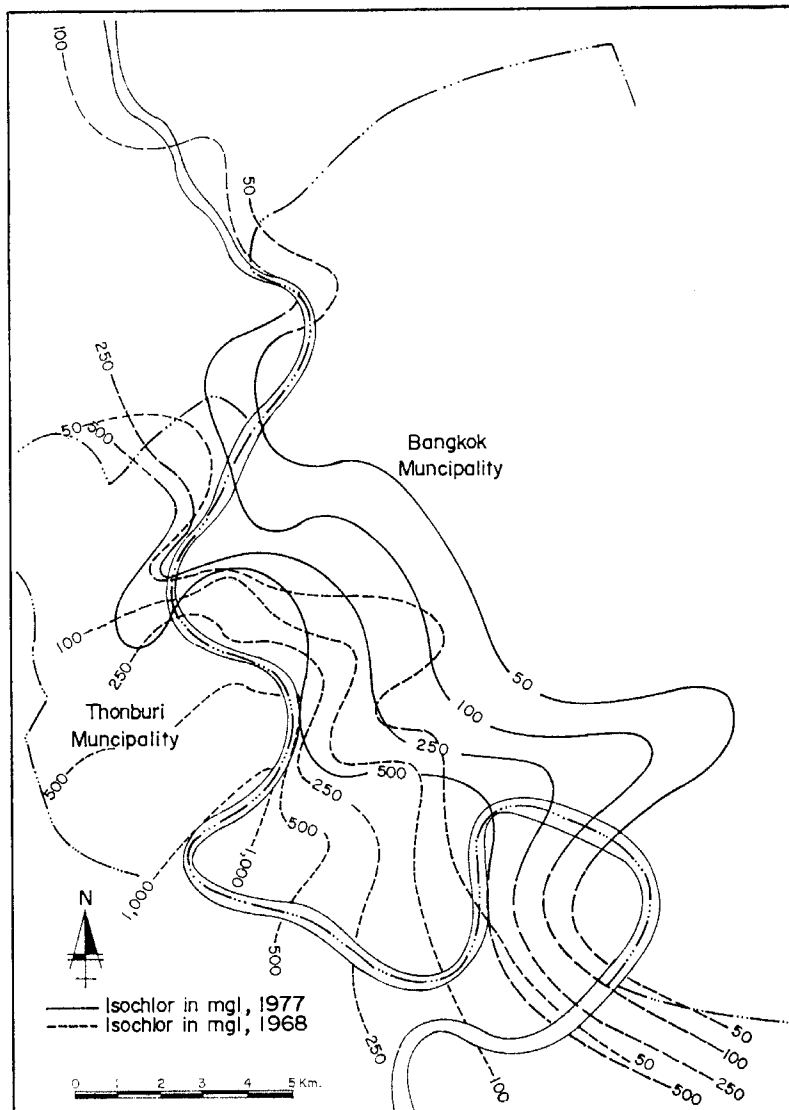
The saltwater intrusion had not been observed until the last 15 or more years when many municipal wells in Thonburi and southern Bangkok yielded



**Fig. 3. Location of water quality sampling wells in Nakhon Luang Aquifer.**

*DAS GUPTA, ARBHABHIRAMA AND AHMAD*

brackish or salty water. Since then more and more wells, especially on the western bank of the Chao Phraya River southward from central Thonburi and southern Bangkok, have been abandoned due to salty water. Groundwater in Bangkok has a chloride content varying from less than 100 mg/l to more than 2000 mg/l. In general, salinity of groundwater in Nakhon Luang Aquifer increases towards west and south but becomes fresh again in the extreme western region. Figure 3 gives the location of wells in Nakhon Luang Aquifer for which water quality data was available from the DMR and the MWWA.



**Fig. 4. Distributed chloride content in Nakhon Luang Aquifer.**

### *PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT*

Very few sampling points are in the region already contaminated by saltwater and there was no regular frequency of water sample collection. The distributed chloride content in the aquifer for specific years is shown in Figure 4.

### SIMPLIFIED ANALYSIS PROCEDURE

#### *Dispersion .*

Primary sources of chloride in groundwater are evaporites, salty connate water, marine water and sea water for coastal aquifers. Invasion of saline water into fresh groundwater due to groundwater withdrawal commonly occurs in coastal areas, where seawater moves inland if groundwater level or piezometric head declines. When poor quality water moves into a freshwater aquifer, the concentration of undesirable solute decreases with increasing distance of flow because of hydraulic dispersion. Longitudinal dispersion occurs in the direction of flow and is caused by different microscopic velocities as some parts of the invading fluid move through wider and less tortuous pores than other parts. Transverse dispersion occurs normal to the direction of flow and results from the repeated splitting and deflection of the flow by the solid particles in the aquifer. Transverse dispersion is effective only at the edges of a poor quality source. When the invading fluid enters over a broad front (for example, seawater intrusion) the effects of transverse dispersion within the contaminated zone cancel each other and only longitudinal dispersion needs to be considered (BOUWER, 1978).

#### *Mathematical Statement*

Complete analysis of the solute transport phenomena for such an extensive aquifer requires a substantial amount of field data on piezometric head fluctuation and water quality covering the period of utilisation of the aquifer. As mentioned earlier, piezometric maps for the Nakhon Luang Aquifer are available only for three years. Water quality measurement started only in the year 1968 for a limited number of wells concentrated in the region of heavy utilisation. No water quality data is available for the regions severely affected by saltwater encroachment. Henceforth, inadequacy of data has restricted the analysis to a simplified one, disregarding nominal details and considering some significant characteristics in order to provide an understanding on the mechanism of saltwater encroachment. A mathematical statement of the problem has been established considering the landward piezometric gradient due to the heavy pumpage in the central part of Bangkok. As the aquifer is of confined nature and is quite extensive, the



effect of the decline of piezometric head is regional, extending up to the coastline. Accordingly, the flow is assumed to be unidirectional from the sea towards the centre of the cone of depression as shown in Figure 5.

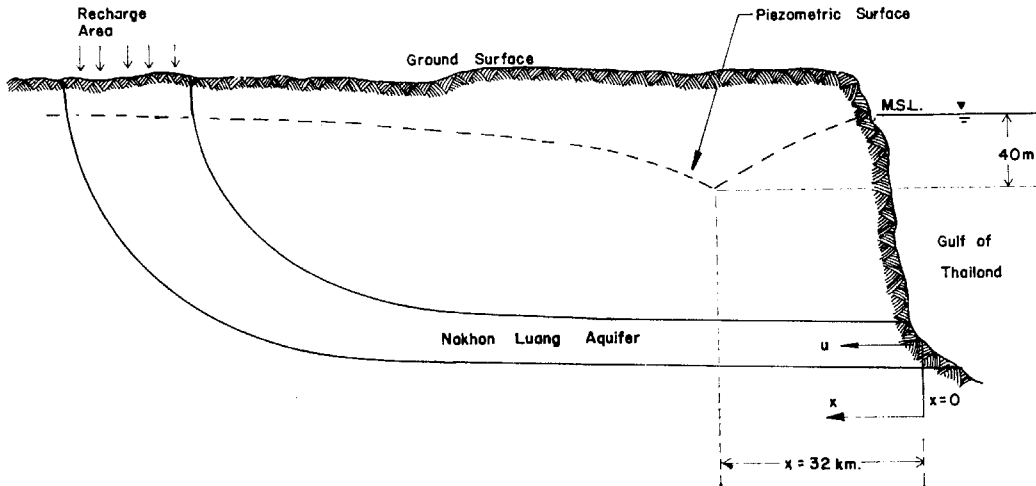


Fig. 5. Schematic representation of the Nakhon Luang Aquifer.

The general equation describing hydrodynamic dispersion in homogeneous, isotropic media (SCHEIDEGGER, 1961; BACHMAT & BEAR, 1964) can be reduced to the following equation for two-dimensional dispersion of a fluid invading a porous medium from a line source normal to steady, one-dimensional flow in the medium (BRUCH & STREET, 1967; HOOPES & HARLEMAN, 1967).

$$D_L \frac{\partial^2 C}{\partial x^2} + D_T \frac{\partial^2 C}{\partial y^2} - u \frac{\partial C}{\partial x} = \frac{\partial C}{\partial t} \dots\dots\dots(1)$$

where  $C$  = concentration of the invading fluid in the original fluid of the medium,

- $D_L$  = longitudinal dispersion coefficient,
- $D_T$  = transverse dispersion coefficient,
- $u$  = fluid velocity through the pore space in  $x$ -direction,
- $x$  = coordinate in the direction of flow,
- $y$  = coordinate normal to flow, and
- $t$  = time since the start of invasion.

If the invading fluid enters the porous medium over a broad front, there is no lateral dispersion and Equation 1 reduces to:

$$D_L \frac{\partial^2 C}{\partial x^2} - u \frac{\partial C}{\partial x} = \frac{\partial C}{\partial t} \dots\dots\dots(2)$$

*PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT*

The dispersion coefficients vary approximately directly with  $u$  as observed experimentally and also in the field by several investigators. The ratio  $D_L/u$  theoretically is a constant, called the dispersivity of the medium with the dimension of length (PINDER, 1973; OAKES & EDWORTH, 1976). In mathematical form:

$$D_L = D |u| \dots\dots\dots (3)$$

and 
$$u = - \frac{K}{\phi} \frac{\partial h}{\partial x} \dots\dots\dots (3)$$

where  $D$  = dispersivity of the medium,  
 $K$  = coefficient of permeability,  
 $\phi$  = porosity of the medium, and  
 $h$  = piezometric head.

Equation (2) was solved analytically by OGATA & BANKS (1961) using Laplace Transform to predict the longitudinal dispersion subject to the following initial and boundary conditions:

$$\begin{aligned} C(x, 0) &= 0 & x &\geq 0 \\ C(0, t) &= C_0 & t &\geq 0 \\ C(\infty, t) &= 0 & t &\geq 0 \end{aligned} \dots\dots (4)$$

and the final form of the solution is:

$$\frac{C}{C_0} = \frac{1}{2} \left[ \operatorname{erfc} \left( \frac{x - ut}{2\sqrt{D_L t}} \right) + \exp \left( \frac{ux}{D_L} \right) \operatorname{erfc} \left( \frac{x + ut}{2\sqrt{D_L t}} \right) \right] \dots\dots (5)$$

where  $C_0$  = concentration at the source,  
 $\operatorname{erfc}$  = complementary error function,  
 $\exp$  = exponential function, and  
 $u$  = average fluid velocity over the period  $t$ .

The above conditions are fairly representative of the flow situation to find the rate of intrusion of saltwater from the sea considering it as a line source along the shoreline. The initiation of intrusion is considered to have taken place in the year 1930 and prior to that, the assumption of zero salinity concentration is fairly representative of the prevailing situation at that time. Furthermore, a substantial period of time precedes the period under investigation so that the effects of this approximation dampen (RUSHTON & WEDDENBURN, 1973).

With specified initial condition along the direction of flow and with time varying boundary conditions, a numerical solution to Equation 2 has been used to simulate the actual field phenomena considering an average piezometric head gradient for the unidirectional velocity of flow. The implicit finite-difference equivalent of Equation 2 is:

$$\frac{C_i^{n+1} - C_i^n}{\Delta t} = D_L \left[ \frac{C_{i+1}^{n+1} - 2C_i^{n+1} + C_{i-1}^{n+1}}{(\Delta x)^2} \right] - u \left[ \frac{C_{i+1}^{n+1} - C_{i-1}^{n+1}}{2\Delta x} \right] \dots (6)$$

where  $i$  and  $n$  = indices for distance and time respectively, and

$\Delta x$  and  $\Delta t$  = increment in distance and time respectively (for this specific problem,  $\Delta x = 1$  km and  $\Delta t = 6$  month).

#### *Application*

In order to apply the solute transport model (Eqn. 2) for the Nakhon Luang Aquifer for approximate understanding of the mechanism of encroachment, a streamline has been selected which is fairly representative of the available piezometric maps and also along which water quality data at some wells are available to check the model prediction with the observed data. Figure 2 gives the selected line of flow of 32 kilometers length from the shoreline up to the centre of the cone of depression. Considering the available piezometric maps and with the interpolated distribution for the periods for which data are not available, the average yearly piezometric head gradient varies from 0.9 to 1.3 meters per kilometer. For the steady one-dimensional flow condition, an average piezometric head gradient of 1.1 meter per kilometer is considered for the invading fluid velocity in the direction of flow. Other pertinent parameters for the analysis are salinity data for three wells (Table 3), permeability  $K = 9.0 \times 10^{-4}$  m/sec, and porosity  $\phi = 0.30$ .

### RESULTS AND DISCUSSION

Considering the sea as the source of saltwater intrusion with a chloride concentration of 30,000 mg/l, and an average piezometric head gradient of 1.1 meters per kilometer along the selected direction of flow, the rate of encroachment is found to be 0.15 kilometer per year. Figure 6 provides a dimensionless distribution of the chloride concentration along the selected direction of flow for various time levels. With this rate of encroachment, approximately 200 years are needed for seawater to reach up to a distance of 30 kilometers from the source. However, the available field data at this location showed a chloride concentration of 250 mg/l in the year 1977 (Fig. 4). Moreover, with the sea as the source, the dispersion zone with chloride concentration ranging



PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT

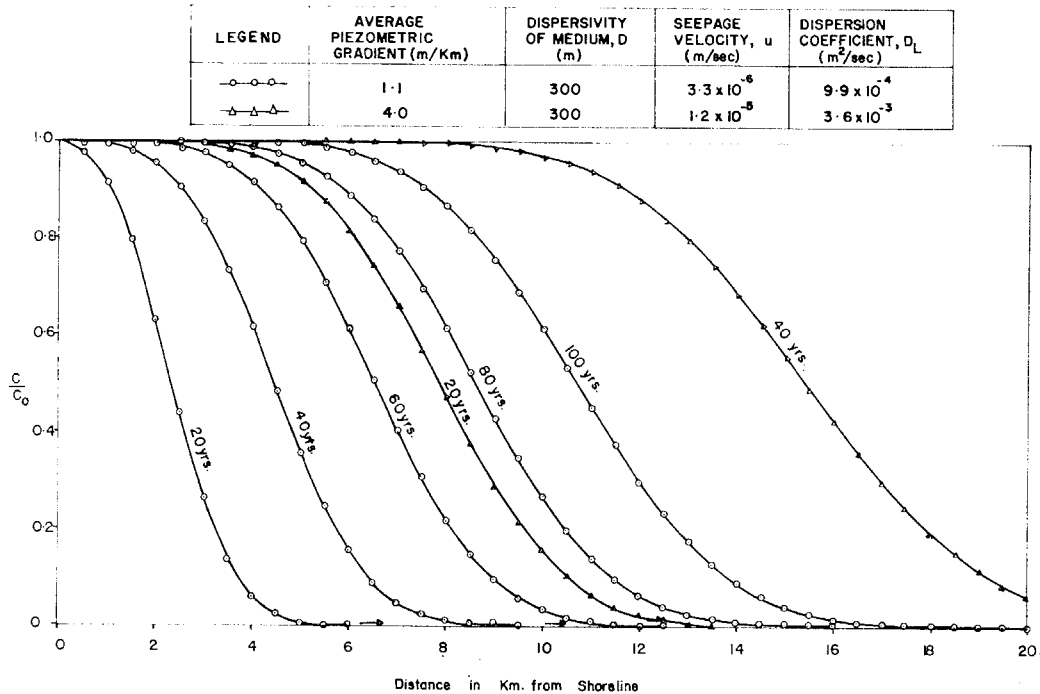


Fig. 6. Salinity distribution considering the sea as the source.

from zero to as high as 30,000 mg/l is relatively small compared to the extent of the saltwater contamination in the actual situation. In addition to this, the maximum chloride content observed in the field is too low to interpret seawater as the main source of intrusion. As an extreme case, considering an average piezometric head gradient of 4 meters per kilometer, the rate of encroachment is found to be 0.5 kilometer per year (Fig. 6). Even with such a high rate of encroachment a period of 60 years is required for the seawater to reach the already affected parts of Bangkok. The history of groundwater development however, reveals that the meaningful extraction of groundwater started in the year 1954. The period of time since then is insufficient for the convection of the contaminant from the sea up to the affected region. Moreover, several wells were abandoned due to salt contamination in the year 1969, only 15 years after groundwater extraction program started. Consequently, it is clearly indicated that the convection of saltwater from the sea is not the predominant source of salt contamination in the aquifer.

The formation of sedimentary layers beneath Bangkok is believed to have taken place in marine environment (CAMP, DRESSER & McKEE, 1969). Therefore, the connate water bodies left in certain relatively impervious layers might be the source of salt contamination. In addition, the increase in manganese

content associated with the increase in chloride content suggests that the source is probably the connate water as manganese is derived from the solution of minerals in the delta sediments under reduced environment produced by the decomposition of ancient organic minerals. Also, the calcium to magnesium ratio does not show any change in wells already abandoned due to saltwater contamination. Based on this evidence, a line source of connate water of chloride concentration 2000 mg/l is assumed to exist on the western side of the Chao Phraya River at a distance of 20 kilometers from the sea along the selected streamline. An attempt has been made to simulate the distribution of chloride along the selected path of flow starting from the year 1969. The model results compared satisfactorily with the field data, which signifies that the connate water body is the predominant source of saltwater encroachment in the Nakhon Luang Aquifer. However, the available field data is inadequate to delineate the region of connate water bodies. Figure 7 shows the comparison of computed chloride concentration at these well locations with the observed field data. The different values of the pertinent parameters used in the analysis

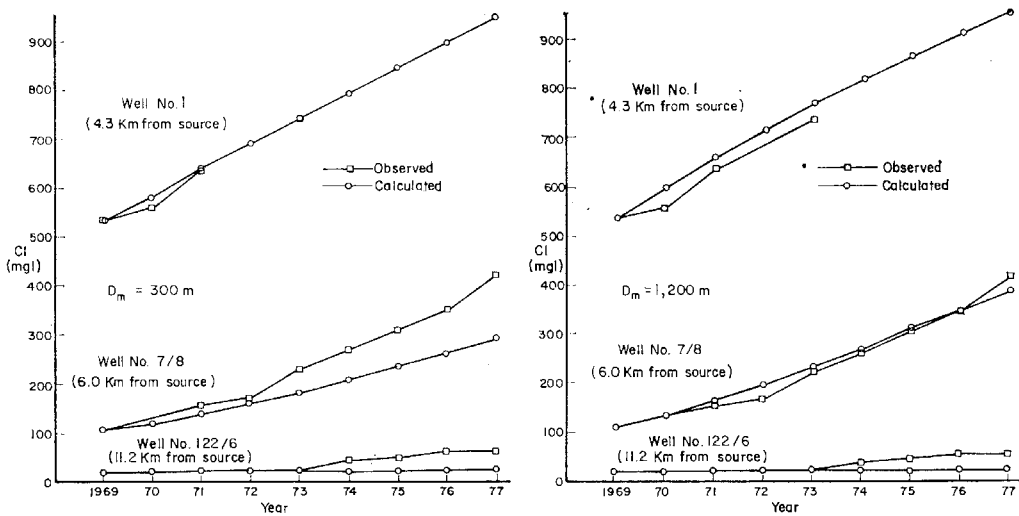


Fig. 7. Salinity distribution considering connate water bodies as the source.

Table 2. Pertinent parameters for analysis.

| Average Piezometric Gradient (m/km) | Dispersivity of the Medium, $D$ (m) | Seepage Velocity $u$ (m/sec) | Dispersion Coefficient $D_L$ (m <sup>2</sup> /sec) |
|-------------------------------------|-------------------------------------|------------------------------|--|
| 1.1                                 | 300                                 | $3.3 \times 10^{-6}$         | $9.9 \times 10^{-4}$                               |
| 1.1                                 | 1200                                | $3.3 \times 10^{-6}$         | $3.96 \times 10^{-3}$                              |

*PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT*

**Table 3. Observed value of chloride concentration at selected locations.**

Well No. 1 Location Co-ordinate 629159

| Date of Sampling | CL (mg/l) | Date of Sampling | Cl (mg/l) |
|------------------|-----------|------------------|-----------|
| 10 Sept. 1969    | 540       | 5 Oct. 1970      | 567       |
| 7 Oct. 1969      | 503       | 8 Dec. 1971      | 638       |
| 20 Aug. 1970     | 549       | 1973             | 736       |

Well No. 7/8 Location Co-ordinate 643169

| Date of Sampling | Cl (mg/l) | Date of Sampling | Cl (mg/l) |
|------------------|-----------|------------------|-----------|
| Aug. 1960        | 20        | 5 Feb. 1973      | 220       |
| Mar. 1968        | 103       | 17 Dec. 1973     | 229       |
| Aug-Sept. 1968   | 106       | 1 Apr. 1974      | 242       |
| Aug-Sept. 1969   | 106       | 22 July 1974     | 267       |
| 14 July 1971     | 157       | 23 Dec. 1974     | 279       |
| 10 Nov. 1971     | 156.5     | 30 June 1975     | 312.5     |
| 4 Jan. 1972      | 169       | 1 Sept. 1975     | 290       |
| 26 Apr. 1972     | 145       | 1 Dec. 1975      | 310       |
| 21 June 1972     | 178       | 8 Mar. 1976      | 332       |
| 16 Aug. 1972     | 182       | 23 Aug. 1976     | 330       |
| 11 Oct. 1972     | 160       | 15 Nov. 1976     | 356       |
| 7 Dec. 1972      | 115       | 21 Feb. 1977     | 414       |

Well No. 122/6 Location Co-ordinate 678206

| Date of Sampling | Cl(mg/l) | Date of Sampling | Cl(mg/l) |
|------------------|----------|------------------|----------|
| 1 Sept. 1971     | 20.5     | 17 Mar. 1975     | 44       |
| 5 Apr. 1972      | 16       | 9 June 1975      | 44       |
| 9 Aug. 1972      | 18       | 25 Aug. 1975     | 51       |
| 20 Sept. 1972    | 25       | 10 Nov. 1975     | 48       |
| 15 Nov. 1972     | 21       | 9 Feb. 1976      | 50       |
| 16 Jan. 1973     | 23       | 4 May 1976       | 61       |
| 25 Apr. 1974     | 54       | 26 July 1976     | 56       |
| 17 July 1974     | 39       | 31 Jan. 1977     | 62       |
| 23 Sept. 1974    | 25       | 13 June 1977     | 52       |
| 16 Dec. 1974     | 40       |                  |          |



and the observed values of chloride concentration at selected locations are given in Tables 2 and 3 respectively.

Another important observation of this analysis is that the leakage of brackish water through the confining bed appears to be quite significant in certain sections of the aquifer. With a dispersivity of the medium of 300 meters, the computed values of concentration agree closely with the observed values at locations nearest to the source, but for locations further away from the source the calculated concentrations are less compared to the field values during the latter period of simulation and the discrepancy increases with time. Even with the increase of dispersivity of the medium up to 1200 meters, the calculated concentrations at the farthest location do not change appreciably and are still less than the observed data, whereas, the calculated values at the nearest location are higher than the observed values. The amount of available data is not enough to ascertain the dispersivity of the medium; even then, a value of 1200 meters is considered to be quite high. Restricting the analysis within the limited data for only the three wells which met the requirements, it is inferred that apart from the migration of chloride from the connate water bodies, vertical leakage of brackish water through the confining media appears to be another source resulting in higher observed chloride concentration in certain regions of the aquifer.

#### CONCLUSIONS

This preliminary investigation with a simplified analysis procedure based on the limited available data leads to the following conclusions:

(1) The saltwater encroachment of the Nakhon Luang Aquifer does not occur only from the sea. Connate water trapped in less permeable sediments subsequent to their time of deposition under marine conditions is the predominant source contaminating the freshwater supply. This connate water body lies on the western side of the Chao Phraya River extending north of the Gulf of Thailand to Amphoe Bang Bua Thong and beyond. However, data available at this stage is not adequate to delineate the distribution of the saline water bodies.

(2) Saltwater contamination is not only due to the lateral movement from the source as mentioned but also appears to be due to the vertical leakage of brackish water from other aquifers and confining layers.

(3) Encroachment is related to the piezometric gradient; a higher piezometric gradient will cause a higher rate of encroachment. The predicted rate of

*PRELIMINARY INVESTIGATION OF SALTWATER ENCROACHMENT*

encroachment of the 250 mg/l isochlor from its observed position in 1977 is as follows:

| Average Piezometric gradient<br>(m/km) | Rate of Encroachment<br>km/year |
|--|---------------------------------|
| 1.1                                    | 0.15                            |
| 2                                      | 0.25                            |
| 3                                      | 0.31                            |
| 4                                      | 0.50                            |

ACKNOWLEDGEMENT

Several government officials and private agencies have furnished information and extended cooperation during the course of this investigation. In particular the authors would like to thank Mr. Charoen Piancharoen, Dr. Vachi Ramnarong and Miss Somkid Buapeng of the Department of Mineral Resources; Mr. Weibul Taweewup of the Metropolitan Water Works Authority; and Mr. Donald Lubke of Metcalf and Eddy Inc.

REFERENCES

- BACHMAT, Y. and BEAR, J. (1964), The General Equations of Hydrodynamic Dispersion in Homogeneous, Isotropic Porous Medium, *Journal of Geophysical Research*, Vol. 69, pp. 2561-2567.
- BOWER, H. (1978), *Groundwater Hydrology*, McGraw-Hill Book Company, New York.
- BRUCH, J. C. and STREET, R. L. (1967), Two-Dimensional Dispersion, *Journal of Sanitary Engineering Division*, American Society of Civil Engineers, Vol. 93, No. SA-6, pp. 17-39.
- CAMP, DRESSER and McKEE (1970), Master Plan, Water Supply and Distribution, Metropolitan Bangkok, Thailand, Vol. I, II, IV, *Report* prepared for Metropolitan Water Works Authority.
- GERAGHTY and MILLER, INC. (1969), Groundwater Resources of Bangkok Metropolitan Area, *Report* prepared for Camp, Dresser and McKee.
- HOOPEs, J. A. and HARLEMAN, D. R. F. (1967), Wastewater Recharge and Dispersion in Porous Media, *Journal of the Hydraulics Division*, American Society of Civil Engineers, Vol. 93, No. HY-5, pp. 51-71.
- METCALF and EDDY, INC. (1977), Groundwater Monitoring, Well Construction and Future Programs, *Report* prepared for Metropolitan Water Works Authority.
- OAKES, D.B. and EDWORTHY, K.J. (1976), Field Measurements of Dispersion Coefficients in the United Kingdom, *Proceedings of the International Conference on Groundwater Quality, Measurement, Prediction and Protection* Water Research Centre, England, Paper No. 12.
- OGATA, A. and BANKS, R.B. (1961), A Solution of the Differential Equation of Longitudinal Dispersion in Porous Media, *Professional Paper 411-A*, United States Geological Survey, Washington D.C.

*DAS GUPTA, ARBHABHIRAMA AND AHMAD*

- PIANCHAROEN, C. and CHUAMTHAISONG, C. (1976), Groundwater of Bangkok Metropolis, Thailand, *Proceedings of the International Conference on Hydrogeology of Great Sedimentary Basins*, Budapest.
- PINDER, G.F. (1973), A Galerkin Finite-Element Simulation of Groundwater Contamination on Long Island, *Water Resources Research*, Vol. 9, pp. 1657-1669.
- RAMNARONG, V. (1976), Pumping Tests for Nakhon Luang and Bangkok Aquifers, *Groundwater Open File Report No. 90*, Department of Mineral Resources, Bangkok, Thailand.
- RUSHTON, K.R. and WEDDENBURN, L.A. (1973), Starting Conditions for Aquifer Simulation, *Groundwater*, Vol. 11, No. 1, pp. 37-42.
- SCHEIDEGGER, A.E. (1961), General Theory of Dispersion in Porous Media, *Journal of Geophysical Research*, Vol. 66, pp. 3273-3278.

## ANALYSIS OF LONG-TERM COMPRESSION OF PEATS

TUNCER B. EDIL\* and ABDULMOHSIN W. DHOWIAN†

### SYNOPSIS

Long-term one-dimensional consolidation tests were performed on peat samples from four different sources and covering a wide compositional variation, including amorphous granular and fibrous peat types. The analysis of the behavior of peats under such conventional test conditions is considered herein; the prediction of field performance is beyond the scope of this paper. Results from the consolidation tests indicate errors may result from extending the linear portion of the secondary compression curve. In general, there is an increase in the compression rate with the logarithm of time giving rise to a "tertiary" compression. A number of empirical and theoretical methods for predicting settlements due to secondary compression are briefly reviewed and the model of Lo (1961) is used to describe the observed response and to determine the parameters needed to analyze the long-term compression of these peats. These parameters are related to the level of consolidation pressure and no apparent consistent trends are found with respect to the type of peat and initial void ratio.

### INTRODUCTION

Peat is generally considered among the worst of foundation materials because of its high water content and compressibility and low bearing capacity. Encountered extensively in many parts of the world, peat is a mixture of fragmented organic material derived from vegetation which has been chemically changed and fossilized. It is formed in wetlands under appropriate climatic and topographic conditions. Today, some of the marginal sites are old wetlands that have been drained. Because of development in some parts of the world where peat deposits are extensive, preloading techniques through surcharging have been employed with some success as a means of *in situ* improvement of engineering properties (LEA & BRAWNER, 1963; WEBER, 1969; SAMSON & LA ROCHELLE, 1972), and this renders them capable of supporting structures without failure or excessive settlement. It is of prime importance to understand and quantify the stress-strain-time response of peat soils in order to employ such techniques and to predict the long-term

\* Associate Professor of Civil & Environmental Engineering and Engineering Mechanics, University of Wisconsin-Madison, Madison, Wisconsin, U.S.A.

† Assistant Professor of Civil Engineering, University of Riyadh, Riyadh, Saudi Arabia, formerly Graduate Student, Dept. of Civil & Environmental Engineering, University of Wisconsin-Madison, Madison, Wisconsin, U.S.A.

settlement of structures supported by these soils; however, there exist relatively few reports of tests on peats. In an attempt in this direction, the results of long-term consolidation tests on "undisturbed" samples of peat are reported herein.

#### EXPERIMENTAL PROGRAM

Fiber content appears to be a major compositional factor in determining the way in which peaty soils behave. The higher the fiber content, the more the peat will differ from an inorganic soil in its behavior. Peats with fiber contents ranging from about 20% to 60% were sampled from four sites in Wisconsin, U.S.A.; Fond du Lac County, City of Portage, Waupaca County, and City of Middleton (DHOWIAN, 1978). The peat samples are designated after these locations. Fond du Lac Peat is an amorphous granular peat in which the soil particles are mainly of colloidal size and the majority of porewater is adsorbed around the grain structure. The remaining three peats are fibrous peats which have essentially an open structure with interstices filled with a secondary structural arrangement of nonwoody, fine fibrous material with most of the water occurring as free water rather than as viscous adsorbed water (MACFARLANE & RADFORTH, 1965).

At all sites the peat deposit was within a meter below the surface and sampling was achieved by using a 76-mm Shelby tube (thin-wall tube sampler with a sharp edge). A portion of the sample in each tube was used to determine the characteristics of the peat. The average properties of the peats are given in Table 1. These properties were determined largely in accordance with the procedures suggested in the Muskeg Engineering Handbook (MACFARLANE, 1969). Fiber content was determined in accordance with the procedure suggested by LYNN, et al (1974).

Table 1. Average Properties of Peat Samples.

| Source of Samples              | Water Content (%) | Unit Weight kN/m <sup>3</sup> | Ash Content (%) | Specific Gravity | Fiber Content (%) | pH   | Vane Shear Strength kPa | Sensitivity | Classification     |
|--------------------------------|-------------------|-------------------------------|-----------------|------------------|-------------------|------|-------------------------|-------------|--------------------|
| Fond du Lac County, Wisconsin] | 240               | 10.2                          | 39.8            | 1.94             | 20                | 6.24 | 27.0                    | 9.0         | Amorphous Granular |
| Portage, Wisconsin             | 600               | 9.6                           | 19.5            | 1.72             | 31                | 7.30 | 14.4                    | 3.0         | Fibrous            |
| Waupaca County, Wisconsin      | 460               | 9.6                           | 15.0            | 1.68             | 50                | 6.20 | 15.0                    | 3.8         | Fibrous            |
| Middleton, Wisconsin           | 510               | 9.1                           | 12.0            | 1.41             | 64                | 7.00 | 22.0                    | 4.4         | Fibrous            |



### COMPRESSION OF PEATS

In order to develop a visual appreciation of the peats their microstructure was examined under a scanning electron microscope. An examination of the scanning electron micrographs of the peat samples, given in Figure 1, indicates that Middleton Peat is made of relatively uniformly shaped fibers with minor amounts of interstitial soil grains or particles. Fond du Lac Peat, on the other hand, has a higher amount of soil grains and particles than the other three types of peat. The differences in fiber content determined in the laboratory can be observed in the micrographs. It was noted that Portage Peat contained certain woody pieces (not shown in Figure 1) larger than the fibers.

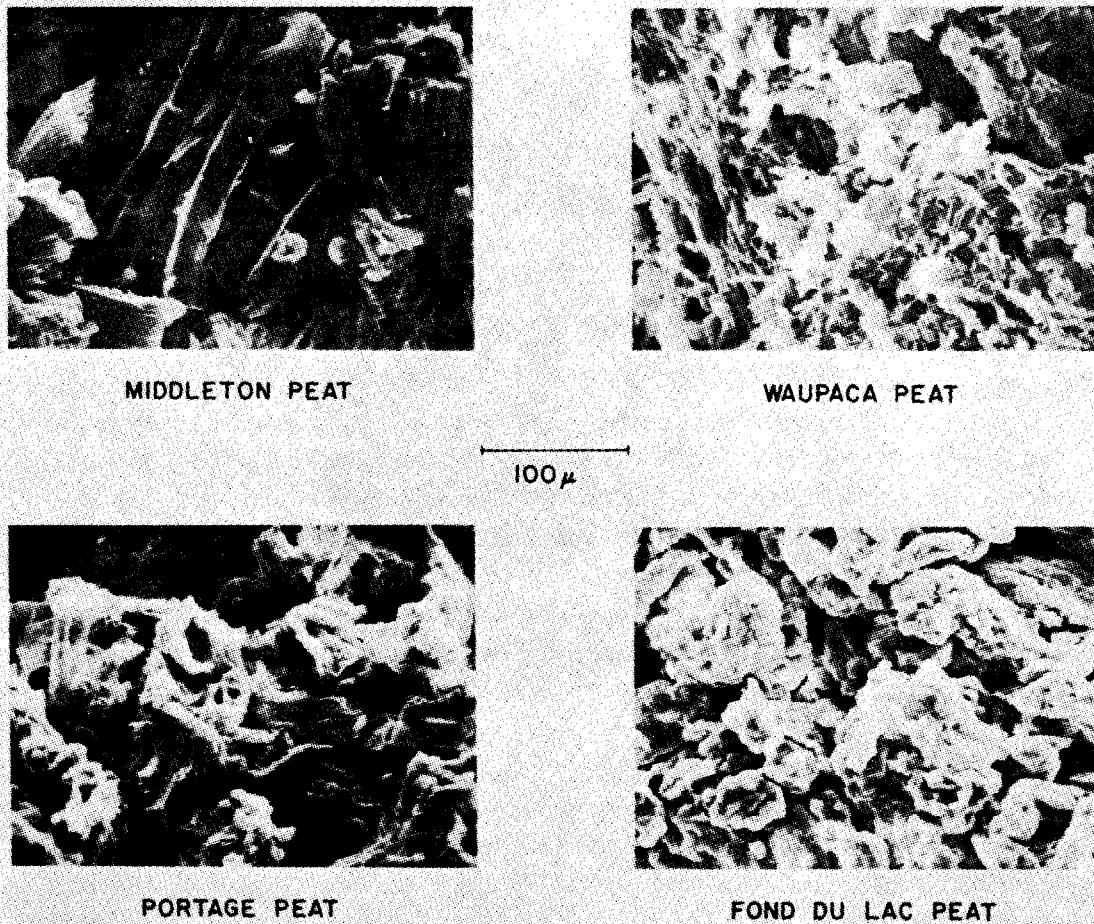


Fig. 1. Scanning electron micrographs of four peat samples.

Fourteen one-dimensional consolidation tests were performed using Anteus consolidometers (modified to allow monitoring of pore water pressure and effluent outflow) as well as conventional fixed-ring consolidometers to determine the strain-time behavior of peats. Portage Peat, an intermediate peat in its characteristics between highly fibrous and amorphous granular peats, was subjected to the largest number of tests in order to establish the general consolidation behavior. An extension of the observed behavior to other types of peat was sought by a series of simple conventional fixed-ring consolidation tests on the other three peat samples. The peats used were normally consolidated.

The test specimens had a diameter of 63 mm and an initial height of 19 mm. The consolidation stresses were applied incrementally starting with 25 kPa and then increased by doubling the existing pressure increment until a pressure of 400 kPa was reached (a pressure increment ratio of 1 was used throughout this testing program). Specimens were allowed to compress under each increment of pressure for a period of 20 days before a new increment of pressure was applied. The experimental work was performed in a constant temperature room. Maintaining constant temperature was necessary in order to avoid erroneous results arising from temperature fluctuations during such long-term tests, especially in view of the sensitivity of secondary compression to temperature changes. The room temperature was maintained at about 20° C,  $\pm 0.5^\circ\text{C}$ .

The relationship between vertical strain and the logarithm of time for the first increment of loading for a Portage Peat specimen is given in Figure 2 along with pore pressure and normalized effluent outflow response. As indicated on Figure 2, the strain-time curve is not similar to those obtained for clay soils and even for some organic soils. The usual primary consolidation curve is not observed. The strain-logarithm of time ( $\epsilon$ -log  $t$  curve) consists of four components of strain; instantaneous strain ( $\epsilon_i$ ), primary strain ( $\epsilon_p$ ), secondary strain ( $\epsilon_s$ ), and tertiary strain ( $\epsilon_t$ ).  $t_a$  and  $t_k$  indicate the beginning of the secondary and the tertiary compression, respectively. The  $\epsilon$ -log  $t$  response shown in Figure 2 was observed, with slight variations, for all peat samples tested in this study and can be considered representative of the general behavior. The excess pore water pressure, as measured at the bottom of the specimen while it was draining from the top, is shown along with the normalized effluent outflow curve (obtained by dividing the effluent outflow by the initial volume of the specimen prior to the application of each stress increment) in Figure 2 for the first increment of stress application, and in

COMPRESSION OF PEATS

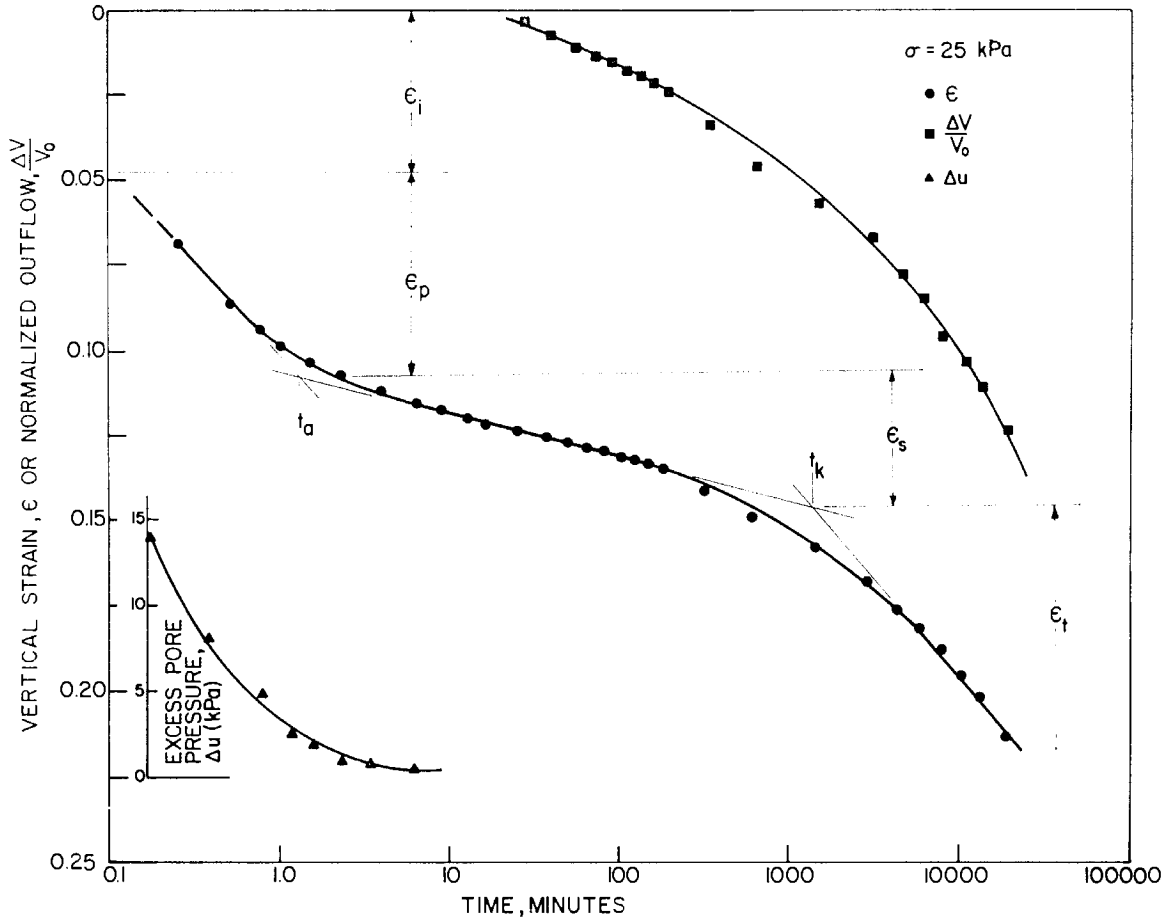


Fig. 2. Vertical strain, pore pressure, and normalized outflow versus time response for a Portage Peat specimen under the first stress increment.

Figure 3 for the last increment of stress application on the same specimen. The vertical strain-logarithm of time curves for most of the pressure increments lack any clear demarcation between the primary consolidation and the secondary compression stages corresponding to the complete dissipation of excess pore water pressure. It is also interesting to observe that water continues flowing out of the specimen even after the dissipation of the measurable pore water pressure implying that the outflow is not totally controlled by the macro-hydrodynamic effects. The four components of strain were found to be quite distinguishable in the  $\epsilon$ -log  $t$  curve for the first stress increment. However, for the advanced stress increments, the strain components were less clearly marked (compare Figures 2 and 3) indicating the influence of stress history.

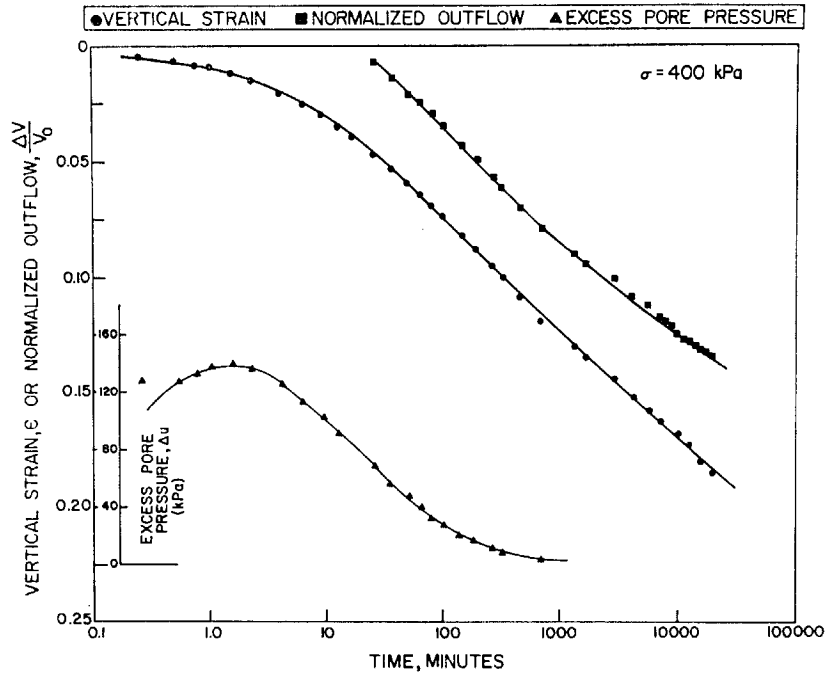


Fig. 3. Consolidation response curve for a Portage Peat specimen under the last stress increment.

THEORETICAL BACKGROUND

The one-dimensional consolidation process is now accepted as continuous, but is traditionally divided into primary and secondary stages. The primary stage is governed by the dissipation of pore pressure and the secondary stage is compression under nearly constant effective stress. One-dimensional compression due to the primary consolidation has been successfully estimated for certain types of soils by use of the Terzaghi theory of consolidation (TERZAGHI, 1925). However, a rational procedure whereby the settlement due to the secondary compression, in addition to the primary consolidation, is far more complicated and involved. Both empirical and theoretical methods have been developed for predicting settlements due to the secondary compression. Several of these methods are briefly discussed herein.

The earliest and perhaps the most commonly used method was suggested by BUISMAN (1936), who assumed that secondary compression rate is constant with the logarithm of time. The Buisman expression to determine secondary compression is given in the form:

$$\rho_s = C_\alpha H_o \log \frac{t_s}{t_p} \dots\dots\dots (1)$$

*COMPRESSION OF PEATS*

where  $\rho_s$  is the secondary compression,  $H_0$  is the initial thickness of the compressible layer,  $C_\alpha$  is the secondary compression coefficient,  $t_p$  is the time required for completion of primary consolidation, and  $t_s$  is the total time after loading into the time range of secondary compression. The coefficient of secondary compression  $C_\alpha$ , is the slope of the linear portion of a vertical strain-logarithm of time plot. It is implicit that  $C_\alpha$  is determined over the appropriate range of stress increase and the compression follows a linear relationship in a plot of strain versus  $\log t$ . A drawback of Buisman's expression is its assumption that compression continues indefinitely, and accordingly a specimen could vanish in a laterally confined condition. Nevertheless, the Buisman theory is widely used in geotechnical engineering applications when an estimate of secondary compression is required. This may be due to the fact that the behavior assumed by this theory, that is, the constant rate of secondary compression with the logarithm of time, is generally encountered for most clays within the range of times considered. This behavior was also reported for peats by a number of investigators for the range of times they considered (BUISMAN, 1936; HANRAHAN, 1954, 1964; ADAMS, 1961, 1963, 1965; SCHROEDER & WILSON, 1962; LEA & BRAWNER, 1963; BARDEN, 1969; BERRY & VICKERS, 1975).

GIBSON & LO (1961) developed a theory in which the structural viscosity was assumed to be linear. The structure of soils exhibiting secondary compression was assumed to behave according to the rheological model shown in Figure 4a and an exact solution was obtained for secondary compression of the soil. For large values of time, the time-dependent strain  $\varepsilon(t)$ , may be written as:

$$\varepsilon(t) = \Delta\sigma [a + b(1 - e^{-\lambda/bt})], t > t_a \dots\dots\dots(2)$$

where  $a$ ,  $b$ , and  $\lambda$  are empirical parameters which can be determined from test data, and  $t_a$  is the time after which the applied stress has become fully effective. Equation 2 has been found to work well for certain shapes of secondary compression curves. However, there are different types of secondary compression curves depending upon the properties of the tested soil and on testing conditions. From the published data, together with the results of consolidation tests performed on several types of clay, remolded and undisturbed, LEONARDS & GIRAULT (1961) and LO (1961) identified three types of secondary compression curves which are common in soils exhibiting secondary compression. The three types are:



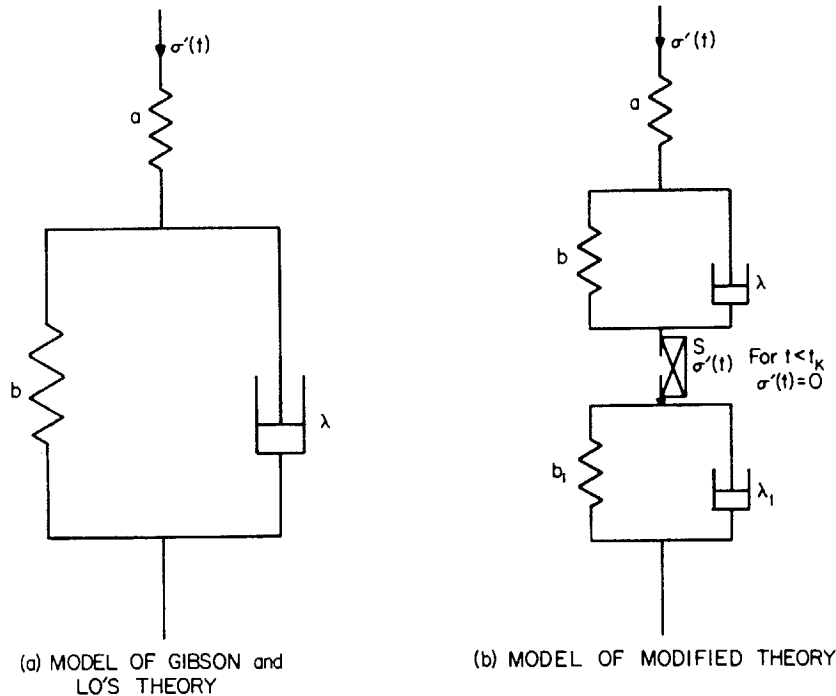


Fig. 4. Schematic representation of secondary compression (after Lo, 1961).

(1) Type 1 curve for which the rate of secondary compression gradually decreases with time until it becomes zero when the ultimate settlement is reached,

(2) Type 2 curve for which the rate of secondary compression is constant with the logarithm of time over a considerable range of time and then decreases rapidly near the ultimate settlement, and

(3) Type 3 curve for which the rate of secondary compression increases with the logarithm of time and then decreases gradually until it vanishes for very large times.

The acceleration of compression can be either abrupt, as in Type 3(a), or gradual as in Type 3(b); Figure 5 illustrates the three types of secondary compression curves outlined. While Equation 2 satisfactorily describes the secondary compression curve for Types 1 and 2, it does not account for the increase in the rate of secondary compression noticed in Type 3. Lo (1961) extended the theory to include the third type of secondary compression by adding another Kelvin element in series with the previous model as shown in Figure 4b. This model describes the three types of secondary compression curves: below a critical value of stress or strain the element

COMPRESSION OF PEATS

$S$  is rigid, the stress is, therefore, sustained by a  $b$ - $\lambda$  element (Types 1 and 2) and Equation 2 is used. However, when the critical value is exceeded, the element  $S$  loses its load carrying capacity and  $\varepsilon(t)$  operates as well in the  $b_1$ - $\lambda_1$  element (Type 3). For such cases, the time-dependent vertical strain is given by the equation:

$$\varepsilon(t) = \Delta\sigma [a + b(1 - e^{-\lambda/bt}) + b_1(1 - e^{-\lambda_1/b_1(t-t_k)})], t \geq t_k \dots\dots(3)$$

where  $b_1$  and  $\lambda_1$  are the parameters of the second element, and  $t_k$  is the time at which an abrupt increase in the observed compression curve occurs. The model described by Equation 3 was applied to cemented materials, clays exhibiting collapsing structures (LO, 1961) and to maintenance dredgings (KRIZEK & SALEM, 1974).

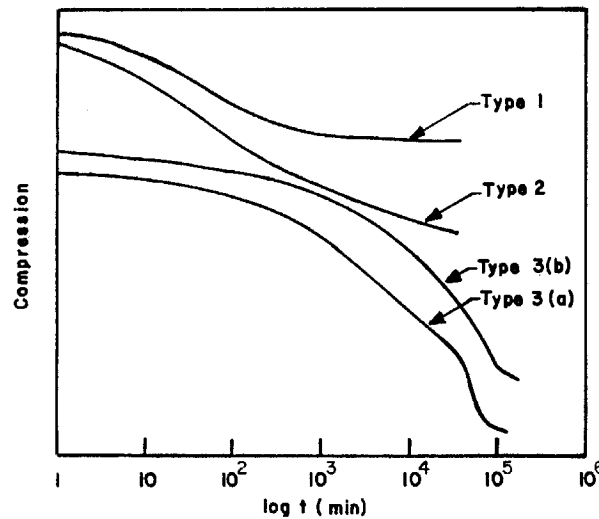


Fig. 5. Types of secondary compression curves (after Lo, 1961).

BARDEN (1965, 1968, 1969) developed a theory which takes non-linear viscosity into account by assuming that the viscous resistance is governed by a power law of the form:

$$\tau = b \left[ \frac{\partial e}{\partial t} \right]^{1/n} \dots\dots\dots(4)$$

where  $\tau$  is the viscous resistance of the adsorbed water causing time lag in secondary compression,  $e$  is the void ratio, and  $b$  and  $n$  are constants. The resulting governing equations of consolidation are non-linear and have to be solved numerically and the parameters can not be precisely evaluated from experimental data.

## EDIL & DHOWIAN

BERRY & POSKITT (1972) developed theories of consolidation for both amorphous granular and fibrous peats. The theories attempt to incorporate most of the variables encountered in the case of peat, such as finite strains, decreasing permeability and compressibility, and secondary compression time effects. Amorphous granular peat, which consists mainly of colloidal size particles and has a similar behavior to clay soils, was treated using a rheological model which is a non-linear form of the Gibson and Lo model. Fibrous peat, which has an essentially two-level structure (micropores and micropores), was modeled using a double Terzaghi pot to account for the physical mechanism involved in the primary and secondary consolidation in terms of a system of micropores draining to a macronetwork. Close agreement was obtained between the predicted and observed rates of settlement and pore pressure dissipation using the Berry and Poskitt theory for certain peats (BERRY & VICKERS, 1975). However, this theory assumes that the rate of secondary compression (expressed in change of void ratio per log cycle of time) is constant with time and with increasing effective pressure. Investigations conducted by many researchers indicated that the rate of secondary compression is not always a constant, but rather is a function of time, effective pressure, or both (THOMPSON & PALMER, 1961; LAKE, 1961; MESRI & GODLEWSKI, 1977; DHOWIAN 1978). The variable rate of secondary compression could, therefore, present limitations, in addition to already existing limitations of using one-dimensional theory, for the use of this theory in predicting settlements for certain soils.

## METHOD OF ANALYSIS

In general, the shape of the compression curves observed for the four peat samples tested in this study exhibit Type 3 behavior and the Lo theory appears applicable to describing their responses. Serious errors may result from extending the linear portion of the secondary compression curve as suggested by the other theories. Furthermore, it appears that it is preferable to retain the simplicity of a linear theory from which the parameters could be readily determined. The variation of these parameters may then be investigated and the non-linearities studied. Non-linear theories may then be adopted, if proved to be necessary. Two methods for the determination of the empirical parameters from long-term one-dimensional consolidation tests have been described by GIBSON & LO (1961) and LO (1961). In the first method a plot of  $\log_{10} \frac{\varepsilon(f) - \varepsilon(t)}{\Delta\sigma}$ , in which  $\varepsilon(f)$  is the final strain, against time  $t$ , provides the necessary information about the parameters. This method

*COMPRESSION OF PEATS*

requires the value of the final strain which may take place after an impractically long period of time. For example, the final strain for two of Portage Peat specimens took place at about 300 days (DHOWIAN, 1978). Therefore, this method presents certain practical limitations, since testing periods used at a given increment were about 20 days. The second method uses a plot of  $\log_{10} (\delta_1 - \delta_2)$  against  $t_1$ , in which  $\delta_2$  and  $\delta_1$  are the settlements at times  $t_2$  and  $t_1$ , respectively, the interval  $\Delta t = t_2 - t_1$  being kept constant at any convenient value. This method can be used without resorting to a final strain, but involves the assumption that  $\lambda/b$  remains constant during the working range. A third method has therefore been developed which is more convenient for the analysis of the data in determining the empirical parameters (DHOWIAN, 1978).

*New Method for Determining Empirical Parameters*

For Type 3 compression, a distinct change in long-term compression takes place at time  $t_k$ . The following asymptotic solutions for large values of time account for the secondary and tertiary compressions ( $\epsilon_s$  and  $\epsilon_t$ ) as defined in Figure 2:

$$\epsilon_s(t) = \Delta\sigma [a + b(1 - e^{-\lambda/b t})]; t_a \leq t \leq t_k \quad \dots\dots\dots(5)$$

and

$$\epsilon_t(t - t_k) = \Delta\sigma [a + b(1 - e^{-\lambda/b t_k}) + b_1(1 - e^{-\lambda_1/b_1(t-t_k)})]; t \geq t_k \quad \dots(6)$$

If Equations 5 and 6 are differentiated with respect to time, the rates of strain obtained are:

$$\frac{\partial \epsilon_s(t)}{\partial t} = \Delta\sigma \lambda e^{-\lambda/b t} \quad \dots\dots\dots(7)$$

and

$$\frac{\partial \epsilon_t(t - t_k)}{\partial (t - t_k)} = \Delta\sigma \lambda_1 e^{-\lambda_1/b_1(t-t_k)} \quad \dots\dots\dots(8)$$

Taking the logarithm of both sides in Equations 7 and 8 the following linear relations are obtained :

$$\log_{10} \frac{\partial \epsilon_s}{\partial t} = \log_{10} \Delta\sigma \lambda - 0.434 \frac{\lambda}{b} t \quad \dots\dots\dots(9)$$

and

$$\log_{10} \frac{\partial \epsilon_t}{\partial (t - t_k)} = \log_{10} \Delta\sigma \lambda_1 - 0.434 \frac{\lambda_1}{b_1} (t - t_k) \quad \dots\dots\dots(10)$$

Therefore, by plotting the logarithm of strain rate versus time from laboratory results of a particular soil under consideration, straight lines should be ob-

tained for the time ranges considered for the secondary and tertiary compressions beyond primary compression, given that the soil conforms with the basic assumptions made in the theory. The slopes and the intercepts yield the values of  $a$ ,  $b$ ,  $b_1$ ,  $\lambda$  and  $\lambda_1$  according to the procedures shown in Figure 6. Parameter  $a$  for primary compressibility is obtained by substituting the value of  $t_k$  and the corresponding strain  $\epsilon(t_k)$  in Equation 5 and solving for  $a$  as:

$$a = \frac{\epsilon(t_k)}{\Delta\sigma} - b + be^{-\lambda/b t_k} \dots\dots\dots(11)$$

A method similar to the one described above was also presented by LO, et al (1976) for Type 2 compression.

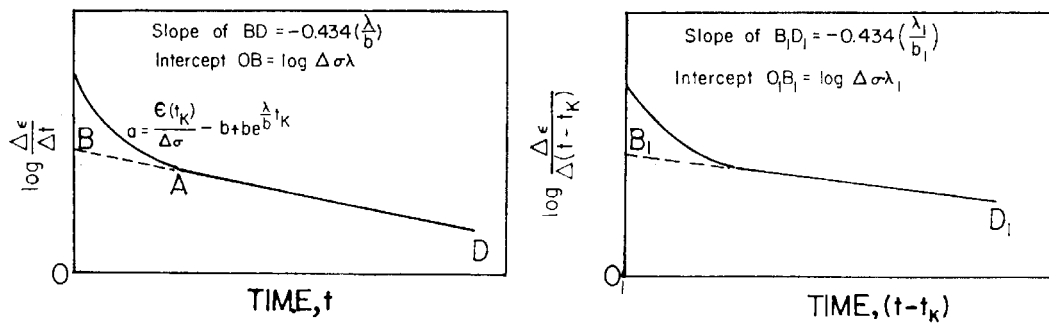


Fig. 6. Modified procedure for determining model parameters from consolidation data.

*Application of the Theory to Peats*

In order to verify the applicability of the Lo theory for the peats tested, the logarithm of strain rate versus time plots, an example of which is shown in Figure 7b and 7c, were constructed with the laboratory data (Figure 7a), and the empirical parameters were determined by use of the method described above. The linearity was obtained for a substantial time interval for all peat samples for all pressure increments, suggesting the applicability of the theory. The theoretical results determined by means of this model are plotted in Figures 8 and 9 for the lowest and the highest increments of pressure for each peat sample subjected to the conventional consolidation tests. Good agreement with experimental data was obtained over much of the time range indicating that the overall shape of the long-term compression curve can be represented adequately by this rheological model.



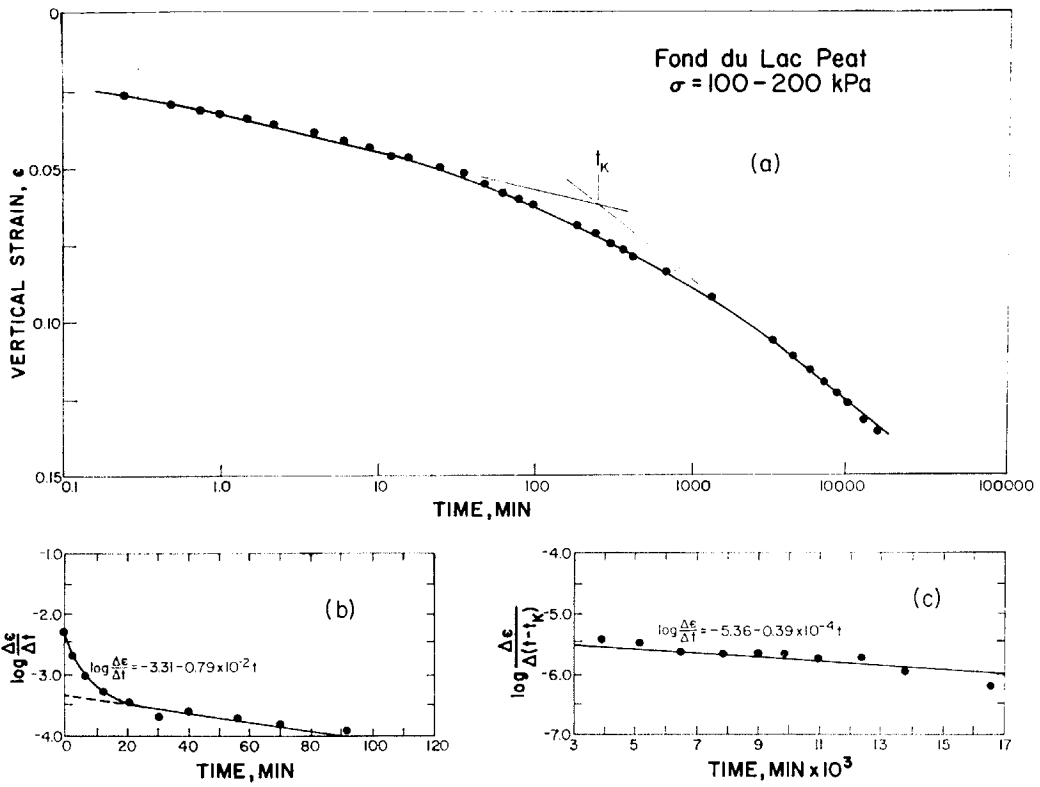


Fig. 7. Determination of model parameters from typical consolidation data.

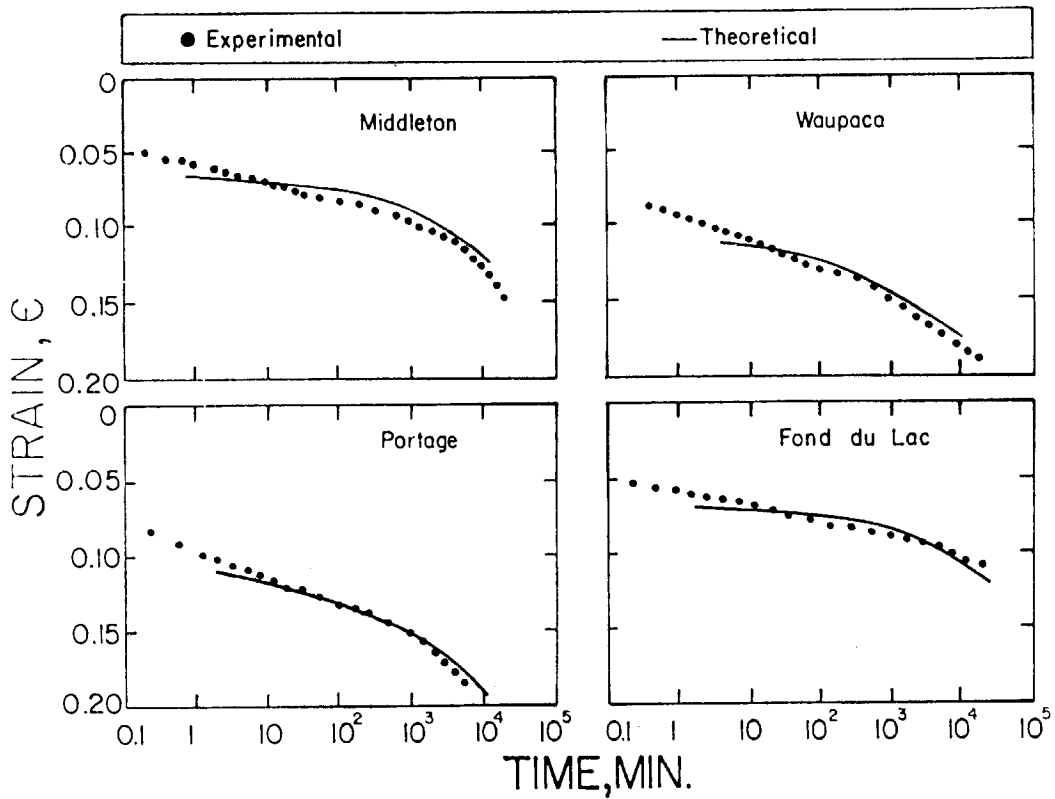


Fig. 8. Predicted and observed compression of peat soils under a pressure increment of 0-25 kPa.

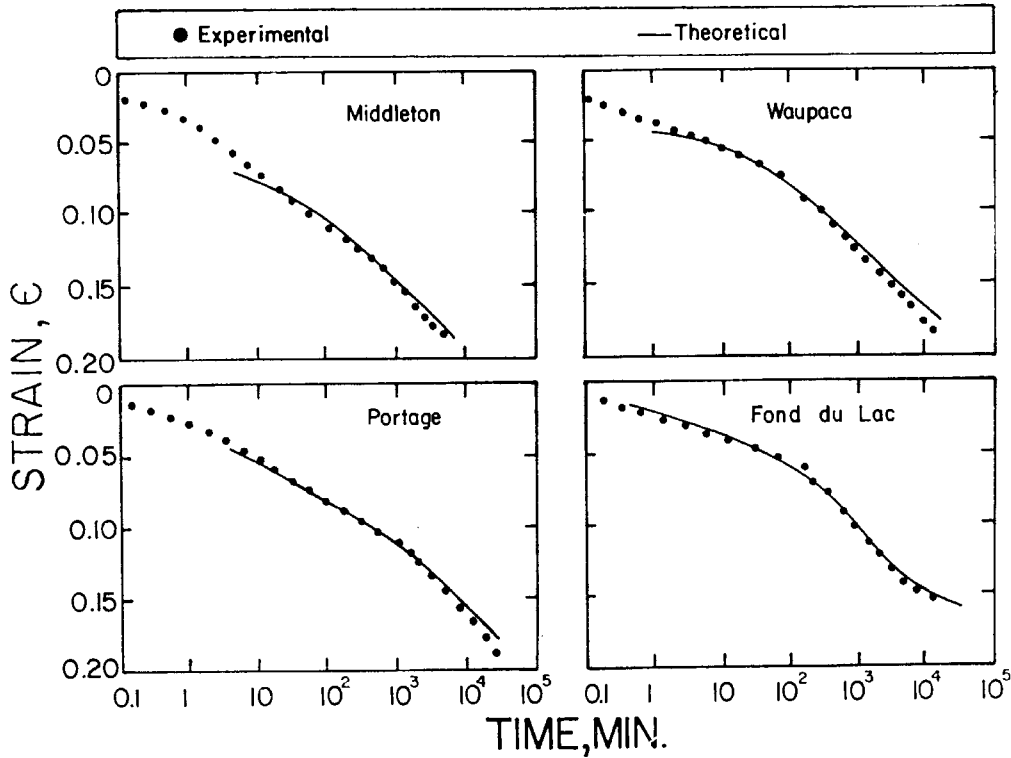


Fig. 9. Predicted and observed compression of peat soils under a pressure increment of 200-400 kPa.

VARIATION OF EMPIRICAL PARAMETERS

The characteristic behavior of peats can be described by the following factors, which are combinations of the empirical parameters.

- $\lambda/b$  represents the rate of “secondary” compression (rate factor for secondary compression),
- $\lambda_1/b_1$  describes the rate of “tertiary” compression, the compression after the increase in the rate of secondary compression curve takes place, for time greater than  $t_k$  (rate factor for tertiary compression),
- $b/a$  is the relative importance of the secondary to primary compression (secondary compressibility ratio),
- $b_1/a$  is the relative importance of the tertiary to primary compression (tertiary compressibility ratio),
- $1/\lambda$  represents the viscosity of soil structure during the secondary compression (secondary viscosity), and
- $1/\lambda_1$  gives the viscosity of soil structure during the tertiary compression (tertiary viscosity).

COMPRESSION OF PEATS

The empirical parameters needed to estimate the secondary compression of peats, both before and after  $t_k$ , are plotted in Figure 10 as a function of consolidation pressure. On this figure, it is also possible to differentiate between different types of peat, i.e., open symbols indicate amorphous granular Fond du Lac Peat; solid symbols indicate fibrous peats. Since the compression rate factors  $\lambda/b$  and  $\lambda_1/b_1$ , which represent the combination of parameters that govern the rate of secondary and tertiary compressions, have average values of about  $18 \times 10^{-3} \text{ min}^{-1}$  and  $15 \times 10^{-5} \text{ min}^{-1}$ , respectively, these peats exhibit higher rates of compression after  $t_k$  than before, i.e., higher tertiary compression rate than secondary compression rate.  $\lambda/b$  increases with increasing consolidation stress up to the last stress increment application (400 kPa) at which it drops and merges with  $\lambda_1/b_1$ ,

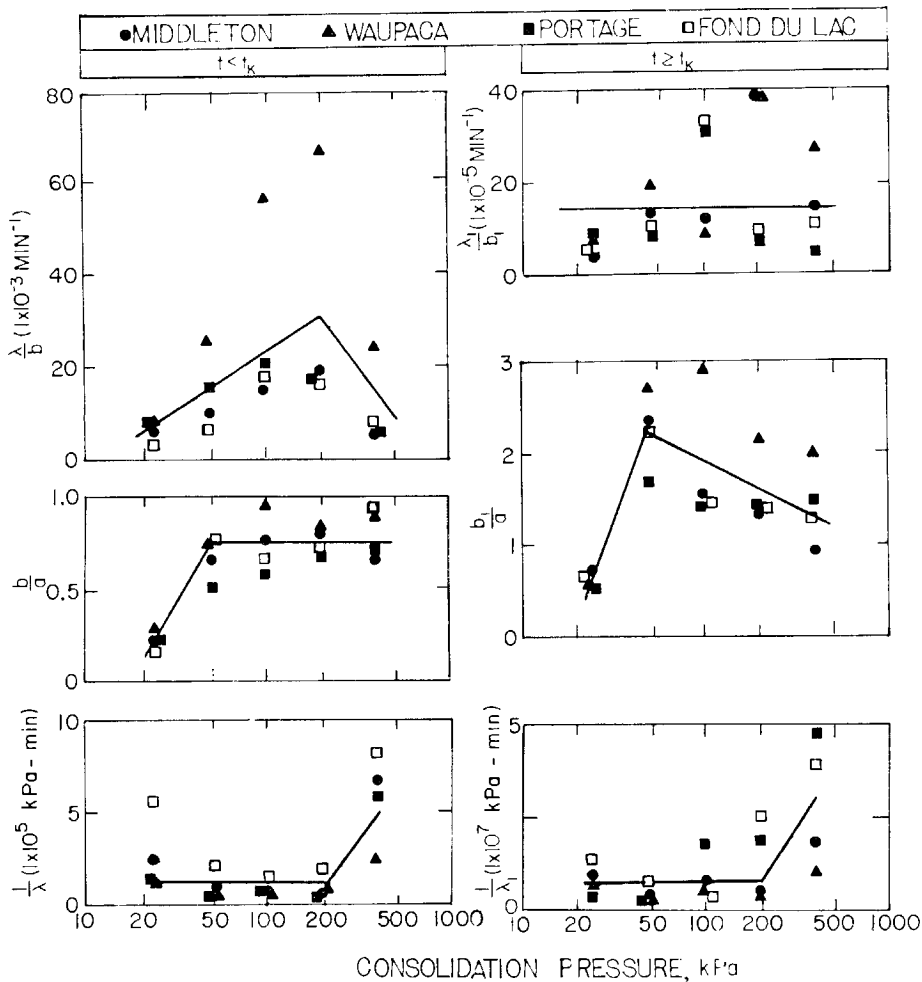


Fig. 10. Empirical parameters as a function of consolidation pressure.

reflecting a major change in microstructural arrangement in peat at high level of stresses, possibly the break-down of the two-level structure.  $\lambda_1/b_1$  appears to be independent of consolidation stress. Amorphous granular peat exhibits the lowest values of  $\lambda/b$  and  $\lambda_1/b_1$  in comparison with the fibrous peats; however, dependency of these parameters on the type of peat and initial void ratio appears to be relatively small.

The compressibility ratios  $b/a$  and  $b_1/a$  have average values of 0.65 and 1.54, respectively, indicating that the tertiary compression is more significant with respect to the primary compression than the secondary compression. Furthermore, there is a dramatic increase in the relative importance of the secondary and tertiary compression with respect to the primary compression after the first stress increment of 0 to 25 kPa, thereon these parameters exhibit little dependence on consolidation stress. There appears no discernible dependency on the type of peat or initial water content.

The viscosity of peat structure during secondary and tertiary compressions is represented by  $1/\lambda$  and  $1/\lambda_1$ , respectively, and the difference between the values of these two parameters indicates the change in the level of the structure which takes the primary part in the compression process after time  $t_k$ . In general, values of  $1/\lambda$  are about 100 fold lower than the values of  $1/\lambda_1$  and both parameters are quite independent of consolidation stress except the last stress increment (400 kPa) at which both tend to increase slightly. Their dependency on the type of peat and initial void ratio is relatively small; however, it is observed that the amorphous granular peat exhibits the highest values of  $1/\lambda$  and  $1/\lambda_1$  in comparison to the fibrous peats.

Time  $t_a$ , which indicates the beginning of secondary compression (as defined by  $\epsilon_s$  in Figure 2), had a value of 1.5 minutes or less for all stress increments except the last one for all peat samples. At the last increment of stress,  $t_a$  increases as  $t_a$  and  $t_k$  merge into each other. The dependency of  $t_k$  on consolidation pressure is shown in Figure 11. For the first increment of pressure  $t_k$  has large values (1000 to 5000 min.), but under subsequent pressure increments it stabilizes at around 200 minutes.

In general, there appears to be no apparent consistent trend in the variation of the factors considered above with respect to the type of peat and initial water content or void ratio. However, it is possible to differentiate broadly between the amorphous granular peat and the fibrous peats consistent with the classification suggested by MACFARLANE & RADFORTH (1965). There also seems to be, in general, little dependence of the parameters on consolidation stress within the range of consolidation pressures applied with the exception

COMPRESSION OF PEATS

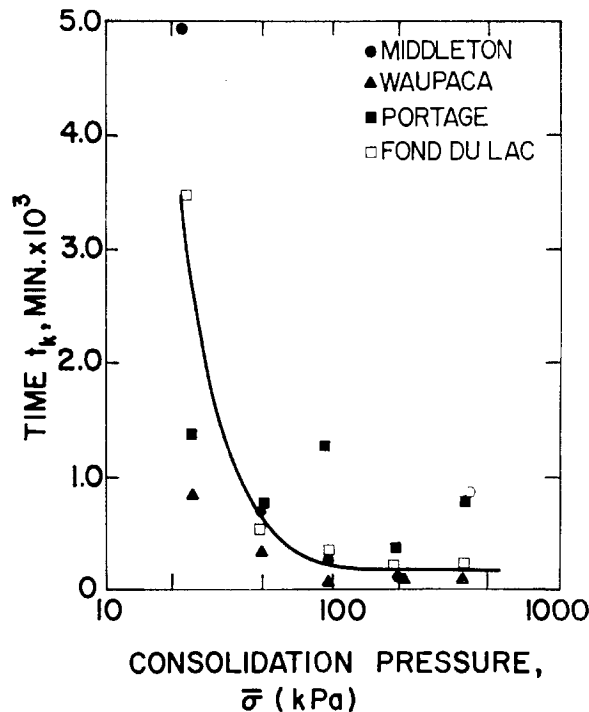


Fig. 11. Time  $t_k$  versus consolidation pressure.

of the noted changes in behavior of certain parameters at the first or the last increment of stress application.

EXAMPLE PROBLEM

Within the range of applicability justified by the limited data presented here, the total compression of an undisturbed specimen of peat subjected to a long-duration one-dimensional consolidation test can be readily estimated, even without conducting a test, provided that certain information about the material and the stress ranges is known. For example, given a peat sample with an initial void ratio of 8.00, water content of 450% and a fiber content of 40% subjected to a consolidation test, the total compression of a specimen 2 cm thick loaded with 100 kPa to 200 kPa at times of 10,000 and 100,000 minutes may be estimated as follows: From Figure 11,  $t_k$  corresponding to this pressure range is about 200 minutes, Figure 10 gives  $1/\lambda = 1 \times 10^5$  kPa-min,  $\lambda/b = 2 \times 10^{-2}$  min<sup>-1</sup>,  $1/\lambda_1 = 1.25 \times 10^7$  kPa-min,  $\lambda_1/b_1 = 1.4 \times 10^{-4}$  min<sup>-1</sup>, and  $b/a = 0.75$ , whereupon  $a = 6.67 \times 10^{-4}$  (kPa)<sup>-1</sup>,  $b = 5 \times 10^{-4}$  (kPa)<sup>-1</sup>, and  $b_1 = 5.7 \times 10^{-4}$  (kPa)<sup>-1</sup>. Hence the use of Equation 3 with the above values and  $\Delta\sigma$  equal to 100 kPa, yields  $\Delta H$  (at 10,000 minutes) = 3.2 mm and  $\Delta H$  (at 100,000 minutes) = 3.5 mm.



#### *EDIL & DHOWIAN*

The method described herein is based strictly on laboratory experiments using one-dimensional consolidation tests. When this method is used for predicting the field settlement of peat deposits, possible errors due to the difference in strain rates between the laboratory and the field may be encountered, in addition to the possible errors known to be inherent in using a one-dimensional theory in certain field conditions.

#### SUMMARY AND CONCLUSIONS

Based on this investigation of the secondary compression of peat soils subjected to long-term conventional consolidation tests the following conclusions can be advanced.

- (1) The compression of all peat samples tended to increase in essentially a linear manner with the logarithm of time for a considerable period of time, after which the rate of secondary compression increased significantly, giving rise to a "tertiary" compression. This type of behavior suggests that a major structural change occurs at some value of strain for each particular load increment. With advanced increments of stress application, this behavior was somewhat modified with secondary and tertiary components merging together, which indicates the influence of stress history on structural changes.
- (2) A quantitative analysis based on the rheological model suggested by Lo (1961) is presented. The method for evaluating the soil parameters has been modified and the new method does not require a knowledge of the final strain which is ordinarily not available for peat tests, since a long period of time is needed to attain the final strain. The Lo model is preferred to other models since it can represent the tertiary compression which is exhibited by the peats tested.
- (3) Relatively good agreement between the strain predicted by the model and the measured strain is obtained for the four types of peat tested, indicating that the model can represent the compression curves adequately as obtained from the experiments.
- (4) The model parameters computed from the experimental data using the new method suggested are related to the consolidation pressure. For all practical purposes, there seems to be relatively little or no dependence of any of the parameters on water content and peat type.

Finally, the prediction of the compression of a peat sample based on a knowledge of the type of peat and the applied stress is illustrated by use of the analytical procedure outlined. Certain errors may be encountered in

## COMPRESSION OF PEATS

using the method described herein for predicting the field performance, which is considered to be beyond the scope of this paper.

## ACKNOWLEDGEMENT

The junior author was supported financially by the University of Riyadh, Saudi Arabia, during this work. The authors would like to thank Professor Gerhard B. Lee and Mr. Norman H. Severson for their interest and assistance.

## REFERENCES

- ADAMS, J. I. (1961), Laboratory Compression Tests on Peat, *Proceedings of the Seventh Muskeg Research Conference*, National Research Council of Canada, Technical Memorandum No. 71, pp. 36-54.
- ADAMS, J. I. (1963), A Comparison of Field and Laboratory Measurements in Peat, *Proceedings of the Ninth Muskeg Research Conference*, National Research Council of Canada, Technical Memorandum No. 81, pp. 117-135.
- ADAMS, J. I. (1965), The Engineering Behavior of A Canadian Muskeg, *Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 3-7.
- BARDEN, L. (1965), Consolidation of Clay with Non-Linear Viscosity, *Geotéchnique*, Vol. 15, No. 6, pp. 345-362.
- BARDEN, L. (1968), Primary and Secondary Consolidation of Clay and Peat, *Geotéchnique*, Vol. 18, pp. 1-24.
- BARDEN, L. (1969), Time Dependent Deformation of Normally Consolidated Clays and Peats, *Journal of the Soil Mechanics and Foundations Division*, American Society of Civil Engineers, Vol. 95, No. SM-1, pp. 1-31.
- BERRY, P. L. and POSKITT, T. J. (1972), The Consolidation of Peat, *Geotéchnique*, Vol. 22, No. 1, pp. 27-52.
- BERRY, P. L. and VICKERS, B. (1975), Consolidation of Fibrous Peat, *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 101, No. GT-8, pp. 741-753.
- BUISMAN, A. S. K. (1936), Results of Long Duration Settlement Tests, *Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 103-106.
- DHOWIAN, A. W. (1978), Consolidation Effects on Properties of Highly Compressible Soils-Peats, *Ph.D. Thesis*, Department of Civil and Environmental Engineering, University of Wisconsin-Madison.
- GIBSON, R. E. and LO, K. Y. (1961), A Theory of Consolidation of Soils Exhibiting Secondary Compression, *Acta Polytechnica Scandinavica*, Ci 10 296, pp. 1-15.
- HANRAHAN, E. T. (1954), An Investigation of Some Physical Properties of Peat, *Geotéchnique*, Vol. 4, No. 3 pp. 108-123.
- HANRAHAN, E. T. (1964), A Road Failure on Peat, *Geotéchnique*, Vol. 14, No. 3, pp. 185-202.
- KRIZEK, R. J. and SALEM, A. M. (1974), Behavior of Dredged Materials in Dyked Containment Areas, *Technical Report No. 5* by Northwestern University, Civil Engineering Department, to the U.S. Environmental Protection Agency.
- LAKE, J. R. (1961), Investigation of the Problem of Constructing Roads on Peat in Scotland, *Proceedings of the Seventh Muskeg Research Conference*, National Research Council of Canada, Technical Memorandum No. 71, pp. 133-148.
- LEA, N. D. and BRAWNER, C. O. (1963), Highway Design and Construction over Peat Deposits in Lower British Columbia, *Highway Research Record No. 7*, pp. 1-32.

EDIL & DHOWIAN

- LEONARDS, G. A. and GIRAULT, P. (1961), Study of the One-Dimensional Consolidation Test, *Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 213-218.
- LO, K. Y. (1961), Secondary Compression of Clays, *Journal of the Soil Mechanics and Foundation Division*, American Society of Civil Engineers, Vol.87, No. SM-4, pp.61-87.
- LYNN, W. C., MCKINZIE, W. E., and GROSSMAN, R. B. (1974), *Field Laboratory Tests for Characterization of Histosols, Histosols: Their Characteristics Classification, and Use*; Soil Science Society of America, Special Publication No. 6.
- MACFARLANE, I. C. (1969), *Muskeg Engineering Handbook*, National Research Council of Canada, University of Toronto Press, Toronto, Canada.
- MACFARLANE, I. C. and RADFORTH, N. W. (1965), A Study of the Physical Behavior of Peat Derivatives Under Compression, *Proceedings of the Tenth Muskeg Research Conference*, National Research Council of Canada, 159 p.
- MESRI, G. and GODLEWSKI, P. M. (1977), Time and Stress-Compressibility Interrelationship, *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 103, No. GT-5, pp. 417-430.
- SAMSON, L. and LA ROCHELLE, P. (1972), Design and Performance of an Expressway Constructed Over Peat by Preloading, *Canadian Geotechnical Journal*, Vol. 9, No. 4, pp. 447-466.
- SCHROEDER, J. and WILSON, N. E. (1962), The Analysis of Secondary Consolidation of Peat, *Proceedings of the Eighth Muskeg Research Conference*, National Research Council of Canada, Technical Memorandum No. 74, pp. 130-142.
- TERZAGHI, K. (1925), *Principles of Soil Mechanics*, Engineering News-Record, Vol. 25, Nos. 19-23, 25-27.
- THOMSON, J. B. and PALMER, L. A. (1951), Report of Consolidation Tests with Peat, *Special Technical Publication No. 126*, American Society for Testing and Materials, pp. 4-8.
- WEBER, W.G. (1969), Performance of Embankments Constructed Over Peat, *Journal of the Soil Mechanics and Foundations Division*, American Society of Civil Engineers, Vol. 95, No. SM-1, pp. 53-76.

# STATE-OF-THE-ART REPORT ON SETTLEMENTS AND TIME RATES OF CONSOLIDATION<sup>+</sup>

TOSHINOBU AKAGI\*

## SYNOPSIS

Although the ultimate consolidation settlement can often be predicted with reasonable accuracy, it has repeatedly been pointed out that the rate of settlement of a structure built on soft clay is almost always much faster than that predicted on the basis of the one-dimensional consolidation theory with the use of oedometer test results. The three-dimensional theory appears to give a promising clue to account for the large difference in time rates when a much greater horizontal permeability is taken into account. In order to make such a theoretical approach practicable, it is vital to establish a reliable geological profile with drainage conditions clearly defined. For this purpose the development of a new technique is required to measure the mass permeability of the compressible stratum. This report reviews the effectiveness of vertical drains in terms of settlement rates and undrained shear strength of the stabilized foundation soil. Both the displacement type and non-displacement type sand drains disturb the soft ground when being installed. The adverse effects of the former type does not appear to be as severe as has been accused. Recent studies even indicate some beneficial aspects of displacement type sand drains. This suggests the necessity of reappraisal of this more economical type.

## INTRODUCTION

During preparation of the general report (AKAGI, 1979a) for Main Session 2 entitled "Problems in Soft Clay/Soil Technology and Stabilization," the Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering, it was noted that among the contributions to this session the problem of greatest concern still lies in the deformation of soft foundation soil, particularly vertical deformation or settlement, and the development of deformation with time or the rate of settlement.

The first group of papers submitted to this session dealt with the settlement of structures such as embankments, oil tanks, etc., on the natural clay foundations. The second group of papers dealt with the settlements of the clay improved or disturbed by the installation of a variety of vertical drains, typically sand drains.

<sup>+</sup> Presented during Main Session 2, Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering, Singapore, July 24-27, 1979. Revised for this publication.

\* Professor, Department of Civil Engineering, Toyo University, Saitama, Japan. Currently on leave at the Asian Institute of Technology, Bangkok, Thailand.

Although some difficulty has been reported with estimating the magnitude of settlements, it appears that the time-settlement relationship is the most difficult one to predict with a reasonable degree of accuracy. The following summarizes the state-of-the-art on this subject and points out some of the important problems which currently appear to be of common interest in our profession.

#### ONE-DIMENSIONAL CONSOLIDATION

Our present capability of handling routine settlement problems at the professional level still appears to be limited basically to that derived from Terzaghi's one-dimensional consolidation theory which is now well over half a century old. As is well known, Terzaghi's solution is by no means almighty, and in fact its limitations are well understood among practitioners. Better theories in more general terms are available, notably for instance, the general theory of consolidation by BIOT (1941). No matter how elegant the theory is in terms of theoretical concept, the profession benefits very little unless the theory offers a practical solution with the use of simple parameters which may be determined with relative ease, confidence and economy.

As a matter of fact the oedometer test is a simple one, although time consuming, and requires a relatively simple apparatus and much less sophisticated technique than the triaxial test, for instance. In actuality this test is a small-scale laboratory model test run specifically to verify Terzaghi's one-dimensional consolidation theory. We obtain the coefficient of volume compressibility  $m_v$ , to support his assumption that the soil specimen behaves elastically, and we determine the coefficient of consolidation  $c_v$ , to verify his assumption that consolidation takes place in accordance with the law of diffusion or flow of heat.

Differing essentially from other tests such as determinations of the physical properties of soil, it may be stated that the oedometer test does not provide the intrinsic properties of soil, but rather how well the small specimen behaves in accordance with Terzaghi's theory. Even if it behaves in a different manner, we force it to behave as if it does, and accordingly assign parameters such as  $m_v$  and  $c_v$ .

Even under one-dimensional loading we have long recognized, to a varying degree, that the primary consolidation defined by Terzaghi is preceded by immediate compression under an undrained condition and succeeded, by secondary compression under a drained condition.

CONSOLIDATION SETTLEMENTS AND TIME RATES

We are interested in the behavior of a small specimen confined in a rigid oedometer ring in the hope that it would show the behavior of the thick compressible layer which is at least a few hundred times as thick as the thin specimen we test. Apart from such problems as sample disturbance and heterogeneity of actual foundation soil, it is interesting and important for us to know how well the small sample represents the thick homogeneous clay stratum under one-dimensional compression. In this connection, interesting data was presented by ABOSHI (1973), Figure 1. Specimens of various sizes were cut out of an artificial clay layer consisting of remolded marine clay which had been consolidated for more than 6 years. The smallest specimen which is of standard size in Japan, is 6 cm in diameter and 2 cm in thickness, while the largest is 300 cm in diameter and 100 cm in height. They were consolidated one-dimensionally for a period as long as several hundred days.

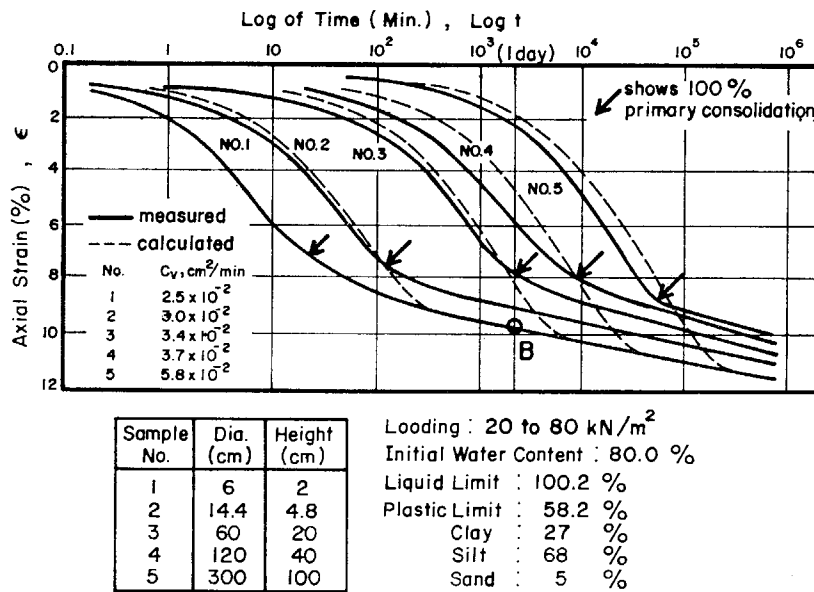


Fig. 1. Time-settlement curves of remolded specimens with various thicknesses (Aboshi, 1973).

These time-settlement curves tell us a number of things concerning one-dimensional consolidation. Firstly, the thicker the specimen, the larger is the axial strain corresponding to the primary consolidation (indicated by arrows in Figure 1). In other words, the value of  $m_v$  increases with the thickness. Secondly, the secondary compression in terms of axial strain proceeds approximately in parallel, regardless of the thickness of specimens, but the total strains at any time  $t$  are smaller when the thickness is great. In the standard oedometer test the 24 hour reading represented by Point B is cus-

AKAGI

tomarily taken to construct the  $e$ -log  $p$  curve (void ratio versus logarithm of consolidation pressure) and this compression would correspond very roughly to the primary compression of a thick homogeneous stratum with thickness exceeding several meters. This seems to be a reason, at least in part, why the small specimen gives a reasonable value for the compression of the thick stratum under one-dimensional condition.

Thirdly, the observed time-settlement curves show that as the thickness increases, consolidation takes place more rapidly than the theoretical curves would indicate. In fact, when the thickness is 100 cm, the value of  $c_v$  is more than twice as much as the  $c_v$  value determined from the small standard size specimen in the oedometer. Incidentally, this reminds us of the fairly consistent differences noted between the  $c_v$  values determined by the log  $t$  method and the  $\sqrt{t}$  method. The latter, more often than not, gives a higher value, approximately twice the former.

With particular reference to the secondary consolidation, it was BJERRUM (1972) who brought a new life to the  $e$ -log  $p$  relationship, extending the old findings of TAYLOR (1942) and incorporating the dimension of time. Figure 2, for instance, eloquently illustrates the effect of preloading on the rate of secondary consolidation.

Such a diagram, clearly demonstrates one of the major reasons why most normally-consolidated clays conspicuously show a critical pressure  $p_c$ , considerably higher than the effective overburden pressure  $p_o$ , during the oedometer test. The diagram also illustrates the significance of  $p_c$ , beyond which the clay exhibits a far greater compressibility; i.e. point (b) to (c). Upon reaching point (d), the preload or surcharge is removed. Then the state of soil is represented by point (e) which corresponds to a much lower rate of secondary consolidation than the rate which may be attained during the same period of time without the use of the preload.

TIME RATE OF ONE-DIMENSIONAL CONSOLIDATION

It has been widely recognized that the observed rate of settlement of structures on soft clay is almost always very much faster than that calculated using the one-dimensional consolidation theory based on oedometer tests carried out on small samples (SIMONS, 1974; LADD, et al 1977). Actually a few papers submitted to this session (e.g., ADACHI & TODO, 1979; TAN & PHANG, 1979) also report that the field  $c_v$  value was several tens of times as large as the laboratory value of  $c_v$ .

CONSOLIDATION SETTLEMENTS AND TIME RATES

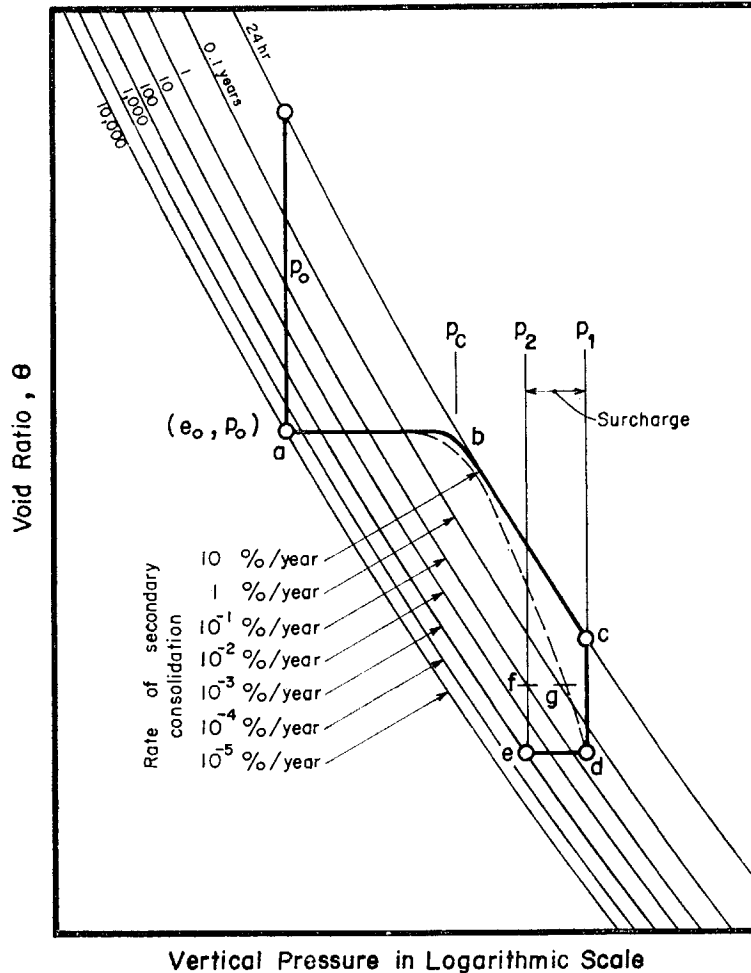


Fig. 2. Preloading effect on  $e$ - $\log p$  relationship with time (Bjerrum, 1972).

A recent study conducted at AIT on several test embankments also indicates similar results, (Table 1). The field  $c_v$  values obtained by the curve-fitting method using Terzaghi's one-dimensional consolidation theory gave values anywhere between a few times and 200 times as great as the  $c_v$  values obtained from the oedometer tests. As summarized in Table 1, these were fairly large embankments built on thick strata of clays having the ratio of the thickness of the compressible layer to the width of embankment  $H/B$ , ranging from  $1/4$  to  $3/4$ .

Figure 3 shows the time-settlement curves for the Atsugi test embankment. It is well known on the basis of oedometer test results that the  $c_v$  value decreases drastically when the stress exceeds the critical pressure, often as much as one order of magnitude or more. Even if some median value of  $c_v$



**Table 1. Curve fitting for time<sup>2</sup>-settlement relationships of test embankments (Samaras nghe, 1979).**

| Case Study  | Atsugi, Japan | Nong Ngoo Hao, Thailand | Pom Prachul, Thailand | Thonburi-Paktho, Thailand |
|---|---------------|-------------------------|-----------------------|---------------------------|
| Embankment Height (m)                                 | 8.5           | 2.9                     | 2.4                   | 3.0                       |
| Size, B (m)   | 203 × 60      | 100 × 40                | 90 × 34               | 65 × 20                   |
| Thickness of compressible Layer, H (m)                | 14.5          | 15.5                    | 17.0                  | 14.5                      |
| H/B   | 0.24          | 0.39                    | 0.50                  | 0.73                      |
| Settlement Records                                    | 10 months     | 8 months                | 1 year                | 4 years                   |
| Lab $c_v$ from oedometer tests (cm <sup>2</sup> /day) | 54-490        | 10-950                  | 17-432                | 9-130                     |
| Field $c_v$ (cm <sup>2</sup> /day)                    | 1,245         | 2,036                   | 1,382                 | 1,050                     |
| Field $c_v$ / lab $c_v$                               | 3-23          | 2-204                   | 3-81                  | 8-122                     |

is chosen, it is evident that the difference is still one order of magnitude. At this site an extensive field and laboratory investigation was conducted, and all the test holes indicated the presence of several distinct sand seams in the stratum of soft organic clay, about 14 m in thickness (JHPC, 1964). It was suspected that some of the sand seams must have served as effective drainage layers while the compressible stratum consolidated. With present-day techniques, however, it is still impossible to pinpoint which seams have horizontal continuity and drainability capable of shortening drastically the time required for consolidation.

As has been pointed out by OLSON & LADD (1979), the finite difference method has proved to be a tool of great versatility to deal numerically with one-dimensional consolidation problems which require consideration for such factors as large and nonuniform strains, nonlinear stress-strain relationships, nonuniform excess porewater pressures, layered systems and variable coefficients of consolidation with time. While part of the disagreement noted between the field and laboratory parameters may be attributable to some of these factors (extremely large strains in particular), which are readily incorporated in the finite difference method, the great difference has not been fully accounted for with convincing evidence.

CONSOLIDATION SETTLEMENTS AND TIME RATES

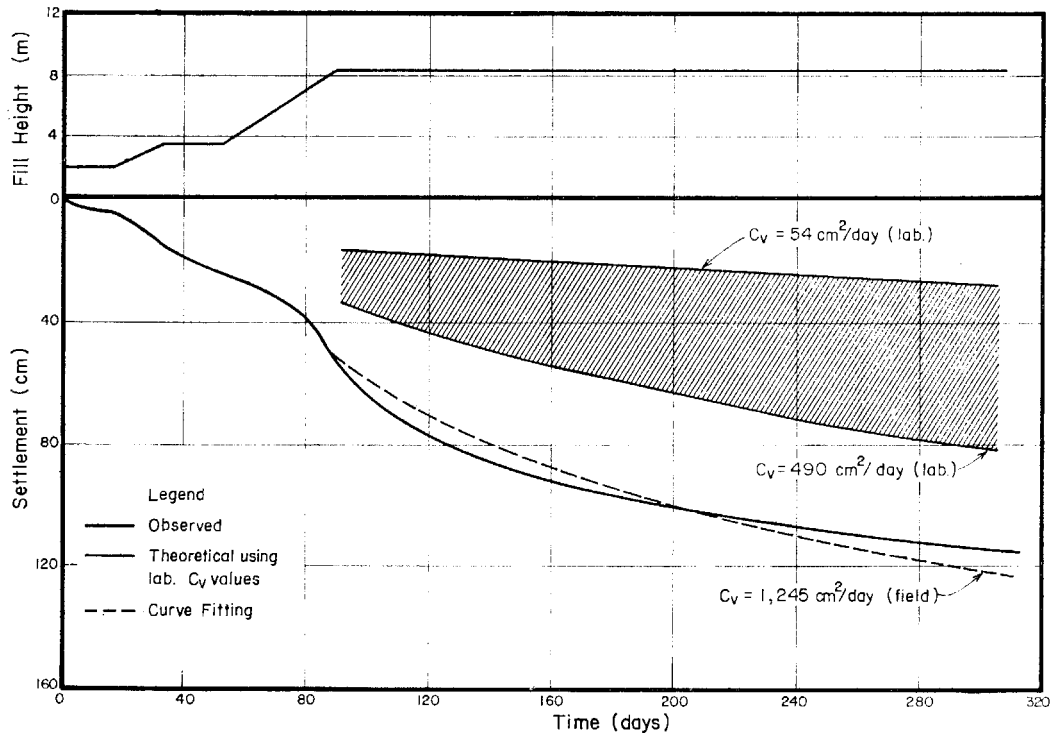


Fig. 3. Time-settlement curves, Atsugi test embankment (Samarasinghe, 1979).

Indeed this is a serious problem and it follows that we are simply incapable of predicting time-settlement relationships, because the error involved is roughly 1 to 2 orders of magnitude in terms of the time required to attain a certain degree of consolidation. Our capability of predicting the ultimate or final settlement has reached the point where we can happily discuss an error on the order of 10 to 20%. Nevertheless, in many cases we are almost at a loss when we have to draw a time-settlement curve. Very fortunately in actual projects, the construction period usually provides us with the opportunity, to correct our erroneous prediction. Also, what we are concerned with is often the post-construction settlements which are less in magnitude and slower in rate. Therefore, luckily, our poor prediction does not look as bad as has just been pointed out. However, such inaccurate prediction is totally unsatisfactory when one has to determine the necessity of vertical drains to accelerate the expected consolidation. Although the wrong prediction of the time-settlement relationship does not usually lead to a dramatic sign of failure such as a catastrophic slide or bearing capacity failure, it is not only embarrassing but costly in many cases.

TWO-AND THREE-DIMENSIONAL CONSOLIDATION

One-dimensional consolidation is considered to be approximately true if we are concerned with the consolidation phenomenon under reclamation fill or at the center of a wide embankment built on a relatively thin compressible stratum. We are well aware, however, that the behavior of soft clay differs considerably when the stress-strain-drainage conditions at the construction site are not simulated by our routine model test on a small specimen confined in an oedometer. In fact, in the majority of actual cases we know an element of soil we consider deforms and drains in all directions. How important is this 2-or 3-dimensional consolidation in terms of vertical settlement and the time-settlement relation?

SKEMPTON & BJERRUM (1957) proposed a practical method to improve the one-dimensional settlement analysis, taking into consideration pore pressures and deformations associated with undrained loadings. Their

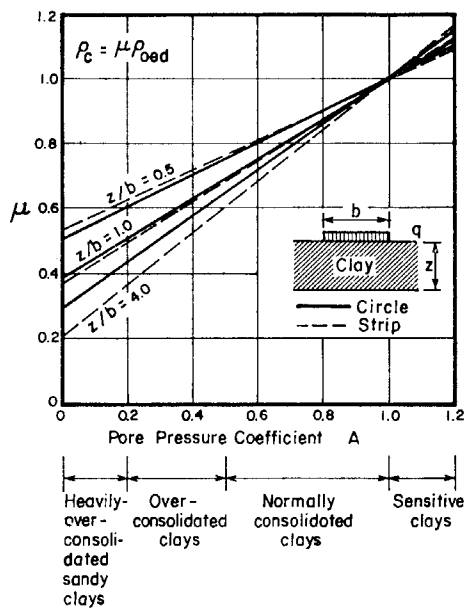


Fig. 4. Computing consolidation settlements from oedometer test results (Skempton & Bjerrum, 1957).

method attempts to make the use of oedometer test results in order to give a better fit to the 3-dimensional problem, ignoring the effects of lateral deformations. For saturated clays, the correction factor  $\mu$  is given by the pore pressure coefficient  $A$  and the ratio of the thickness of the compressible layer  $z$  to the width of the loaded area  $b$ , Figure 4.

Skempton and Bjerrum stated that the total settlement  $\rho_t$  at any time  $t$  after the load application may be defined by the expression:

$$\rho_t = \rho_i + U \mu \rho_{oed}$$

where  $\rho_i$  is the immediated settlement, and  $\rho_{oed}$  and  $U$  are the settlement and the average degree of consolidation, respectively, as evaluated from the

theory of one-dimensional consolidation. In spite of its relative simplicity, their method is still often ignored in routine practice and we have not accumulated sufficient experience to support the usefulness of this procedure. Unfortunately, the comparison of the time-settlement relationships between

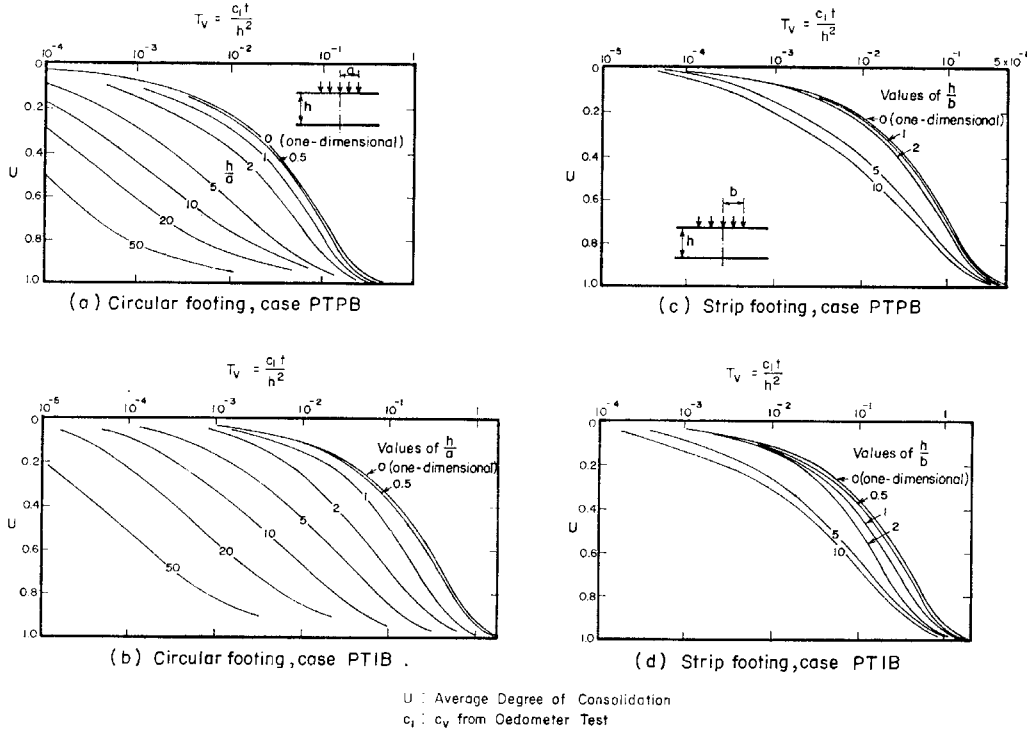
### CONSOLIDATION SETTLEMENTS AND TIME RATES

the observed and the calculated given in their original paper were not altogether convincing.

The effects of lateral deformations which were not taken into account in the Skempton-Bjerrum approach may be investigated by the stress path method developed by LAMBE (1967, 1979). The investigation carried out by means of this powerful concept (SIMONS & SOM, 1969) suggests that axial strains which are usually of practical interest are highly stress path dependent. That is, the effect of 2-or 3-dimensional loading situation may have a significant bearing on the settlement. Unfortunately again, very few data are available to check the applicability of such methods of settlement analysis to our actual complex problems.

Perhaps the most significant theoretical contribution made in the last decade or so is the effect of lateral drainage on consolidation rates. As reported by LADD, et al (1977), various numerical techniques recently applied to this problem have allowed development of chart solutions for a wide range of loading geometries and drainage problems including consideration for anisotropic permeabilities, for instance, the solutions presented by DAVIS & POULOS (1972). Figure 5 shows the effects of lateral drainage on consolidation rates beneath the center of a circular load and a strip load on an isotropic clay layer, with and without bottom drainage. These curves correspond to time-settlement curves of circular and strip footings of different sizes with respect to the thickness of the compressible layers with different drainage conditions. Figures 5a and 5b are for circular footings, whereas Figures 5c and 5d are for strip footings: PTPB represents the two-way drainage, i.e. the permeable top and permeable bottom, and PTIB, the one-way drainage, i.e. the permeable top and impervious bottom. The influence of the hydraulic boundary conditions is indeed remarkable.

It is interesting to note that the layer depth has a much greater influence on the degree of consolidation for circular footings than for strip footings. For a layer of constant depth, as the footing size decreases, the 3-dimensional effects become increasingly apparent and the ability of the pore pressures to dissipate laterally as well as vertically results in a considerable increase in the average rate of pore pressure dissipation, even for relatively shallow layers. For the case of a circular load with the ratio of the thickness to the radius,  $h/a$ , being equal to 5, consolidation occurs about 10 times more rapidly than predicted by Terzaghi's 1-dimensional theory. As the  $h/a$  ratio becomes greater or the thickness of the compressible stratum increases, settlement takes place much more rapidly. An embankment may roughly be simulated by a strip footing, but the effect of 3-dimensional consolidation on a strip



**Fig. 5. Time factor versus average degree of consolidation beneath the center of circular and strip footings, lateral drainage permitted (Davis & Poulos, 1972).**

loading is not very pronounced, particularly when the ratio of the thickness to one half of the width of the loading,  $h/b$ , is small. This ratio ranges from 0.5 to 1.5 for the embankment analysed recently at AIT, Table 1, and does not account for the time difference as much as 1 to 2 orders of magnitude.

The diagram in Figure 6, given also by DAVIS & POULOS (1972), illustrates the effect of horizontal permeability in the 3-dimensional consolidation under a circular load. The ordinate shows the ratio of the time required for 50% consolidation for the isotropic case to the time required for the same degree of consolidation for an anisotropic soil with an anisotropic permeability ratio indicated on the abscissa, which is represented by  $c_h/c_v$ . Figure 6 (a) shows the case  $h/a = 1$  and the case of a semi-infinite mass for which the ratio  $h/a$  equals infinity with a permeable upper surface, while the corresponding relationships for an impervious footing are shown in Figure 6 (b). For the case of PTPB, for instance, if  $c_h/c_v = 100$ , there will be a ten-fold difference in the time required for 50% consolidation. In other words, the anisotropic clay having a horizontal permeability 100 times greater than the vertical permeability will consolidate 10 times faster than an isotropic clay.

CONSOLIDATION SETTLEMENTS AND TIME RATES

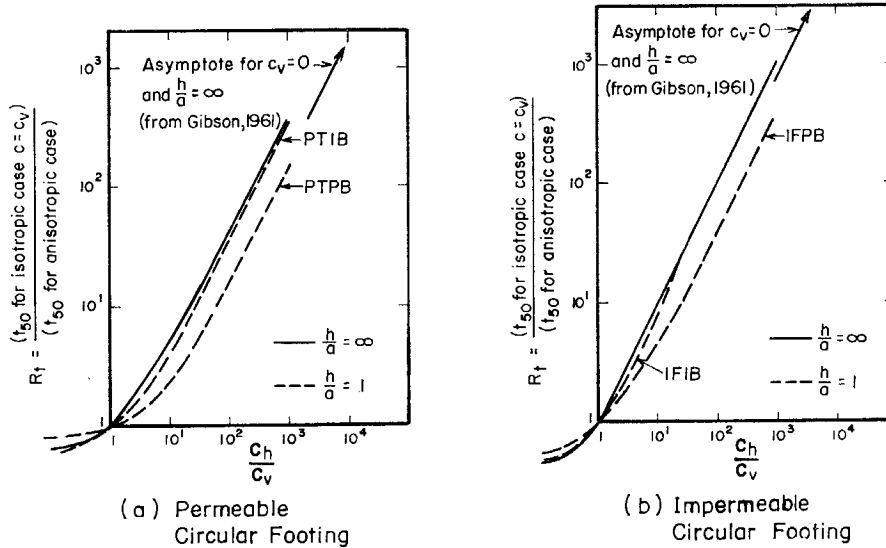


Fig. 6. Effect of anisotropic permeability on rate of settlement (Davis & Poulos, 1972).

This ratio of  $t_{50}$  values may be taken to hold approximately for earlier and later times. Furthermore, this leads to a simple method of modifying the basic isotropic results to allow for anisotropy, which according to Davis and Poulos, should be sufficiently accurate for calculating the average degree of consolidation and for shapes other than circles. In order to account for the time difference of several tens to a few hundred times, their theory dictates that the horizontal permeability should be a few hundred to several hundred times greater than the vertical permeability, i.e., the  $c_h/c_v$  ratio being something in this order. This may or may not be the case.

CONCLUDING REMARKS ON TIME-SETTLEMENT RELATIONSHIP

Experience tells us  $c_h$  is generally only a few times greater than  $c_v$  in many alluvial or marine clays on the basis of oedometer test results on small specimens. It is to be noted, however, that in accumulating such experience we have the habit of running oedometer tests on "good looking" portions of clay excluding intentionally the portions containing deficiencies such as cracks, fissures, silt seams, sand pockets, decayed vegetation, etc. It should be remembered that these defects may have a profound influence over the horizontal permeability of the clay mass. At the present time we do not have sufficient knowledge of the actual mass permeability of the whole stratum of thick soft clay and should be in search of factors resulting in serious discrepancy. Previous well-documented case studies should be reinvestigated, in

#### AKAGI

the light of recent theoretical studies. Investigation should be conducted on such problems as the effects of the soil layering, which should determine the drainage conditions of the whole stratum, and geological features, including presence of fissures, thin layers of sand and silt, etc. ROWE (1972), for example, reported that the *in situ* permeabilities were one or more orders of magnitude larger than measured in small laboratory samples for many of the clays typically encountered in England.

It is indeed important for us to establish whether this common discrepancy is due to an underestimate of the *in situ* coefficient of consolidation, or due to the lateral drainage effects, or due to the combined effect of both factors possibly including some others. In spite of the availability of such theoretical solutions and our increasing computational capabilities by means of computers, it appears that our profession has benefited little in the recent years. There is apparently a gap between the theory and the practice. Obviously the practice also needs considerable improvement, including more careful subsurface investigations, development of new *in situ* testing technique of mass permeability in the vertical and horizontal directions, and reliable field observations.

This gap may be closed only by alertness of practicing engineers and accumulation of high quality case histories. Every soils engineer should know that refinement of a theoretical analysis alone will never solve our problem, but such attempts coupled with systematic accumulation of local experiences would certainly improve our capability of predicting the time-settlement relationship.

#### VERTICAL DRAINS

We seem to have enough problems in natural deposits, but we also have to face the problems brought up by vertical drains which bring more complications to the complexity of the natural ground. Various types of vertical drains have been used for nearly as many years as the Terzaghi theory has. The purpose of installing vertical drains is two-fold, that is, firstly, to accelerate the consolidation process of soft clay by providing horizontal drainage, and secondly, to gain rapid strength increase to improve the stability of weak clay foundation. In the following brief comments will be made on sand drains which are still the most widely used type of vertical drains.

Very roughly, there are two categories of installation methods, namely, a displacement type which means either driving or pushing a closed-end

### *CONSOLIDATION SETTLEMENTS AND TIME RATES*

mandrel displacing soft clay both laterally and vertically. The other is a nondisplacement type which requires drilling a hole commonly by auger or water-jet, involving no brutal process of displacement, and hence is considered to have less disturbing effects on the soft clay. Nondisplacement type drains are however more expensive, because the installation is time consuming and requires disposal of the messy excavated material. On the other hand, the displacement type is neat, more efficient and therefore more economical. The recent trend seems to be that more designers prefer the more expensive nondisplacement type. They believe the displacement type has detrimental effects on the properties of soft clay to be stabilized, that is, the displacement type would remold the clay so badly it would reduce both the coefficient of consolidation and the shear strength to an undesirable degree.

This belief no doubt comes from the results of some case studies and published data, notably the paper by CASAGRANDE & POULOS (1969). After reviewing several documented sand drain installations, they concluded that "driven sand drains are harmful in many cases when used in soft sensitive clays, and disturbance due to driving of drains causes the shear strength and the horizontal permeability of soft sensitive soils to be reduced. Therefore, installation of driven drains may be self-defeating".

On the other hand, JOHNSON (1970) pointed out that the nondisplacement type is by no means nondisturbing and cast the following questions.

- (1) Can beneficial results be obtained if displacement type sand drains are used?
- (2) Do other installation methods result in a significantly improved performance?
- (3) Are other installation methods preferable even though more costly?

It appears that conflicting views still exist among researchers on the disturbance effects on the coefficient of consolidation and the shear strength of soft clays due to driving of a closed-end mandrel. It is at least the author's observation that none of the arguments were sufficiently quantitative when some designers started favoring the nondisplacement type sand drains approximately 10 years ago.

### *SETTLEMENT RECORDS IN THE SAND DRAIN AREA*

Since we fully realize the difficulty when dealing with the time-settlement relationship of natural soft clay strata, it appears almost formidable to predict the time-settlement relationship when vertical drains are installed. It would



require no complex theory, however, to answer the first question cast by Johnson if we have reliable field observations showing the time-settlement relations in the area where sand drains were installed and in the area where no sand drains were installed, provided that the loading and geological conditions are the same. A few such examples are given in what follows.

By far the most famous case is the one in Skå Edeby in Sweden (HANSBO, 1960; HOLTZ & BROMS, 1972). The driven sand drains, 18 cm in diameter and about 12 m in length, were arranged in triangular patterns at three different spacings in Area I, as indicated in Figure 7. They were effective and certainly brought beneficial results, as compared with the settlement record taken in the non sand-drained area, Area IV. Another instance is the somewhat controversial record available in the Bangkok area, Thailand. Sand drains, 20 cm in diameter, were installed by the displacement method on 2 m spacings to various depths (EIDE & ANDRESEN, 1977). Although the sand drains appear to have accelerated the consolidation rate in the early stage as may be seen in Figure 8, no significant benefit is apparent for the post-construction settlement, because these curves are practically all parallel.

This case reminds us of our empirical rule that sand drains have practically no effect on secondary consolidation. Although it is unlikely to be the case here, sand drains are not effective for the clay having a high value of critical

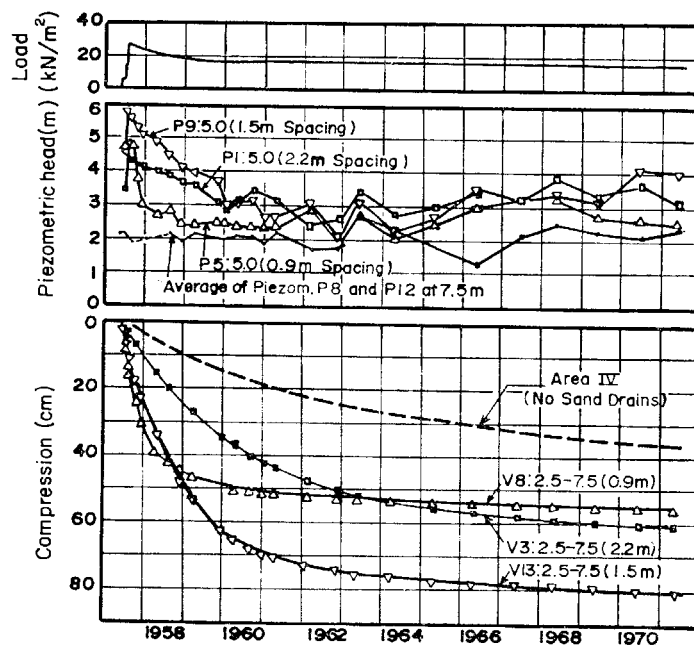


Fig. 7. Time versus observed settlement and piezometric head in Area I, Skå Edeby (Holtz & Broms, 1972).

CONSOLIDATION SETTLEMENTS AND TIME RATES

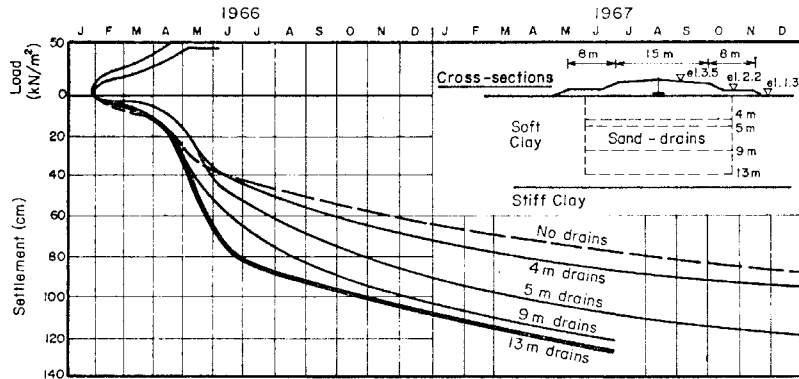


Fig. 8. Time versus observed settlement, Bangkok (Eide & Andresen, 1977).

pressure or an overconsolidated clay. Incidentally the third rule is that sand drains do not accelerate the rate of settlement in peat and highly organic clay.

Another example is the Japanese experience in the old days when these rules were not well recognized. Closely spaced sand drains were driven into thick layers of peat and organic clay, and test embankments were constructed in the sand drain area and in the adjacent untreated area in Aiko, Japan (JHPC, 1966). The sand drains were 40 cm in diameter and were driven to a depth of 10 m on 1.2 m spacings in triangular patterns. The time-settlement curves obtained in the sand drained area and in the adjacent untreated area are nearly identical showing dramatically the ineffectiveness of sand drains, Figure 9.

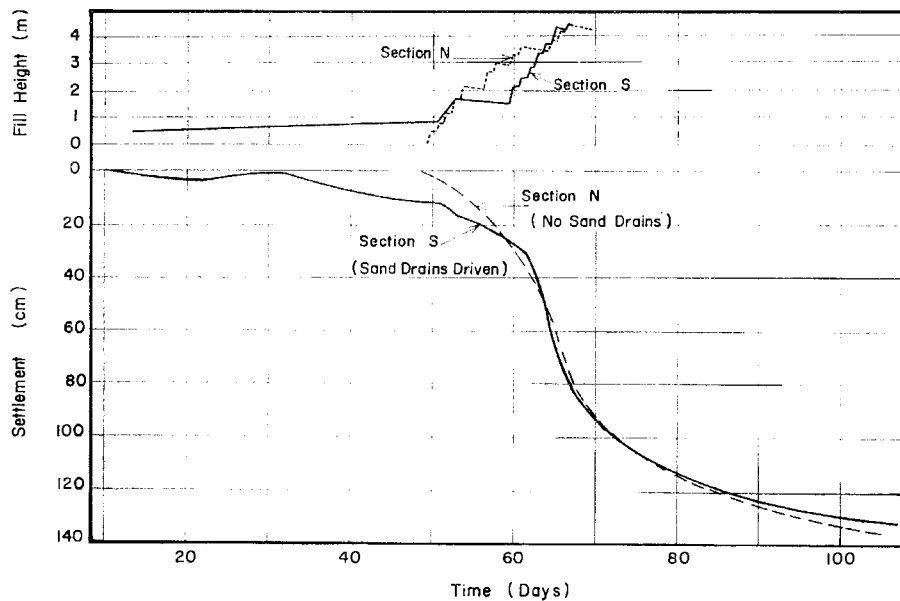


Fig. 9. Time versus observed settlement, Aiko test embankments (Akagi, 1977b).

SHEAR STRENGTH OF THE SAND DRAINED FOUNDATION

The vertical settlement, however, should not be mistaken for being synonymous with the consolidation taking place in the compressible layer (AKAGI, 1977a). Figure 10a shows the changes in water content and undrained shear strength, about 1.5 months (S1) and 5 months (S2) after the installation of sand drains in comparison with the changes noted in the adjacent untreated area (N1 and N2) of the Aiko test embankments. The decrease in water content in the sand drain area was substantially greater than that in the non-sand drain area, in spite of the fact that the vertical settlements observed were about the same when these water contents were determined. The undrained shear strength appears to have increased roughly to the same degree both in the sand drained area and the non-sand drain area. When one considers a number of sand columns driven into the soft peat having a marginal initial strength of 15 kN/m<sup>2</sup> (S0 and N0), it may be stated that the installation of driven sand drains has served the purpose of improving the stability of the embankment, which is often the important purpose of installing sand drains.

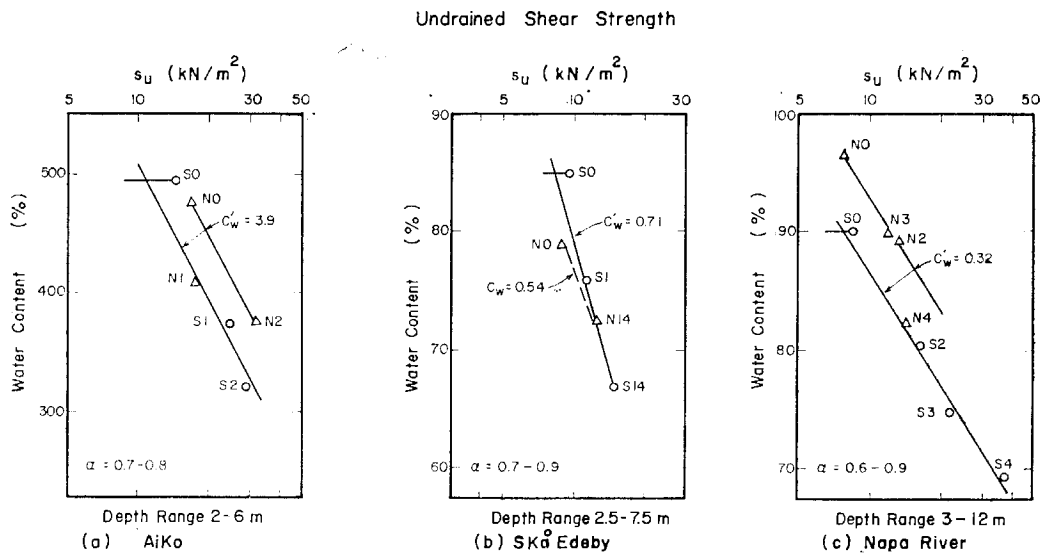


Fig. 10. Relationships between water content and undrained shear strength (Akagi, 1977b).

In a more successful case such as the Skå Edeby case, Figure 10b, the undrained shear strength 1 year after the installation (S1) slightly exceeded the original strength (S0) and the strength 14 years after (S14) was greater in the sand drain area than that obtained in the non-sand drained area (N14), not only because the measured settlement was greater but also more consolidation took place in the soft clay due to the installation of displacement

#### *CONSOLIDATION SETTLEMENTS AND TIME RATES*

type sand drains. Based on the results of a series of check borings made frequently after the sand drain installation, Figure 10c indicates that a similar relation is evident in the soft clay in the Napa River area California, stabilized by mandrel-driven sand drains (WEBER, 1966).

It is interesting to note that the installation of displacement type sand drains alone causes the consolidation of soft clay because of large stresses induced by sand column installation (AKAGI, 1979b). When a closed-end mandrel is driven, it displaces a volume of soft clay equal to that of the mandrel. Since it behaves incompressibly, the driving of the mandrel generates excess pore pressures which may even exceed the overburden pressure as have been observed and reported by many investigators in connection with the driving of displacement type piles and sand drains (e.g., Brenner, et al 1979). As these excess pore pressures dissipate very rapidly, principally in radial directions, the soft clay consolidates and regains its strength, eventually exceeding its undrained strength considerably under a newly placed embankment.

Regarding the effect of driving on the permeability, we still know very little. MASSARSCH (1976, 1978) conducted a study on displacement and hydraulic fracturing of soft clays due to pile driving, and summarizing the Scandinavian experiences, concluded that "during driving of sand drains, cracks open and can be filled with sand. In this way the effective drainage area of driven sand drains, and thus their efficiency, is increased."

The indications from the recent studies are that the disturbance effects of displacement type sand drains may not be as bad as has been accused. It is strongly felt that it is high time for us to give another critical look at the performance of the displacement type vertical drains.

#### *CONCLUDING REMARKS ON VERTICAL DRAINS*

A large number of displacement type sand drains are still being installed, because of their simplicity and economy, without fully realizing their advantages and disadvantages. In addition, an increasing number of pre-fabricated vertical drains made of various artificial materials are also being installed. They are indeed displacement type vertical drains, because they are either driven or pushed, employing a mandrel having a much larger cross-sectional area. It is also true that a number of nondisplacement type sand drains are being installed in spite of their inferior efficiency and higher cost ascertaining fully that the merits anticipated justify the extra cost.

## AKAGI

We have no simple answer for the three questions cast earlier by Johnson, other than realizing that we need more research oriented case studies in which logical comparisons are made between the performance of displacement type and nondisplacement type vertical drains together with the behavior of the untreated area.

In actuality, the reliable observational data are surprisingly few where rational comparison is possible. When we consider the inaccuracy of 1 to 2 orders of magnitude involved in our prediction of the time-settlement relationship, there is even a suspicion that we may be installing vertical drains where they are not required at all. Even where such is needed with the sheer necessity ascertained, we may still be wasting money by employing more expensive types of drains. It is one of our urgent tasks to respond to the questions set forth by Johnson some 10 years ago in addition to the immediate demand to improve our capability of predicting the time-settlement relationship.

## ACKNOWLEDGEMENTS

In preparing this report, the author is indebted considerably to the comprehensive state-of-the-art report presented by LADD, et al (1977) during the Ninth International Conference on Soil Mechanics and Foundation Engineering in Tokyo. Readers interested in the developments prior to the Tokyo Conference are urged to read it for the further details. Grateful appreciation is expressed to Dr. K.V. Campbell for his tireless scrutiny and editing of the manuscript, far beyond his normal duty as Editor of this journal.

## REFERENCES

- ABOSHI, H. (1973), An Experimental Investigation on the Similitude on the Consolidation of a Soft Clay Including the Secondary Creep Settlement, *Proceedings of the Eighth International Conference on Soil Mechanics and Foundation Engineering*, Moscow, Vol. 4.3, p. 88.
- ADACHI, K. and TODO, H. (1979), A Case Study on Settlement of Soft Clay in Penang, *Proceedings of the Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Singapore, Vol. 1, pp. 117-120.
- AKAGI, T. (1977a), Effect of Mandrel-Driven Sand Drains on Strength, *Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering*, Tokyo, Vol. 1, pp. 3-6.
- AKAGI, T. (1977b), Effect of Displacement Type Sand Drains on Strength and Compressibility of Soft Clays, *Publication from the Department of Civil Engineering*, Toyo University, Saitama, Japan, 403 p.
- AKAGI, T. (1979a), General Report: Problems in Soft Clay/Soil Technology and Stabilization, *Proceedings of the Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Singapore, Vol. 2, pp. 133-142.

### CONSOLIDATION SETTLEMENTS AND TIME RATES

- AKAGI, T. (1979 b), Consolidation Caused by Mandrel-Driven Sand Drains, *Proceedings of the Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Singapore, Vol. 1, pp. 125-128.
- BICT, M. A. (1941), General Theory of Three-Dimensional Consolidation, *Journal of Applied Physics*, Vol. 12, pp. 155-164.
- BJERRUM, L. (1972), Embankments on Soft Ground, *Proceedings of the Specialty Conference on Performance of Earth and Earth-Supported Structures*, American Society of Civil Engineers, Vol. 2, pp. 1-54.
- BRENNER, R. P., BALASUBRAMANIAM, A. S., CHOTIVITTAYATHANIN, R. and PANANOOKOOLN, P. (1979), Pore Pressures from Pile Driving in Bangkok Clay, *Proceedings of the Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Singapore, Vol. 1, pp. 133-136.
- CASAGRANDE, L. and POULOS, S. (1969), On the Effectiveness of Sand Drains, *Canadian Geotechnical Journal*, Vol. 6, No. 3, pp. 287-326.
- DAVIS, E. H. and POULCS, H. G. (1972), Rate of Settlement Under Two-and Three-Dimensional Conditions, *Geotéchnique*, Vol. 22, No. 1, pp. 95-114.
- EIDE, O. and ANDRESEN, A. (1977), Exploration, Sampling and In-Situ Testing of Soft Clay, *Preprint, International Symposium on Soft Clay*, Bangkok, Report 3, 74 p.
- HANSBO, S. (1960), Consolidation of Clay, with Special Reference to Influence of Vertical Sand Drains, *Swedish Geotechnical Institute Proceedings*, No. 18, 159 p.
- HOLTZ, R. D. and BROMS, B. (1972), Long-Term Loading Tests at Skå-Edeby, Sweden, *Proceedings of the Specialty Conference on Performance of Earth and Earth-Supported Structures*, American Society of Civil Engineers, Vol. 1, Part 1, pp. 435-464.
- JAPAN HIGHWAY PUBLIC CORPORATION (1964), *Report on Atsugi Test Embankments, Tomei Highway*, Part I, 268 p. (in Japanese).
- JAPAN HIGHWAY PUBLIC CORPORATION (1966), *Report on Aiko Test Embankments, Tomei Highway*, Part I, 248 p. (in Japanese).
- JOHNSON, S. J. (1970), Foundation Precompression with Vertical Sand Drains, *Journal of the Soil Mechanics and Foundation Division*, American Society of Civil Engineers, Vol. 96, SM-1, pp. 145-175.
- LADD, C. C., FOOTT, R., ISHIHARA, K., SCHLOSSER, F. and POULOS, H. G. (1977), Stress-Deformation and Strength Characteristics, State-of-the-Art Report, *Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering*, Tokyo, Vol. 2, pp. 421-494.
- LAMBE, T. W. (1967), The Stress Path Method, *Journal of the Soil Mechanics and Foundation Division*, American Society of Civil Engineers, Vol. 93, SM-6, pp. 309-331.
- LAMBE, T. W. (1979), Stress Path Method: Second Edition, *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 105, GT-6, pp. 727-738.
- MASSARSCH, K. R. (1976), Soil Movements Caused by Pile Driving in Clay, *Rapport 51*, Royal Swedish Academy of Engineering Sciences, 261 p.
- MASSARSCH, K. R. (1978), New Aspects of Soil Fracturing in Clay, *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 104, GT-8, pp. 1109-1123.
- OLSON, R. E. and LADD, C. C. (1979), One-Dimensional Consolidation Problems, *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 105, GT-1, pp. 11-30.
- ROWE, P. W. (1972), The Relevance of Soil Fabric to Site Investigation Practice, Twelfth Rankine Lecture, *Geotéchnique*, Vol. 22 No. 2, pp. 195-300.
- SAMARASINGHE, M. F. (1979), Review of Time-Settlement Records of Test Embankments on Soft Clays, *M. Eng. Thesis*, Asian Institute of Technology Bangkok, 285 p.

AKAGI

- SIMONS, N. E. (1974), Normally Consolidated and Lightly Over-Consolidated Cohesive Materials, Review Paper, *Settlement of Structures*, British Geotechnical Society, Pentech Press, London, pp. 500-530.
- SIMONS, N. E. and SOM, N. N. (1969), The Influence of Lateral Stresses on the Stress Deformation Characteristics of London Clay, *Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering*, Mexico, Vol. 1, pp. 369-377.
- SKEMPTON, A. W. and BJERRUM, L. (1957), A Contribution to the Settlement Analysis of Foundations on Clay, *Geotéchnique*, Vol. 7, pp. 168-178.
- TAN, S. B. and PHANG, C. P. (1979), Performance of Sand Drains (at Taxiway Changi Airport), *Proceedings of the Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Singapore, Vol. 1, pp. 183-186.
- TAYLOR, D. W. (1942), Research on Consolidation of Clays, *Publication from the Department of Civil and Sanitary Engineering*, Serial 82, Massachusetts Institute of Technology, 147 p.
- WEBER, W. G. (1966), Experimental Sand Drain Fill at Napa River, *Highway Research Record*, No. 133, pp. 23-44.

## APPLICATIONS OF STEEL PIPE PILES IN JAPAN

MASATOSHI SAWAGUCHI\*

### SYNOPSIS

This paper describes and illustrates the utilization of steel pipe piles in foundations built on soft soils in Japan, including the techniques of sheet pile construction and prepacked concrete foundations. To counteract the effects of negative skin friction on pipe piles, the current practice in Japan is to apply a special viscoelastic asphalt coating. The paper also discusses methods of corrosion control and their effectiveness, noise pollution control, and outlines the use of an automatic welding machine.

### INTRODUCTION

In Japan there are a great number of cases where pile foundations are adopted for support of structures to be built on soft soils. The areas occupied by soft soils include deltas developed near river mouths and sea bed soils in their vicinity, wet lands associated with river embankments, and marshes or lake bottoms where marl has been deposited. However, several problems result when pile foundations are built in such soft soils. These include the disturbance of soft clays due to pile driving and the succeeding reduction of their shear strength, negative skin friction developed on the surface of piles driven into an underconsolidated clay because of its consolidation phenomenon, bending of installed piles attributed to the movement of surrounding soils under action of external forces, and so forth. A great deal of research for solving such difficult problems has been performed, but no perfect solutions have yet been achieved.

As is well known, the procedure of pile installation can be classified into two general techniques; driving of prefabricated piles and setting of cast-in-place piles. Wood, precast concrete, or steel piles are most commonly used for the first procedure. Among these, steel piles have been favorably adopted for foundations in soft clays in Japan. In particular, steel pipe piles have been utilized to a remarkably large extent. The major reasons are as follows.

- (1) Steel pipe piles can be driven firmly into a stiff bearing stratum to develop a comparatively large bearing capacity.

†Presented at the Seminar on Geotechnical Engineering in Practice, Bangkok, July 20th and 21st, 1979 under the title: Pile Foundations in Soft Clay.

\*Professor, Institute of Structural Engineering, University of Tsukuba, 1-1-1, Tenohdai, Sakura-mura, Niihari-gun, Ibaragi-ken, Japan.



*SAWAGUCHI*

- (2) In clayey soils steel pipe piles can be closely spaced because the soils are less disturbed owing to the smaller amount of soil displacement.
- (3) With their great resistance to bending moment, steel pipe piles can withstand large horizontal loads.

In Japan both the dimensions and the qualities of steel pipe piles have been standardized and so they are quite appropriate for foundation design and handling during construction in soft clayey ground.

EXAMPLES OF STEEL PIPE PILE APPLICATIONS

Figure 1 shows steel pipe pile foundations to support apartment buildings under construction at a site underlain by alluvial clay strata; i.e. at the Kōtō delta developed in the eastern part of Tokyo. The thickness of the clay layer in this area is 40 to 50 m and considerable subsidence, due to ground-water pumping, occurred until several years ago. At that time the maximum rate of settlement amounted to more than ten centimeter per year and a reinforced concrete building had collapsed due to negative skin friction. Recently, pumping of ground water has been significantly restrained and the subsidence has ceased.



Fig. 1. Pile foundation to support apartment buildings.

Figure 2 shows a steel pipe pile foundation supporting a liquified natural gas reservoir tank. By extending the reinforcing bars welded to the pile tops, it is not difficult to connect the piles with a base slab of the tank. The capacity of each tank is 800,000 kiloliters and there are 18 tanks.

## STEEL PIPE PILES

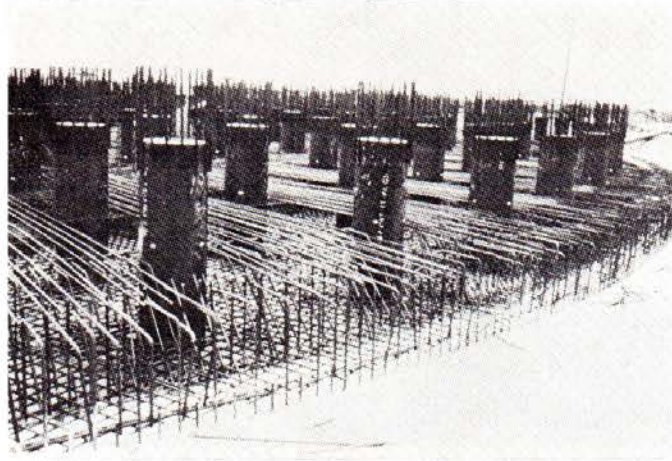


Fig. 2. Pile foundation to support a liquified natural gas reservoir tank.

Figure 3 shows steel pipe piles employed for a sea-berth. Such a sea-berth requires long, large diameter piles because it is quite a distance from the coast and because a great magnitude of bending moment occurs in the piles under the action of external horizontal forces.

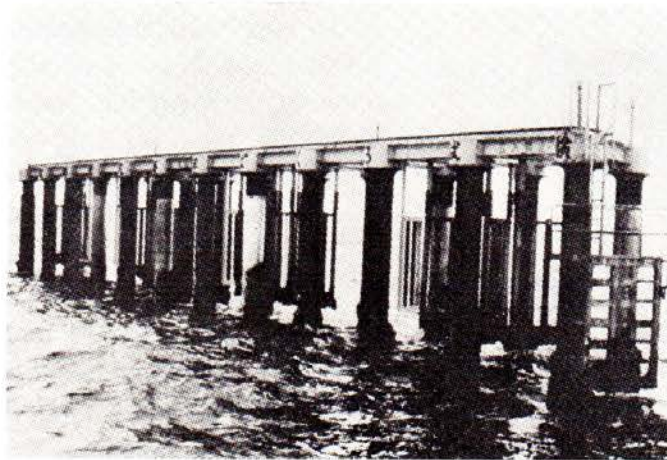


Fig. 3. Steel pipe piles employed for a sea-berth.

Figure 4 shows Rokko Bridge which connects the mainland with reclaimed Rokko Island, located in the eastern part of Kobe Port. Pipe piles arranged in sheets (termed “steel sheet pipe piles”, described below) were used as components of the foundations, which extend 8 m from the sea bottom. The geological sequence is a 20 m layer of alluvial clay underlain by a 15 m of alluvial gravelly sand and 20 m of diluvial stiff clay. Steel piles were driven as deeply as 33 m into the gravelly sand layer.





Fig. 4. Rokko Bridge.

Figure 5 shows piles driving of the sea-berth at Mindanao by the Philippine Sinster Corporation; another successful application of steel pipe piles. This sea-berth was built at a depth of 23 m for loading and unloading operations of 6,000 dead weight ton barges and bulk ore carriers of up to 250,000 dead weight tons. Large diameter (1,016 to 1,212 mm) steel pipe piles with 14 and

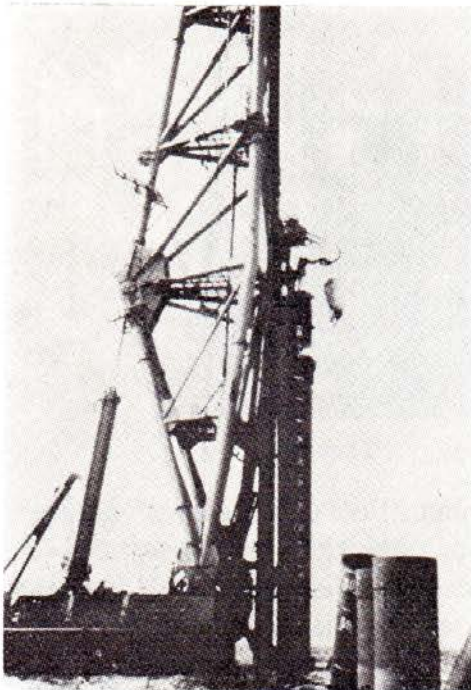


Fig. 5. Driving steel pipe piles for a sea-berth.

16 mm wall thicknesses were used. As many as 300 pipe piles (8,300 tons) were used for this construction. The raking piles for this sea-berth were driven in by a large pile driving barge equipped with a steam hammer MRB No. 1500.

The tidal and splash zone of the piles was coated with one primary coat of zinc-rich epoxide resin paint and two or three finishing coats of tar-epoxide resin paint. The piles were further protected with a galvanic anode system applied to the immersed portion or the portion under water. The pile tops were rigidly covered with concrete pile caps and the pile caps were connected with concrete beams and steel shapes. Figure 6 shows the completed sea-berth and a 250,000 ton bulk ore carrier moored at the side.

## STEEL PIPE PILES

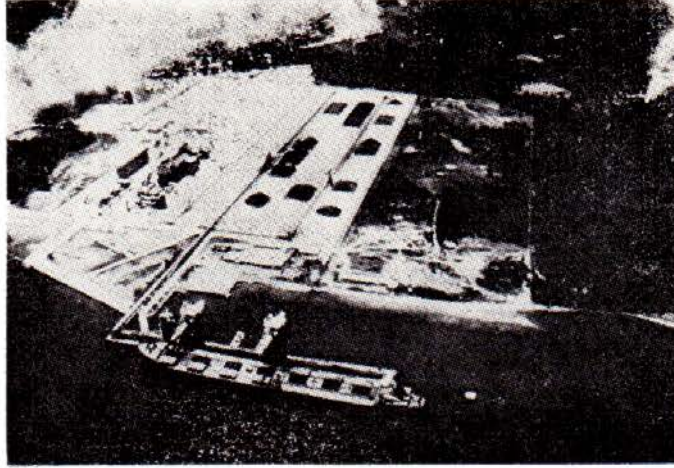


Fig. 6. General view of the sea-berth.

## SHEET PILE FOUNDATION

A sheet pile foundation is defined as a structural form in which steel pipe piles are set up adjacent to one another in round, oval, rectangular and other enclosed shapes. The rows of sheets of piles are made more rigid by reinforcing the tops and joints, if necessary, to acquire a higher horizontal resistance and vertical bearing capacity. Since this method is executed by driving pipe piles one by one with a pile driver, there is very little danger as there is in the case of installing caisson foundations. Even in soft soils or deep bearing strata safe and sure construction can be undertaken. Speedy construction and labor saving can also be achieved in the same way as the pile foundations.

Figures 7 and 8 show the successive construction process of a double sheet of piles into an enclosure where industrial and domestic disposals were to be dumped. This type of structure may be commonly used for a cutoff wall to make the inside space available for dry work. In Japan however, it has at times been adopted for a sheet pile enclosure such as is shown in these figures, where the inside sea level is lowered in advance of dumping the waste so that dirty or detrimental liquid staying inside after rainfalls will not overflow across the top of sheet piles. It goes without saying that the structure should be safe enough against lateral pressure due to the waste or the sea water, and also impermeable enough to prevent inside liquid percolating through the sheet pile walls. For these reasons sheet piles with a relatively large rigidity, e.g. steel sheet pipe piles, are often adopted when the waste is to be thrown into the sea where soft soils are deposited, such as in Tokyo Bay.



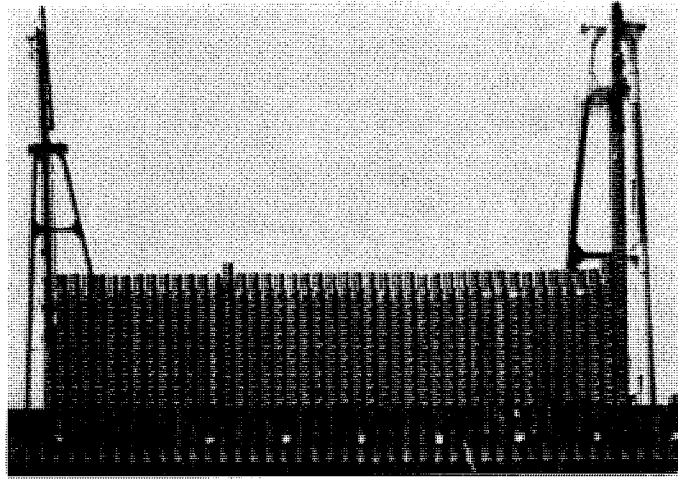


Fig. 7. Driving sheet piles in panels.

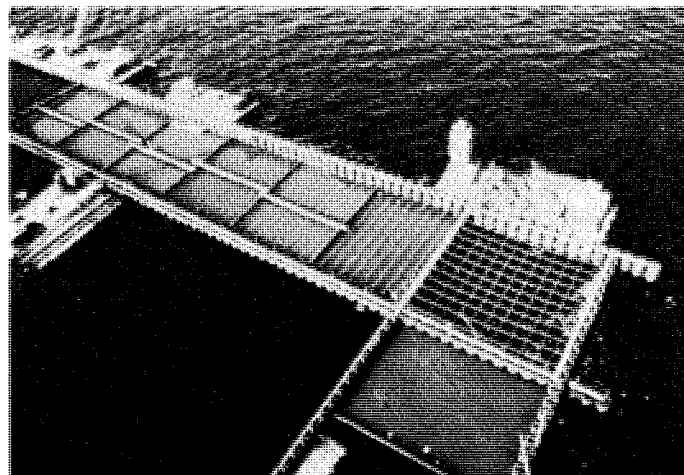


Fig. 8. Appearance of double sheet piles.

The geological conditions at the site shown in Figure 7 are very unfavorable for such an enclosure construction. A 26 m thick layer of very soft marine mud overlies a 15 m thick layer of medium or soft clay and silt. A dense sand and gravel layer, with more than 50 standard penetration blow counts, has been encountered as deeply as 41 m below the sea bottom.

Figure 7 shows one of driving techniques of sheet pipe piles, called “driving-in-panels”, where a given number of pipe piles are driven in a group to equal penetration up to a half-way stage. If each pile is driven up to its maximum penetration one by one, the sheet may be in danger of inclination along its alignment. The pile hammer on the right side of the picture is driving a final pile from an elevation one stage higher. The pile hammer on the left side is driving the first pile in the sheet advanced to a lower stage elevation.

### STEEL PIPE PILES

The main dimensions of double sheet piles in Figure 8 are a width or distance from the front wall to the rear wall of 20 m, a steel pipe pile diameter of 1,320 mm, a wall thickness of 12 to 16 mm and a total length of 48 to 46 m. The portion driven into the sea bottom is 42 m long. Sheets of steel pipe piles have been successfully applied to this project. Even a sand compaction technique, at present considered to be most efficient for stabilizing soft soils, has proven to be inadequate to achieve a safety factor of 1.5 against sliding rupture. In addition, water pollution has been strictly restrained by law for at least 50 years.

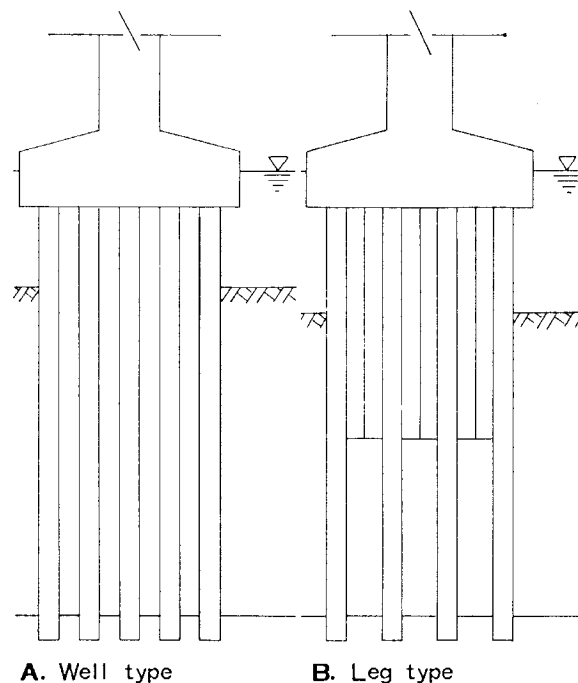


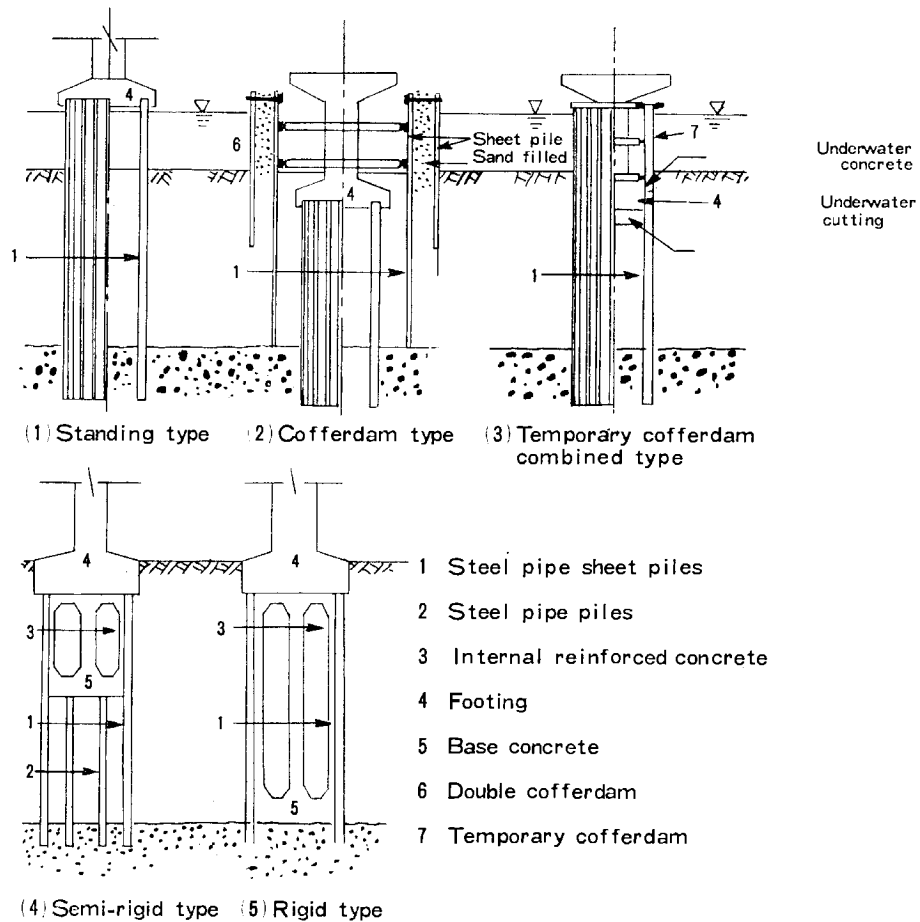
Fig. 9. Types of sheet pile foundations.

Figure 9 illustrates a classification of sheet pile foundations in terms of structural form. Type A is called a well type. The well type is the most basic type of the sheet pile foundation. All the piles are driven into the bearing stratum to the same depth, and the top is rigid structure by means of the footing. Type B, the leg type, is used when there is more vertical bearing capacity in reserve than horizontal bearing capacity, the bearing stratum being at a comparatively deep place. About a half of the pipe piles are driven into the bearing stratum and the remaining piles are stopped at a comparatively good,

*SAWAGUCHI*

intermediate bearing stratum. As a result, the quantity of steel materials can be small with the added merit of easy pile driving. The leg type is more economical than the well type.

Figure 10 shows 5 types of construction methods. These are described below.



**Fig. 10. Types of construction methods.**

*Standing Type*

A standing type has the pipe pile well structure extending above the water surface. Though this method is advantageous in terms of construction period and cost, its sites are limited to rivers that do not set limits to the section of flowing water or harbors that do not restrict the width of shipping routes.

## STEEL PIPE PILES

### *Cofferdam Type*

A cofferdam type is suitable when the foundation has to be built underwater due to restrictions on the cross sections of running water or the widths of shipping routes. Steel sheet piles are used to erect the doublewall cofferdam, whereby required work space is secured underwater, and a pipe pile wall is built in the space. However, in the case of performing cofferdam work by sheet piles at areas whose subsoil is composed of widely distributed thick, soft layers, there are inherent problems of hazardous work, occupying a large area exclusively during work, and the necessity of completely removing the sheet piles after construction since they are temporary structures. Hence, in the event of constructing a bridge foundation during a limited dry period, some inconvenience in the construction period and increased cost often results.

### *Temporary Cofferdam Combined Type*

In this type the pipe pile will itself extend above the water surface, its joints being filled with cutoff materials to serve as temporary cofferdam. The inside of the well structure is dried and after a footing and a pier are erected there, the temporary pipe pile cofferdam above the top end of the footing is cut and removed. Since the body of the well structure and the temporary cofferdam can be worked on simultaneously, the period of construction can be shortened. It is possible to complete construction of the foundation within a limited period, such as within a dry season. The other salient features of this method are the minimum obstruction to river passages or shipping routes during work and execution of this method adjacent to existing structures.

### *Semi-Rigid Type*

A semi-rigid type is so structured that the inside of the well is partially excavated and cast with concrete. This method is most appropriate where:

- (1) the vertical and horizontal loads are large. Nevertheless, since the soils above the bearing stratum are soft, it is not quite safe to let the soil bear a part of the load.
- (2) The volume inside the wall is so large that it is uneconomical to replace weak soil with good soil, or
- (3) the inside of the wall is to be made hollow in order to utilize it for underground rooms or tanks.



### SAWAGUCHI

In addition to the head of the pipe well, which is made rigid, the intermediate section of the pipe pile well is also made rigid by means of connecting members and base concrete and the space between is further reinforced by internal walls and bulkheads. When the soil remaining below the bottom base concrete is so soft that a problem may result under an application of stress, group piles, as shown in Figure 10, may be used to transmit the load directly to the bearing stratum.

#### *Rigid Type*

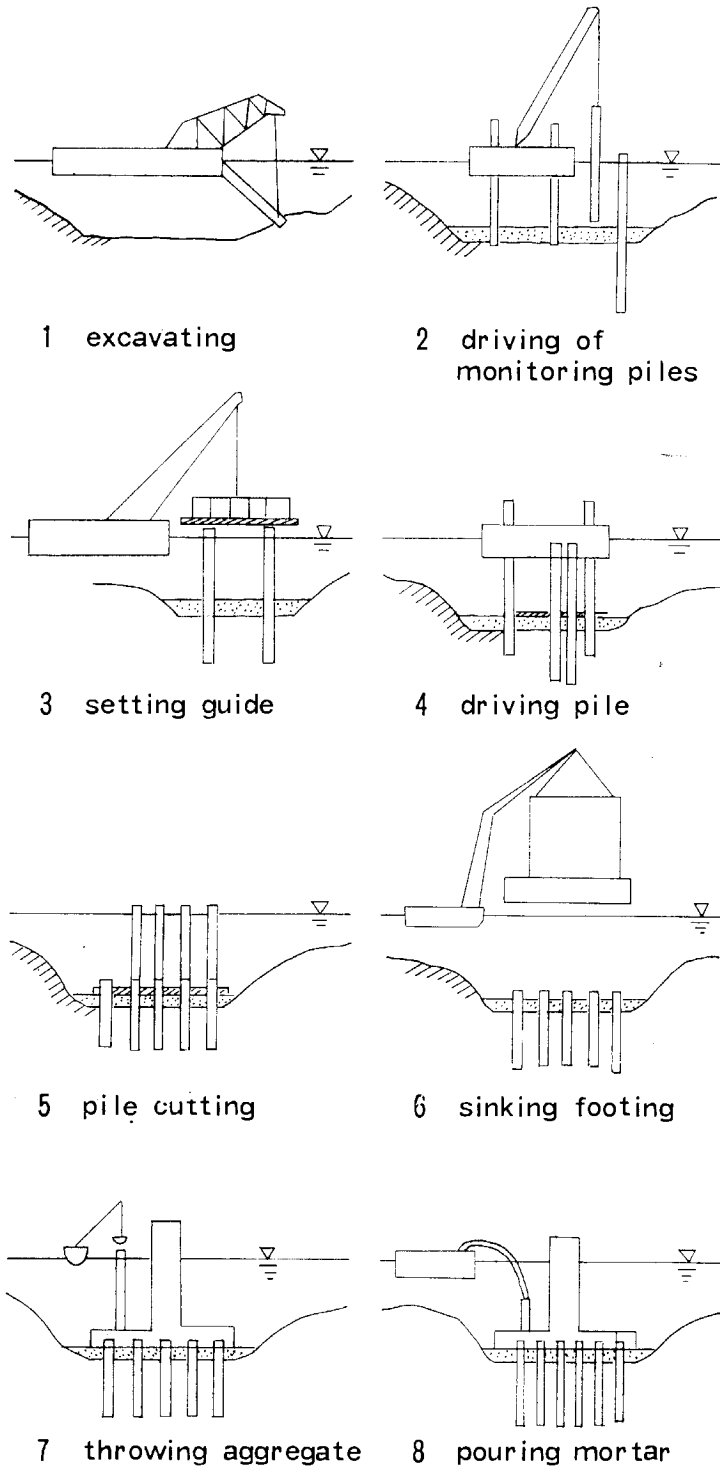
A rigid type is employed when it is considered necessary to spread and transmit both vertical and horizontal loads over a broad area above the bearing stratum because the vertical and horizontal loads are extremely large or because the soils above the bearing stratum are much softer than those in the semi-rigid case. After the inside of the well is nearly all excavated a concrete base is placed directly above the bearing stratum. The base is reinforced by the connecting members and base concrete for the head and bottom, by the connecting members for internal walls and bulkheads for the intermediate section, and if necessary, by filling with good earth and sand.

### PREPACKED CONCRETE FOOTING

This type of foundation is principally employed for bridge supports. The process of construction is as follows (Fig. 11). First a dredging or excavation is performed to make the river bed quite even at a certain elevation. After its completion a ballast with about 1.0 m thickness is placed, followed by the installation of four monitoring steel pipe piles. Then, a temporary guide frame for pile driving is sunk down onto the surface of the ballast along the monitoring piles.

After sinking the guide frame, steel pipe piles are driven, with careful attention to erection accuracy. After completion of the pile driving the temporary guide frame is removed leaving the piles in place. The water left inside the piles is pumped out and replaced with cast-in-place concrete up to the required elevation. The remainder of the pipe piles are then cut off by divers. After a completion of such prerequisites, a bridge pier connected in a body with a steel skeleton footing, which has been fabricated at the factory in advance and carried to the site by a floating crane, is sunk down to the proper location. Finally the aggregates for prepacked concrete are thrown into the skeleton and then prepacked mortar, supplied for example from a floating concrete plant, is poured.

*STEEL PIPE PILES*



**Fig. 11. Prepacked concrete footing construction technique.**

### SAWAGUCHI

The advantages of this technique are that

- (1) the construction can safely proceed all year regardless of weather as temporary works for its operation are not needed,
- (2) the construction period can be abbreviated, and
- (3) the cost can be reduced owing to its labor-saving practice.

The most important concern is to avoid a weak bond between the surface of piles and the prepacked concrete. This may result if soft muds are deposited and cover the surface of piles. To avoid such objectionable matter the method of washing away their muds with the use of water jetting has been adopted.

### NEGATIVE SKIN FRICTION AND THE SLIP LAYER (SL) METHOD

Piles installed in underconsolidated clay may be subject to a dragdown force, termed negative skin friction, during settlement of the surrounding soil so that the superstructures supported by them will sometimes suffer from functional damage due to their insufficient bearing capacity or excessive settlement.

The functional damage that a fixed jib crane, supported by a pile foundation, suffered due to negative skin friction is described as an example. The geological conditions of the site were as follows. A 12.20 m layer of fine sand was located just below the ground surface followed by a 6.6 m layer of silty sand or silty gravel, and 12.5 m of silt underlain by a gravelly mudstone. The average confined compressive strength of the silt was 9.0 t/m<sup>2</sup>. The base slab of the crane was supported by four H-shaped steel piles with dimensions of 304 × 301 × 11 × 17 mm and a length of 37 m. The piles had been driven immediately after raising a sand fill on the original ground as high as 4.60 m. After 4.5 years lapse a gap of 25 cm occurred between the bottom of the base slab and the ground surface. The relative settlement of four pile tops to the ground surface became 0.1 cm, 7.1 cm, 0 cm, and 8.3 cm, respectively, resulting in a maximum inclination of about one in fifty. Consequently, the operation of turning the arm during lifting of a heavy object became hazardous. Another pile foundation was made and the crane machine was moved. The original piles of the damaged structure were pulled out for inspection. Several crooked sections of the pile shafts were discovered in positions except for welded joints. Such bends might have been caused partly by faulty driving, but the action of negative skin friction is thought to be principally responsible.

### STEEL PIPE PILES

To prevent a pile from experiencing negative skin friction the asphalt slip layer (SL) method, developed in Holland, has been put into practice in Japan over the last several years. An outline of the present situation of application of the SL method in Japan follows.

A special asphalt, whose physical properties depend on the shearing velocity, is coated on the surface of a pile that will possibly be subjected to negative skin friction. When an instantaneous load acts on a pile, especially at the time of pile driving, the velocity of shearing developed on the pile surface exhibits an elastic behaviour. In this case, a great shear resistance attributable to the elastic property enables the pile to be driven without any slippage of the slip layer. On the other hand, where a pile is subject to a slow ground movement such as land subsidence, the velocity of shearing developed on the pile surface is very low and thus asphalt applied on the pile surface exhibits a viscous behaviour. In this case, slippage occurring in the slip layer due to the subsidence serves to prevent shearing force from being transmitted to the pile, thus permitting negative skin friction to be reduced.

To determine the effect of reduction in negative skin friction, slip layer coated piles (SL pile) and plain piles were driven into ground experiencing substantial land subsidence in various parts of Japan. The axial force distribution over the piles subject to negative skin friction was then observed over a long time period. Some of the experimental results are shown in Figure 12. As is apparent, the axial force on SL piles is much smaller than that on plain piles (1/7 or 1/10) and thus SL piles exhibit remarkable effects of reduction in negative skin friction. This means little negative skin friction remained on SL piles.

### CORROSION CONTROL

Several methods of investigation have been proposed for measuring the corrosiveness for steel pipe piles. Basic standards for corrosiveness have also been established. However, these methods for investigation and measurement are able to judge only the tendency to corrosiveness. Also, these estimated corrosion rates differ from the actual rates. Accordingly, in Japan, the best method of estimating corrosion is considered to be the one of referring to data from similar environmental conditions.

Generally, the corrosion environment of steel pipe piles is roughly classified into corrosion in soil, sea water, or air. Corrosion in air, mentioned above, can easily be controlled by painting, etc. Maintenance is also easy, as mentioned above. Usually, as the tops of steel piles are embedded in concrete

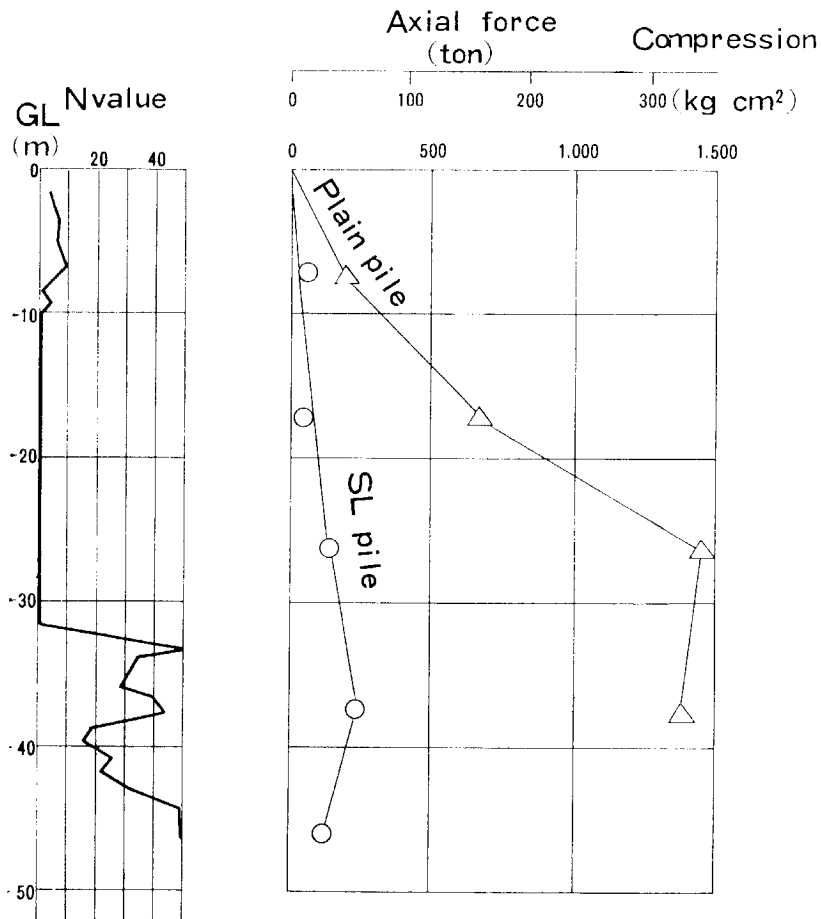


Fig. 12. Comparison of axial forces on SL pile and plain pile. Slip layer thickness = 10 mm, pile diameter = 711.2 mm and wall thickness = 11.0 mm. GL refers to ground level, N to number of blow counts experienced.

footing, corrosion in the air is already controlled. The corrosion rate in sea water differs accordingly to the position and to the state of the sea. In many cases structures in sea are treated with special corrosion control techniques from the upper positions to the sea bottom.

Normal steel piles are driven into undisturbed soil which differs from the case of piles driven into disturbed soil. Generally therefore, the corrosivity is far smaller than anticipated; on the order of 0.01 to 0.02 mm/year. If the corrosion allowance is 2 mm, and the rate is 0.02 mm/year, the service life of steel pile is 100 years before the allowance is consumed. This is sufficient for the service life of upper portions of structures. Also, as the inside surface of steel pipe piles is sealed from the outside, the surface is free

### STEEL PIPE PILES

from corrosion. Therefore, the corrosion allowance generally refers to the outside surface only. Accordingly, normal steel piles do not need special corrosion control. If the corrosion allowance is 2 mm of the pipe wall thickness, it is sufficient to cope with corrosion. Therefore in Japan, the corrosion of steel piles is not worried about except in special environments, such as in sea water, etc.

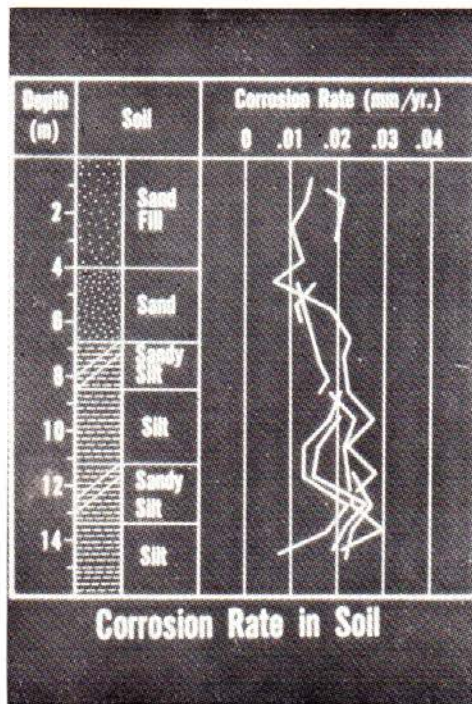


Fig. 13. Corrosion rate in soil (Kawasaki Steel Corporation).

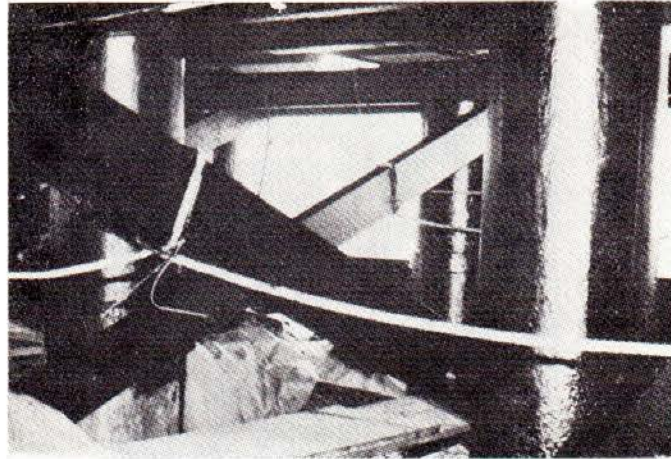
Figure 13 shows corrosion rates of steel test piles in soils at a reclaimed land site. The tops of some test piles were embedded into concrete footing. If the steel piles and steel reinforcing bar in the footing are electrically coupled, it might have a harmful influence on the corrosion. To investigate the corrosion effect of a galvanic anode system, 1 m long aluminum anodes were attached 1.25 m and 7.5 m below the tops of a half number of piles. However, there was no proof of significant difference in the corrosion of the two groups. Nevertheless, the corrosion rate of steel piles in sea water is never negligible, so that some kind of protective treatment should be adopted.

Figure 14 shows a commonly used method of coating steel piles above the sea water level. The coating starts with organic zinc-rich primer after blast-cleaning of the surface of steel pipe piles. Over the primer are the intermediate and finishing coats of coal-tar epoxy. Figure 15 shows a new corrosion



*SAWAGUCHI*

protective method. It consists of petrolatum paste, synthetic textile tape and an outer covering of fiberglass reinforced plastic. A diver is here shown applying petrolatum paste on pipe piles. The repair work for the splash zone is finished with this ERP covering.



**Fig. 14. Corrosion protection by coating.**



**Fig. 15. New corrosion protection method.**

**A PILE DRIVER MUFFLER**

One of the most difficult but urgent problems in pile driving is how to dampen the intolerable extent of noise or vibration during pile driving. Today, in densely populated districts of Japan, pile driving is not permitted unless there are effective noise pollution controls. A pile driver muffler has

### STEEL PIPE PILES

been developed and introduced in Japan to comply with such legal regulations. The conditions that must be met for such a muffler are:

- (1) conspicuous improvement for the reduction of noises,
- (2) speedy movability, and
- (3) simplicity in execution control.

A better improvement of its efficiency may require an expedient selection of the proper quality of muffler constituents. The wall of the most successful muffler ever manufactured in Japan is constituted of (Fig. 16) an outer sheath termed a dumbplate, which is made of a special damping material 0.2 mm in thickness sandwiched between two steel plates of 0.5 mm thickness, and glasswool sandwiched between a metal lath and a perforated aluminum plate. The two materials are connected by steel channels. In addition, the muffler is provided with both an air supplying pipe at the lower end and an air exhausting pipe at the upper end.

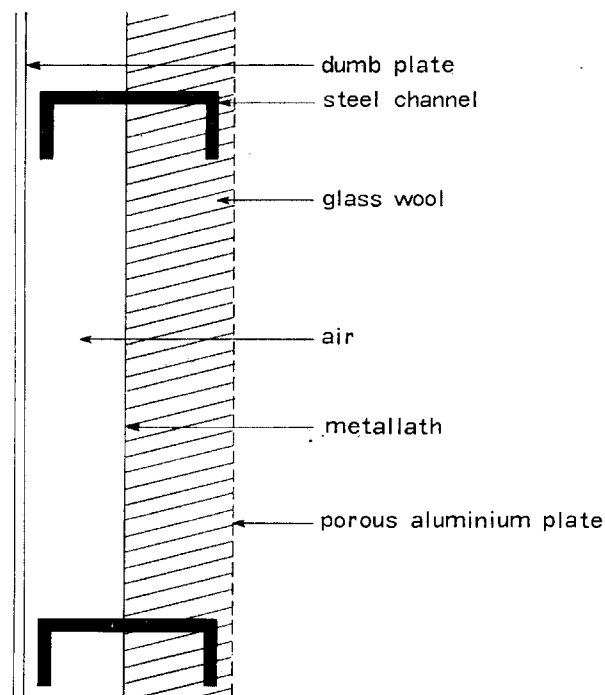


Fig. 16. Outer sheath of muffler for pile driver.

For knowledge of the character pertaining to this type of muffler, several measurements on the extent of noises have been made at several points apart from the pile driver covered with the muffler. The results are shown in Figure 17. It is apparent from this plot that the extent of noises during pile



SAWAGUCHI

driving with a muffler covering is at every point as much as 20 dB (A) lower than the pile without a muffler. Figure 18 shows the appearance of the muffler opened and then shut, its operation being readily possible with the use of air-cylinder systems.

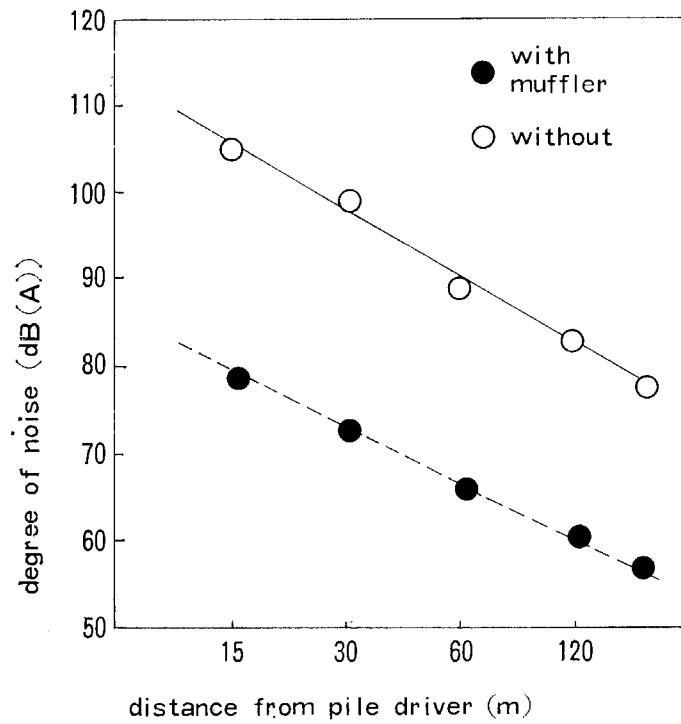


Fig. 17. Comparison of extent of noises.

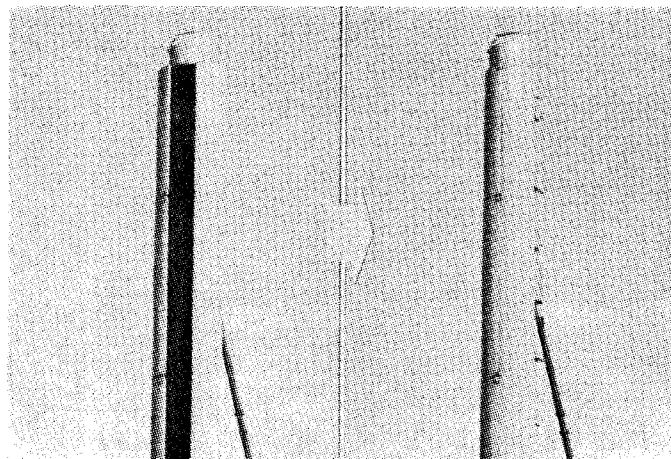


Fig. 18. Appearance of muffler.

## STEEL PIPE PILES

### AUTOMATIC WELDING MACHINE

Welding work on joints of steel pipe piles in earlier times was carried out manually. However, this type of welding displays a wide scatter of joint strengths and moreover took much time. For these reasons it has been an essential task to develop an automatic welding machine, resulting in a uniformity of joint strengths. In Japan, a semiautomatic welding machine was developed in 1965. Its principle is such that a welding wire is automatically sent out from the machine but the welding itself is done in the usual manner. Even such a semiautomatic machine can abbreviate a welding hour to a significant degree and reduce a scatter of strength in welded parts. A more advanced welding machine, termed a fully automatic welding machine, has been recently developed and put to a practical use. A joint of steel pipe piles can be automatically welded as shown in Figure 19, where a welding machine mounted on the guide rail is moving along the seam of joint around a steel pipe pile. Such a machine has further advantageous features in its operation and efficiency.

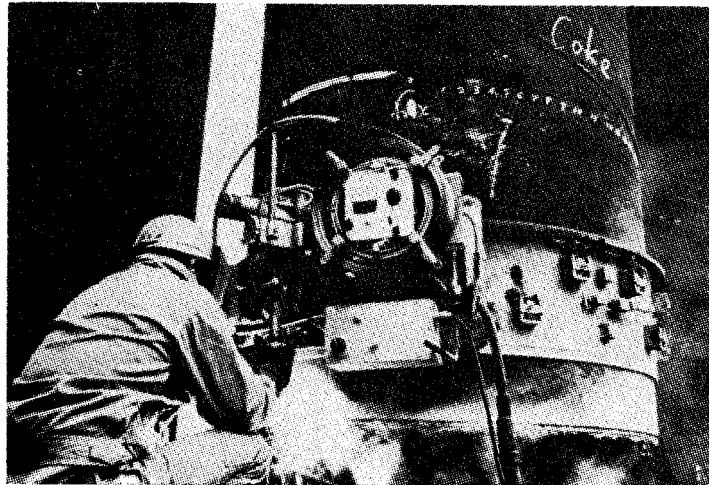


Fig. 19. Automatic welding machine.

### FINAL REMARKS

In Japan, where urban or local developments have been advanced to a considerable degree, engineers are forced to construct such public facilities as buildings, roads, railways, ports, or reclaimed land on soft soils in spite of full recognition of their unsuitable properties. In these projects a vast quantity of pipe piles have been satisfactorily utilized in order to offer stout support to quite massive structures; i.e. tall buildings, the abutments or

*SAWAGUCHI*

piers of a bridge, water-gates, and other marine structures. The tendency of increasing utility of steel pipe piles will probably continue in the future in Japan. It is heartily hoped that this description will become an aid to a better understanding of the present situation as to pile foundation in soft soils in Japan. It is a great pleasure to acknowledge the considerable assistance of Japanese Association of Steel Pipe Piles.

## REVIEW OF JAPANESE SUBSURFACE INVESTIGATION TECHNIQUES†

HIROSHI MORI\*

### SYNOPSIS

The paper reviews the current practice of subsurface investigation in Japan, with emphasis on particular techniques used during the preliminary and detailed phases of investigation. The method of standard penetration testing and its reliability with respect to testing depth and hammer type must be considered in the interpretation of blow-count values. Borehole geophysical methods, primarily seismic, provide a way to identify low velocity zones beneath a higher velocity zone during the preliminary phase of investigation. Innovations in undisturbed sampling techniques of cohesionless soils, borehole shear testing devices, and *in situ* measurement of earth pressure at rest are examples of continuing refinement in detailed geotechnical investigation methods used in Japan.

### INTRODUCTION

The classical techniques of subsurface investigation developed more than thirty years ago, such as various penetration tests, vane tests, and boring and sampling procedures, are still useful methods. However, they must be refined in the light of modern scientific knowledge. Recent developments in geotechnical analysis of soil mechanics problems have resulted in the development of new techniques that can obtain parameters representing strength and deformation characteristics of soils and rocks with higher accuracy and reproducibility than the classical methods.

Reviewing the broad scope of subsurface investigation the author attempted to classify various methods in regards to the different phases of investigation, for example, reconnaissance, preliminary investigation, and detailed investigation. In each of these phases the geotechnical information required is different depending upon the nature of the decision to be made in each phase. The information required in each phase of investigation is first discussed in this paper. Then the methods of investigation to meet the needs of each phase are collected and classified into several groups.

It is evident that the space of this paper does not allow the author to discuss each of the investigation methods in entirety. For this reason the focus of the paper is on particular subsurface techniques; borehole seismic exploration

†Presented at the Seminar on Geotechnical Engineering in Practice, Bangkok, July 20th and 21st, 1979.

\*Consulting Engineer, President of Kiso-Jiban Consultants Co., Ltd. Tokyo, Japan.

## MORI

and the standard penetration test during the preliminary investigation and undisturbed sampling of cohesionless soil, borehole shear testing and *in situ* measurement of earth pressure at rest during the detailed investigation.

### GENERAL DESCRIPTION ON SUBSURFACE INVESTIGATION

The object of subsurface investigation is to obtain geotechnical information required for design and construction decisions. The information required is completely different between the phase where the site feasibility is to be studied and the phase where the feasibility has been established and final dimensions of structures and the plan of construction are to be determined.

In order to classify various methods of subsurface investigation in a systematic framework of knowledge, subsurface investigation is divided into five phases; reconnaissance, preliminary investigation, detailed investigation, control tests and observation during construction, and investigation after construction completion.

#### *Reconnaissance*

The following procedures are normally applied in this phase.

- (1) The collection of information on topography, geology, ground water and seismicity from literature and documented materials.
- (2) A site visit to learn the regional conditions from people having local experience.
- (3) An investigation of geological structure, drainage pattern, and the distribution of soils from the interpretation of aerial photographs.
- (4) Obtaining information which may critically influence the design such as landslides, or cavities in the ground caused by mining, etc.
- (5) An investigation of possible sources of construction materials.

It is evident from many case records that faults of design and construction problems have often been caused by insufficient geological information. For a site located in an urban area, topography and subsurface conditions have usually been changed by man. In such an urban area site the use of an old map assists in recognizing important geological information which could have completely disappeared as a result of man's activity.

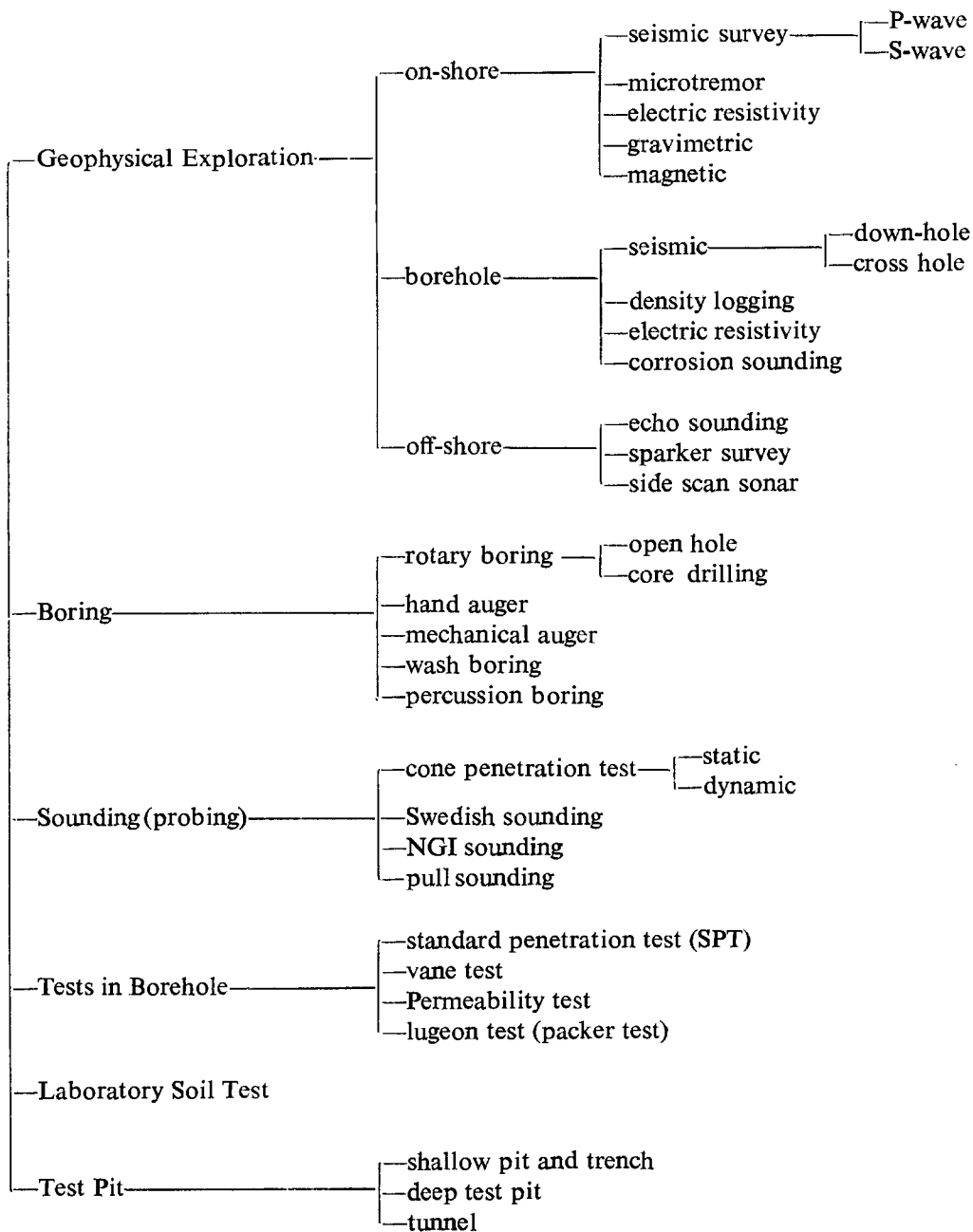
#### *Preliminary investigation*

In the preliminary investigation phase the decision is still concerned with feasibility. However, in this phase the owner wishes to establish feasibility

*SUBSURFACE INVESTIGATION*

more quantitatively. He needs to know whether or not economic benefits produced from the project can be balanced with his investment. This requires a preliminary design with a roughly estimated cost. The preliminary investigation aims at preparing the information necessary for this preliminary design.

**Table 1. Classification of subsurface investigation. Preliminary Investigation.**



The methods of subsurface investigation applied to the preliminary phase are summarized in Table 1. Among the various methods in Table 1 the standard penetration test (SPT) and geophysical exploration are selected here as topics.

*Standard Penetration Test.* As far as the author knows, in Japan more than 90% of boring during the preliminary investigation phase are carried out with the SPT. The SPT has been used in Japan for about twenty six years, since its first application by the author (MORI, 1953). The reasons that the SPT has not lost its value in 30 years are that:

- (1) representative samples having sufficient quality for visual inspection can be obtained,
- (2) the blow count ( $N$ -value) can be taken as a numerical index representing relative density of cohesionless soil or the consistency of cohesive soil, and
- (3) the correlation of  $N$ -value with practical problems in soil mechanics, such as settlement of foundations or bearing capacity of piles, was a useful guide to the design of foundations.

However, uncertainties of the SPT results caused by variable operational procedures of boring and testing have been pointed out in recent years. It is widely recognized that the imbalance of water level in a borehole causes disturbance of soil at the bottom of the hole, which results in a considerable reduction of  $N$ -value.

Recent tests by KOVACS (1979) indicate that the hammer velocity at impact, using a cathead with 2 wraps of manila rope, produces about 2/3 of the theoretical energy. It is found that the velocity of a free-drop hammer at impact is close to the theoretical free fall velocity. In consequence, automatic hammers with 100% efficiency will produce about 3/4 the  $N$ -value obtained from the SPT with a rope wrapped twice around a cathead (SOWERS, 1979).

Another problem is the reliability of the SPT results with respect to the depth of testing. The measurement of stress due to impact on a drill rod by TAKENAKA (1965) suggests that  $N$ -value can be valid without correction to the depth of 30 m in dense sandy ground. Mathematical calculations of the efficiency of energy transmitted to the bottom of a drill rod having different lengths have been made by a number of authors. The calculated efficiency is variable, 95% to 60%, for a rod 30 m long, depending upon the assumption of the mode of wave propagation and boundary conditions. UTO & FUYUKI (1977a) concluded from the results of experiments and analysis that the efficiency of energy within the length of 20 m is affected by a complex

*SUBSURFACE INVESTIGATION*

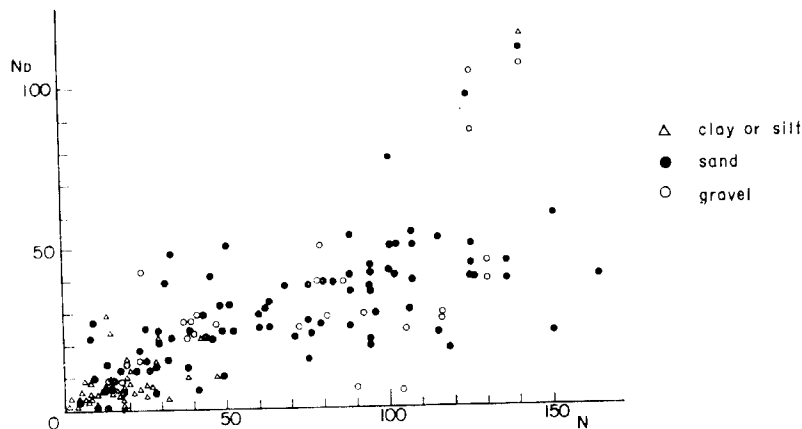
combination of the length of rod and rigidity of ground represented by a parameter called the reflection factor. In general, according to these authors, the attenuation of energy in a rod is small due to the effect of the reflecting wave in a rod. Uto and Fuyuki proposed a correction of  $N$ -value for a rod longer than 20 m with the equation:

$$N = N' (1.06 - 0.003l) \dots\dots\dots (1)$$

where  $N$  is the corrected  $N$ -value,  $N'$  is the measured  $N$ -value and  $l$  is the length of rod in m.

The attenuation of impact energy in a rod does not appear to be significant. Therefore, a substantial correction of  $N$ -value for depths within about 50 m will not be required. According to UTO & FUYUKI (1977a), the buckling of a long rod is not likely to affect the  $N$ -value, since lateral deformation of a rod happens after a sampler has penetrated into the ground.

In Japanese Standard (JISA1219-1961) it is stated that number of blows ( $N$ -value) shall be counted to the limit of 50 unless particularly specified otherwise. This clause suggests that a  $N$ -value over 50 does not always represent relative density of cohesionless soil or consistency of cohesive soil. Penetration tests using a sampler having an inside diameter of 50 mm and a hammer of 100 kg were made by KAITO, et al (1971) and UTO & FUYUKI (1977b). The blow-count obtained from the large penetration test ( $N_D$ ) is plotted against  $N$ -value by the SPT at the equivalent location and depth, as shown in Figure 1. Plotted points are widely scattered but the value of  $N_D$  roughly correlates in a linear fashion with  $N$ -values less than 50. For the  $N$ -values more than 50, no such linear relationship is recognized. Particularly,



**Fig. 1. Relationship between  $N_D$  and  $N$ -value (Uto & Fuyuki, 1977a).**



### MORI

the value of  $N_D$  in gravel is almost unchanged while  $N$ -value increases from 50 to 150. This means that  $N$ -values higher than 50 in gravel do not imply a higher density.

UTO & FUYUKI (1977a) suggested that the rebound height of a hammer after it strikes the top of a drill rod can be a parameter to identify rigidity of the ground. He proposed a method to compute a reflection factor from height of rebound or the time from the first to the second to when the rebounded hammer strikes against a rod.

*Geophysical exploration.* Borehole geophysical methods have developed greatly in recent years. If a low velocity zone exists beneath a high velocity zone, or if such a zone of low velocity is contained locally, borehole logging is the only way to detect such a feature.

The use of shear waves in seismic exploration has become more popular in soil mechanics. Shear waves are propagated through soil fabric and ground water does not influence their velocity. Furthermore, shear wave velocity is a function of the shear modulus which is an important factor governing the dynamic response of the ground. The study of seismic exploration by shear waves have therefore developed to meet the practical demands of earthquake engineering.

Borehole seismic methods may be divided into cross-hole, down-hole and up-hole methods. The cross-hole method consists of creating a seismic impulse at a given elevation in one borehole and recording shear wave arrivals at the same elevation in two adjacent boreholes. Three holes are used in order to eliminate possible nonlinearities at the source and to avoid uncertainties in start-time measurements. The borehole spacing should be far enough to give a discernible difference in shear wave travel time and should be close enough to reduce the possibility of picking up refracted waves from adjacent layers. According to MCLAMORE, et al (1978), refraction effects may become particularly significant when spacing exceeds 12 to 15 m. AULD (1977) suggested that a 5 m spacing between boreholes produces the best results.

NISHIKI, et al (1979) attempted to apply a cross-hole method to the evaluation of soil improvement. The shear wave velocity in soft ground before improvement was compared to the velocity after installing compacted sand piles of 700 mm in diameter at a spacing of 1.5 m. The results are summarized in Figure 2. A considerable difference in shear wave velocity is recorded before and after improvement. This result suggests that the cross-hole method has a possible application in controlling improvement, although the increase of

224

### SUBSURFACE INVESTIGATION

shear wave velocity shown in Figure 2 is supposed to be considerably affected by sand columns replaced with soft soil.

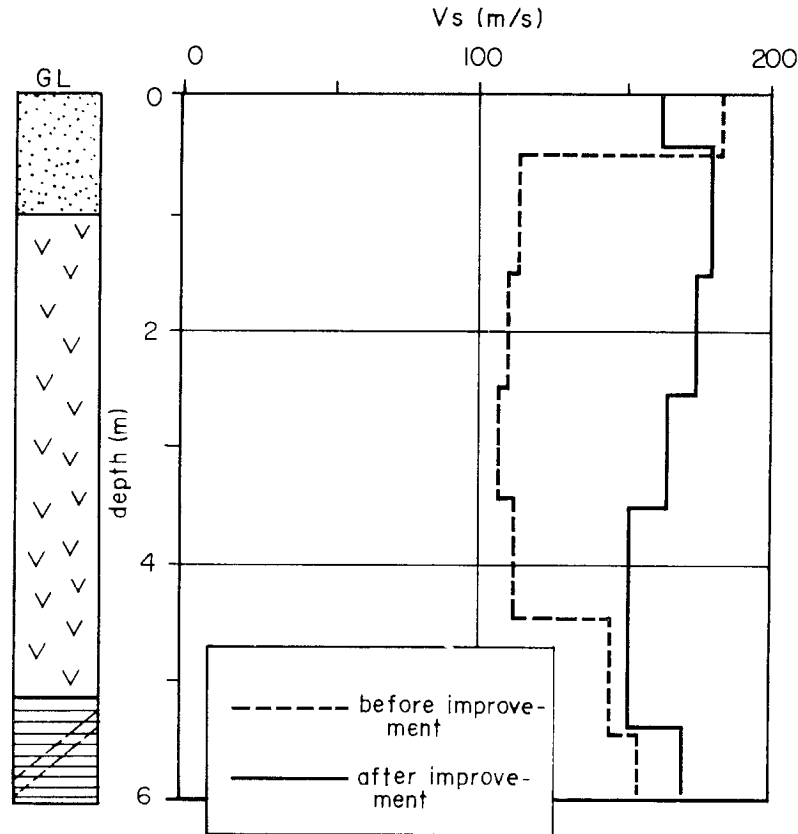
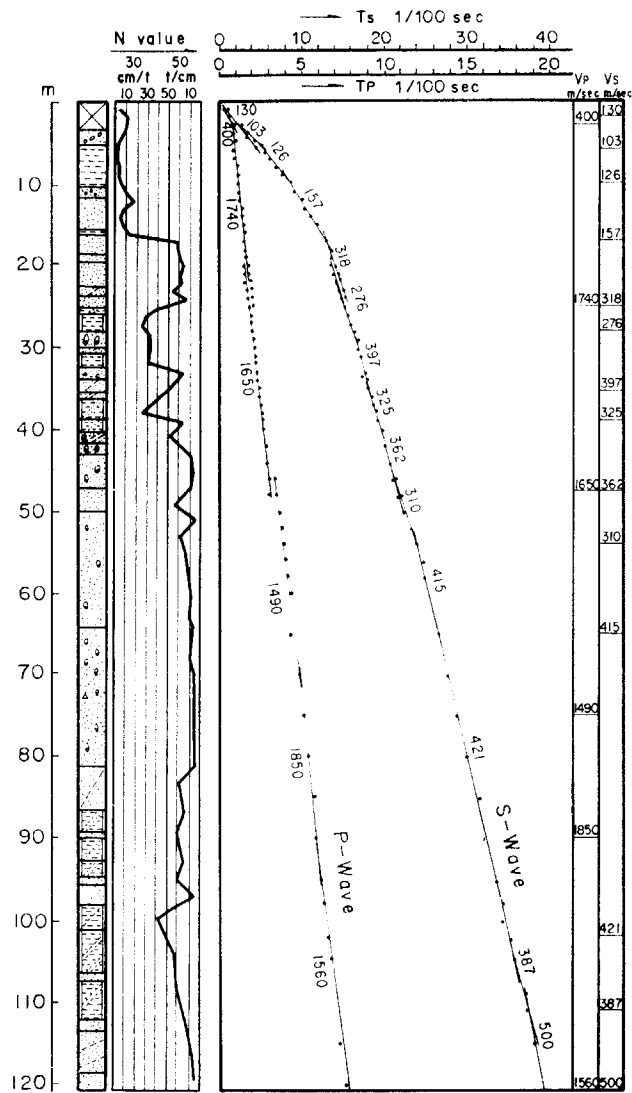


Fig. 2. Shear wave velocity in the ground improved with compacted sand piles (Nishiki et al, 1979).

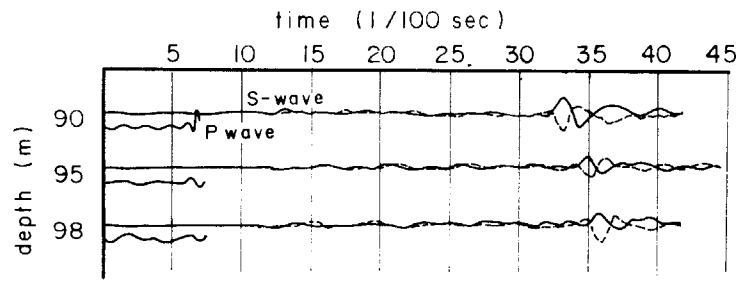
The velocities of P-waves and S-waves can also be measured by applying a shock on the ground surface and receiving elastic waves by instruments arranged in a drill hole. In this system, called the down-hole method, P-waves are generated by a drop hammer and the S-waves are produced by a horizontal impact to a plate placed on the ground surface. A typical shear wave record is shown in Figure 3. The velocity of shear waves in the range between 130 to 500 m/sec has been measured successfully to a depth of 120 m (MORI, 1977a).

In general, the cross-hole technique provides greater measurement accuracy. The observed waves travel through a particular stratum with less interference from nearby refracting horizons (AULD, 1977). The down-hole method may be

MORI



( a )



( b )

Fig. 3. Distribution of shear wave velocity obtained from the down-hole technique .

## *SUBSURFACE INVESTIGATION*

less accurate, but the results can be obtained from a single borehole instead of a minimum of three holes required in the cross-hole method. The down-hole technique has the advantage that the path of the wave is identical to the vertical path of refracted earthquake wave.

### *Detailed Investigation*

The decisions to be made in this phase of investigation are concerned with the final dimensions of structures, construction aspects, and maintenance. Relevant government agencies will require information on public security and environmental impact. Information showing that the adopted design meets regulatory requirements must be prepared.

Methods of investigation applied in this phase are classified into three major groups; laboratory tests of undisturbed samples, tests in borehole, and large scale tests. Laboratory tests have the advantage that testing conditions of specimens can be controlled so as to simulate the geotechnical environment, such as stress in the ground or pore water pressure caused by structures or excavation. However, in order to make parameters obtained from laboratory tests meaningful, undisturbed samples must be obtained.

Before 1970, tube samples of sand were mainly used for identification of stratification, though block samples were used for laboratory tests. After 1970, sampling techniques were improved as specimens for dynamic triaxial tests were demanded in order to solve soil dynamic problems. The techniques to obtain undisturbed samples of sand will continue to be refined in the future.

Various tests in boreholes can be applied to the soil whose mechanical properties can not be accurately obtained from laboratory tests because of sample disturbance. Even borehole tests can not be free from the disturbance of soil. However, disturbance of soil around a borehole is relatively minor compared to the disturbance of samples of cohesionless soil or fissured clay.

Borehole tests are not the final solution to control the geotechnical environment. For example, from a strength parameter obtained from borehole tests in virgin ground, it is hard to predict the value of the same parameter to be obtained from the bottom of an excavation when overburden soil is removed and the water table is drawn down. Combined laboratory and borehole testing supplements the disadvantage of each method.

Large scale field tests can simulate the effect of proposed structures by full scale tests or large scale model tests. The parameters obtained either from laboratory tests or borehole tests can be corrected prior to application of

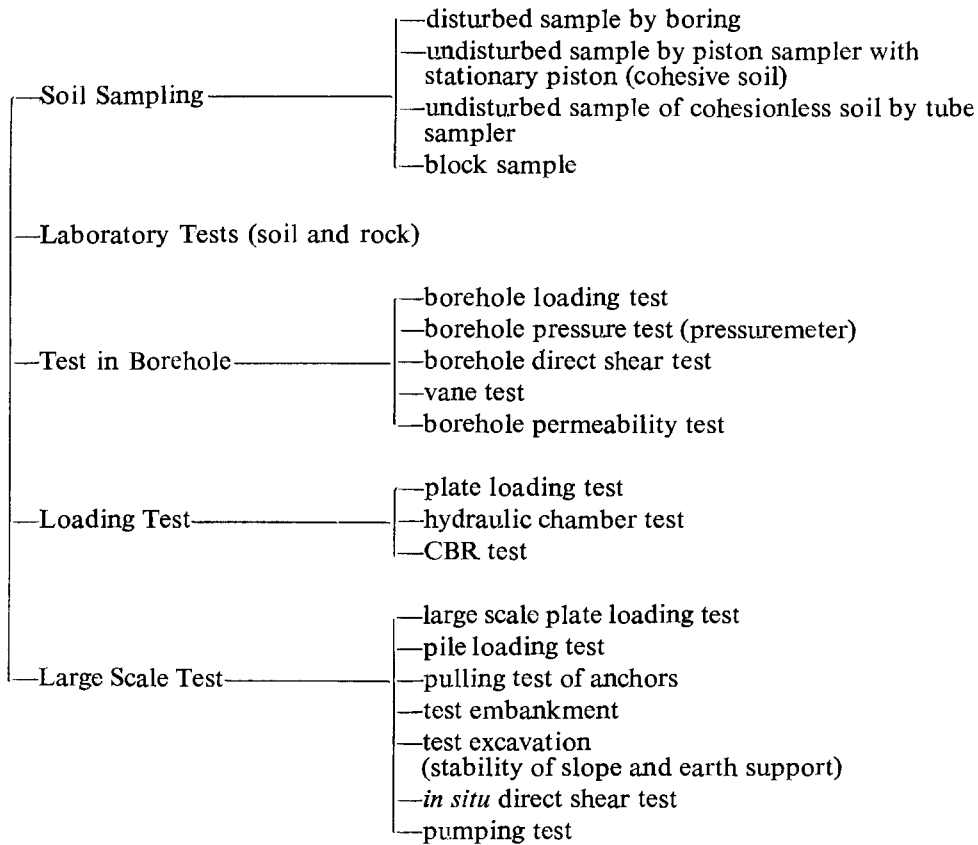
MORI

those values to the design by correlating the measured values from a large scale test with the values predicted from the results of laboratory or borehole tests.

Large scale tests simulating only a limited portion of the proposed structure do not yield sufficient information required for a design. Therefore the contribution of large scale tests will be limited unless their results are properly interpreted in light of various test results in the laboratory or borehole.

Methods applicable to the detailed investigation phase are summarized in Table 2. Three of the techniques are discussed below.

**Table 2. Classification of subsurface investigation. Detailed Investigation.**

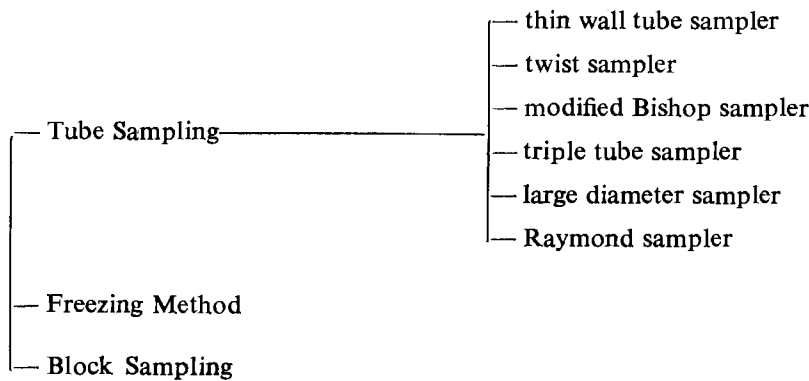


*Undisturbed sampling of cohesionless soil.* The techniques of sampling popularly used in Japan for sandy soil are classified as shown in Table 3. Both the freezing and block sampling methods have the advantage of avoiding disturbance of the sample caused when a sample tube is forced into sandy

*SUBSURFACE INVESTIGATION*

soil. The freezing method can not be applied to silty sands because of volume changes caused by freezing. Taking block samples below ground water table after dewatering may cause a change of the void ratio of silty sand.

**Table 3. Classification of sampling techniques.**



Recent techniques of tube sampling are more advanced than the primitive methods, such as an open tube sampler driven by a drop hammer or air hammer. The efforts to reduce disturbance of a sample by refined samplers are presently continuing. The dimensions of samplers popularly used in Japan are summarized in Table 4.

**Table 4. Dimensions of samplers.**

| Name of Sampler         | Diameter of Sampler (mm) | Length of Sample(cm) | Area ratio(%) | Suitable Soil Conditions               |
|-------------------------|--------------------------|----------------------|---------------|--|
| Thin wall sampler       | 75                       | 30                   | 11            | loose sand $5 < N < 10$                |
| Twist sampler           | 50                       | 70                   | 96            | very loose to medium sand $0 < N < 20$ |
|                         | 70                       | 70                   | 40            |  |
| Modified Bishop sampler | 53                       | 50                   | 62            | loose sand $5 < N < 10$                |
| Triple tube sampler     | 82                       | 95                   | 13            | loose to dense sand $5 < N = 50$       |
| Large diameter sampler  | 200                      | 100                  | 15            | loose sand $5 < N \leq 10$             |

The objective of sampling technique improvement is to minimize the influence of sample disturbance on the results of laboratory tests. If we know how the disturbance of samples influences the results of laboratory tests, we can determine from test results whether the samples used are of good quality or disturbed.

MORI

In Figure 4, the angle of internal friction obtained from drained triaxial tests is plotted against relative density of dense sand of a Tertiary formation. It is noted from Figure 4 that the difference in the angle  $\phi_d$  between undisturbed and reconstituted samples is in general less than one degree. Reviewing data published or unpublished, the author considers that the angle  $\phi_d$  is not sensitively influenced by the sample disturbance. The values of the parameter

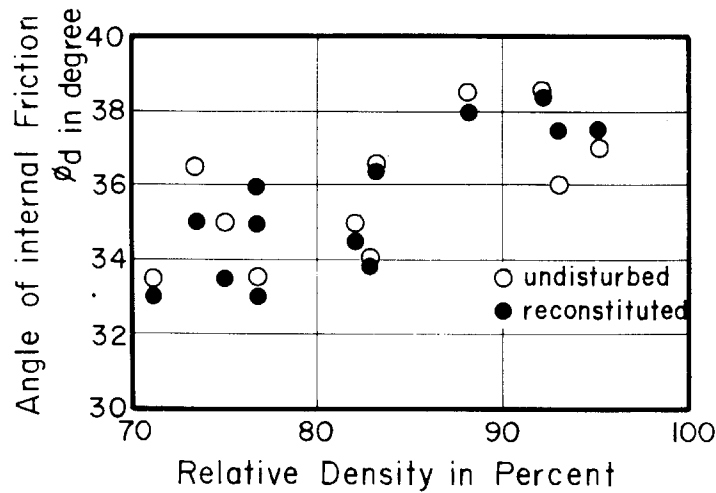


Fig. 4. Angle of internal friction for undisturbed and reconstituted samples.

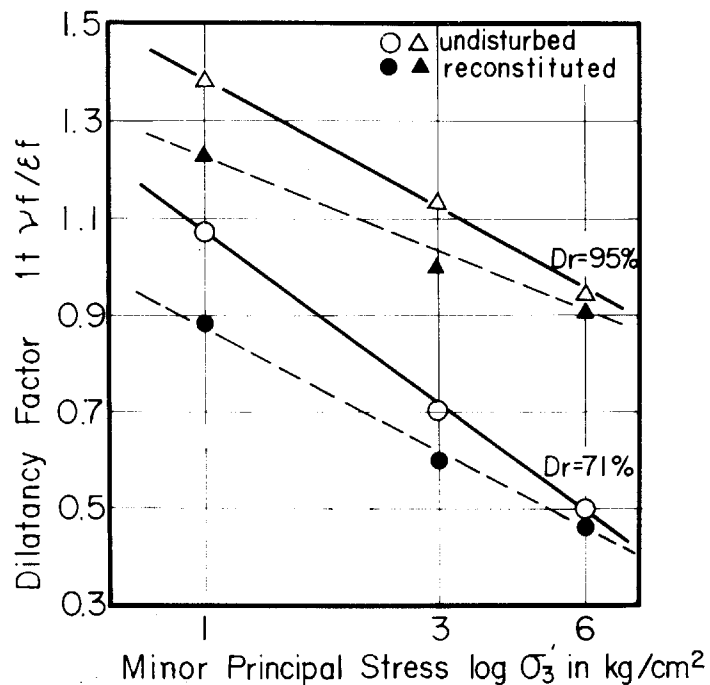


Fig. 5. Dilatancy factor for undisturbed and reconstituted samples.

*SUBSURFACE INVESTIGATION*

$\phi_d$  seem more affected by grain size distribution, grain shape, or other factors which are not related to the disturbance of the sample.

For the same sand as depicted in Figure 4, the dilatancy factor at failure observed during drained triaxial tests is plotted against effective confining pressure as shown in Figure 5. The dilatancy factor at failure is defined as  $1 + v_f / \xi_f$ , where  $v_f$  is the volumetric strain at failure and  $\xi_f$  is the axial strain at failure. It is noted that the dilatancy factor for reconstituted samples is considerably smaller than that for undisturbed samples. The dilatancy factor against relatively low minor principal stress is more sensitive for the disturbance of sand than its drained shear strength.

Recent studies on liquefaction and cyclic deformation characteristics have shown that the dynamic strength will be influenced by the disturbance of soil fabric and dynamic stress history which may occur in the course of sampling (LADD, 1974; SEED & LEE, 1966).

The typical reduction of dynamic strength of silty sand is depicted in Figure 6 (OHASHI et al, 1976). The reclaimed silty sand, having an average grain size  $D_{50} = 1.1 \sim 1.5$  mm and a fines content of about 15 to 30%, was subjected to a cyclic triaxial test at a relative density of 76%. Dynamic strength,

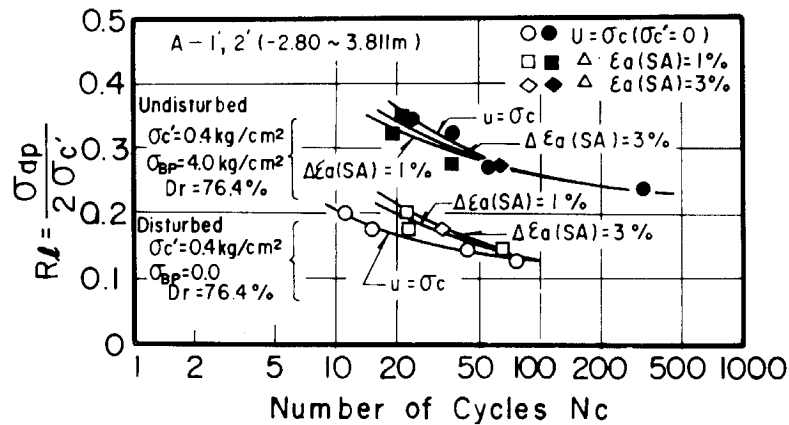


Fig. 6. Influence of sample disturbance to the dynamic strength of silty sand (Ohashi et al, 1976).

defined for initial liquefaction, 1% and 3% strain in single amplitude is plotted against the number of cycles. It is evident that sample disturbance has significant influence on the dynamic strength of silty sand. Therefore it is meaningless to evaluate liquefaction potential of this type of soil from the results of cyclic triaxial tests of disturbed samples.



*Borehole shear test.* An instrument for direct shear testing in a borehole, named *in situ* shear testing device (IST), was built by the author and his colleagues. As schematically illustrated in Figure 7, the device is a cylindrical probe consisting of 6 segments surrounding a flexible rubber tube and a pulling device consists of a center hole jack and load cell to measure the resistance of the shear probe against pulling force. The IST has been used in number of project sites since 1970.

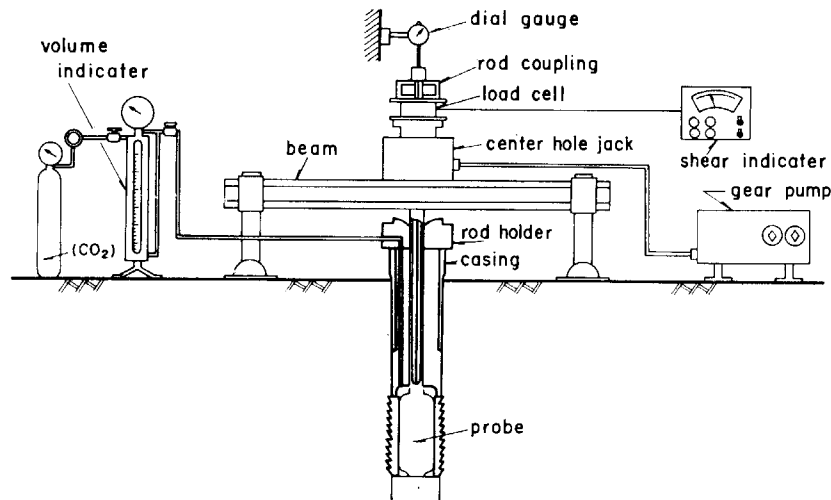
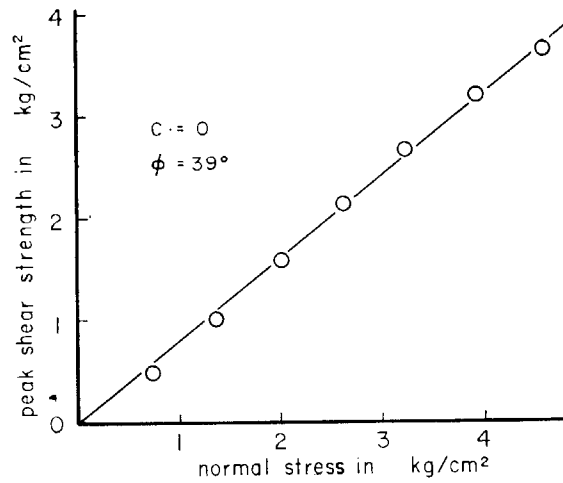


Fig. 7. Layout of in-situ shear test device (IST).

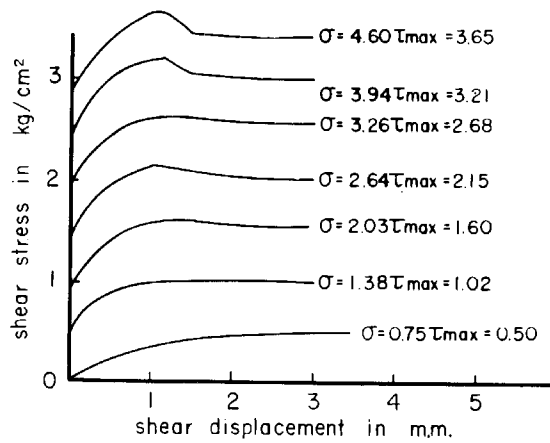
In the United States an instrument for borehole shear test was devised by LOHNES & HANDY (1968). The instrument, named a borehole shear device (BHS), is presently used in the United States. The probe of the BHS has two shear plates connected with a pneumatic piston.

An example of the data obtained from the IST is shown in Figure 8. The test was made in saturated dense sand of a Pleistocene formation at the depth of 24 m. The  $N$ -value of SPT at the equivalent depth was 48, and the rate of shear displacement was 1 mm/min. By plotting peak shear strength against normal stress an angle of internal friction of 39 degrees was obtained. It is noted that the points for normal pressure less than 2 kg/cm<sup>2</sup> plot slightly below Mohr's envelope. In the case of dense sand or stiff clay, the shear strength against normal stress less than 1.0 kg/cm<sup>2</sup> falls generally lower than the Mohr's envelope obtained from higher normal stress. For lower normal stress the teeth of shear plates are not completely seated in the side of borehole. In case of sand containing gravel, the teeth can not seat in soil, but they slip along the surface of gravel. For such soil, the angle  $\phi$  obtained by IST becomes unreasonably low.

**SUBSURFACE INVESTIGATION**



(a)



(b)

**Fig. 8. Results of the IST for dense sand; (a) Mohr's envelope (b) stress-displacement curves.**

The results of the BHS presented by WINELAND (1975) appear too low in cohesion and too high in the angle  $\phi$ . This may result from the range of normal stress, 28 to 104 kN/m<sup>2</sup>. The two shear plates mechanism of the BHS may not allow higher normal stress to be applied as soil behind the plate may cause shear failure.

The results of the IST in soft cohesive soil often give lower values of  $c$  and higher values of  $\phi$  as compared to the results of consolidated undrained shear tests in the laboratory using either a triaxial cell or a simple shear device. Suppose a granular soil containing a substantial amount of fine grained soil is subjected to the IST. The consolidated undrained shear strength of this soil is assumed to be represented by the Mohr's envelope shown by the

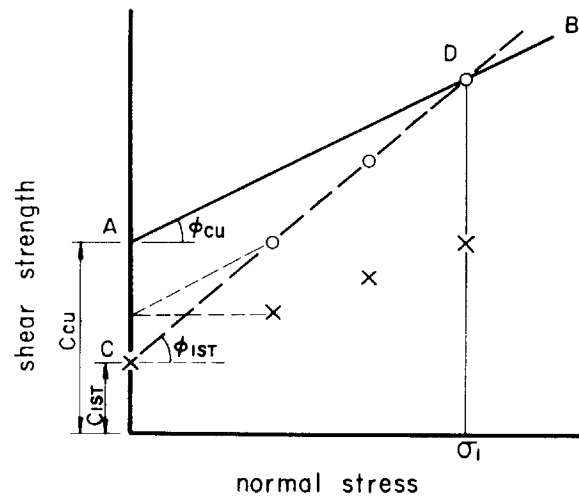


Fig. 9. Shear strength obtained from the IST compared to the true strength of soft cohesive soil.

solid line  $AB$  in Figure 9. The cohesion of soil around a borehole is decreased due to disturbance and stress reduction caused by boring. The soil behind the shear plate then regains its cohesion by consolidation while the plate is forced into soil. If the soil is assumed to have recovered its original cohesion by normal stress  $\phi_1$ , the shear strength will reach point  $D$ . The Mohr's envelope obtained from the IST will become as the broken line  $CD$ , which gives higher value of  $\phi$  and lower value of  $c$ . If the IST is made under a normal stress higher than  $\phi_1$ , it will produce the Mohr's envelope close to true  $\phi_{cu}$  and  $c_{cu}$ . However, a normal stress higher than  $\phi_1$  may exceed yield stress of soft soil around a borehole. One of the results of the IST for soft plastic clay of volcanic origin is shown in Table 5. The parameters obtained from the IST are close to those obtained by direct shear tests in the laboratory, but with higher  $\phi$  and lower  $c$ .

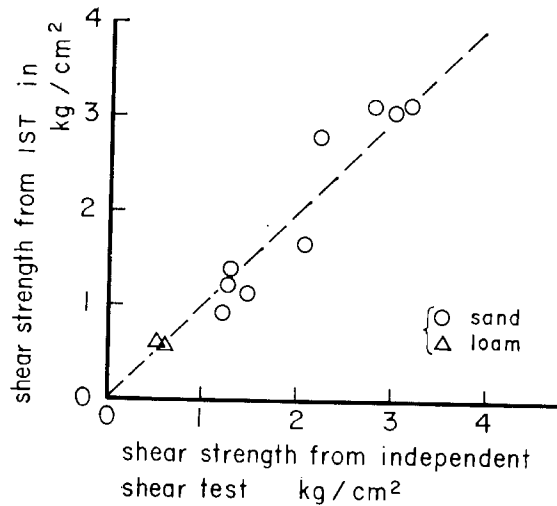
Table 5. Comparison of the IST results with laboratory test results.

| Parameters          | Triaxial Test ( $cu$ ) | Direct Shear Test | IST  |
|---------------------|------------------------|-------------------|------|
| $c(\text{kg/cm}^2)$ | 0.05                   | 0.21              | 0.08 |
| $\phi$ (degree)     | 19                     | 22                | 24   |

In the IST, shear tests are repeated several times at almost same elevation in a borehole. The influence of repeated shear testing was studied by comparing the results of the IST to those obtained from an independent shear test

*SUBSURFACE INVESTIGATION*

without repetition. The independent shear test was done about 20 cm away from the IST. The shear strength obtained from the independent shear test was compared to the shear strength corresponding to the same normal stress obtained from the IST. The results are illustrated in Figure 10.



**Fig. 10. Influence of repeated shear test.**

With respect to the statistical correlation between the parameter  $\phi$  and  $N$ -value of SPT, the author cannot conclude a relationship as he has not collected a sufficient amount of reliable data. From a correlation of the present data, he assumes an approximate relationship between  $\phi$  of IST and  $N$ -value as follows:

$$\phi_{IST} = 0.3N + 26 \quad \dots\dots\dots (2)$$

or 
$$\phi_{IST} = \sqrt{15N} + 15 \quad \dots\dots\dots (3)$$

The above empirical relationship is illustrated in Figure 11 together with the relationship obtained by DUNHAM (1954), OSAKI (1959), and TERZAGHI & PECK (1948). From Figure 11, it is noted that the parameter  $\phi$ , obtained from IST, is close to the correlation of Terzaghi and Peck and the lower boundary of Dunham, but that Osaki's curve and the higher boundary of Dunham give higher values of  $\phi$  than predicted by the IST.

It is concluded that the IST is a simple field test which can be made without spending time and cost for laboratory testing. The method is adopted for cohesionless soil whose parameters  $\phi$  and  $c$ , may seriously be influenced by disturbance of soil samples. This method can be applied to cohesive soil

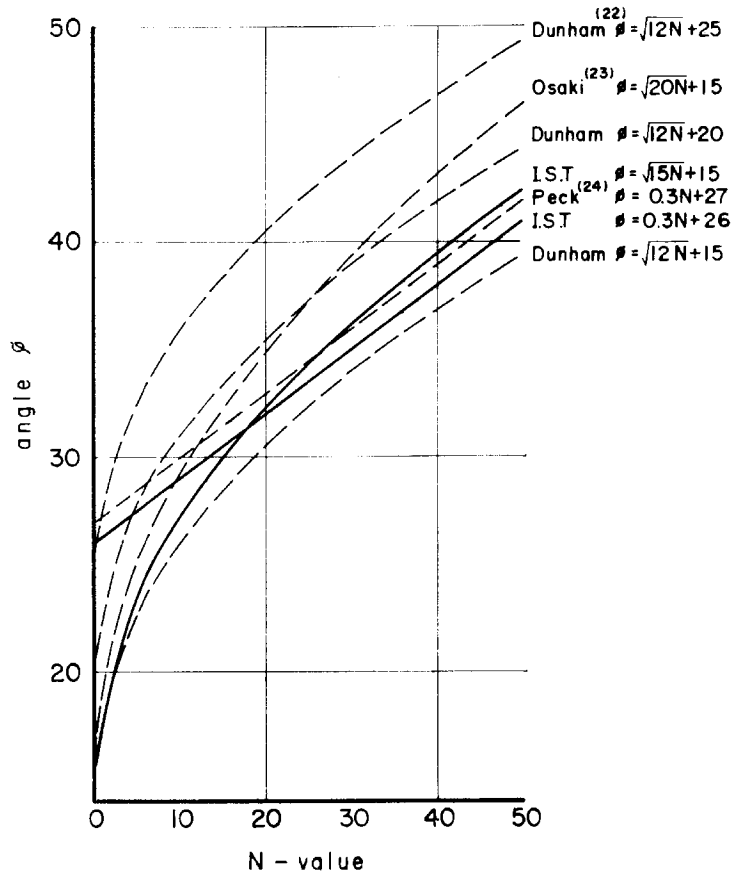


Fig. 11. Correlation between  $\phi$  and N-value.

but care is needed in order to avoid disturbance of soil along a borehole. The application of this method to soft cohesive soil is difficult, because of uncertainty related with the setting-up of pore water pressures and their dissipation.

*In Situ Measurement of Earth Pressure at Rest.* Recent techniques in geo-technical analysis, such as the finite element method or the evaluation of liquefaction potential of sand during an earthquake, requires an estimation of initial *in situ* stresses in the ground. However, in spite of its importance in soil mechanics, proper estimation of horizontal stress or earth pressure at rest requires highly sophisticated techniques, most of which need to be refined.

The earth pressure at rest,  $p'_o = p_o - u$ , where  $p_o$  is total horizontal stress at rest and  $u$  is pore water pressure, may be estimated from a value of coefficient of earth pressure at rest defined by the equation:

*SUBSURFACE INVESTIGATION*

$$K_o = \frac{p'_o}{q'_o} = \frac{p_o - u}{q_o - u} \dots\dots\dots (4)$$

where  $q_o$  is total vertical stress and  $q'_o$  is effective vertical stress.

The value of  $K_o$  is measured by so-called  $K_o$  consolidation tests in a triaxial cell. The coefficient  $K_o$  is also estimated from an empirical equation of JAKY (1944) and other equations of similar nature.

$$K_o = 1 - \sin \phi' \dots\dots\dots (5)$$

The empirical formula (Equation 5) is roughly applicable to normally consolidated clay, but it is evident from the field measurements of MASSARCH, et al (1975) and BAGUELIN, et al (1978) that  $K_o$  for over-consolidated clay is much higher than the value given by Equation 5.

Direct *in situ* measurements of the coefficient may be classified into three groups:

- (1) hydraulic fracture technique,
- (2) total earth pressure cell, and
- (3) the pressuremeter and its modifications.

BJERRUM & ANDERSEN (1972) first reported using the hydraulic fracture technique to determine the coefficient  $K_o$  in natural deposits of soft Norwegian silty clays and quick clays. Basically, this method is to form vertical cracks or failure plane around a cylindrical piezometer in normally consolidated clay by forcing water into surrounding soil. A falling-head permeability test is then performed and quantity of water flowing out into surrounding soil is measured. As the head decreases, a point is reached when a sudden reduction in flow occurs as cracks close. This "close-up" point of pressure should correspond to total lateral earth pressure at rest.

MASSARCH, et al (1975) compared the coefficient  $K_o$ , obtained from the hydraulic fracture technique in several sites in glacial clay in Sweden to that obtained from measurement of horizontal stress by a total pressure cell of 4 mm thick, named a "Glotzel earth pressure cell". He reported that the agreement between the two methods was rather poor; only at one site was the agreement within 10%.

TAVENAS, et al (1975) compared the results of three different methods; hydraulic fracture technique, total earth pressure cell, and pressuremeter. The investigation was carried out in soft sensitive clay located about 80 km west of Quebec, Canada. These authors used a hydraulic fracture technique as proposed by Bjerrum and Andersen but with an electronic transducer to

record pressure head. As a total earth pressure cell, they selected a spike-like cell 12 mm thick driven into clay. For the measurement of  $p_o$  by pressuremeter, standard Menard pressuremeters with probe diameters of 44 mm and 60 mm were applied. The authors reported that the reproducibility of the hydraulic fracture technique was very good, but disturbance of sensitive clay associated with creep when a piezometer was installed caused significant variation of horizontal stress. Total stress cell measurements were shown to be satisfactory both in terms of limited disturbance of clay during installation and good reproducibility. The results of pressuremeter tests were shown to be questionable in quality. The agreement among three methods was not encouraging.

Menard proposed a method to measure pressure at rest using a flexible rubber membrane called a "geocell". Improving the simple device proposed by Menard, a modified type of geocell was built in Japan (MORI, 1977b). A geocell consists of a simple cylindrical probe connected with a device to apply hydraulic pressure to the probe.

The results of observation on the horizontal stress variation during the improvement of loose sandy ground with compacted sand piles are illustrated in Figure 12. The site was covered with loose sand or sandy silt whose  $N$ -value was from 2 to 12 to the depth of 6 m. The sandy soil was underlain by plastic clay or silt. Compacted sand piles of 50 cm in diameter were driven

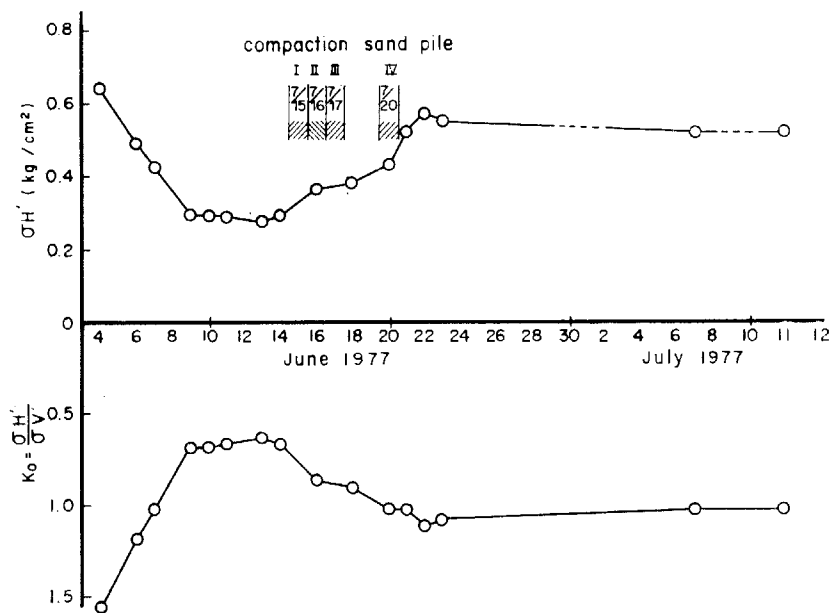


Fig. 12. Variation of horizontal stress during construction.

### *SUBSURFACE INVESTIGATION*

to the depth of 6 m in order to prevent liquefaction of the sandy soil during an earthquake. Sand piles were arranged in square grids of 2 m spacing. The geocell was installed in the center of a grid at a depth of 3 m.

It is noted from Figure 12 that effective horizontal stress was stabilized in about 10 days after installation and then increased as sand piles were driven around the geocell. After completion of sand piling the horizontal stress decreased slightly, but then remained constant. Subsequently, the value of  $K_o$  increased from its initial value of 0.65 to 1.1 during sand piling. The coefficient  $K_o$  was maintained at approximately 1.0 after the soil was compacted. The increase of  $K_o$  has practical significance in increasing the dynamic shear strength of sand against liquefaction.

The knowledge of horizontal stress in the ground is not only useful for geotechnical analysis but it has a possibility of practical application to the control of construction and the observation of the influence of excavation or tunnelling in adjacent areas.

### CONCLUSION

The paper is concluded with a final review on the flow of subsurface investigation throughout the phase of reconnaissance, preliminary investigation, and detailed investigation. The flow of work is summarized in Table 6. At the completion of the detailed design based on the results of detailed investigation, the design is evaluated. As one of the points in evaluation it is reviewed whether or not data required for construction are sufficiently provided. If data are insufficient additional investigation will be needed.

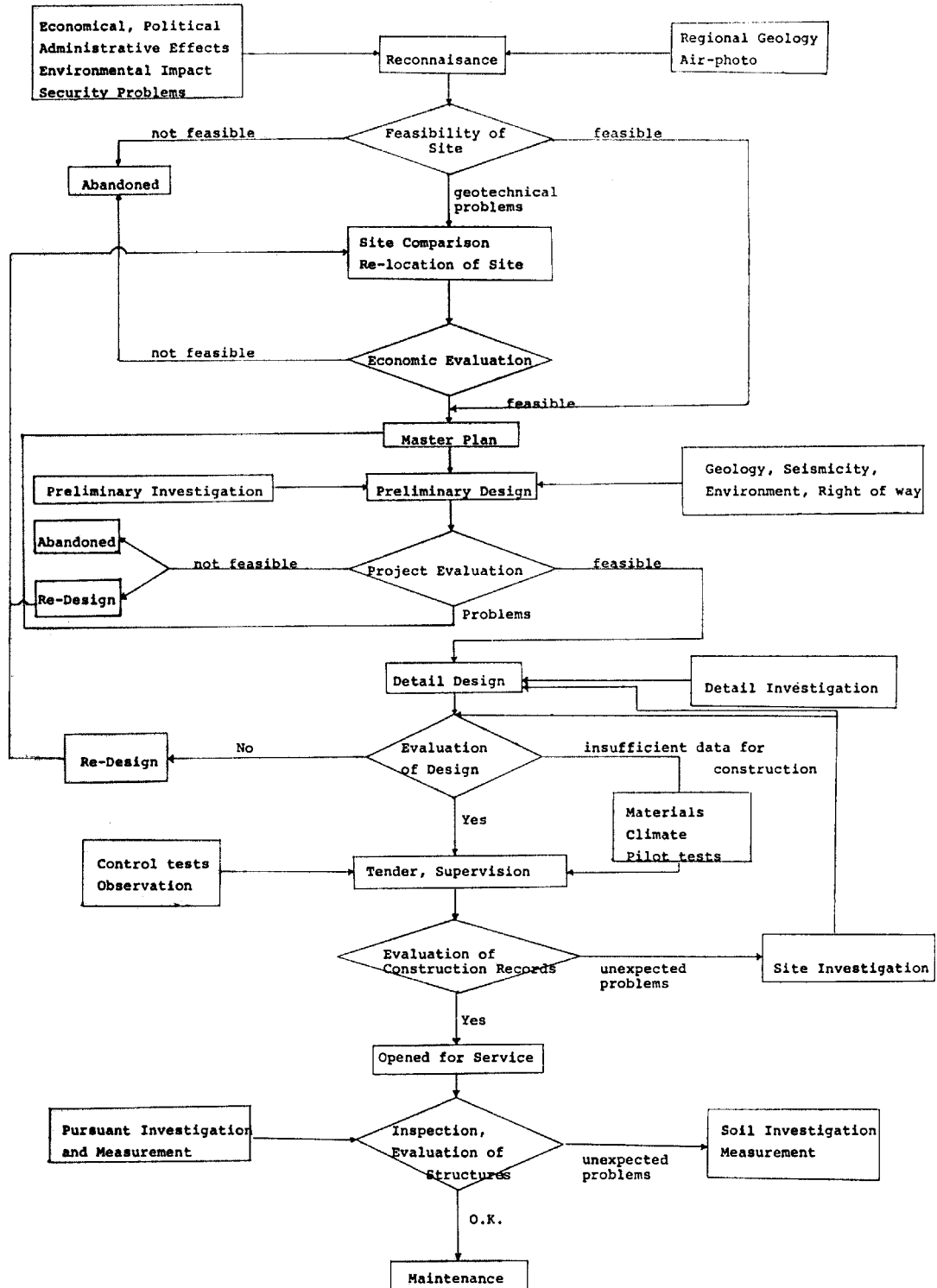
It is important to note that no one should consider that soil investigation has terminated when design is submitted to tender. In reality, full scale tests do not start until the moment when construction commences. No one can confirm if the ground model set forth was correct or if the parameters determined were adequate unless one observes what happens during construction. Catastrophic accidents are often caused by overlooking unusual ground phenomena appearing during construction which are not expected by either a designer or a supervisor.

Foundations or embankments commonly continue to settle even after the completion of structures. There are a number of examples of moving retaining walls, settling and cracking pavements, or leaking water from dams. Such cases may require enormous cost of maintenance and redesign of structures. The subsurface investigation and measurement after construction will furnish



MORI

Table 6. Flow chart of subsurface investigation.



## SUBSURFACE INVESTIGATION

proper information as how to maintain or redesign structures and how to prevent catastrophic phenomena which may occur unless the behavior of structures constructed are looked after by geotechnical engineers.

## ACKNOWLEDGEMENTS

The author acknowledges the assistance of Mr. Tsuchiya and other staff of his firm in the experiments presented in this paper, and presents his appreciation to them.

## REFERENCES

- AULD, B. (1977), Cross-Hole and Down-Hole Vs by Mechanical Impulse, *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 103, GT-12, pp. 1381-1397.
- BAGUELIN, F., JEZQUEL, J.F., SHIELDS, D.H. (1978), *The Pressuremeter and Foundation Engineering*, Trans Tech Publications, Aedermansdorf, Switzerland.
- BJERRUM, L. and ANDERSEN, K.H. (1972), In-Situ Measurement of Lateral Pressure in Clay, *Proceedings of the Fifth European Conference on Soil Mechanics and Foundation Engineering*, Madrid, Vol.1, pp. 11-20.
- DUNHAM, J.W. (1954), Pile Foundation for Building, *Journal of the Soil Mechanics Division*, American Society of Civil Engineers, Vol.80,SM-1, No. 385, pp.1-21.
- JAKY, J. (1944), *The Coefficient of Earth Pressure at Rest*, Journal of the Society of Hungarian Architects and Engineers.
- KAITO, T., SAKAGUCHI, S., NISHIGAKI, Y., MIKI, K. and YUKAMI, H. (1971), Large Penetration Test, *Journal of Japanese Society of Soil Mechanics and Foundation Engineering* (in Japanese), Vol.19, No.7, pp. 15-21.
- KOVACS, W.D. (1979), Velocity Measurement of Free-Fall SPT Hammer, *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol.105, GT-1, pp. 1-10.
- LADD, R.S. (1974), Specimen Preparation and Liquefaction of Sands, *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol.100, No. GT-10, Technical Notes, pp. 1180-1184.
- LOHNES, R.A. and HANDY, R.L. (1968), Slope Angle in Friable Loess, *Journal of Geology*, Vol.76, No.3, pp. 247-258.
- MASSARCH, K.R., HOLTZ, R.D., HOLM, B.G. and FREDRIKSSON, A. (1975), Measurement of Horizontal In-Situ Stresses, *Proceedings of the Conference on In-Situ Measurement of Soil Properties*, Raleigh, North Carolins, American Society of Civil Engineers, Vol. I, pp. 266-286.
- McLAMORE, V. R., ANDERSON, D. G. and ESPANA, C. (1978), Cross-hole Testing Using Explosive and Mechanical Energy Sources, *Special Technical Publication 654*, American Society for Testing and Materials, pp.30-55.
- MENARD, L. (1957), *Travail personnel sur le pressiometre*, Ecole Nationale des Ponts et Chaussees, Paris, France.
- MORI, H. (1953), Two Innovative Methods in Soil Exploration, *Journal of Japanese Society of Soil Mechanics and Foundation Engineering*, in Japanese, Vol, 1, No. 1, pp.25-31.
- MORI, H. (1977a), *Technical Report* of Kiso-Jiban Consultants Co., Ltd.
- MORI, H. (1977b), *Unpublished Report* of Mori Geotechnique Inc. Tokyo.
- NISHIKI, T., ITO, K. and KATO, S. (1979), Application of Cross-Hole to the Evaluation of Soil Improvement, *Fourteenth Annual Convention of Japanese Society of Soil Mechanics and Foundation Engineering* (in Japanese), pp. 541-544.

MORI

- OHASHI, A., IWASAKI, T., TATSUOKA, F. and MIYATA, K. (1976), Investigation of Earthquake Resistance along Shore Line Highway of Tokyo Bay, *Public Works Research Institute*, Data No. 1170, Ministry of Construction (in Japanese).
- OSAKI (1959), *Soil Map of Tokyo*, Gihodo Co., pp. 18.
- SEED, H. B. and LEE, K. L. (1966), Liquefaction of Saturated Sand during Cyclic Loading, *Journal of the Soil Mechanics Division*, American Society of Civil Engineers, Vol. 92 SM-6, pp. 105-134.
- SOWERS, G.F. (1979). Application of Results of Exploratory Borings and Index properties to Soil Engineering Problems, *Proceedings of the International Symposium on Soil Mechanics*, Oaxaca, Mexico, Vol. I, pp. 3-16.
- TAKENAKA, J. (1965), 'Method to Correct N-value for Deep Layer of Sandy Gravel' *Journal of Japanese Society of Soil Mechanics and Foundation Engineering* (in Japanese), Vol.13, No.2, pp.34-39.
- TAVENAS, F.A., BLANCHETTE, G., LEROUEIL, S., ROYARD, M. and ROCHELLE, P.L.A. (1975), Difficulties in the In-Situ Determination of  $K_o$  in Soft Sensitive Clays, *Proceedings of the Conference on In-situ Measurement of Soil Properties*, Raleigh, North Carolina, American Society of Civil Engineers, Vol.1, pp. 450-476.
- TERZAGHI and PECK (1948), *Soil Mechanics in Engineering Practice*, John Wiley, N. Y. New York.
- UTO, T. and FUYUKI, M. (1977a), Problems in the SPT from View Point of Penetration Mechanics, *Twelveth Annual Convention of Japanese Society of Soil Mechanics and Foundation Engineering*, pp. 37-40.
- UTO, K. and FUKUKI, M. (1977b), An Experiment on Dynamic Penetration Test, Congress of Kanto Branch, *Japanese Society of Civil Engineers* (in Japanese), pp. 111-112.
- WINELAND, J.O. (1975). Borehole Shear Device, In-Situ Measurement of Soil Properties, *North Carolina Specialty Conference of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 1, pp. 511-522.

## THE INFLUENCE OF FABRIC ON THE SHRINKAGE LIMIT OF CLAY

S. NARASIMHA RAO

### SYNOPSIS

This investigation brings out that the shrinkage limit of clays can also be used to identify the fabric. The previous attempts to use shrinkage limit in characterizing the fabric in clays have been reviewed. The experimental results obtained from kaolinite reveal that the clay mineral and the particle size distribution being the same, the shrinkage limit has been affected by the changes in the fabric brought about by the changes in the initial water content, induced shearing strains, drainage conditions during the application of shear stress and stress level. Several triaxial shear tests under both drained and undrained conditions on kaolinitic and montmorillonitic clays have also been conducted and shrinkage limits of these sheared samples have been determined. These strength results show the possibility of arriving at a unique relationship between effective angle of shearing resistance  $\phi'$ , and shrinkage limit.

### INTRODUCTION

The arrangement of soil particles influences the response of the soil to an external set of constraints. The term 'fabric' is generally used to denote the geometrical arrangement of the constituent mineral particles, including the void spaces (YONG & WARKENTIN, 1975; MITCHELL, 1976). The shape and size of the particles in a particular type of clay control the general arrangement of particles under the specified environmental conditions. The mechanism by which soil fabric is finally realized dictates the arrangement of particles. The influence of soil fabric on the three basic engineering properties, viz., strength, compressibility and permeability, has been reported in the literature by several investigators. It is an observed phenomenon that the soil fabric is affected by the initial moulding water content, type of consolidation, stress level, induced shear strains and the type of disturbance during the preparation of the sample. Some indirect methods based on the measurement of certain soil properties such as shrinkage, strength, and hydraulic conductivity, have come into practice to characterize soil fabric. Instead of using a single property, two or three properties can be used in combination for the measurement of soil fabric. In view of the small size of the particles involved in clays, the direct methods require a very involved procedure of specimen preparation and instrumentation involving a considerable investment. In this paper an

---

Lecturer, Department of Civil Engineering, Indian Institute of Technology, Madras, 600 036, India.

attempt has been made to trace the variation of shrinkage limit with the initial water content, the shear strains induced and the consolidation pressure. The possibility of uniquely relating shrinkage limit with the average angle of shearing resistance ( $\phi'$ ) in clays has also been brought out.

BRIEF REVIEW OF EARLIER WORK ON THE USE OF SHRINKAGE  
FOR IDENTIFYING SOIL FABRIC

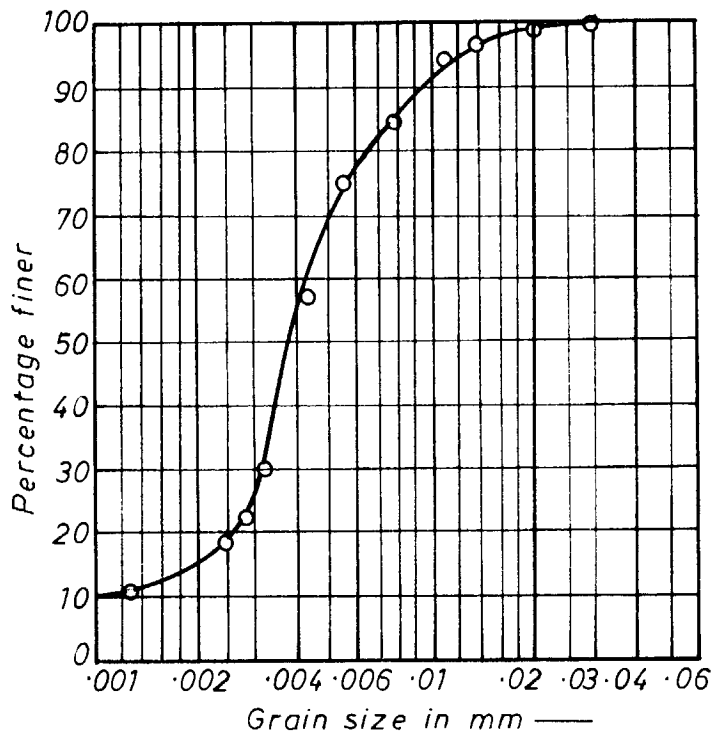
Though shrinkage has quite often been used as an index in classifying soils, it has also been used as an index to measure soil fabric. The work of HAINES (1923), which presented the concepts of shrinkage, led several others to formulate the basic mechanisms involved in the shrinkage phenomenon. But mostly, the work of LAMBE (1958) has brought out the usefulness of shrinkage limit in measuring the soil fabric. Apart from LAMBE (1958), several others have also brought out its usefulness. The work of MELLER (1932) and WILLIAMSON (1947, 1954) shows that a greater degree of shrinkage could occur in a direction perpendicular to the broad surfaces and that the differential shrinkage has been correlated with particle orientation. The work of LAMBE (1958), SEED & CHAN (1959) and SEED, et al (1960) made use of shrinkage to characterize the particle orientation in compacted clays. Their studies show that the more nearly parallel the particles are, the greater the shrinkage of the soil upon drying. A more random arrangement of soil particles results in a higher value of the shrinkage limit and a more parallel arrangement of particles reduces the shrinkage limit. Their linear shrinkage measurements also show a similar trend as that of MELLER (1932) and WILLIAMSON (1947, 1954). The results of WARKENTIN & BOZOZUK (1961) indicate that where there is a probable breakdown of the edge-to-face particle arrangement during remoulding to a more parallel arrangement, the value of shrinkage limit has been reduced significantly. The reported results of YONG & WARKENTIN (1975) obtained for a marine clay suggest that the shrinkage limit values are lower, by about 30%, for remoulded samples in comparison to the values of undisturbed samples.

From the above review it is clear that when the clay mineralogy and the particle size distribution remain constant, the shrinkage limit has been affected by the changes in fabric brought about by the changes in water content, remoulding, stress history, induced shear strains etc. Hence an attempt has been made in this paper to study the variation in shrinkage limit brought about by the changes in initial water content, shearing strains and consolidation pressure. The strength parameter  $\phi'$ , measured at the respective fabric conditions has been related to the shrinkage limit.

**SHRINKAGE LIMIT**  
**EXPERIMENTAL WORK**

*Materials Tested*

The present investigation was carried out on commercially available pure kaolinite clay (LL = 49%, PL = 29%; specific gravity = 2.57). The grain size distribution curve obtained for this clay is shown in Figure 1. A few test results obtained from some montmorillonitic clays by the author (RAO, 1973) are also made use of to examine the possibility of relating shrinkage limit with the effective angle of shearing resistance.



**Fig. 1. The particle size distribution curve for kaolinite clay.**

*Procedure for Measuring the Shrinkage Limit*

Two procedures have been followed to measure the shrinkage limit. In the first series, the shrinkage limit tests have been conducted on the thoroughly remoulded kaolinite clay slurry. About 50 gm of air-dried soil passing through B.S. sieve No. 40 was used for each test and thoroughly mixed with a sufficient quantity of distilled water to make the soil pasty enough to be readily worked in the shrinkage dish without the inclusion of air bubbles. The shrinkage dish used was similar to the one described by the ASTM designation D427-61 (1964). The same procedure was adopted in filling up the dish and measuring

RAO

the initial volume and final volume. The volume changes are calculated from the weight of the mercury displaced. This procedure has been adopted by several others including HAINES (1923). In all these cases, the shrinkage limit arrived at is computed from an average of 3 sets of observations. By this method, it is possible to measure shrinkage limit to an accuracy of 1%. The initial placement water content was varied from 40% to 75% to study the influence of initial water content on shrinkage limit. The soil with an initial water content less than 40% could not be packed into the dish in the slurry form without the possible entry of air bubbles.

In the second series, the shrinkage tests were conducted on the soil specimens obtained from the sheared samples. After carrying out the triaxial shear tests on 1½ in. (3.8 cm) diameter x 3 in. (7.6 cm) high samples to the failure stage, the tests were stopped and the samples were carefully removed from the cell. Two specimens of about ¾ in. (1.9 cm) long x ½ in. (1.27 cm) wide x ⅜ in. (0.96 cm) thick were cut with a sharp blade from the failure plane zone in each of the sheared samples. After taking the initial measurement of the weight and volume, the samples were allowed to dry first in an air controlled chamber before placing them in an oven for drying at 105°C to 110°C for 24 hours. The entire drying process took about 15 days for montmorillonitic clays and only about 3 days for kaolinite. This slow process of drying was done to prevent any possible occurrence of hairline cracks. A set of shrinkage measurements were also conducted on kaolinite clay subjected to different magnitudes of vertical consolidation pressure under  $K_0$ -conditions in a conventional oedometer.

Triaxial shear compression tests were conducted on samples moulded at different initial water contents. Both consolidated undrained and drained triaxial compression tests were carried out. All the shear testing procedures correspond to the methods detailed by BISHOP & HENKEL (1962).

#### TEST RESULTS AND DISCUSSION

The results of shrinkage test conducted on kaolinite clay remoulded at different initial water contents are presented in Figure 2. It can be seen that the shrinkage limit decreases with increase in the initial water content in the range of 40 % to 60 %. Beyond 60 % there is no further reduction in the value of shrinkage limit. The increase in the water content leads to a change in the particle arrangement from a relatively random arrangement to a parallel arrangement. A more random arrangement increases the shrinkage limit; a more parallel arrangement decreases it (YONG & WARKENTIN, 1975).

### SHRINKAGE LIMIT

Hence, these changes in the shrinkage limit values with initial water content can be attributed to the changes in fabric.

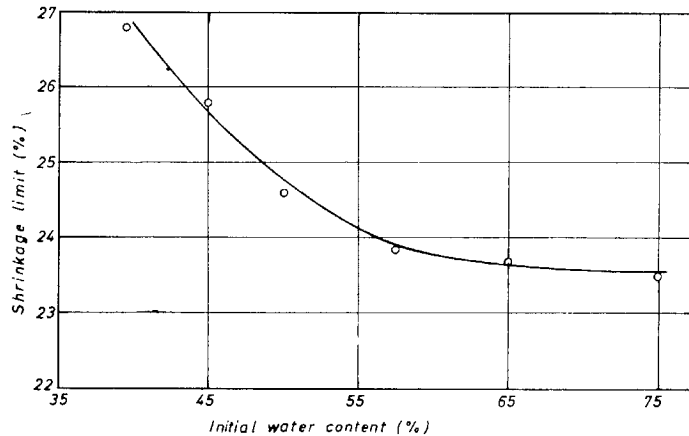


Fig. 2. The effect of initial water content on shrinkage limit in kaolinite clay.

Figures 3 and 4 show the results of shrinkage limit tests conducted on kaolinite clay specimens taken from samples sheared in triaxial tests. Figure 3 represents the data obtained from undrained shear tests conducted on two series of samples moulded at two different initial water contents. In both the series, the shrinkage limit decreases with increase in consolidation pressure. The isotropic consolidation as such may not bring about significant changes in fabric; but only the subsequent shearing strains induced during shear testing are expected to bring about the significant changes in soil fabric. During shearing, the particles around the failure plane are expected to align

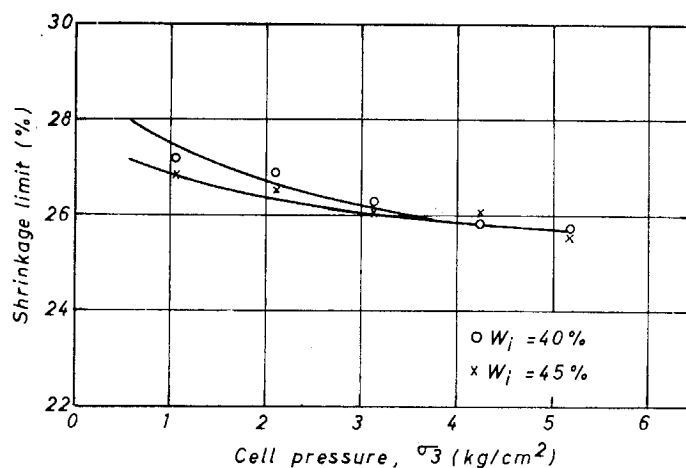


Fig. 3. The effect of consolidation (cell) pressure on shrinkage limit of samples sheared in undrained conditions in kaolinite clay.



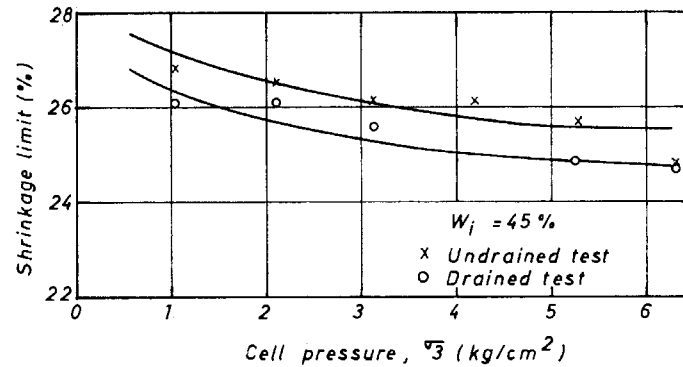


Fig. 4. Comparison between the values of shrinkage limit obtained from samples sheared in undrained and drained conditions in kaolinite clay.

themselves in a direction parallel to the failure plane (SEED et al, 1960; YONG & WARKENTIN, 1975). With increase in cell pressure (i.e., isotropic consolidation pressure), the shear stresses mobilised to cause failure are also increased and as such a greater degree of particle orientation with a consequent reduction in shrinkage limit can be expected with higher consolidation pressures. A sample moulded with higher initial water content gives lower values of shrinkage limit in the lower pressure range, but the difference in the values gradually decreases with increase in the consolidation pressure (Figure 3). Figure 4 compares the shrinkage limit values as obtained from samples sheared in both undrained and drained conditions. The samples for both the types of tests are moulded at the same initial water content. From the figure it can be seen that for the pressure ranges considered in this investigation the specimens taken from the samples sheared in drained conditions give less values of shrinkage limit. It may be mentioned that during the undrained triaxial shear tests an axial strain of 12-15% was required to reach the failure conditions as defined by the maximum deviatoric stress. In the case of drained tests the failure conditions were reached at an axial strain of 20-25%. This increase in the axial strain facilitates a more parallel oriented fabric in failure zone under drained conditions and hence lower values of shrinkage limit can be expected under these conditions (SEED et al, 1960).

Figure 5 shows the values of shrinkage limit obtained from kaolinite samples consolidated to different magnitudes of vertical pressure under  $K_0$ -conditions in a conventional oedometer. One-dimensional consolidation under  $K_0$ -conditions is known to cause a better orientation of particles. Hence shrinkage limit is expected to be diminished with consolidation pressure. The X-ray diffraction studies of QUIGLEY & THOMPSON (1966)

### SHRINKAGE LIMIT

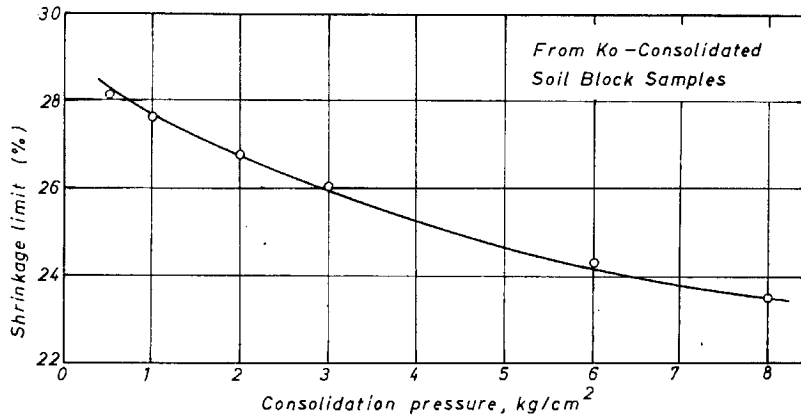


Fig. 5. Variation of shrinkage limit with consolidation pressure in kaolinite clay.

conducted on samples subjected to one-dimensional consolidation bring forth the parallel oriented fabric in the consolidated sediments.

In all three cases (Figure 3, 4 and 5) described above the shrinkage limit is observed to have decreased with consolidation pressure. The consolidation preceding shear, due to an all-round cell pressure, brings down the water content. The decrease in water content is known to increase the shrinkage limit (as established by Figure 2) under the normal circumstances, but this reduction in water content is brought about by an isotropic consolidation (all-round cell) pressure and this only brings the particles together without any change in the orientation of the particles. For the reasons stated earlier, the shear stresses of higher magnitude are mobilized at higher values of cell pressures, the effect of which is to result in a more parallel oriented fabric. The net effect of both reduction in moisture content and shear strains is only to result in a particle system tending to a parallel orientation and hence the reduction in shrinkage limit values are expected with increase in cell pressure.

From the above results and discussion it can be seen that shrinkage limit is not a constant for any soil; but is significantly influenced by fabric. The value of shrinkage limit decreases with increase in particle orientation. In general, it is observed that the particle orientation decreases the strength. If the angle of shearing resistance  $\phi'$ , obtained from a normally consolidated clay is considered, it is more for a random oriented system and less for a parallel oriented system. It may be mentioned that the angle of shearing resistance obtained is only an average value obtained from tests conducted on samples subjected to different consolidation pressures in the triaxial cell. The strength results obtained yield lower average value of  $\phi'$

and lower average values of shrinkage limit for the samples moulded at higher water contents. If the values of shrinkage limit obtained from sheared samples are considered, the average value of  $\phi'$  should increase with the value of shrinkage limit. To examine any possibility of relating  $\phi'$  with shrinkage limit, the results of kaolinite and montmorillonitic clays obtained from samples with different initial conditions are plotted in Figure 6. From the limited results presented in Figure 6, it can be observed that all the results fall on a single line without any noticeable scatter. With further work on several types of clays with wide variation in shrinkage limit values, the possibility of developing a unique relationship between  $\phi'$  and shrinkage limit can be examined. This brings out the possibility of predicting the strength parameter  $\phi'$ , from a relatively simple shrinkage limit test in saturated clays at any fabric conditions.

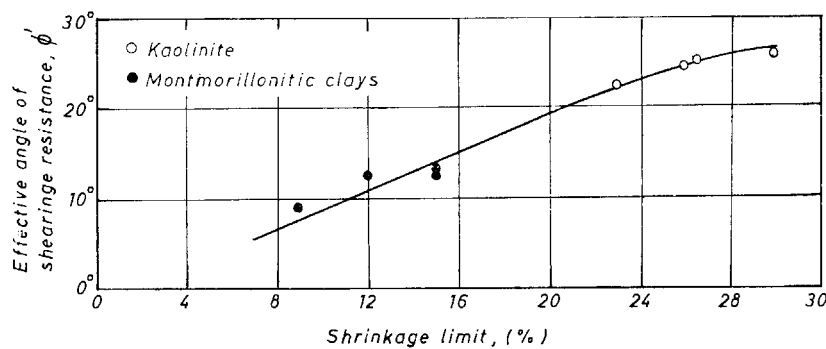


Fig. 6. Relationship between effective angle of shearing resistance and shrinkage limit for clays.

### CONCLUSIONS

This investigation shows that clay mineral and the particle size distribution being the same, the shrinkage limit has been affected by the changes in fabric brought about by the change in the initial water content, induced shearing strains and stress level.

This investigation also brings out the possibility of arriving at a unique relationship between effective angle of shearing resistance  $\phi'$ , and shrinkage limit.

### ACKNOWLEDGEMENTS

The author thanks Dr. K.S. Sankaran, Professor of Soil Engineering for his encouragement and timely help. The facilities extended by the Department of Civil Engineering, Indian Institute of Technology, Madras, are gratefully acknowledged.

## SHRINKAGE LIMIT

### REFERENCE

- AMERICAN SOCIETY FOR TESTING AND MATERIALS (1964), *Procedures for Testing Soils*, Designation D427-61.
- BISHOP, A.W. and HENKEL, D.J. (1962), *The Measurement of Soil Properties in the Triaxial Test*, 2nd ed., Edward Arnold and Co., London.
- HAINES, W.B. (1923), The Volume Changes Associated with Variations of Water Content in Soil, *Journal of Agricultural Science*, Vol. 13, pp. 296-310.
- LAMBE, T.W. (1958), The Structure of Compacted clay, *Journal of the Soil Mechanics Division*, American Society of Civil Engineers Vol. 84, SM-2, pp. 1-34.
- MELLER J. (1932), Some Notes on the Shrinkage of Clays During Drying, *Transactions of Ceramic Society*, Vol. 32. pp. 455-471.
- MITCHELL, J.K. (1976), *Fundamentals of Soil Behaviour*, John Wiley and Sons, Inc., New York.
- QUIGLEY, R.M. and THOMPSON, C.D. (1966), The Fabric of Anisotropically Consolidated Sensitive Marine Clay, *Canadian Geotechnical Journal*, Vol. 3, pp. 61-73.
- RAO, S.N. (1973), Strength Behaviour Including Stress-Strain Relationships of Clays, *Ph.D. Thesis*, Indian Institute of Science, Bangalore.
- SEED, H.B. and CHAN, C.K. (1959), Structure and Strength Characteristics of Compacted Clays, *Journal of the Soil Mechanics Division*, American Society of Civil Engineers, Vol. 85, SM-5, 87-128.
- SEED, H.B., MITCHELL, J.K. and CHAN, C.K. (1960), The Strength of Compacted Cohesive Soils, *Proceedings of the Research Conference on Shear Strength of Cohesive Soils*, American Society of Civil Engineers, Colorado, pp. 877-964.
- WARKENTIN, B.P. and BOZOUK, M. (1961), Shrinking and Swelling Properties of Two Canadian Clays, *Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering*, Paris, Vol. 1, pp. 851-855.
- WILLIAMSON, W.O. (1947), The Fabric, Water-Distribution, Drying-Shrinkage, and Porosity of Some Shaped Discs of Clay, *American Journal of Science*, Vol. 245, pp. 645-662.
- WILLIAMSON, W.O., (1954), The Effect of Rotational Rolling on the Fabric and Drying Shrinkage of Clay, *American Journal of Science*, Vol. 252, pp. 129-143.
- YONG, R.N. and WARKENTIN, B.P. (1975), *Soil Properties and Behaviour*, Elsevier Scientific Publishing Co., Amsterdam.

## DYNAMIC RESPONSE OF FOOTINGS IN A SATURATED SOIL MEDIUM

K.S. SANKARAN\*, N.R. KRISHNASWAMY\*\* and  
P.G. BHASKARAN NAIR†

### INTRODUCTION

There has been very little experimental study on the influence of water on the dynamics of embedded foundations. BARKAN (1962) investigated the effect of water on the dynamic response of footings subjected to vertical mode of vibrations. He observed an increase of damping for backfilled condition as well as for the case of flooding the sides with water compared to the condition where foundation sides were exposed. PATERSON (1955) observed a decrease of compression wave velocity in saturated sand compared to that in dry sand. BIOT (1956) considered a general three-dimensional propagation of both compression and shear waves in a fluid saturated porous medium. His theory points out the strong influence of the structural coupling involved in the compression waves and the lack of coupling for the shear waves. HARDIN (1961) has shown that the velocity of compression wave in water of a saturated medium is higher than that in water and the velocity in the elastic structure is slightly lower than that in dry condition. HARDIN & RICHART (1963) have observed that the presence of moisture in a sand slightly reduces the velocity of shear wave propagation. KRISHNASWAMY & ANANDAKRISHNAN (1975) studied the influence of soil moisture on the vertical response of a footing and correlated the results with those available from Biot's 1956 theory.

### EXPERIMENTAL INVESTIGATIONS

Field tests to investigate the influence of soil moisture on the steady-state torsional response of embedded foundations have not been reported in literature.

Field vibratory tests were carried out with the aim of collecting valuable data with regard to the following aspects in the vertical and torsional modes of vibration.

- (1) Embedded footing with the sides exposed.
- (2) Backfilling the sides with dry sand.

---

\*Professor, \*\*Lecturer, †Research Scholar, Civil Engineering Department, Indian Institute of Technology, Madras, India.

(3) Footing sides not backfilled, but flooded with water.

(4) Backfilling the sides with saturated sand.

The soil at the test site at the Indian Institute of Technology, Madras, is a silty sand with some clay (Table 1). The average water content and bulk density of the soil are 11 % and 1970 kg/m<sup>3</sup> respectively. The investigations were made in a test pit of size 1.60 × 1.60 × 0.50 m. The lateral dimensions of the pit were more than 2.25 times the corresponding dimensions of the footing, so that reflections of elastic waves from the sides of the pit were insignificant as reported by SANKARAN, et al (1978a).

**Table 1. Soil Particulars.**

| Particle Size<br>(mm) | Percentage finer                     |                                |
|-----------------------|--------------------------------------|--------------------------------|
|                       | A. Soil at Test Site<br>(silty sand) | B. Backfill Material<br>(sand) |
| 4.0                   | 100.0                                | 99.0                           |
| 2.0                   | 100.0                                | 99.0                           |
| 0.6                   | 71.5                                 | 56.0                           |
| 0.2                   | 38.0                                 | 7.0                            |
| 0.06                  | 17.5                                 | --                             |
| 0.02                  | 10.0                                 | --                             |
| 0.006                 | 6.0                                  | --                             |
| 0.002                 | 3.0                                  | --                             |

Void ratio of A = 0.50

Void ratio of B = 0.66

A Lazan type mechanical vibrator of weight 40.6 kg was used. The eccentric masses positioned at 8.3 cm from the vertical axis had a total weight of 2.98 kg and an eccentricity of 3.89 cm. The overall weight was increased by adding over the vibrator six cast iron plates of total weight equal to 580 kg. The vibrator was run by a 5 H.P. motor and the two were connected through a flexible shaft.

Electrodynamic vibration pick-ups of natural frequency 13 cps were fixed to the wooden brackets which in turn were screwed down to wooden plugs embedded in the footings. The pick-ups were used in conjunction with the amplitude measuring apparatus. With this combination, it was possible to measure the vibration amplitudes. An electronic speed indicating tacheometer was used to indicate the spot speeds of the revolving shafts.

Two concrete footings of sizes 50 × 50 × 50 cm (Base 1) and 60 × 60 × 45 cm (Base 2) were used in this study. For all tests the bases were placed centrally

#### TECHNICAL NOTE ON FOUNDATION DYNAMICS

at the bottom of the pit. The footing, the base plate, the vibrator and the additional weights were firmly fixed together, ensuring that their centres of gravity were located in the same vertical line. Backfilling was done by using clean dry river sand (Table 1) and compacting it to obtain a uniform bulk density of  $1600 \text{ kg/m}^3$ . Flooding of the test pit was done by a continuous supply of water for a very long time so as to achieve a condition almost equal to rising of the water table in the neighbourhood of the test footing. Backfilling with saturated sand was achieved by keeping dry sand backfill and saturating it with a continuous supply of water for a long time.

Vertical and torsional vibration tests were conducted on the two footings to study the four aspects listed previously. The frequency of vibration was gradually increased and the amplitude was measured in each case.

#### VERTICAL VIBRATION TEST RESULTS

Response curves for the vertical vibration tests are presented in Figures 1 and 2. Curves 1 and 2 indicate a remarkable decrease in the resonant amplitude and increase in the resonant frequency for the backfilled case. This is due to the increased values of damping and stiffness in the presence of the backfill. Curves 1 and 3 signify an appreciable reduction in amplitude (22% and 24% for the two bases) and a slight decrease in the resonant frequency for the flooding case compared to that of exposed sides. The reduction in amplitude is due to the increase in damping at the sides through water. The reduction in frequency is due to the reduction in stiffness of the soil in the presence of moisture.

Comparison of curves 2 and 3 reveals that the amplitude has a little increase in the case of water around the footing compared to the case of dry backfill. This is because the energy transmission in water is only through compression waves and that in the soil is through compression and shear waves. It is observed, from a comparison of curves 2 and 4, that the saturated sand backfill has not yielded any significant difference in amplitude from the dry backfill case, indicating nearly the same damping in both cases. This is probably because the additional damping through water and the reduction in damping in the soil structure of the saturated soil have had compensating effects. The reduction in frequency for the saturated case is due to the reduction in stiffness in the presence of water. An observation of curves 3 and 4 shows that the saturated backfill yields a lower amplitude than the case of sides flooded with water. The lower amplitude is due to the higher damping through soil and water on the sides compared to the damping in water alone in the other case.

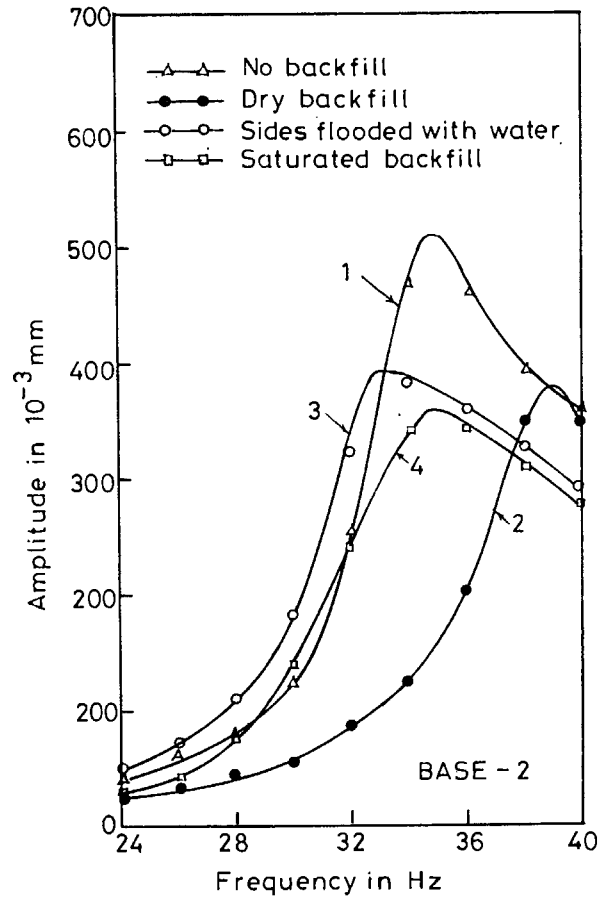
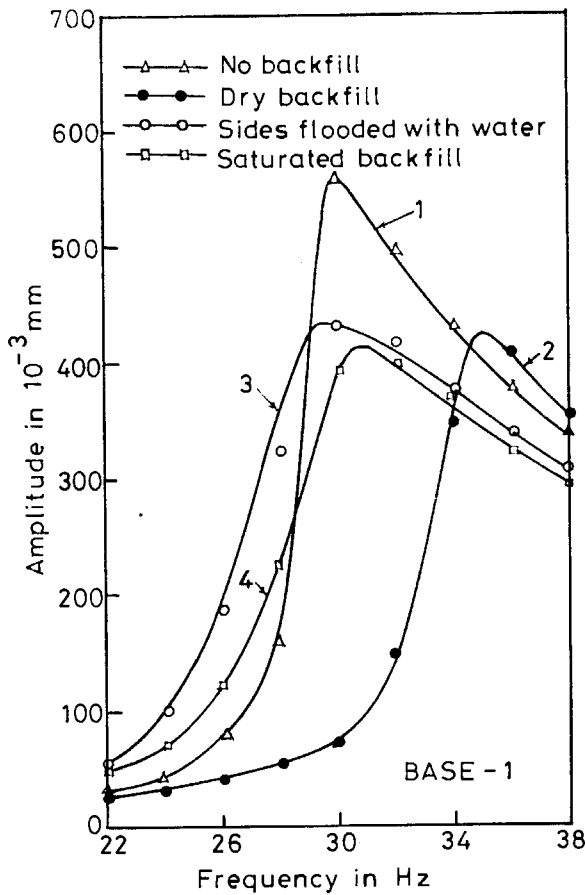


Fig. 1. Response curves for vertical vibration tests—Base 1.

Fig. 2. Response curves for vertical vibration tests—Base 2.

TORSIONAL VIBRATION TEST RESULTS

Response curves for the torsional vibration tests are presented in Figures 3 and 4, from which it is seen that backfilling has a remarkable influence on the torsional response of embedded footings. This has been illustrated in detail by SANKARAN, et al (1978b, 1979) with regard to footings of various sizes and shapes. Curves 1 and 3 indicate that there is only a slight reduction in resonant amplitude (5% and 8%) for the flooding condition compared to the exposed condition. This is because in the torsional vibration mode the energy is dissipated through shear waves only (REISSNER & SAGOCI, 1944) and water cannot transmit shear waves.

Comparison of curves 2 and 3 shows a higher amplitude for the flooding case than for backfill of dry sand. This is due to the absence of energy dissipa-



TECHNICAL NOTE ON FOUNDATION DYNAMICS

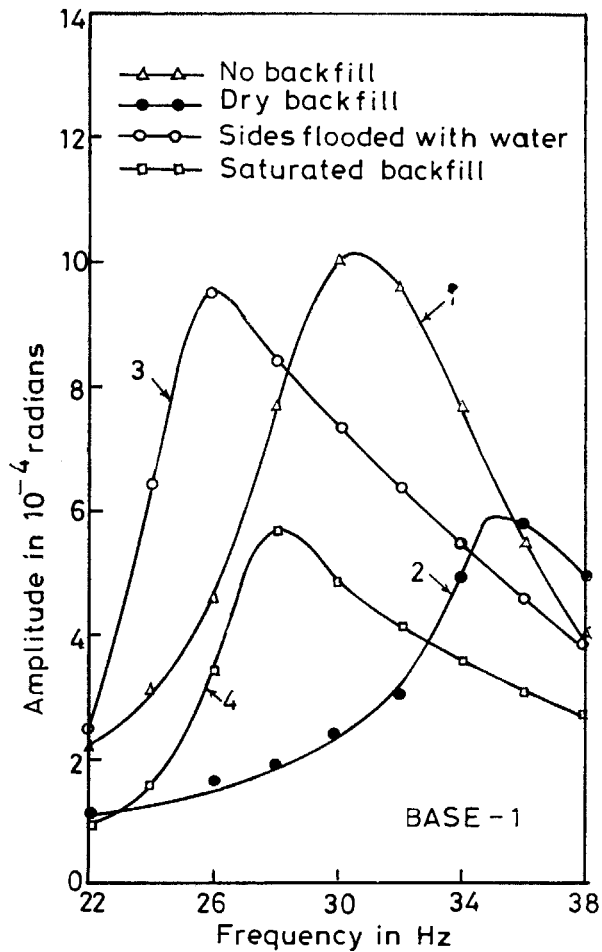


Fig. 3. Response curves for torsional vibration tests—Base 1.

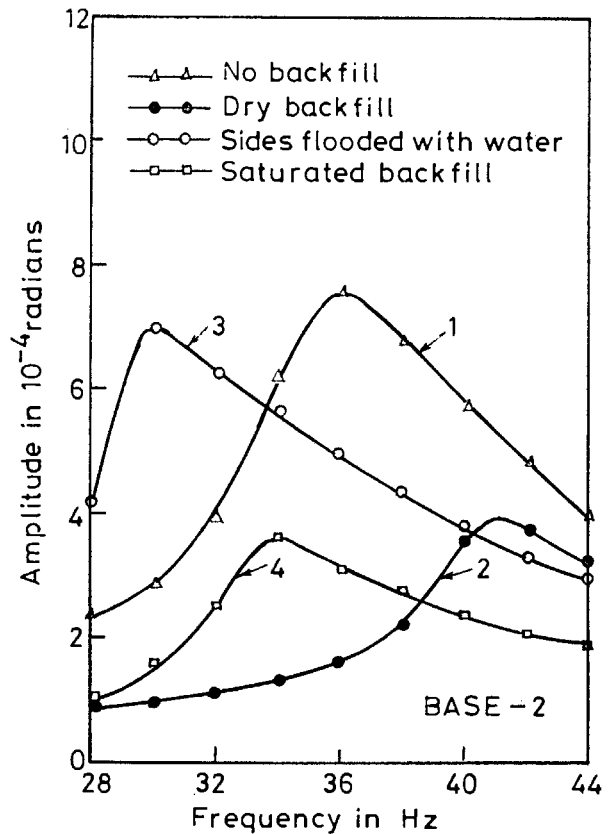


Fig. 4. Response curves for torsional vibration tests—Base 2.

tion in water at the sides. Comparison of curves 2 and 4 indicates nearly the same values for the amplitude, illustrating that shear wave velocities are insensitive to pore water.

There are no experimental results reported in literature to compare with the authors' results on the influence of soil moisture on torsional vibrations.

CONCLUSIONS

An extensive field investigation programme on the vertical and torsional response of two embedded footings was carried out to study the effect of soil moisture. The presence of moisture reduces soil stiffness and consequently the resonant frequency. Flooding of the sides of the foundation reduced

SANKARAN, KRISHNASWAMY & BHASKARAN NAIR

vertical vibration amplitudes by 22 and 24% while the corresponding reductions were only 5 and 8 % for torsional vibration amplitudes. The dry and saturated backfill did not show appreciable difference in the resonant amplitudes in both the modes.

REFERENCES

- BARKAN, D.D. (1962), *Dynamics of Bases and Foundations*, McGraw Hill Book Co., New York.
- BIOT, M.A. (1956), Theory of Propagation of Elastic Waves in a Fluid-Saturated Porous Solid, *Journal of the Acoustical Society of America*, Vol. 28, pp. 168-191.
- HARDIN, B.O. (1961), Study of Elastic Wave Propagation and Damping in Granular Materials, *Ph.D. Thesis*, University of Florida.
- HARDIN, B.O. and RICHART, F.E., Jr. (1963), Elastic Wave Velocities in Granular Soils, *Journal of the Soil Mechanics and Foundations Division*, American Society of Civil Engineers, Vol. 89, SM-1, pp. 33-65.
- KRISHNASWAMY, N.R. and ANANDAKRISHNAN, M. (1975), Influence of Soil Moisture on Footing Vibrations, *Proceedings of the Fifth Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Vol.1, pp. 295-298.
- PATERSON, N.R. (1955), Elastic Wave Propagation in Granular Media, *Ph.D. Thesis*, Department of Physics, University of Toronto.
- REISSNER, E. and SAGOCCI, H.F. (1944), Forced Torsional Oscillations of an Elastic Half-Space, *Journal of Applied Physics*, Vol. 15, pp. 652-662.
- SANKARAN, K.S., KRISHNASWAMY, N.R. and BHASKARAN NAIR, P.G. (1978a), Influence of Trench Size on Dynamic Response of Footings, *Research Report No.2*, Soil Engineering Laboratory, Indian Institute of Technology, Madras.
- SANKARAN, K.S., KRISHNASWAMY, N.R. and BHASKARAN NAIR, P.G. (1978b), Rotational Vibrations of Embedded Foundations, *Proceedings of the Sixth Symposium on Earthquake Engineering*, University of Roorkee, Vol. 1, pp. 219-224.
- SANKARAN, K.S., KRISHNASWAMY, N.R. and BHASKARAN NAIR, P.G. (1979), Response of Embedded Foundations to Torsional Vibrations, accepted for presentation at the Seventh Canadian Congress of Applied Mechanics (CANCAM 79).

---

## BOOK REVIEWS

---

**Terrain Analysis**, 2nd edition, by Douglas S. Way, McGraw-Hill Book Co., 1221 Avenue of the Americas, New York, N. Y. 10010, U.S.A., 1978, 438 pp., about US \$40.00.

Professor Way, of the Graduate School of Design, Harvard University, has produced a revision of his 1973 text **Terrain Analysis, A Guide to Site Selection Using Aerial Photographic Interpretation**. The author states in the Preface that the text's goals are two-fold. The first is to establish a systematic approach to terrain analysis which planners and other professionals could follow in order to achieve an understanding of land surface and subsurface conditions. The second goal is to provide a reference for planners, architects, and other professionals or students who are not specialists in geology or geomorphology but who wish to learn an interdisciplinary approach to site investigation. This reviewer considers that the first goal has been achieved, the text being the best available work on air photo interpretation applied to site investigation. As such it is highly recommended for engineering geologists and students of geology, geomorphology, and civil engineering. Whether or not the second goal is reached is questionable, as planners and architects could have difficulty with many of the terms and concepts expressed unless they refer to standard introductory geology texts. However, the geological topics that are included are stated in a very clear fashion.

There are twelve chapters, a glossary, and five appendices in this book. The chapters can be separated into three sections: remote sensing and landform interpretation (three chapters); landforms and issues of site development (eight chapters); and case studies (one chapter).

The first chapter, Remote Sensing, will be a disappointment to anyone who has seen F. Sabins' text, 'Remote Sensing, Principles and Interpretation' (1978). At most, this chapter is of value only to someone who has little or no knowledge of aerial photography, non-photographic sensors, and the current space program. There is nothing described in this chapter to enable an interested person to manipulate air photos or Landsat imagery. The information on the Landsat return beam vidicon (RBV) system is outdated, if not erroneous with its castigation of the usefulness of this system. Contrary to what the author states, RBV imagery presently available is surpassing expectations with its clarity and detail. Engineering geomorphological mapping techniques have achieved new impetus from the availability of the RBV data. The multispectral concept of Landsat applications is also

poorly presented. Radar sensors and thermal infrared scanners are briefly passed over and the reader is left ignorant of recent successful applications of these systems to engineering geological mapping.

Chapter 2, Landforms and Aerial Photographic Terrain Analysis, is the start of the principal contribution of the text; the explanation of the methodology of site analysis by remotely sensed data. Physical site factors; of geology, soils, water, vegetation, and minerals, are analysed through systematic visual examination of landscape units. There are two common approaches of terrain analysis that are introduced in this chapter. The Australian "terrain unit" parametric scheme is based on the definition of an area by its topographic slope, structure, lithology, soils, vegetation, and hydrologic characteristics. The American "landform approach" is based on the identification of specific landforms that then lead to interpretations of physical properties and soil types. This latter scheme is used in this text.

After this brief introduction to the philosophy of analysis and a chapter on air photo interpretation of soils, the text reviews the general requirements of the following engineering and construction categories: sewage and solid waste disposal, trenching, excavation and grading, dewatering, construction materials, soil compaction, groundwater supply, pond or lake construction, foundations, and highway construction.

Subsequent chapters, forming about two-thirds of the text, are superbly presented. The genesis and classification of sedimentary, igneous, and metamorphic rocks; and glacial, eolian, and fluvial landforms are first outlined. Then specific rocks and landform types are described and discussed. Characteristic features of recognition in both humid and arid climates are given along with remarks on soil types to be expected, location of the water table, drainage, and soil depth to bedrock. Conclusions, recommendations on procedures, and potential problems for each of the issues of site development follow. The outstanding feature of these descriptive chapters is the synthesis of stereograms, topographic maps, block diagrams, characteristic soil profiles, drainage patterns, and topographic profiles developed on specific rock types or landforms in both humid and arid climates.

The chapter on case studies consists of five very brief project summaries complemented with photographs and small maps. These studies are so brief that they serve little purpose to someone wanting to know more about practical applications of Professor Way's terrain analysis methods. This chapter contains no references.

The Appendices include: Soil Classification Systems, Data and Aerial Photographic Sources, Published Soil Surveys (U.S.), State Geologic Maps'

and Additional Maps. Apart from the first appendix all are relevant only to persons working in the U.S.

If one already has the 1st edition of this book little would be gained in replacing it with the 2nd edition. Apart from the additional section on the two current terrain analysis approaches, not much of consequence has been added to the 1973 text. In the descriptive core of the book, the chapters on landforms and issues of site development, the only revision this reviewer noted was a change in one block diagram.

The book (either edition) is highly recommended, for students of air photographic interpretation and for professionals looking for a method of applying terrain analysis to site investigation.

*K.V. Campbell*

**Methods of Geological Engineering in Discontinuous Rocks**, by Richard E. Goodman, West Publishing Company, P.O. Box 3526, St Paul, MN 55165, U.S.A., 1976, 472 pp, US \$27.95.

This text contains material from Professor Goodman's courses in engineering geology and applied rock mechanics at Berkeley. The topic of the text, structural discontinuities of rocks, is considered from an engineering geological viewpoint and is an excellent meld of both civil engineering quantifying practices and subjective geological observations.

The text is meant to serve as a supplementary source for engineering geology and applied rock mechanics courses, as well as a text for geological engineering analysis. It discusses analytical methods and procedures to assess the influence of discontinuities on the behavior of rocks in engineering applications. Such discontinuity analyses are a prerequisite to the design of a stable rock face, as it is the presence and character of structural discontinuities in a rock mass that largely determine the physical and mechanical properties of that mass. Chapter topics include rock classification, principles of stereographic projection and joint surveys, exploration of rock conditions, mechanical properties of discontinuities, applications of stereographic projection in mechanics of discontinuous rocks, physical models, and the finite element method.

There are several highlights of the text that will make it a useful reference for students of engineering geology. These include the following: a review of the degree of fissuring and its relation to compressibility, wave propagation, ratio of strengths determined by direct and indirect tension tests, and radial permeability tests; an explanation of the method and an application of

terrestrial photogrammetry using a conventional 35 mm camera; drilling methods and techniques of orientating structures in a drill core; laboratory and *in situ* testing of joint strength properties; use of kinematic and physically scaled models to determine modes of failure; and a thorough explanation of the finite element method as applied to displacements in heterogenous or discontinuous rock masses. A computer program accompanies this explanation.

The text has numerous up-to-date (1976) references and is written with a clear, no nonsense style. The line drawings and photographs are generally good, but this reviewer's students found some of the figure notations and explanation of stereographic projection procedures to be ambiguous. A student without experience in stereographic projections would have to refer to other standard works for complete comprehension of this subject.

Those topics, listed above as highlights of the book, are not described as well or as concisely in any other one text on engineering geology. This reviewer recommends **Methods of Geological Engineering** as a supplementary reference to all who are interested in engineering problems in discontinuous rocks.

*K.V. Campbell*

---

## **Book Received**

---

**What Every Engineer Should Know About Patents**, by William G. Konold Bruce Tittel, Donald F. Frei, and David S. Stallard, Marcel Dekker, Inc., 270 Madison Avenue, New York, N. Y. 10016, 1979, 124 pp., SFr. 24.00

Written to take much of the mystery out of intellectual property law **What Every Engineer Should Know About Patents** discusses patents, trade secrets, copyrights, and trademarks.

Written by experts in the field of intellectual property law, the information contained in this book is intended to help the reader take the proper steps to avoid a loss of rights and establish valid rights in inventions, trademarks, and writings, and business information generally. In addition, the fund of knowledge contained in this book should enable the reader to ask informed questions and to communicate more effectively with a patent lawyer when the need arises saving both the patent lawyer and the client much valuable time and expense.

Students and teachers of engineering, practicing engineers, patent liaison persons in corporate engineering departments, investors, general attorneys, and businessmen will find in **What Every Engineer Should Know About Patents** the information necessary to resolve many of the problems they may encounter.

*(from accompanying summary)*

---

## CONFERENCE NEWS

---

**International Symposium on Soils Under Cyclic and Transient Loading**, Swansea, U.K., January 7-11, 1980. All enquiries to: Dr. G.N. Pande, Dept. of Civil Engineering, University College of Swansea, Swansea SA2 8PP, U.K.

**Fourth African Highway Conference**, Nairobi, Kenya, January 20-25, 1980. All enquiries to: International Road Federation, 1023 Washington Building, Washington, D.C. 20005, U.S.A.

**Third International Symposium on Dredging Technology**, Bordeaux, France, March 5-7, 1980. All enquiries to: Organizing Secretary, BHRA Fluid Engineering, Cranfield, Bedford MK34 OAJ, U.K.

**Seventeenth International Coastal Engineering Conference**, Sydney, Australia, March 23-27, 1980. All enquiries to: Secretary, Coastal Engineering Research Council, 412 O'Brien Hall, University of California, Berkeley, CA 94720, U.S.A.

**Second International Seminar on Recent Advances in Boundary Element Methods**, Southampton, U.K., March 25-27, 1980. All enquiries to: Dr. C. Brebbia, Seminar Director, Dept. of Civil Engineering, University of Southampton, Southampton S09 5NH, U.K.

**International Symposium on Landslides**, New Delhi, India, April 7-11, 1980. All enquiries to: The Organizing Secretary, International Symposium on Landslides, P.O. Central Road Research Institute, New Delhi 110 020, India.

**Second Conference on Ground Movements and Structures**, Cardiff, U.K., April 14-17, 1980. All enquiries to: Conference Secretary, Dept. of Civil Engineering and Building Technology, UWIST, King Edward VII Avenue, Cardiff, CF1 3NV, South Glamorgan, U.K.

**Symposium on Appropriate Technology in Civil Engineering**, London, U.K., April 15-17, 1980. All enquiries to: Secretary (Appropriate Technology in Civil Engineering), Institution of Civil Engineers, 1-7 Great George St., London SW1P 3AA, U.K.

**International Conference in Compaction**, Paris, France, April 22-24, 1980. All enquiries to: Colloque (Compactage), Ecole Nationale des Ponts et Chaussées, Direction de la Formation Continue, 28 Rue des Saints-Peres, 75007 Paris, France.



**Conference on Structural Foundations on Rock**, Sydney, Australia, May 7-9, 1980. All enquiries to: Conference Manager, Structural Foundations on Rock 1980, The Institution of Engineers, Australia, 11 National Circuit, Barton, ACT 2600, Australia.

**Third Australia - New Zealand Geomechanics Conference**, Wellington, New Zealand, May 12-16, 1980. All enquiries to: Organizing Secretary, The 3rd Australia-New Zealand Geomechanics Conference, P.O. Box 243, Wellington, New Zealand.

**Sixth Southeast Asian Conference on Soil Engineering**, Taipei, Taiwan, R.O.C. May 19-23, 1980. All enquiries to: Secretary General, Organizing Committee, 6th SEACSE, c/o Moh and Associates, 6-1, Lane 137, Yen Chi Street Taipei, Taiwan, Republic of China.

**Conference on Safety of Underground Works under Construction and in Service**, Brussels, Belgium, May 19-23, 1980. All enquiries to: Center International de Conferences de Bruxelles, Parc des Expositions, B-1020 Bruxelles, Belgium.

**Seventh African Regional Conference**, Accra, Ghana, June 1-6, 1980. All enquiries to Secretary, Ghana Geotechnical Society, c/o Building and Road Research Institute, P.O. Box 40, Kumasi, Ghana.

**International Seminar on the Application of Stress Wave Theory on Piles**, Stockholm, Sweden, June 4-5, 1980. All enquiries to: Royal Institute of Technology KTH, Dept of Soil and Rock Mechanics, Seminar on Stress wave Theory on Piles, Fack, 10044 Stockholm, Sweden.

**Fourth International Conference on Expansive Soils**, Denver, U.S.A., June 16-18, 1980. All enquiries to: Dr. Donald R. Snethen, U.S. Army Engineer Waterways Experiment Station, Attn: WESGE, P.O. Box 631, Vicksburg, MS 39180, U.S.A.

**Second International Symposium on Innovative Numerical Analysis in Applied Engineering Science**, Montreal, Canada, June 16-19, 1980. All enquiries to: Prof. A.A. Lakis, Dept. of Mechanical Engineering, Ecole Polytechnique de Montreal, C.P. 6079, Station A, Montreal, Quebec H3C 3A7, Canada.

**International Symposium on Sub-surface Space**, Stockholm, Sweden, June 23-27, 1980. All enquiries to: Rockstore 80, c/o Stockholm Convention Bureau, Jakob Torg 3, S-111 52 Stockholm, Sweden.

**Seventh World Conference on Earthquake Engineering**, Istanbul, Turkey, September 8-13, 1980. All enquiries to: Organizing Committee, 7 WCEE, Deprem Arastirma Enstitusu, Yuksel Caddesi, 7/B, Ankara, Turkey.

**Sixth Danube European Conference on Soil Mechanics and Foundation Engineering**, Varna, Bulgaria, September, 1980. All enquiries to: Secretary, 6th Danube-European Conference SMFE, Scientific-Technical Unions in Bulgaria, 108 Rakovski St., 1000 Sofia, Bulgaria.

**Underground Space Conference and Exposition**, Kansas City, U.S.A., June 9-12, 1981. All enquiries to: Exposition Director, National Exposition Service, Inc., 2461 East Grand Blvd., Detroit, Michigan 48211, U.S.A.

**Tenth International Conference of the ISSMFE**, June 15-19, 1981. All enquiries to: Secretary General X ICSMFE, Jakobs Torg 3, S-111 52 Stockholm, Sweden.

**International Symposium on Weak Rock**, Tokyo, Japan, September 1981. All enquiries to: Secretariat of the International Symposium on Weak Rock, c/o Japan Society of Civil Engineers, Yotsuya 1-Chome, Tokyo 160, Japan.

---

## **NEWS OF SOUTHEAST ASIAN SOCIETY OF SOIL ENGINEERING**

---

### **Professor W. Fisher Cassie Honoured**

Professor W. Fisher Cassie was recently awarded the Honorary Degree of Doctor of Technology in recognition of his contributions which led to the birth of the Asian Institute of Technology. The Honorary degree was conferred on Professor Cassie during graduation ceremonies at AIT on 27th August, 1979, by Professor Robert B. Banks, President of AIT.

### **Sixth Southeast Asian Conference on Soil Engineering**

The Sixth Southeast Asian Conference on Soil Engineering will take place in Taipei from 19-23 May 1980, with post Conference tours scheduled for 24-26 May 1980. The Conference will be held in the Conference Hall of the Grand Hotel, located on the northern edge of Taipei City.

Summaries of about 100 papers were received in response to the call for papers announced in Bulletin No. 1. Some 60 papers by authors from 18 countries have been tentatively accepted. The Conference Program will be divided into five main sessions ;

- (i) Soil Behaviour
- (ii) Foundations
- (iii) Stability and Excavations
- (iv) Soil Improvement and Pavement
- (v) Engineering Geology and Rock Mechanics

Two world renowned authorities on geotechnical engineering will be invited to give guest lectures. Further details of the conference may be obtained from:

Secretary-General,  
Organizing Committee, 6SEACSE,  
c/o Moh and Associates,  
6-1, Lane 137, Yen Chi Street,  
Taipei, Taiwan,  
Republic of China

### **Asian Remote Sensing Training Center Established**

On 30th August, 1979 an agreement was signed between AIT and the U.S. Government in which the latter agreed to provide 5.6 million dollars over a six year period for the establishment of a Remote Sensing Training Center at AIT. The Center will provide training to selected students from the U.N.'s ESCAP region on the analysis of data from Landsat satellites for earth resources applications in the fields of forestry, geology, agriculture, urban growth, etc.

The first faculty to join AIT and specialized in Remote Sensing is Dr. Kaew Nualchawee who obtained his Ph.D. from Colorado State University. Not yet selected is the five-man project team, which will be composed of a Project Leader, Systems Analyst, Cartographer, Land Use Specialist, and Hydrologist/Geologist.

The first group of 15 to 30 students are expected to begin studies in the Center in May 1981.

### **Southeast Asian Group of Rock Mechanics & Mining Engineering**

At its recent council meeting held in Singapore in July 1979, the General Committee of SEASSE approved the formation of a Regional Group in Rock Mechanics and Mining Engineering, which is to be affiliated to the International Society for Rock Mechanics. The Governing Board and the Governing Council of the International Society for Rock Mechanics unanimously approved at their recent meeting in Montreux in September 1979 the affiliation of the Southeast Asian Society of Soil Engineering as a Regional Group with the creation of a Rock Mechanics and Mining Engineering Section. Members of SEASSE should be grateful to the past President Prof. Pierre Habib, Secretary General Dr. Arnold Silveira, Vice-President for Asia Dr. Yoshida. The members will also be pleased to know the new President of ISRM is Prof. Dr. Walter Witke from Germany.

The General Committee of SEASSE is now working on the office bearers of this Rock Mechanics and Mining Engineering Section. It is anticipated that the Governing Committee will include Directors of Hydro-electric Construction and Mining Industry from Countries of SEASSE and especially from Malaysia, Singapore, Indonesia, Philippines, Taiwan, Hong Kong, Thailand, etc. These details will be announced to members of our Society in January 1980.

### **Environmental Control in Geotechnical Engineering**

For the Session 6 of X ICSMFE, a State-of-the-Art Report concerning "Environmental Control in Geotechnical Engineering" is being prepared.

In order to make this report comprehensive members are requested to provide the General Reporters of the Session a list of papers and publications written on the following topics:

- fluids withdraw from the ground
- fluids disposal into the ground
- solids extraction from the ground
- solids introduction into the ground
- solids accumulation on surface
- solids removal from surface
- changing physio-chemical properties of soils
- changing water levels drainage and drainage conditions
- geotechnical maps for environmental assessment.

It may be appropriate to specify that the General Reporters intend:

- to deal only with geotechnical subjects/procedures measurements
- to consider only activities leading to environmental problems rather than their isolated consequences.
- to limit their interests to what a soil engineer can control.

Please mail your contributions before 1st December 1979 to:

Dr. Piero Sembenelli,  
c/o Elc-Electro Consult,  
Via Chiabrera 8,  
20151 Milano,  
Italy

### **Southeast Asian Regional Symposium on Problems of Soil Erosion and Sedimentation**

The Southeast Asian Regional Symposium on Problems of Soil Erosion and Sedimentation will be held at the Conference Centre of the Asian Institute of Technology, Bangkok, Thailand, from 27-29th January 1981. The symposium is intended to promote the sharing of experiences of soil erosion and its consequences in the Southeast Asian Region. The organizers seek contributions and participations by engineers, scientists, research workers and planners concerned with combating these problems. The processes that are to be considered in the symposium cover several disciplines, including agriculture, forestry, livestock management, river hydraulics, civil engineering and land use planning. The aim is that the symposium shall reflect this multidisciplinary character, and that all relevant disciplines shall be represented.

Soil erosion is a process that is accelerating in most developing countries under the impact of population expansion, changes of land use, reduction of forest cover, and numerous other influences that are related to economic development.

Intending participants should contact:

Organizing Secretary,  
P. S. E. S. Symposium,  
Asian Institute of Technology,  
P. O. Box 2754, Bangkok, Thailand

#### **SEASSE and other National Societies of Asia Endorses AGE**

At the 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering held in Singapore on 24-27 July 1979, it was decided that AGE is now *officially endorsed* by the Southeast Asian Society of Soil Engineering and by the National Societies of ISSMFE from the following Asian countries: China, India, Indonesia, Iran, Israel, Japan, Pakistan, S.E. Asia and Syria.

This official endorsement of AGE is a much appreciated recognition of the usefulness of AGE and of its continuous efforts to provide the geotechnical community, especially in developing countries, with the information it needs at a price it can afford, despite the considerable difficulties arising from the low membership fees charged versus the rapidly increasing costs due to inflation.

Since its creation in 1973, AGE has progressively developed to what has become today the most comprehensive and by far the cheapest existing geotechnical information system

For 1980 AGE plans to further improve its services, for instance, by airmailing at no extra cost its Newsletter and its Current Awareness Service which aim at providing current information. This was felt necessary considering the long delays and unreliability of surface mailing.

It is sincerely hoped that AGE membership will increase further to ensure its survival and its continuous improvement for the benefit of the geotechnical community worldwide and particularly for the engineers and institutions who cannot afford far more expensive information sources and who therefore have no other alternative than AGE to get the information they need.

## **Graduate Programmes in Geotechnical & Transportation Engineering at the Asian Institute of Technology**

The Asian Institute of Technology was established to help meet the growing needs for advanced engineering education in Asia. The Institute was founded in 1959. The Institute provides advanced education in engineering, science and allied fields and the academic programs lead to the Diploma of the Asian Institute of Technology and to the degrees of Master of Engineering, Master of Science, Doctor of Engineering and Doctor of Technical Sciences. The Institute's academic programs are related closely to the needs of Asia. The student enrollment of more than 400 is drawn from across the region; faculty are recruited internationally.

The consolidation of geotechnical and transportation engineering activities at the Institute has led to the following four fields of studies in the Division of Geotechnical & Transportation Engineering at AIT.

- (a) Soil Engineering
- (b) Engineering Geology
- (c) Transportation Engineering
- (d) Transportation Systems

The principles of geotechnical engineering find wide scope for application to problems of transportation infrastructure: roads, railways airports and seaports. Concomitantly, transportation engineering provides knowledge and skills in planning, design and economics which are necessary for the formulation and selection of the infrastructure projects which the geotechnical engineer designs and builds.

Studies in geotechnical engineering cover soil mechanics, rock mechanics, soil engineering, foundation engineering, and engineering geology. The essential focus of teaching and research in transportation engineering is on the planning, design and performance of highways, railways, airports and seaports. The program builds on basic knowledge in civil engineering-including surveying, photogrammetry, airphoto interpretation and construction materials. The courses to transportation systems are concerned with the study of the basic laws related to traffic in its broadest sense, and the application of this knowledge to professional practice for the planning, design and operation of transportation systems.

The Geotechnical and Transportation Engineering Division maintains outstanding laboratory facilities which occupy a large area and which are excellently equipped for both teaching and research. The laboratory facilities cater for soil engineering, engineering geology, rock mechanics, airphoto interpretation, traffic engineering and highway materials.

Most students apply for the Master's degree program. This is a full time course of 16 months minimum duration. Application is open to those who hold or expect to receive a good degree in Civil Engineering, Geology or Mining Engineering, The main intake of students for the graduate programs is in September each year, and the closing date for the receipt of applications for admission is April 15. There are many comprehensive scholarships available at the Institute.

For the complete AIT Prospectus, application forms, and scholarship information write:

The Deputy Registrar for Admissions,  
Asian Institute of Technology,  
P.O. Box 2754,  
Bangkok, Thailand

#### **Society Publications**

The following Society publications can be ordered from the Division of Geotechnical and Transportation Engineering, Asian Institute of Technology, P.O. Box 2754, Bangkok Thailand. Cheques should be made payable to "ASIAN INSTITUTE OF TECHNOLOGY".

| Proceedings  | Price including postage<br>by surface mail |
|--|--|
| Proceedings of the Second Southeast Asian Conference on Soil Engineering held in Singapore in June, 1970 | US\$ 22                                    |
| Proceedings of the 3rd Southeast Asian Conference on Soil Engineering held in Hong Kong in 1972          | US\$ 25                                    |
| Proceedings of the 4th Southeast Asian Conference on Soil Engineering held in Kuala Lumpur in 1975       | US\$ 30                                    |
| Proceedings of the 5th Southeast Asian Conference on Soil Engineering held in Bangkok in 1977            | US\$ 40                                    |



**Publications Asian Institute of Technology Related to Geotechnical Engineering**

| Proceedings   | Price including postage<br>by surface mail |
|---|--|
| Proceedings of the Fourth Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, Thailand (1971)  | US\$ 35                                    |
| Proceedings of the Specialty Session on Lateritic Soils, held at the Seventh International Conference on Soil Mechanics and Foundation Engineering, held in Mexico City in 1969 | US\$ 25                                    |

Orders should be placed to the Director, Library & Information Center Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

**General Committee of the Southeast Asian Society of Soil Engineering for the period 1978-80.**

|                                    |                     |
|------------------------------------|---------------------|
| Dr. Tan Swan Beng                  | President           |
| Prof. A.S. Balasubramaniam         | Secretary-Treasurer |
| Dr. Za-Chieh Moh                   | Founder-President   |
| Prof. Chin Fung Kee                | Past-President      |
| Prof. Peter Lumb                   | Past-President      |
| Dr. Chai Muktabhant (Thailand)     |                     |
| Mr. S.G. Elliott (Hong Kong)       |                     |
| Prof. Salvador Reyes (Philippines) |                     |
| Prof. J.J. Hung (Taiwan, R.O.C.)   |                     |
| Dr. Ting Wen Hui (Malaysia)        |                     |
| Mr. Nibon Rananand (Thailand)      |                     |
| Dr. Edward W. Brand (Hong Kong)    |                     |
| Prof. Seng Lip Lee (Singapore)     |                     |
| Dr. Vincent Campbell               | Editor              |

The membership application forms and other details can be obtained from:

The Secretary, SEASSE,  
c/o Division of Geotechnical & Transportation  
Engineering,  
Asian Institute of Technology,  
P.O. Box 2754,  
Bangkok, Thailand

## SI UNITS AND SYMBOLS

The following list of quantities, SI (Système International) units and SI symbols, are recommended for use in Geotechnical Engineering.

| Quantities  | Units                      | Symbols                  |
|---|----------------------------|--------------------------|
| Length  | kilometre                  | km                       |
|   | metre                      | m                        |
|   | centimetre                 | cm                       |
|   | millimetre                 | mm                       |
| Area  | micrometre                 | $\mu\text{m}$            |
|   | square kilometre           | $\text{km}^2$            |
|   | square metre               | $\text{m}^2$             |
|   | square centimetre          | $\text{cm}^2$            |
| Volume  | square millimetre          | $\text{mm}^2$            |
|   | cubic metre                | $\text{m}^3$             |
|   | cubic centimetre           | $\text{cm}^3$            |
| Mass  | cubic millimetre           | $\text{mm}^3$            |
|   | tonne                      | t                        |
|   | kilogram                   | kg                       |
| Density $\rho$ (mass density)                                     | gram                       | g                        |
|   | tonne per cubic metre      | $\text{t/m}^3$           |
|   | kilogram per cubic metre   | $\text{kg/m}^3$          |
|   | gram per cubic centimetre  | $\text{g/cm}^3$          |
| Unit weight $\gamma$ (weight density)                             | kilonewton per cubic metre | $\text{kN/m}^3$          |
| Force   | meganewton                 | MN                       |
|   | kilonewton                 | kN                       |
|   | newton                     | N                        |
| Pressure  | megapascal                 | MPa                      |
|   | kilopascal                 | kPa                      |
| Energy  | megajoule                  | MJ                       |
|   | kilojoule                  | kJ                       |
|   | joule                      | J                        |
| Coefficient of volume compressibility or swelling $m_v$           | 1/megapascal               | MPa <sup>-1</sup>        |
|   | 1/kilopascal               | kPa <sup>-1</sup>        |
| Coefficient of consolidation or swelling $c_v$                    | square metre per second    | $\text{m}^2/\text{s}$    |
|   | square metre per year      | $\text{m}^2/\text{year}$ |
|   | square metre per second    | $\text{cm}^2/\text{s}$   |
| Hydraulic conductivity $k$ (formerly coefficient of permeability) | metre per second           | $\text{m/s}$             |
|   | centimetre per second      | $\text{cm/s}$            |

NOTES: The term specific gravity is obsolete and is replaced by relative density. The former term relative density  $(e_{\text{max}} - e)/(e_{\text{max}} - e_{\text{min}})$  is replaced by the term density index,  $I_D$ .

## **AFTERWORD TO VOLUME 10**

**BY THE PRESIDENT OF THE SOCIETY**

The Southeast Asian Society of Soil Engineering was founded in 1967. The Society's main objectives were the promotion of closer co-operation the interchange of ideas, information and experiences among researchers, practising engineers and engineering geologists in the field of geotechnical engineering. To fulfill these objectives, the Society has in the past twelve years organised numerous lectures by prominent engineers, five Southeast Asian Conferences on Soil Engineering and two Asian Regional Conferences on Soil Mechanics and Foundation Engineering. The proceedings of these Conferences have been published in several volumes. They have been circulated not only to the Region's engineers, but also internationally.

To provide geotechnical information between the regular conferences the Society publishes its official journal "Geotechnical Engineering". This issue marks the tenth anniversary of "Geotechnical Engineering". Within these ten years, "Geotechnical Engineering" has become a well known and respected international journal. For this, the Society expresses its deep appreciation to the former editors, Professor E. W. Brand and Professor A.S. Balasubramaniam; and the present editor, Dr. K. V. Campbell.

The Society in 1978 established a Southeast Asian Regional Engineering Geology Group and affiliated it with the International Association of Engineering Geology (IAEG). The formation of a Rock Mechanics Group is now in progress. It is hoped that with the addition of these Groups, the Society and "Geotechnical Engineering" will become the Region's forum for the exchange of experiences and information in the field of geomechanics.

**DR. TAN SWAN BENG**

---

## AUTHOR INDEX

### VOLUME 10, 1979

---

- AHMAD, B. *See* DAS GUPTA (1979)
- AKAGI, T. (1979), Some Land Subsidence Experiences in Japan and Their Relevance to Subsidence in Bangkok, Thailand: v.10, n.1, 1-48
- AKAGI, T. (1979), State-of-the-Art Report on Settlements and Time Rates of Consolidation: v.10, n.2, 179-198.
- ARBHAHBIRAMA, A. *See* DAS GUPTA (1979)
- BALASUBRAMANIAM, A.S. (1979), Slope Analysis: *Book Reviewed*, v. 10, n. 1, 105-106
- BALASUBRAMANIAM, A.S. (1979), Lectures of the Seminar 'Failures of Large Dams-Reasons and Remedial Measures': *Book Reviewed*, v. 10, n. 1, 117-118
- BRENNER, R.P. (1979), Stress in Subsoil and Methods of Final Settlement Calculation: *Book Reviewed*, v. 10, n.1, 120-121
- BRENNER, R.P. (1979), Soil Mechanics and Foundation Engineering: *Book Reviewed*, v.10, n.1, 121
- BRENNER, R.P. (1979), Ring Shear Tests and Determination of the Bearing Capacity and Pile Driving Resistance of Piles Using Soundings: *Book Reviewed*, v.10, n.1, 123-124
- BRENNER, R.P. (1979), Soil Mechanics For Off-Road Vehicle Engineering: *Book Reviewed* v.10, n.1, 124-125
- CAMPBELL, K.V. (1979), Remote Sensing, Principles and Interpretation: *Book Reviewed*, v.10, n.1, 108-109
- CAMPBELL, K.V. (1979), Rockslides and Avalanches, 1: *Book Reviewed*, v.10, n.1, 109-115
- CAMPBELL, K.V. (1979), Terrain Analysis: *Book Reviewed*, v. 10, n. 2, 259-263
- CAMPBELL, K.V. (1979), Methods of Geological Engineering in Discontinuous Rocks: *Book Reviewed*, v. 10, n. 2, 263-265
- DAKSHANAMURTHY, V. (1979), A New Method to Predict Swelling Using a Hyperbolic Equation : *Erratum*, v.10, n.1, 103
- DAS GUPTA, A., ARBHABHIRAMA, A. and AHMAD, B. (1979), Preliminary Investigation of Saltwater Encroachment into the Nakhon Luang Aquifer, Bangkok, Thailand, v.10, n.2, 141-158
- DHOWIAN, A.W. (1979) *See* EDIL (1979)
- EDIL, T.B. and DHOWIAN, A.W. (1979), Analysis of Long-Term Compression of Peats v.10, n.2, 159-178
- HOLMBERG, S. (1979), Bridge Approaches on Soft Clay Supported by Embankment Piles: v. 10, n.1, 77-89
- ITO, M. (1979) *See* NISHITANI (1979)
- KRISHNASWAMY, K.S. (1979) *See* SANKARAN (1979)
- MORI, H. (1979), Review of Japanese Subsurface Investigation Techniques: v.10, n.2, 219-242
- NAIR, P.G.B. (1979) *See* SANKARAN (1979)
- NISHITANI, T. and ITO, M. (1979), Stress and Deformation Characteristics of an Elastic Punch in an Elastoviscoplastic Medium: *Technical Note*, v.10, n.1, 91-101.
- PREMCHITT, J. (1979), Land Subsidence in Bangkok, Thailand: Results of Initial Investigation, 1978: v. 10, n. 1, 49-76
- RAO, S.N. (1979), The Influence of Fabric on the Shrinkage Limit of Clay: v. 10, n.2, 243-251
- SANKARAN K.S., KRISHNASWAMY, N.R. and NAIR, P.G.B. (1979) Dynamic Response of Footings in a Saturated Soil Medium: *Technical Note*, v.10, n.2, 253-258
- SAWAGUCHI, M. (1979) Applications of Steel Pipe Piles in Japan: v. 10, n, 2, 199-218
- TAN, S.B. (1979) Afterword to Volume 10, v. 10, n. 2, 281

---

## SUBJECT INDEX†

### VOLUME 10, 1979

---

#### A. General

##### 04. *Textbooks, Handbooks and Geotechnical Periodicals*

- Slope Analysis: *Book Reviewed*, Balasubramaniam, A.S. (1979), v.10, n.1, 105-106
- Remote Sensing, Principles and Interpretation: *Book Reviewed*, Campbell, K.V. (1979), v.10, n.1, 108-109
- Rockslides and Avalanches, I: *Book Reviewed*, Campbell, K.V. (1979), v. 10, n. 1, 109-115
- Lectures of the Seminar 'Failures of Large Dams - Reasons and Remedial Measures': *Book Reviewed*, Balasubramaniam, A.S. (1979), v.10, n.1, 117-118
- Stress in Subsoil and Methods of Final Settlement Calculation: *Book Reviewed*, Brenner, R.P., (1979), v.10, n.1, 120-121
- Soil Mechanics and Foundation Engineering: *Book Reviewed*, Brenner, R.P.(1979), v.10, n.1, 121
- Ring Shear Tests on Clay and Determination of the Bearing Capacity and Pile Driving Resistance of Piles Using Soundings: *Book Reviewed*, Brenner, R.P. (1979), v.10, n.1, 123-124
- Soil Mechanics For Off-Road Vehicle Engineering: *Book Reviewed*, Brenner, R.P. (1979), v.10, n.1, 124-125
- Terrain Analysis: *Book Reviewed*, Campbell, K.V. (1979), v.10, n.2, 259-263
- Methods of Geological Engineering in Discontinuous Rocks: *Book Reviewed*, Campbell, K.V. (1979), v.10, n.2, 263-265

#### B. Engineering Geology

##### 00. *General*

- Methods of Geological Engineering in Discontinuous Rocks: *Book Reviewed*, Campbell, K.V. (1979), v.10, n.2, 263-265

##### 02. *Hydrogeology*

- Some Land Subsidence Experiences in Japan and Their Relevance to Subsidence in Bangkok, Thailand, T. (1979), v.10, n.1, 1-48
- Land Subsidence in Bangkok, Thailand: Results of Initial Investigation, 1978: Premchitt, J. (1979), v.10, n.1, 49-76
- Preliminary Investigation of Saltwater Encroachment into the Nakhon Luang Aquifer, Bangkok, Thailand: Das Gupta, A., Arbhahirama, A. and Ahmad, B. (1979), v.10, n.2, 141-158

##### 03. *Mass Movements and Subsidence*

- Some Land Subsidence Experiences in Japan and Their Relevance to Subsidence in Bangkok, Thailand: Akagi T. (1979), v.10, n.1, 1-48

---

† Based on the International Geotechnical Classification System (IGC) prepared by the International Society for Soil Mechanics and Foundation Engineering, revised version, 1973.

- Land Subsidence in Bangkok, Thailand: Results of Initial Investigation, 1978: Premchitt, J. (1979), v.10, n.1, 49-76
- Rockslides and Avalanches, 1: *Book Reviewed*, Campbell, K.V. (1979), v.10, n.1, 109-115

### C. Site Investigations

#### 00. *General*

- Review of Japanese Subsurface Investigation Techniques: Mori, H. (1979), v.10, n.2, 219-242
- Terrain Analysis: *Book Reviewed*, Campbell, K.V. (1979), v.10, n.2, 259-263

#### 07. *Measurement of Field Conditions*

- Land Subsidence in Bangkok, Thailand: Results of Initial Investigation, 1978: Premchitt, J. (1979), v.10, n.1, 49-76

#### 08. *Field Testing (excluding tests for engineering properties)*

- Bridge Approaches on Soft Clay Supported by Embankment Piles: Holmberg, S. (1979), v.10, n.1, 77-89
- Ring Shear Tests on Clay and Determination of the Bearing Capacity and Driving Resistance of Piles Using Soundings: *Book Reviewed*, Brenner, R.P. (1979), v.10, n.1, 123-124

### D. Soil Properties: Laboratory and Field Determinations

#### 03. *Composition, Structure, and Density*

- The Influence of Fabric on the Shrinkage Limit of Clay : Rao, S.N., (1979), v.10, n.2, 249-251

#### 05. *Compressibility*

- Analysis of Long-Term Compression of Peats : Edil, T.B. and Dhowian, A.W. (1979), v.10, n.2, 159-178

#### 06. *Shear-deformation and Strength Properties*

- A New Method to Predict Swelling Using a Hyperbolic Equation: *Erratum*, Dakshanamurthy, V. (1979), v.10, n.1, 103
- Ring Shear Tests on Clay and Determination of the Bearing Capacity and Pile Driving Resistance of Piles Using Soundings: *Book Reviewed*, Brenner, R.P. (1979), v.10, n.1, 123-124

### E. Analysis of Soil-Engineering Problems

#### 01. *In Situ Stresses Caused by Gravity, Applied Loads, and Excavations*

- Stress and Deformation Characteristics of an Elastic Punch in an Elastoviscoplastic Medium: *Technical Note*, Nishitani, T. and Ito, M. (1979), v.10, n.1, 91-101

02. *Deformation and settlement Problems*

A New Method to Predict Swelling Using a Hyperbolic Equation : *Erratum*, Dakshanamurthy V. (1979), v.10, n.1, 103

Stress in Subsoil and Methods of Final Settlement Calculation; *Book Reviewed*, Brenner, R.P. (1979), v.10, n.1, 120-121

Analysis of Long-Term Compression of Peats: Edil, T.B. and Dhowian, A.W. (1979), v.10, n.2, 159-178

State-of-the-Art Report on Settlements and Time Rates of Consolidation: Akagi, T. (1979), v.10, n.2, 179-198

04. *Bearing Capacity of Piles*

Ring Shear Test on Clay and Determination of the Bearing Capacity and Pile Driving Resistance of Piles Using Soundings: *Book Reviewed*, Brenner, R.P. (1979), v.10, n.1, 123-124

06. *Stability of Slopes, Fills, Cuts and Excavations*

Slope Analysis: *Book Reviewed*, Balasubramaniam, A.S. (1979), v.10, n.1, 105-106

08. *Dynamic Problems*

Dynamic Response of Footings in a Saturated Soil Medium: *Technical Note*, Sankaran, K.S., Krishnaswamy, N.R. and Nair, P.G.B. (1979), v.10, n.2, 253-258

11. *Soil-Vehicle Interaction (Trafficability)*

Soil Mechanics For Off-Road Vehicle Engineering: *Book Reviewed*, Brenner, R.P. (1979), v.10, n.1, 124-125

## **H. Design, Construction and Behaviour of Engineering Works**

01. *Foundations of Structures*

Bridge Approaches on Soft Clay Supported by Embankment Piles: Holmberg, S. (1979), v.10, n.1, 77-89

02. *Retaining Structures, Cut-off Walls and Concrete Dams*

Lectures of the Seminar 'Failures of Large Dams-Reasons and Remedial Measures': *Book Reviewed*, Balasubramaniam, A.S. (1979), v.10, n.1, 117-118

Applications of Steel Pipe Piles in Japan: Sawaguchi, M. (1979), v.10, n.2, 199-218

## **K. Construction Methods and Equipment**

07. *Piles and Pile Driving*

Bridge Approaches on Soft Clay Supported by Embankment Piles: Holmberg, S. (1979), v.10, n.1, 77-89

Applications of Steel Pipe Piles in Japan: Sawaguchi, M. (1979). v.10, n.2, 199-218

## CONTENTS

---

| Number 1  | June 1979     | Page |
|---|---------------|------|
| <b>Papers:</b>  |               |      |
| <b>Some Land Subsidence Experiences in Japan and Their Relevance to Subsidence in Bangkok, Thailand</b> |               |      |
| TOSHINOBU AKAGI . . . . .   |               | 1    |
| <b>Land Subsidence in Bangkok, Thailand: Results of Initial Investigation, 1978</b>                     |               |      |
| JERASAK PREMCHIT . . . . .  |               | 49   |
| <b>Bridge Approaches on Soft Clay Supported by Embankment Piles</b>                                     |               |      |
| SOREN HOLMBERG . . . . .  |               | 77   |
| <b>Technical Note:</b>  |               |      |
| <b>Stress and Deformation Characteristics of an Elastic Punch in an Elasto-Viscoplastic Medium</b>      |               |      |
| T. NISHITANI and M. ITO . . . . .   |               | 91   |
| Erratum . . . . .   |               | 103  |
| Book Reviews . . . . .  |               | 105  |
| Conference News . . . . .   |               | 127  |
| News of Publications . . . . .  |               | 129  |
| News of Southeast Asian Society of Soil Engineering . . . . .   |               | 130  |
| SI Units and Symbols . . . . .  |               | 139  |
| <br>  |               |      |
| Number 2  | December 1979 |      |
| <b>Papers:</b>  |               |      |
| <b>Preliminary Investigation of Saltwater Encroachment into the Nakhon Luang Aquifer, Bangkok</b>       |               |      |
| A. DAS GUPTA, A. ARBHABHIRAMA and B. AHMAD . . . . .  |               | 141  |
| <b>Analysis of Long-Term Compression of Peats</b>   |               |      |
| T.B. EDIL and ABDULMOHSIN W. DHOWIAN . . . . .  |               | 159  |
| <b>State-of-the-Art Report on Settlements and Time Rates of Consolidation</b>                           |               |      |
| T. AKAGI . . . . .  |               | 179  |
| <b>Applications of Steel Pipe Piles in Japan</b>  |               |      |
| M. SAWAGUCHI . . . . .  |               | 199  |
| <b>Review of Japanese Subsurface Investigation Techniques</b>   |               |      |
| H. MORI . . . . .   |               | 219  |
| <b>The Influence of Fabric on Shrinkage Limit in Clay</b>   |               |      |
| S. NARASIMHA RAO . . . . .  |               | 243  |
| <b>Technical Note :</b>   |               |      |
| <b>Dynamic Response of Footings in a Saturated Soil Medium</b>  |               |      |
| K.S. SANKARAN, N.R. KRISHNASWAMY and P.G. BHASKARAN NAIR . . . . .                                      |               | 253  |
| Book Reviews . . . . .  |               | 259  |
| Conference News . . . . .   |               | 268  |
| News of Southeast Asian Society of Soil Engineering . . . . .   |               | 271  |
| SI Units and Symbols . . . . .  |               | 280  |
| Afterword to Volume 10 . . . . .  |               | 281  |
| Author Index: Volume 10, 1979 . . . . .   |               | 282  |
| Subject Index: Volume 10, 1979 . . . . .  |               | 283  |

---