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PILE DAMAGE DURING INSTALLATION
— A CASE STUDY

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SYNOPSIS

Damage to a number of precast, prestressed concrete piles occurred during the construction of a wharf structure in the Republic of Singapore. Probable causes of the damage were investigated in an attempt to rectify the problem. The causes of pile damage during driving were inferred to have been a function of both the pile driving hammer used and the geology of the area.

INTRODUCTION

Located along the south-western coast of the Republic of Singapore, the Pasir Panjang Port provides a larges warehousing centre as well as modern cargo handling facilities at the same location. Built on reclaimed land, this new gateway to the Port of Singapore provides 640 m long deep water berths, 896 m long coastal berths, 684 m long lighter and lash barge wharves and a mooring basin for lighters. The wharves have been constructed to berth three ocean-going ships, nine coastal vessels, thirty lighters and 6 lash barges at the same time. Warehousing facilities include 246,000 sq. m of covered storage and 52,200 sq. m of open storage space. Figure 1 shows the location of this gateway.

WHARF DESIGN

Part of the wharf structure is designed as a relieving platform type of structure, about 540 m long and about 32.2 m wide. The wharf deck consists of precast, prestressed concrete beams and slabs made monolithic by in-situ concrete topping. The deck is supported on precast and prestressed concrete hollow piles made by a spinning process. A steel sheet pile wall was constructed behind the wharf structure |and was built into the deck to facilitate sand filling behind the wharf structure in order to provide adequate land area to build the warehouses. Layout and cross-section of the wharf structure are shown in Figures 2 and 3 respectively.

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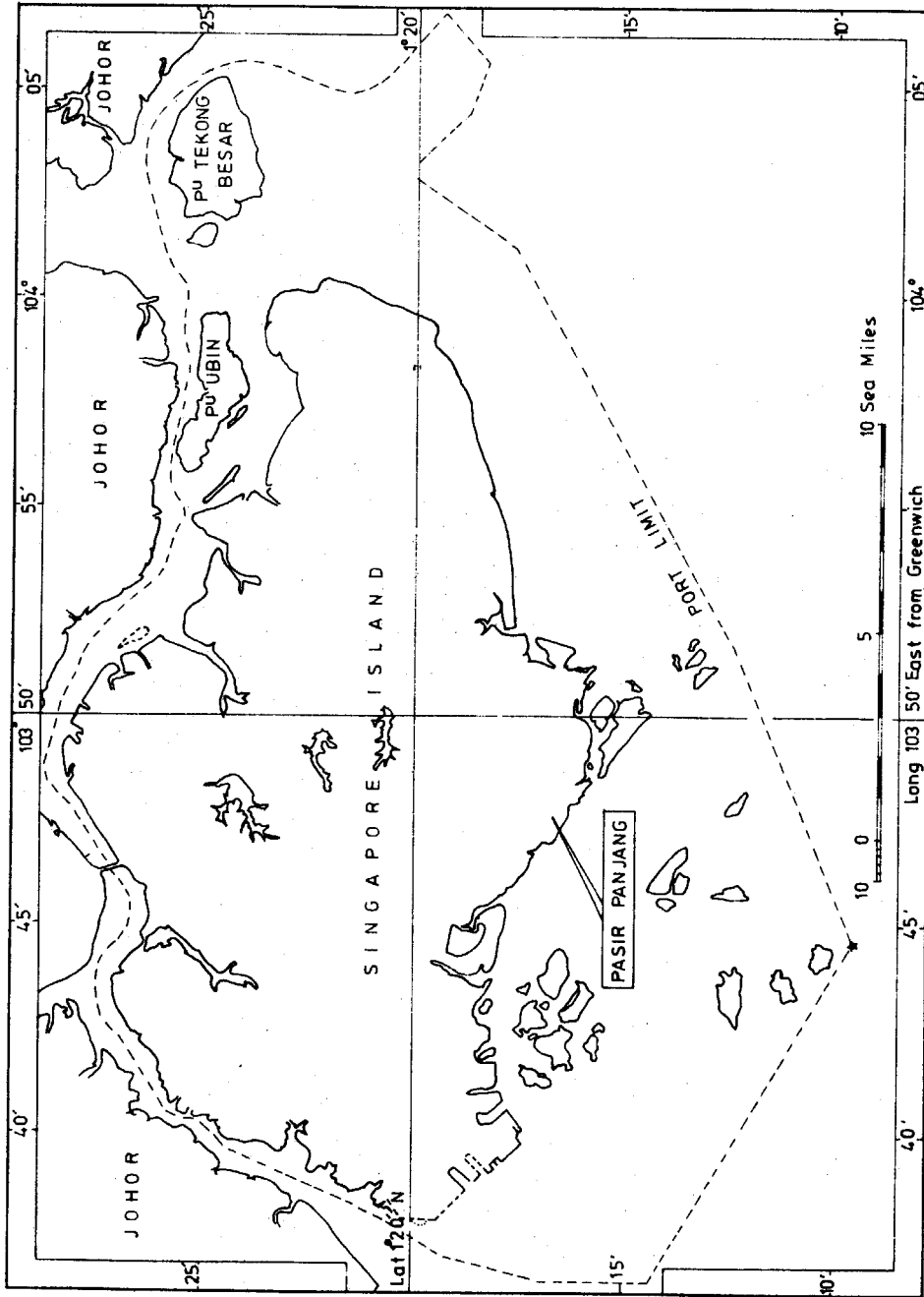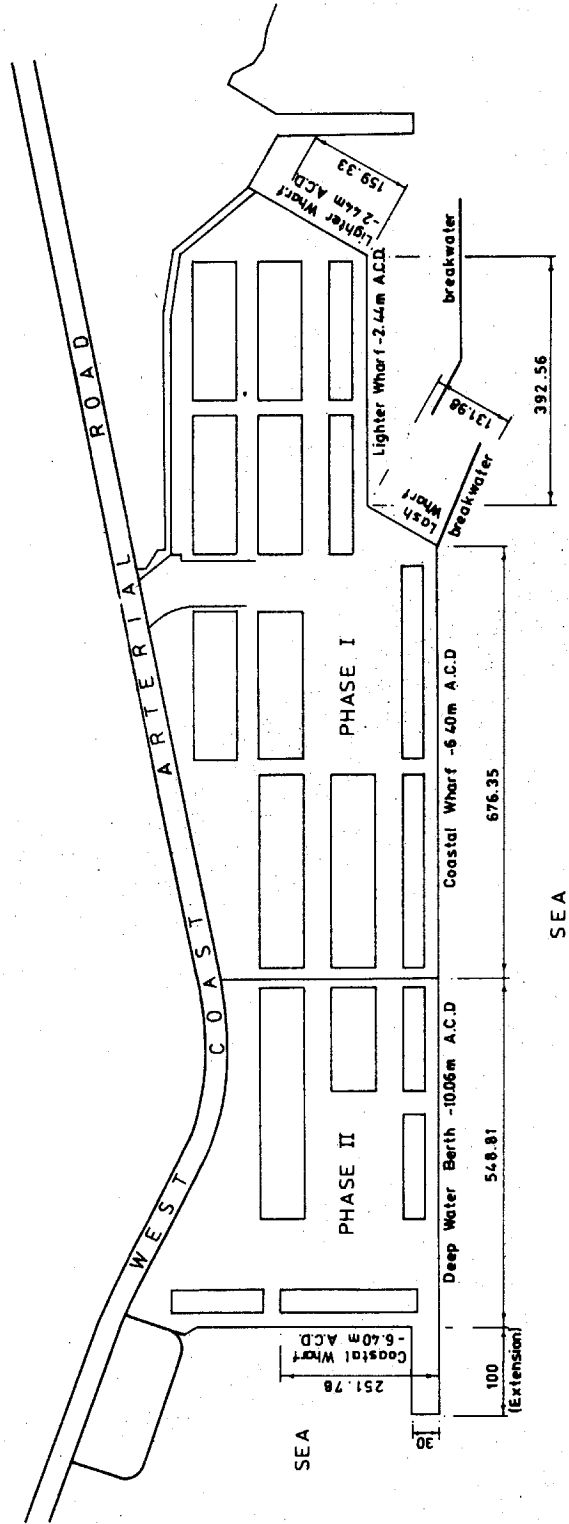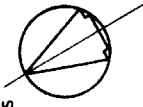*** Executive Engineer, Engineering Division, Port of Singapore Authority.

Fig. 1. Location of Wharf Site.

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Fig. 2. Layout of Pasir Panjang Wharves.

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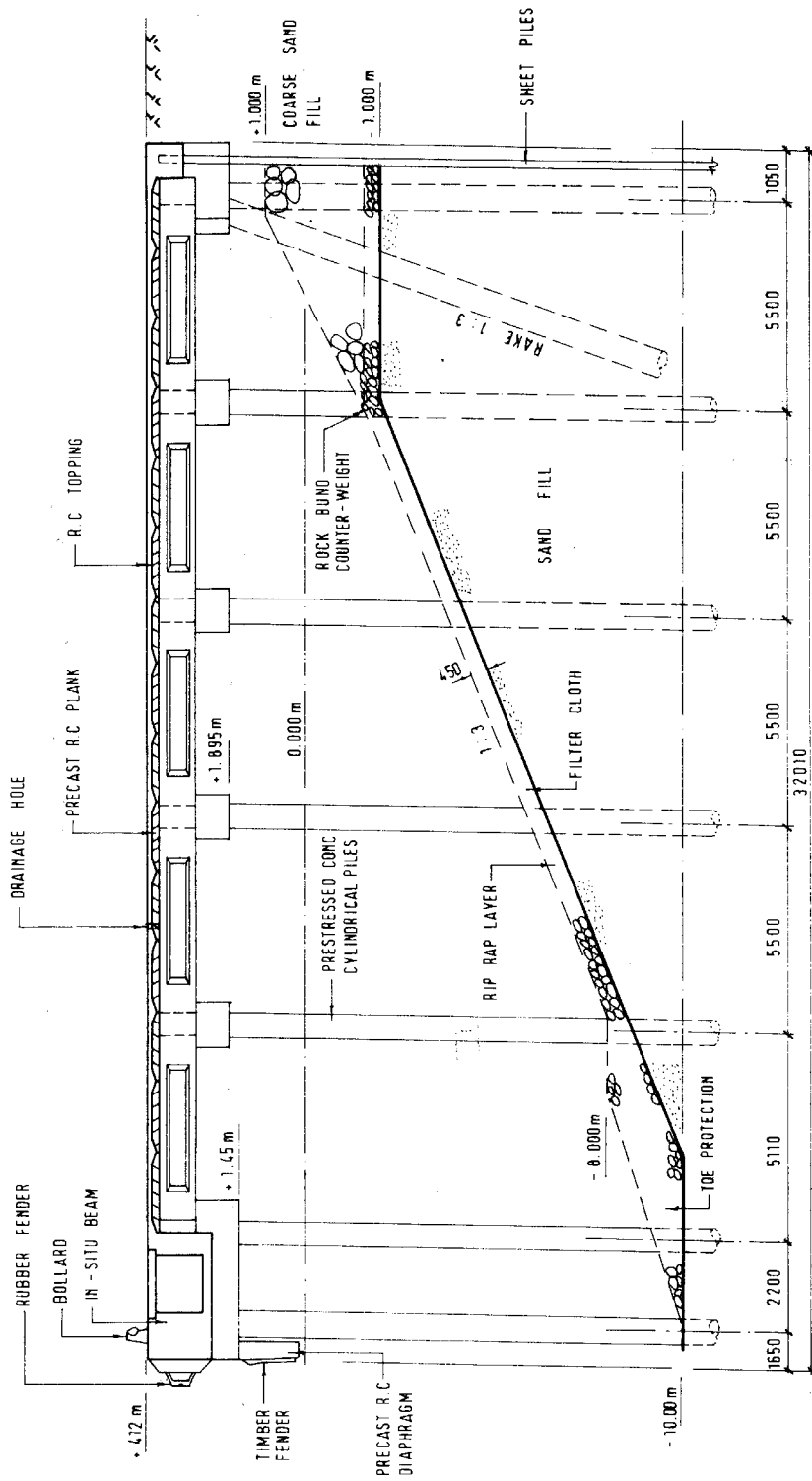
Fig. 3. Cross Section of Wharf Structure.

PILE DAMAGE

PRESTRESSED SPUN PILES

About 1,200 precast and prestressed concrete piles had to be driven to support the wharf deck. Two sizes of hollow piles with outer diameters of 700 mm or 840 mm and both with wall thicknesses of 120 mm were used. The piles were, manufactured using a spinning process, were closed at the pile toe and were steam-cured. Low carbon, heat treated prestressing steel (SCHUPACK, 1976) bars of 11 mm diameter were used as prestressing tendons. The prestressing steel bars were bound by mild steel helical ties using a warm temperature spot welding process in a mechanised pile cage making machine (NETUREN). High strength concrete of 49 N/mm² at 28 days was used to manufacture the piles. A super plasticiser was added to the concrete to achieve low water-cement ratio and high early strength which cut down the time required for stripping the pile moulds.

The piles were spun in a specially prepared spinning yard for a period of approximately 20 minutes at speeds varying from 50 RPM to 300 RPM. Figure 4 shows the pile spinning yard. Single pile lengths varied from 18 m to 46 m to accommodate widely varying depths to the hard sub-bottom.
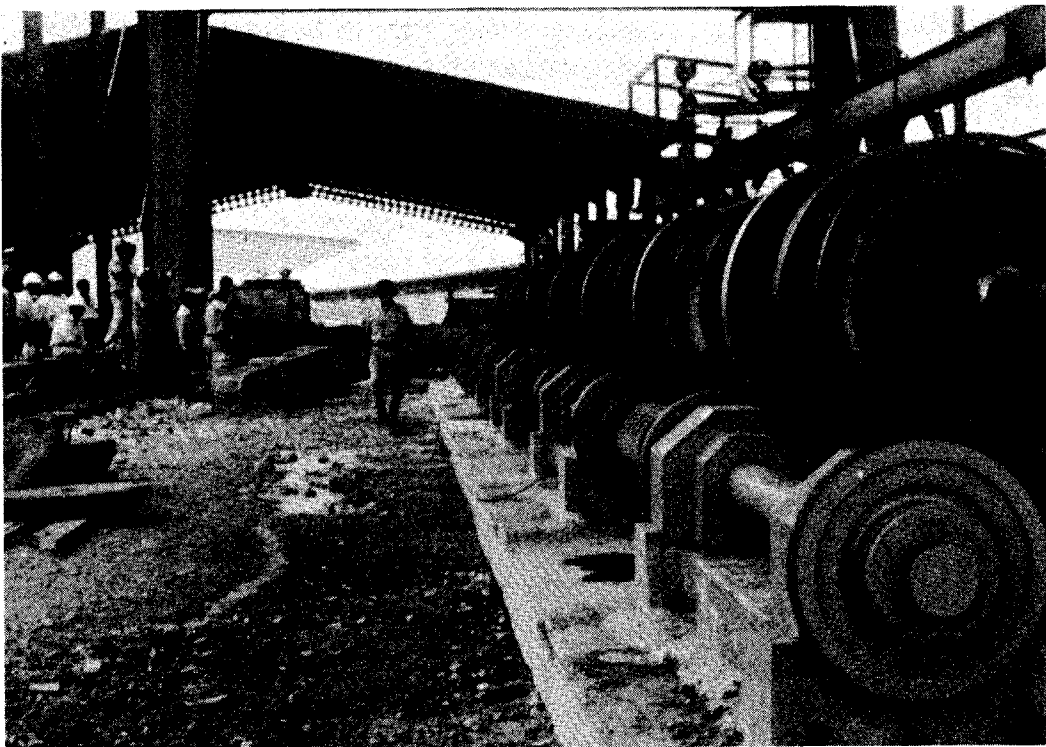
Fig. 4. Pile Spinning Yard.

The piles were steam-cured at atmospheric pressure for a period of 8 hours before demoulding. After demoulding, the piles were water-cured in large pile curing tanks. The concrete had to attain a strength of 28 N/mm^2 before demoulding was permitted. The piles were designed to have a net prestress of 5 N/mm^2 after accounting for losses.

PILE DAMAGE

The prestressed concrete hollow piles were driven with a diesel pile driving hammer from a piling pontoon. During the pile driving it was noticed that a number of piles developed cracks on the pile body. Measurements inside the pile hollow after pile driving showed large differences in pile lengths suggesting serious pile toe damage. These measurements showed that many piles had become much shorter after pile driving by lengths in excess of 10 m and in some cases as much as 20 m or more. These observations prompted a detailed investigation to determine the extent and probable causes of pile damage.

Some minor differences in lengths could have resulted from accumulation of laitance at the bottom of the pile hollow. Measurements within the pile hollow before driving showed that a maximum variation of 2.5 m could occur as a result of accumulation of laitance inside the pile hollow. Table 1 shows the summary of differences in measured lengths of piles. It may be noted that with the exception of piles with a variation in length of up to 2.5 m, which might have been caused by laitance accumulation, most of the other piles had final lengths shortened by about 10 m. Such significant variations in pile lengths after driving could have occurred only because of damage to the pile toe during driving and probable crushing of the lower parts of the pile.

Table 1. Summary of Pile Length Differences.

Difference in pile length	No. of piles
0 m - 2,5 m	1,332
2.5 m - 6 m	231
6 m - 10 m	96
10 m - 15 m	27
15 m - 20 m	9
more than 20 m	5
TOTAL	1,700

To investigate further the cause of the damage, it was decided to drill through the hollow of piles nos C-8 and C-12, located as shown in Figure 8. Pile C-8 had shown damage in excess of 20 m and pile C-12, about 18 m. Drill logs for these pile locations are presented in Figure 5.

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PILE NO. C-8

Elevation in m	Depth in m	Thickness	Legend	Description of soil/rock	General remarks
-19.70	0.00	0.00			
				Fragments of concrete.	Fine and coarse aggregate.
-23.20	3.50	3.50			
				Limestone. Fractured (∅=20-50mm), fresh and hard rock.	Steel reinforcement bar (L=550mm) found at depth of 4.3m
-25.50	5.80	2.30			
				Silty Clay with fragments of lime-stone.	Silty Clay is heavily weathered shale.
-26.80	7.10	1.30			
				Yellowish brown to pinkish brown Silty Clay (heavily weathered shale).	
-32.00	12.30	5.20			
				Light brown limestone.	Each piece of core is 2-5 cm long.
-37.50	17.80	5.50			
				Yellowish brown to pinkish brown Silty Clay with limestone.	19.1m: Expected depth for the tip of pile if pile is not broken. Very hard at the depth of 18.8m.
-41.00	21.30	3.50			

End of drilling

PILE NO. C-12

Elevation in m	Depth in m	Thickness	Legend	Description of soil/rock	General remarks
-26.00	0.00	0.00			
				Dark grey coarse sand with yellowish brown Silty Clay.	With fine aggregate of concrete.
-27.50	1.50	1.50			
				Fragments of concrete.	With fine and coarse aggregate.
-29.30	3.30	1.80			
				Limestone	With concrete fragments.
-30.00	4.00	0.70			
				Alteration of limestone, very hard sandstone and Silty Clay with gravel (∅=20-50mm)	Silty Clay with gravel is heavily weathered. 17.3m: Expected depth for the tip of pile if pile is not broken.
-44.00	18.00	14.00			

End of drilling

Fig. 5. Drill logs of Piles C-8 and C-12.

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A summary of findings from the two drill holes were as follows.

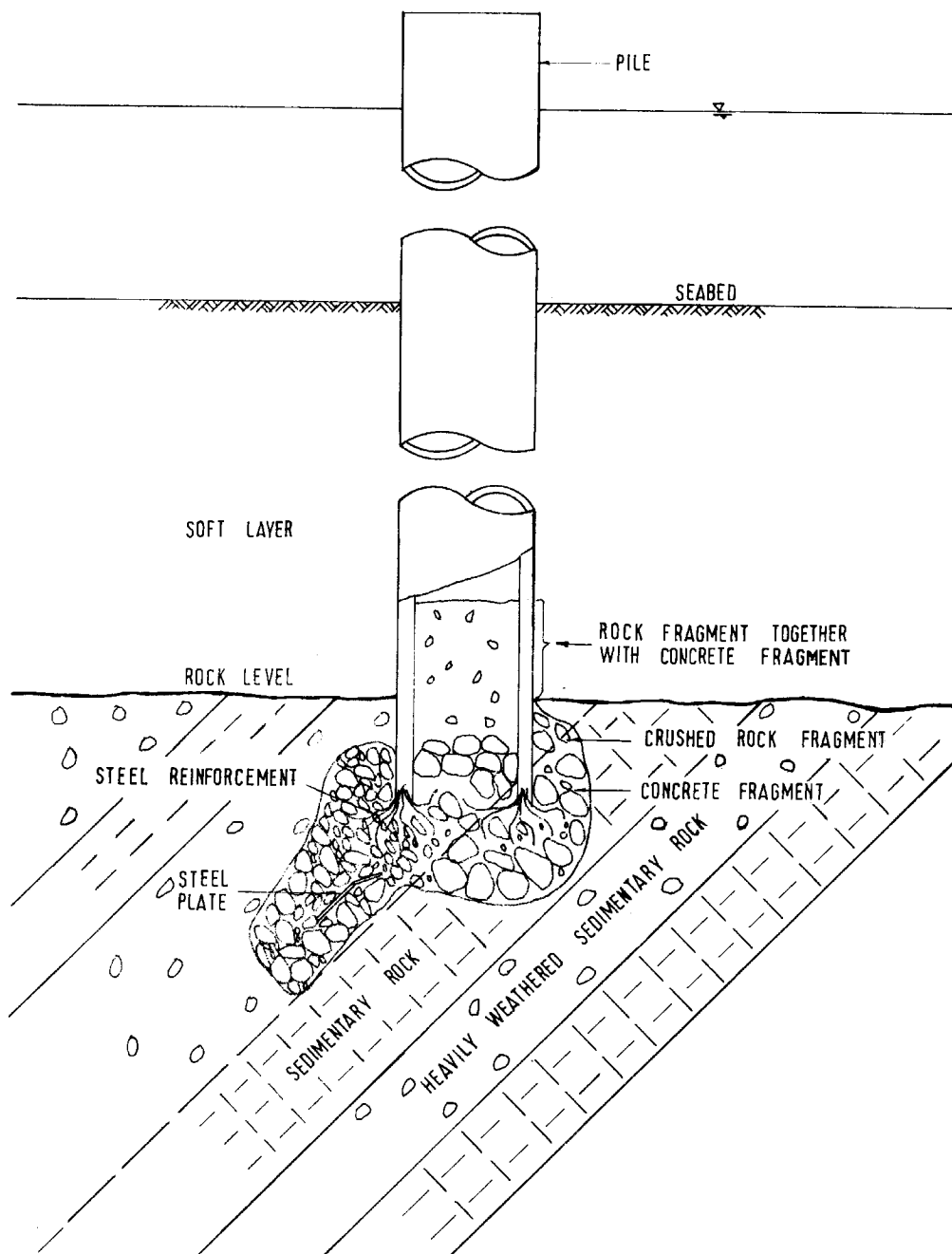(a) About 3 m to 4 m below the inside bottom surface of the pile, fragments of concrete were found.

Fig. 6. Probable condition of Damaged Piles.

(b) 1 m to 2 m below this level were found to consist a mixture of fragments of rocks (20 to 50 mm size) and fragments of concrete. A length of steel reinforcement bar 550 m long was found at this depth at the location of pile no C-8.

(c) Below this level a weathered sedimentary rock formation with alternating layers of limestone, sandstone and shale were noted. The rock cores were hard and were generally of size 50 mm to 200 mm in length. Heavily weathered portions of the sedimentary rocks contained brownish silty clay and gravels of size 20 to 50 mm.

(d) The steel pile driving shoe was not encountered at either drill hole.

Figure 6 shows the probable condition of the damaged piles as inferred from the drill holes. It was obvious that the lower portions of the two piles investigated were severely damaged and the piles had become shorter than their original lengths. The tips of the shortened piles rested probably on hard sedimentary rocks. Concrete fragments from the broken portion of the piles were probably forced to move upward inside the piles or outward outside the piles. The reinforcing steel bars were severely bent. It was considered that the piles had probably broken at a short depth below the measured elevation of the pile bottom inside the pile.

GEOLOGY OF THE AREA

The sedimentary rock formations at the site belong to the Jurong Formation (P.W.D., 1976) and is covered by recent deposits of sand, clay and lateritic soils. The Jurong Formation consists of a series of sedimentary rocks believed to have been formed in the late Triassic to early Jurassic period (about 220 million years ago). Severe folding and faulting of the Jurong Formation is believed to have taken place subsequent to deposition, when the southern sedimentary formations moved against the granitic mass in the north of the Republic. In the present state it therefore exists in alternate layers of mudstone/shale, sandstone, tuff and some conglomerate steeply dipping at places and in various states of weathering. A 'colluvium' type of loose stratum consisting of sand, gravel and very sticky clay with large numbers of rounded boulders forms the overburden above the sedimentary rock. The thickness of the loose colluvium overburden varies considerably from about 15 m to over 30 m. Standard Penetration Test (SPT)N values in the colluvium were found to vary from about 10 to 20. At some places SPT values below 10 were noted at large depths indicating pockets of very loose material in the colluvium.

The colluvium deposited on top of the sedimentary rock formation at the site is believed to have resulted from erosion of the nearby Pasir Panjang Hill. The areas adjacent on either side of the site are known to have deep layers of soft marine clays deposited over the sedimentary rock formation. The colluvium could therefore have resulted only as a result of erosion and transportation of the soil from the land well before the deposition of marine clay in the area took place. This erosion is considered to have taken place at the time of lower sea levels near the end of the Pleistocene epoch. Subsequent rise and fall of the sea level would then have caused the finer marine sediments to be deposited in the surrounding areas. Large movement of soil and rock is associated with cutting deep erosional channels in the rock bed exposing harder rock below. Figure 7 is a contour map of the sedimentary rock surface as inferred from the piling records. Many deep narrow channels on the rock surface may be noted traversing in a general direction perpendicular to the coast line which is the most probable direction the erosion channels could have been cut because of the general manner of transportation of soil from Pasir Panjang Hill.

(Contours in metres below A.C.D)
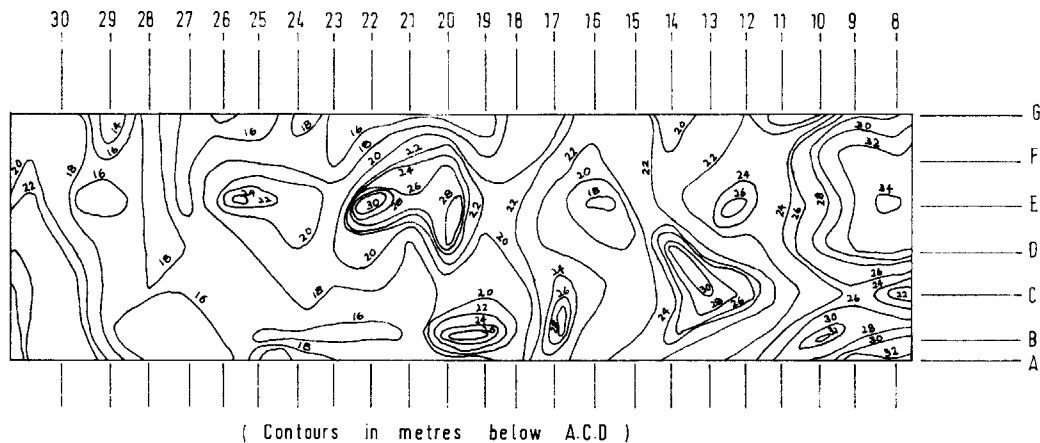
Fig. 7. Contour Map of Suggested Hard Strata.

Figures 8 and 9 show longitudinal sections of the wharf along pile grid C, where the two piles were drilled through, and pile grid E. The extremely undulating hard sub-bottom, where most of the piles were terminated, is evident. The damaged piles along these grids, with more than 2.5 m difference in length when measured inside the pile hollow, are indicated in these figures.
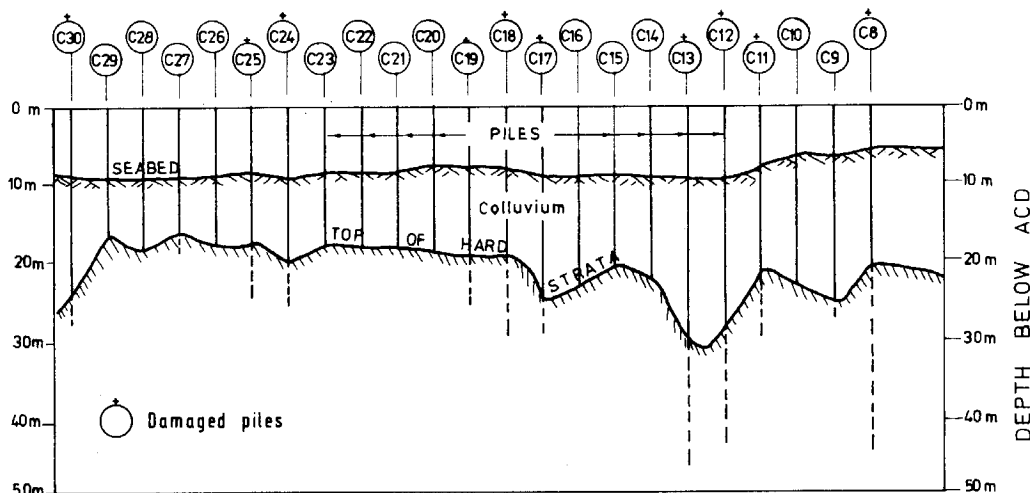
Fig. 8. Cross Section along Pile Grid C.
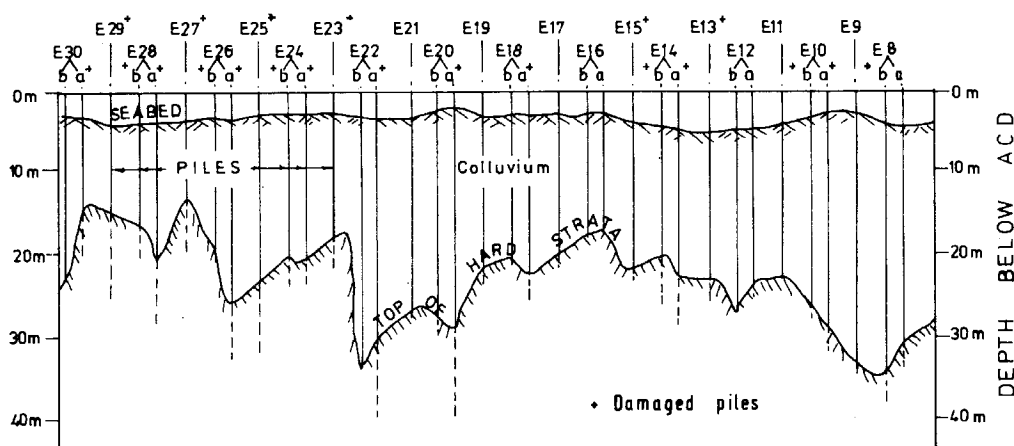
Fig. 9. Cross Section along Pile Grid E.

LOW CARBON HEAT TREATED PRESTRESSING STEEL

In prestressed concrete piles usually high tensile steel strands are used as prestressing tendons. For the manufacture of the hollow prestressed concrete piles at Pasir Panjang, low carbon, heat treated prestressing steel was used for prestressing tendons. The low carbon prestressing steel was manufactured by special techniques using high frequency induction heating of low alloy steel. The pile reinforcing cage was prepared by welding the helical bars around the high tensile steel bars by a warm temperature spot welding method in a

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mechanised cage making machine. Loss in tensile strength of the prestressing steel arising from the welding process was suspected and to determine this effect several pieces of the prestressing steel were tested for tensile strength with and without welding. The results are summarised in Table 2. These tests showed that a very slight reduction in tensile strength could have resulted in the welding process. It was also noted that the welded bars under test failed before reaching the proof stress indicating that, comparatively, the welded bars could have become slightly more brittle as a result of the welding process. However, slight variation in the properties of the steel was not considered sufficient to seriously affect the strength of the piles.

Table 2. Tensile strength of low carbon, heat treated prestressing steel with and without welding.

With Welding		Without Welding	
Effective Area (mm^2)	Tensile Strength (N/mm^2)	Effective Area (mm^2)	Tensile Strength (N/mm^2)
61.20	1542	61.32	1595
61.20	1598	61.32	1587
61.32	1539	61.20	1570
60.18	1547	61.21	1555
60.18	1544	60.35	1551
60.34	1551	60.35	1551

PILE DRIVING HAMMER

The prestressed concrete piles were driven with a Kobe KB-42 diesel hammer. The operation of diesel hammers is different from that of other types of impact hammers. In diesel pile driving hammers, the ram impacts with a steel anvil. Initially, the ram is raised to the start position and dropped. Prior to impact air is compressed between the falling ram and anvil and fuel is injected. Upon impact the fuel-air mixture ignites and the resulting combustion force drives the ram upwards to the starting position again and the cycle continues. In very soft ground, where the pile penetration resistance is small, repeated explosions cannot occur and hence continued operation of the hammer is not possible. As penetration resistance increases, more fuel is injected and better atomization of fuel takes place increasing the energy generated from combustion. Greater combustion energy results in larger ram strokes and increased energy input into the pile. If the pile penetration is negligible because of very hard sub-bottom conditions, there is a possibility for very high compressive stresses to build up close to the pile toe under large ram strokes resulting in cracks or even crushing of the pile toe. Continued cracking and crushing of the lower portion of the pile is therefore

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possible if the hammer operation is not controlled as soon as the pile toe reaches the hard sub-bottom.

In a case where the pile penetrates progressively through relatively harder layers until it reaches the rock layer, a gradual build up in pile resistance is likely. However, in the present case the pile penetrated easily through the loose top colluvium layer with a small number of hammer blows and pile penetration was reduced abruptly when the pile toe reached the very hard rock below. The pile driving was controlled by the number of blows required for 0.5 m pile penetration. When a pile comes to set suddenly, a number of hammer blows might result in the case of rapid acting diesel hammers (60 blows/minute) before the hammer operation is stopped. In such cases, large compressive stresses could build up at a pile toe for short periods of time possibly resulting in serious pile toe damage. These stresses could be even greater if the pile toe comes to rest on hard pinnacles or steep slopes of an undulating sub-bottom as shown in Figures 7 and 8.

REMEDIAL WORKS

A number of additional piles had to be driven where severe pile damage was suspected. Better control in pile driving and strengthening of the pile toe reduced the number of damaged piles in the remaining piles driven at the site. An extension of the wharf became necessary soon after the completion of the Phase II wharves to provide an additional ocean-going berth. Soil investigation showed similar sub-soil conditions in the area. The contractor for the wharf extension used similar spun piles of the same size for this work but these piles were driven with a steam-activated single acting impact type of pile hammer. The steam hammers are slower than diesel hammers (20 to 30 blows/minute), have less complicated mechanical and thermodynamic cycles and are therefore easier to control. The contractor also acted on his previous experiences at the site to guide him and the work was done with very little damage to piles during this extension of the work.

CONCLUSIONS

Detailed geological investigation and careful selection of a suitable pile driving hammer play an important role in successful pile installation. A case study of extensive pile damage suffered during the construction of a wharf structure is described to illustrate these points prior to pile installation. A compatible pile-soil-hammer system has to be first chosen and such a system has then to be further evaluated in the light of the geology of the area.

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PREDICTION OF MOISTURE FLOW
AND RELATED SWELLING OR SHRINKING
IN UNSATURATED SOILS

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SYNOPSIS

The engineering behavior of unsaturated soils is often most easily understood in terms of changes in natural moisture content. Therefore, the prediction of moisture flow and related swelling or shrinking is of importance to engineers concerned with the design of shallow foundations and other related structures. A theoretical model is presented to predict the moisture flow in unsaturated soil continua as the result of hydraulic, vapour and thermal gradients. Experimentally verified constitutive relationships are used to define the volume changes of the unsaturated soil. A partial differential heat flow equation (i.e., for above freezing conditions) and two partial differential transient flow equations (i.e., one for the water phase and the other for the air phase) are derived and solved simultaneously. The solutions give the temperature and the pore pressure distributions with space and time in an unsaturated soil. The computed pore pressures allow the computation of the change in soil suction with time. The computed pressure changes are used to determine the quantity of moisture flow and to predict the rate of swelling and/or shrinking during the transient process as well as the ultimate swell and/or shrinkage in an unsaturated soil. The main part of the proposed analytical model is tested using a case study from the city of Regina, Saskatchewan, Canada. Reasonable agreement is noted with the observed rate and amount of swelling.

INTRODUCTION

A large number of engineering structures founded near the surface of deposits of swelling and shrinking clay soils undergo significant ground movements and pose a serious problem to engineers around the world. In the Prairie regions of Western Canada the glacio-lacustrine soil deposits cover several thousand square miles. In South-Central Saskatchewan there are thick deposits of clay of high plasticity, which are often identified as "problem soils". These soils are generally unsaturated for varying depths for part or all of the year because the moisture received in the form of total precipitation is less than the amount of moisture lost by evaporation and evapotranspiration. As a result, the pore-water pressure is negative and the soil is said to have "suction". The engineering behaviour of these soils is commonly described in terms of changes in natural water content.

There are a number of engineering problems in which the movement of moisture through unsaturated soils play a primary role. However, these

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problems are difficult to analyze since a satisfactory moisture flow formulation, which is coupled with realistic microclimatic boundary conditions, has not been available. The microclimatic condition can be characterized in terms of factors such as temperature, relative humidity, wind velocity, precipitation and evaporation. These factors must be quantified as boundary conditions and used in conjunction with the analytical formulation which considers moisture movement in both the liquid and vapour phases.

An attempt is made in this paper to present briefly the theoretical formulations developed for the moisture flow in the liquid and vapour phases of an unsaturated soil. The use of the proposed analytical model is demonstrated using a study from the city of Regina, Saskatchewan. The case study measurements were made by the Division of Building Research, National Research Council, Saskatoon. The study consisted of observing the vertical slab and subsoil movement of an instrumented slab-on-grade building in Regina, Saskatchewan. This building suffered serious floor movement as the result of a plumbing leak below the floor slab. Periodic records of ground movements were made over a period of several years (1961-1972).

THEORY (REQUIRED PHYSICS)

One-dimensional transient flow equations for saturated soils have basically involved equating the time differential of the constitutive equation for the soil structure to the divergence of the velocity of flow of water from an element (TERZAGHI, 1943). Two- and three-dimensional seepage analyses for saturated soils have been either coupled or uncoupled from the equilibrium requirements. Unfortunately, a similar type of formulation for seepage analysis in unsaturated soils has been lacking. This has been the result of an incomplete understanding of the stress state, constitutive relations and other physical relations for an unsaturated soil.

This paper will first outline the necessary physical relations for an unsaturated soil and provide a transient flow model for an unsaturated soil which is consistent with that commonly used for saturated soils. The formulations are for the one-dimensional case.

Stress State Variables

An element of an unsaturated soil is considered to be a mixture with two phases (i.e., soil particles and contractile skin) that come to equilibrium under an applied stress gradient and two phases (i.e., the air and water) that flow under applied pressure gradients. The air phase is assumed to be continuous.

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$(\sigma_y - u_a)$ and $(u_a - u_w)$ are used as independent stress state variables for an unsaturated soil where $\sigma_y =$ total stress in the y-direction, $u_a =$ pore-air pressure, and $u_w =$ pore-water pressure (FREDLUND & MORGENSTERN, 1977). These stress state variables have been tested by means of "null type" tests and have become widely used for describing the behaviour of unsaturated soils.

Continuity Equation

The deformation state variables required to describe changes in volumes associated with each part of an element must be consistent with the continuity requirement for an unsaturated soil. If the soil particles are assumed to be incompressible and the volume change of the contractile skin assumed internal to the element, the continuity equation is then written,

$$\frac{\Delta V}{V} = \frac{\Delta V_w}{V} + \frac{\Delta V_a}{V} \qquad \dots (1)$$

where: V = total volume of the element,

ΔV = change in volume of soil structure or overall element,

ΔV_w = change in volume of water in the element, and

ΔV_a = change in volume of air in the element.

Constitutive Relations

The volumetric continuity requirement for an unsaturated soil shows that at least two constitutive relations are required to define volume weight relations.

The constitutive relation for an unsaturated soil can be summarized as follows (FREDLUND & MORGENSTERN, 1976).

Soil Structure

$$\Delta V = [m_1^s \, d(\sigma_y - u_a) + m_2^s \, d(u_a - u_w)] \, V \qquad \dots (2)$$

where: m_1^s = compressibility modulus of soil structure when $d(u_a - u_w)$ is zero.

m_2^s = compressibility modulus of soil structure when $d(\sigma_y - u_a)$ is zero.

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Water Phase

$$\Delta V_w = [m_1^w d(\sigma_y - u_a) + m_2^w d(u_a - u_w)]V \dots\dots\dots\dots\dots\dots (3)$$

where: m_1^w = slope of the $d(\sigma_y - u_a)$ versus volume of water plot when $d(u_a - u_w)$ is zero.

m_2^w = slope of the $d(u_a - u_w)$ versus volume of water plot when $d(\sigma_y - u_a)$ is zero.

Air Phase

$$\Delta V_a = [m_1^a d(\sigma_y - u_a) + m_2^a d(u_a - u_w)]V \dots\dots\dots\dots\dots (4)$$

where: m_1^a = slope of the $d(\sigma_y - u_a)$ versus volume of air plot when $d(u_a - u_w)$ is zero, and

m_2^a = slope of the $d(u_a - u_w)$ versus volume of air plot when $d(\sigma_y - u_a)$ is zero.

Any two of the three constitutive relations can be used in an independent manner. Quantitatively, the air phase constitutive relationship is equal to the difference between the soil structure constitutive relationship and the water phase constitutive relationship.

Flow Laws

Flow equations are required for the water and air phases.

Darcy's Law

Darcy's law can be used to describe the flow under a hydraulic gradient in an unsaturated soil (CHILDS & COLLIS-GEORGE, 1950).

$$q_1 = -\left[k_w \frac{\partial h_w}{\partial y}\right] \dots\dots\dots\dots\dots (5)$$

where: q_1 = (liquid) water flux,

k_w = coefficient of hydraulic conductivity with respect to the water phase,

h_w = hydraulic head in the water phase $\left(\text{i.e., } \frac{u_w}{g\rho_w} + \gamma\right)$,

ρ_w = density of liquid water,

g = acceleration due to gravity,

γ = elevation head (is assumed zero in this analysis), and

y = direction of flow.

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The coefficient of hydraulic conductivity is shown as a constant; however, it can be made a function of degree of saturation, void ratio, water content or any combination of these properties (COREY, 1957). Its magnitude must simply be revised as the numerical analysis proceeds.

Fick's Law

BLIGHT (1971) used a simple form of Fick's law to describe the mass of air flowing through an unsaturated soil continuum.

$$\frac{\partial m}{\partial t} = - D \frac{\partial p}{\partial y} \qquad \text{...................... (6)}$$

where: m = mass of air in the element,

D = transmission constant having the same units as coefficient of hydraulic conductivity,

p = absolute air pressure (i.e., $u_a + u_{atm}$),

u_{atm} = atmospheric air pressure, and

t = time.

Modified Fick's Law

DE VRIES (1975) used a modified Fick's law to describe the vapour flux under conditions of uniform and constant total pressure,

$$q_v = \frac{-1}{g\rho_{wv}} \left[D_{vap}\left(\frac{\omega^*}{R\theta}\right) \frac{\partial p_v}{\partial y} \right] \qquad \text{...................... (7)}$$

where: q_v = water vapour flux,

D_{vap} = molecular diffusivity of water vapour in air,

ω^* = molecular weight of water vapour,

R = universal gas constant,

θ = absolute temperature,

p_v = vapour pressure, and

ρ_{wv} = density of water vapour.

Pore Pressure Parameters

FREDLUND (1976) and HASAN & FREDLUND (1980) defined a pore pressure parameter, B_{aw}, to predict the changes in pore-water pressure resulting form a change in pore-air pressure. The change in pore-air pressure of interest in this paper is the result of the thermal gradient imposed on the soil.

$$B_{aw} = \frac{\Delta u_a}{\Delta u_w} \qquad\qquad \text{................ (8)}$$

where: Δu_a = change in pore-air pressure, and

Δu_w = change in pore-water pressure.

The B_{aw} pore pressure parameter is a function of the compressibility of the air-water mixture and the compressibility of the soil (FREDLUND, 1976).

THEORY (TRANSIENT PARTIAL DIFFERENTIAL EQUATIONS)

Three partial differential equations are required for the rigorous non-isothermal analysis of an unsaturated soil.

Heat Flow Equation

The Fourier diffusion equation is used to describe conductive heat transfer for above freezing conditions in the soil mass. The equation expresses the heat flow rate in terms of the thermal conductivity and the temperature gradient. The thermal conductivity and heat capacity values used in this analysis are given in Tables 1 and 2.

$$\frac{\partial \theta}{\partial t} = \alpha \frac{\partial^2 \theta}{\partial y^2} \qquad\qquad \text{............... (9)}$$

where: θ = temperature,

α = thermal diffusivity factor = $\lambda/c\rho_s$,

λ = thermal conductivity,

c = heat capacity, and

ρ_s = soil bulk density.

The heat flow equation is solved using an explicit finite difference method. The solution gives the temperature dissipation with space and time and must be solved at least one time step in advance of the water and air phase equations.

Water Phase Partial Differential Equation

The water phase partial differential equation considers the total flux of (liquid) water in an element of an unsaturated soil continuum. The net flux of (liquid) water is the result of imposed hydraulic and vapour pressure (or in other words the relative humidity) gradients at the surface boundary of the element. Equating the net flux of (liquid) water to the time differential of the water phase constitutive relationship, rearranging and simplifying,

gives the water phase partial differential equation (DAKSHANAMURTHY & FREDLUND, 1981b).

$$\frac{\partial u_w}{\partial t} = C_w \frac{\partial u_a}{\partial t} + c_{vI}^w \frac{\partial^2 u_w}{\partial y^2} + c_{vv}^w \frac{\partial^2 p_v}{\partial y^2} \dots\dots\dots (10)$$

where: $C_w = -(1 - m_2^w/m_1^w) / (m_2^w/m_1^w)$ and is called the interactive constant associated with the water phase equation. This equation is further simplified by letting $R_w = m_2^w/m_1^w$. When the soil is saturated, R_w approaches unity,

$$c_{vI}^w = \frac{1}{R_w} \frac{k_w}{\rho_w g} \frac{1}{m_1^w} \dots\dots\dots\dots (11)$$

The coefficient of consolidation for the water (liquid) phase, and

$$c_{vv}^w = \frac{1}{R_w} D_{vap} \left(\frac{\omega^*}{R\theta}\right) \frac{1}{\rho_{wv} g} \frac{1}{m_1^w} \dots\dots\dots\dots (12)$$

The coefficient of consolidation for the water (vapour) phase.

The vapour pressure in Equation 10 can be expressed as the product of the saturation vapour pressure, p_v^s, and the relative humidity, h (DE VRIES, 1975).

$$p_v = p_v^s h \dots\dots\dots\dots (13)$$

The relative humity, h, can also be expressed in terms of total potential, ϕ, molecular weight of water vapour, ω^*, specific volume of water vapour, V_w^o, universal gas constant, R, and the absolute temperature, θ (VAN HAVEREN & BROWN, 1972).

$$h = e \left(\frac{\phi \omega^* V_w^o}{R\theta}\right) \dots\dots\dots\dots (13)$$

where: $\phi = [(u_a - u_w) + \pi]$ (i.e., total potential)

π = osmotic suction; assumed to be constant and equal to 102 kPa for this study.

Air Phase Partial Differential Equation

The air phase is compressible and flow occurs in response to a pressure gradient (i.e., the gravity term is negligible). The constitutive relationship for the air phase defines the volume of air in the element for any combination of the total, water and air pressures. Equating the net mass flux of dry or moist

air through the element to the time differential of the air phase constitutive relationship, rearranging and simplifying, gives the modified air phase partial differential equation (DAKSHANAMURTHY & FREDLUND, 1981a).

$$\frac{\partial u_a}{\partial t} = C_a \frac{\partial u_w}{\partial t} + C_\theta \frac{\partial \theta}{\partial t} + c_v^a \frac{\partial^2 u_a}{\partial y^2} \quad \dots\dots\dots\dots\dots (15)$$

$$\text{where } C_a = \frac{- m_2^a/m_1^a}{(1 - m_2^a/m_1^a) + \dfrac{(1 - S)n}{m_1^a (\Delta u_a + u_{atm})}} \quad \dots\dots\dots\dots\dots (16)$$

the interactive pressure constant associated with the air phase equation. This equation is further simplified by letting $R_a = m_2^a/m_1^a$,

Δu_a = the difference between the pore-air pressure and standard atmospheric conditions.

$$C_\theta = \frac{1}{\theta} \left[\frac{(1 - S)n (\Delta u_a + u_{atm})}{(1 - R_a) (\Delta u_a + u_{atm})m_1^a + (1 - S)n} \right] \quad \dots\dots\dots\dots\dots (17)$$

the interactive thermal constant associated with the air phase equation,

where S = degree of saturation, and
      n = porosity.

$$c_v^a = \frac{DR\theta}{\omega} \left[\frac{1}{(1 - R_a) (\Delta u_a + u_{atm})m_1^a + (1 - S)n} \right] \dots\dots\dots\dots\dots (18)$$

the coefficient of consolidation for the air phase.

The dissipation of the excess pressures of the pore-air and pore-water phases are obtained by solving Equations 10 and 15 simultaneously. The magnitudes of coefficients of consolidation for the water and air phases viz., c_{v1}^w, c_{vv}^w and c_v^a are primarily dependent upon the coefficients of hydraulic conductivity values for the water and air phases.

TRANSLATION OF PORE PRESSURES TO VOLUME-WEIGHT SOIL PROPERTIES

The solution to Equations 9, 10 and 15 enables the prediction of changes in the temperature, the pore-water pressure, the pore-air pressure, the vapour pressure, the partial pressure during the transient process under complex environmental changes at the boundary. The pressure changes are, in turn,

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used to determine the change in void ratio, degree of saturation, and the moisture content. The volume change of an unsaturated soil can be converted into a prediction of swell or shrinkage by substituting the change in stress state variables, for a given period of time, into the respective constitutive relations.

Void Ratio (Swell or Shrinkage)

The change in volume of the element computed as per Equation 2 is equal to the change in void ratio, Δe. The total volume of the element, V, is (1+e). The void ratio at various times during the transient process can be written.

$$e = e_i + \Delta e \qquad \qquad \text{............} \quad (19)$$

where: e = void ratio at any time,
e_i = void ratio at the beginning of the transient process, and
Δe = change in void ratio during the transient process.

The change in void ratio enables the computation of the amount of swell or shrinkage, ΔH, during the transient process.

$$\Delta H = H_i \left[\frac{\Delta e}{1 + e_i} \right] \qquad \qquad \text{............} \quad (20)$$

where H_i = initial thickness of the soil layer.

Water Content

The change in volume of water in a unit volume of soil can also be computed by substituting the change in stress state variables, at any time, into the water phase constitutive relation as given in Equation 3.

The gravimetric water content at any time during the transient process can then be written:

$$w = w_i + \Delta V_w \frac{\rho_w}{\rho_d} \qquad \qquad \text{............} \quad (21)$$

where: w = water content at any time,
w_i = water content (gravimetric) at the beginning of the transient process, and
ρ_d = dry density of the soil.

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Degree of Saturation

The degree of saturation is calculated using the following- volume-weight relationship.

$$Se = wG_s \qquad \text{...................... (22)}$$

where G_s = specific gravity of the soil solids.

A CASE STUDY

The main part of the theoretical model was checked using a case study from the city of Regina, Saskatchewan. The study was conducted by the Division of Building Research, National Research Council, Saskatoon, starting in 1961 (HAMILTON, 1965). The case study analyzed involved the movement of the concrete floor slab of a light industrial structure, Lorne Street, North Central Regina, Saskatchewan. The site was instrumented with neutron moisture meter access tubes to estimate the moisture content profile in-situ. Three vertical ground movement, spiral foot movement gauges were also installed and referenced to a deep bench mark. Several tensiometers were installed and surveys were performed on the floor levels. The movement gauges were embedded at 0.6 m, 0.9 m and 2.6 m depths, respectively, below the concrete floor slab. The vertical ground movements were recorded periodically over the period 1961 to 1972. In 1962 a rather sudden floor heave of 8.9 cm had developed in the central portion of the slab directly above the centre line of a subfloor plumbing trench. This heave caused a maximum angular distortion of 1/20 and seriously damaged interior partitions. At the point of maximum heave a leak was discovered in the hot water line from which it was estimated that some 7,500 gallons of water had escaped. The ground under the floor slab became saturated and the water migrated for a considerable distance along the length of the back-filled plumbing trench.

EXAMPLE PROBLEMS

Measurements from the case study did not allow the verification of all aspects of the proposed analytical model. Therefore, several examples were also considered to demonstrate the solution of the three partial differential transient flow equations.

The simultaneous solutions of all three differential equations enables the prediction of the transient and ultimate values of ground movements (i.e., swelling or shrinking) and the resulting volume-weight soil properties during the transient process. Four examples are solved in this paper. These are:

i) isothermal swelling, ii) non-isothermal swelling, iii) isothermal consolidation, and iv) non-isothermal consolidation. Example No. 1 (i.e., isothermal swelling) is used to test the theoretical analysis by attempting to fit the analysis with the field data observed in a case study carried out at Regina, Saskatchewan. The other three examples are used to demonstrate the effect of environmental changes such as imposed thermal gradients and vapour pressure gradients. No attempt has been made in this paper, for lack of data, to compare the solution of these three examples with independently observed values.

Table 1. Summary of classification tests, compressibility and hydraulic conductivity values of Regina clay.

Specific Gravity $\qquad$ 2.73

Atterberg Limits
Liquid Limit $\qquad$ 82%
Plastic Limit $\qquad$ 33.7%
Shrinkage Limit $\qquad$ 11.9%
Plasticity Index $\qquad$ 48.3%

Grain Size Distribution
% Sand Sizes $\qquad$ 41%
% Silt Sizes $\qquad$ 10%
% Clay Sizes $\qquad$ 49%

*Compressibility**
$m_i^w = 0.000170/\text{kPa}$
$m_i^a = 0.00004/\text{kPa}$
$R_w = 0.95, \quad R_a = -2.75$

*Hydraulic Conductivity**
$k_w = 0.8 \times 10^{-9}$ m/sec
$D_{vap} = D_{air} = 3.0 \times 10^{-9}$ m²/sec

Volume-Weight Properties
Soil Density, $\rho_s = 1.8$ g/cc
Porosity, $n = 52.56\%$
Degree of Saturation, $S = 52\%$

Thermal Properties
λ = Thermal Conductivity = 0.457 cal/m. sec. °K
c = Heat Capacity = 30 cal/g °K

Physico Chemical Properties
ω^* = Molecular weight of water vapour = 18.015 g/mole
ω = Molecular weight of moist air (80% of water vapour and 20% of air) = 20.2048 g/mole
R = Universal gas constant = 847.825 g.m/mole/°K

*Estimated values from constant volume oedometer tests performed on samples from Lorne Street, North Central Regina.

The first two examples (i.e., isothermal swelling and non-isothermal swelling) were solved assuming a two metres thick layer of Regina clay. In addition, the last two examples (i.e., isothermal consolidation and non-isothermal consolidation) were solved assuming a ten centimetre thick Regina clay. The soil properties of the first series of examples are shown in Table 1. The soil properties of the second series of examples are shown in Table 2. The index properties and other classification tests were carried out by FREDLUND (1964). Both soils are similar. As a result of solving many examples it was observed that only if a relatively small thickness of soil layer was assumed in the analysis, was it possible

Table 2. Summary of classification tests, compressibility and hydraulic conductivity values of Regina Clay.

Specific Gravity	2.83
Atterberg Limits	
Liquid Limit	75.5%
Plastic Limit	24.9%
Shrinkage Limit	13.1%
Plasticity Index	50.6%
Grain Size Distribution	
% Sand Sizes	8
% Silt Sizes	41
% Clay Sizes	51

Compressibility*

$m_1^w = 0.0007614/kPa$

$m_1^a = 0.0003263/kPa$

$R_w = 0.7, R = -0.01$

Hydraulic Conductivity*
  $k_w = 0.6 \times 10^{-10}$ m/sec
  $D_{vap} = D_{air} = 1.0 \times 10^{-9}$ m²/sec

Volume-Weight Properties
  Soil Density, $\rho_s = 1.8$ g/cc
  Porosity, n = 50%
  Degree of Saturation, S = 70%

Thermal Properties
  λ = Thermal Conductivity = 0.4574 cal/m. sec.°K
  c = Heat Capacity = 30 cal/gm °K

Physico Chemical Properties
  ω^* = Molecular weight of water vapour = 18.015 g/mole
  ω = Molecular weight of moist air (80% of water vapour and 20% of air) = 20.2048 g/mole
  R = Universal gas constant = 847.825 g./mole/°K

*Estimated values.

to clearly show the effects of thermal and vapour pressure gradients. There-fore, a ten centimetre thick soil layer was assumed in the second series of the examples.

In all the above examples it was assumed that the subsoil was initially in a known state of stress. It was then assumed that a sudden environmental change such as flooding (i.e., infiltration) or drying (i.e., evaporation) was imposed at the boundary. The boundary conditions used in the examples are given in Tables 3 to 8. Sudden environmental changes such as infiltration or evaporation create a change in the initial boundary conditions. For non-isothermal conditions it is assumed that the pore-air pressure can change as a result of a temperature change and the corresponding change in the pore-water pressure is computed using the B_{aw} parameter. B_{aw} values used in the examples are 0.1 and 0.3. The unsaturated soil mass will eventually equilibrate to the new boundary conditions. The solution showing the temperature, the pore pressure changes with time is obtained by solving Equations 9, 10 and 15. A computer programme was developed to solve these partial differential equations (DAKSHANAMURTHY & FRED-LUND, 1980a).

RESULTS AND DISCUSSION

Based on the solution to Equations 9, 10 and 15, the distribution of pore-water pressure, pore-air pressure, vapour pressure and partial pressure can be computed. The void ratio corresponding to any time can be computed from

Table 3. Boundary conditions for Example No. 1 (Isothermal Swelling).

Variable	Depth (m)	Boundary Conditions	
		Initial	Final
Pore-Water (kPa) (absolute)	0	-816	102
	2	-200	102
Pore-Air (kPa) (absolute)	0	102	102
	2	102	102
Temperature (°C)	0	20	20
	2	20	20
Relative Humidity (%)	0	99.2480	99.9246
	2	99.7015	99.9246
Vapour Pressure (kPa)	0	2.338	2.338
	2	2.338	2.338

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Table 4. Boundary conditions for Example No. 2 (Non-isothermal swelling).

Variable	Depth (m)	Boundary Conditions	
		Initial	Final
Pore-Water (kPa) (absolute)	0	-816	102
	2	-200	102
Pore-Air (kPa) (absolute)	0	102	102
	2	-102	102
Temperature (°C)	0	10	25
	2	10	25
Relative Humidity (%)	0	99.2278	99.9256
	2	99.6894	99.9256
Vapour Pressure (kPa)	0	1.210	3.156
	2	1.220	3.156

Table 5. Boundary conditions for Example 3 (a) (Isothermal consolidation).

Variable	Boundary Conditions	
	Initial	Final
Pore-Water (kPa) (absolute)	-280	-420
Pore-Air (kPa) (absolute)	102	102
Temperature (°C)	20	20
Relative Humidity (%)	99.6425	99.5393
Vapour Pressure (kPa)	2.294	2.294

Equations 2 and 19. Subsequently, the amount of heave can be computed from Equation 20. The water content is computed from Equations 3 and 21 and the degree of saturation from Equation 22.

Example No. 1 (Table 3)

Figure 1 shows the pore-water pressure distribution throughout the clay layer as the result of sudden flooding at the surface. The initial (i.e., equilibrium) pore-water pressure was assumed to be −816 kPa at ground surface

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Table 6. Boundary conditions for Example 3(b) (Isothermal swelling).

Variable	Boundary Conditions	
	Initial	Final
Pore-Water (kPa) (absolute)	-420	-280
Pore-Air (kPa) (absolute)	102	102
Temperature (°C)	20	20
Relative Humidity (%)	99.5393	99.6425
Vapour Pressure (kPa)	2.294	2.294

Table 7. Boundary conditions for Example 4(a) (Non-Isothermal consolidation).

Variable	Boundary Conditions	
	Initial	Final
Pore-Water (kPa) (absolute)	-280	-420
Pore-Air (kPa) (absolute)	102	102
Temperature (°C)	10	25
Relative Humidity (%)	99.6328	99.5455
Vapour Pressure (kPa)	1.188	3.156

and the upper boundary was then instantaneously changed to a value of 102 kPa (i.e., atmospheric pressure) at the surface. It was assumed that the initial pore-water pressure varied linearly with depth and that the initial pore-water pressure at two metres depth was −200 kPa. Although there is no direct verification of these initial boundary values, they are consistent with laboratory test results from one-dimensional, oedometer tests at the light industrial structure, Lorne Street, North Central Regina (YOSHIDA et al., 1981). The relative humidity at the surface was assumed to be essentially the same as

Table 8.   Boundary conditions for Example 4(b) (Non-Isothermal Swelling).

Variable	Boundary Conditions	
	Initial	Final
Pore-Water (kPa) (absolute)	-420	-280
Pore-Air (kPa) (absolute)	102	102
Temperature (°C)	10	25
Relative Humidity (%)	99.5268	99.6473
Vapour Pressure (kPa)	1.188	3.156

the relative humidity at the soil boundary. In other words, it was assumed that there was no water vapour movement when the site was flooded. In addition, under isothermal conditions the pore-air pressure was assumed to remain essentially unchanged.

Figure 2 shows the distribution of (gravimetric) water content throughout the soil layer, during the swelling process. The final water content throughout the soil layer reaches equilibrium at a final value in response to the change in pore-water pressure and the soil modulus. Figure 2 also shows the observed water content profile after an elasped time period of one year, during which time the plumbing leak was noticed. The observed water content shows a reasonable agreement between the predicted and the observed values. The scatter seen in Figure 2 can be attributed to local test plot conditions, such as the presence of extensive shrinkage cracks and fissures in the subsoil at the site. These conditions are responsible for a greater and faster absorption of water in the soil.

Figures 3 and 4 show the distribution of degree of saturation, S, and the void ratio, e, during the transient process under isothermal conditions. The changes in the degree of saturation depend upon the moduli used for soil structure (i.e., m_1^s and m_2^s).

Figure 5 shows the prediction of the total heave for the entire two metres thick soil layer during the transient process. The predicted values of net heave compare well with the test site data (HAMILTON, 1969).
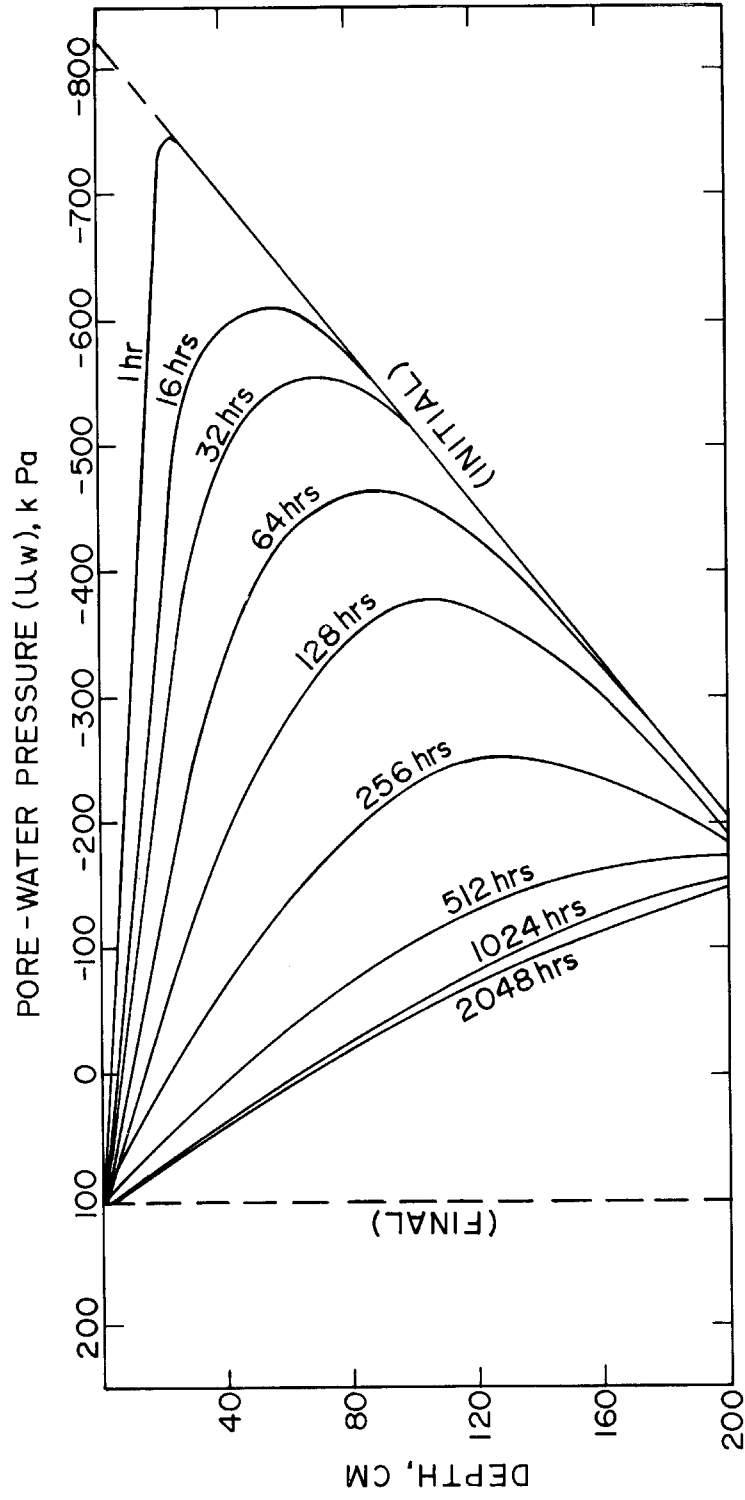
Fig 1. Pore-water pressure distribution for Example No. 1.
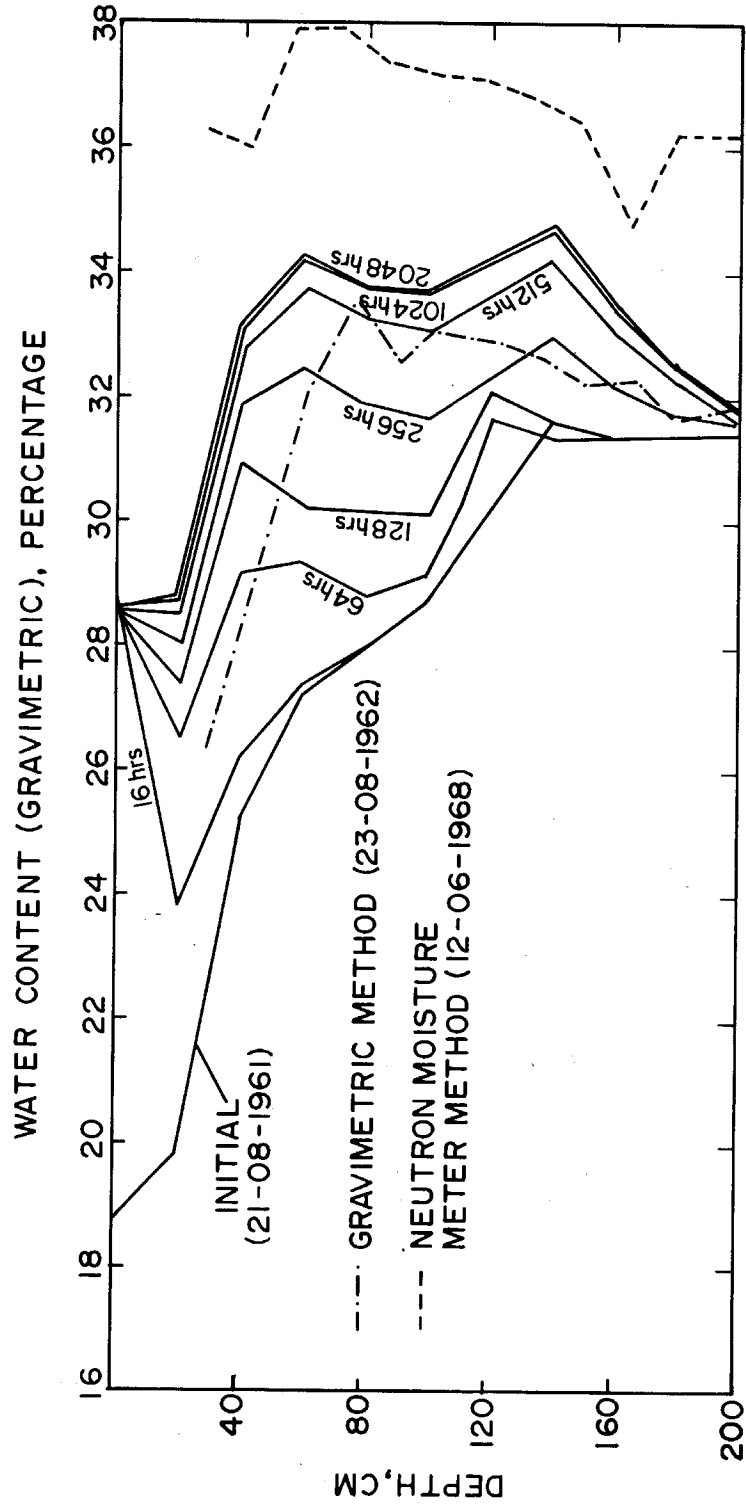
Fig. 2. Water content (gravimetric) change throughout the soil layer for Example No. 1.
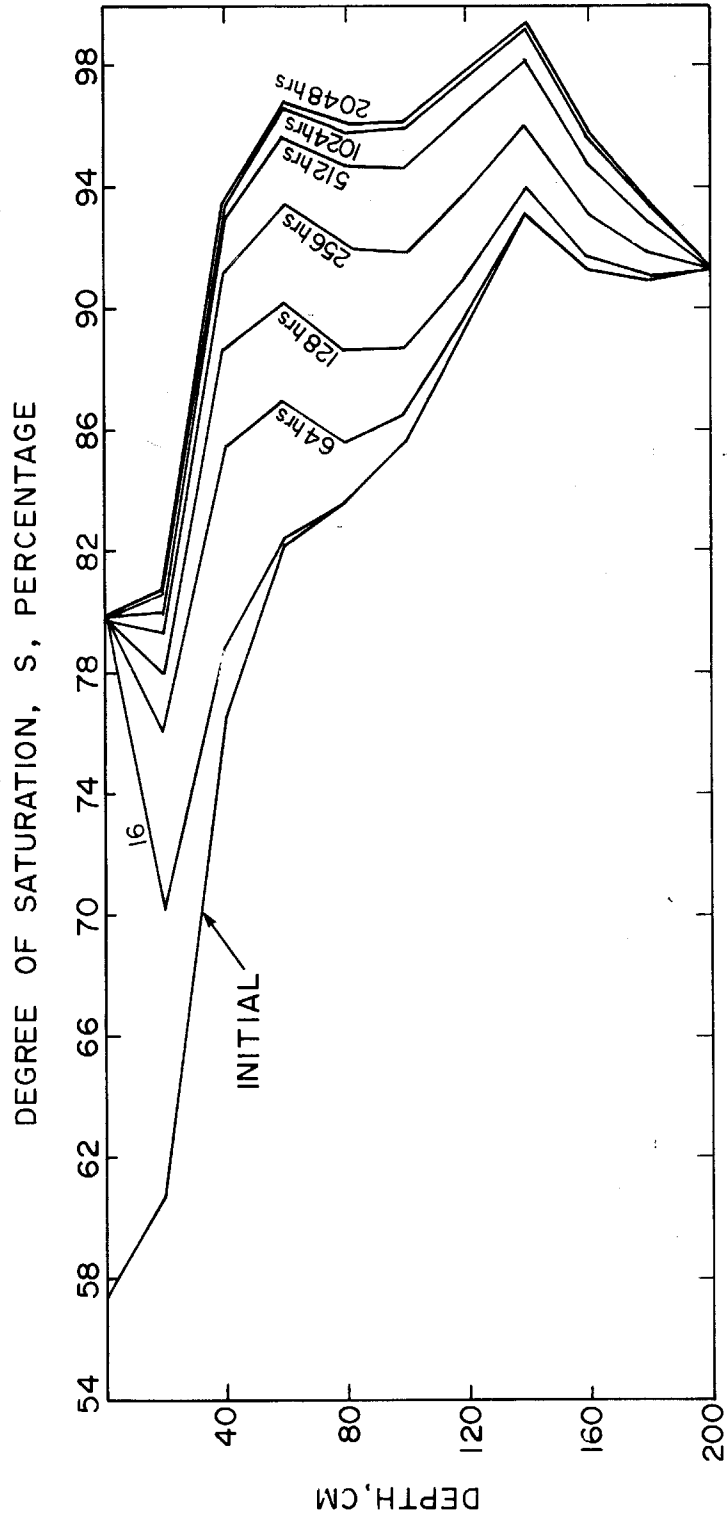
Fig. 3. Degree of saturation change throughout the soil layer for Example No. 1.
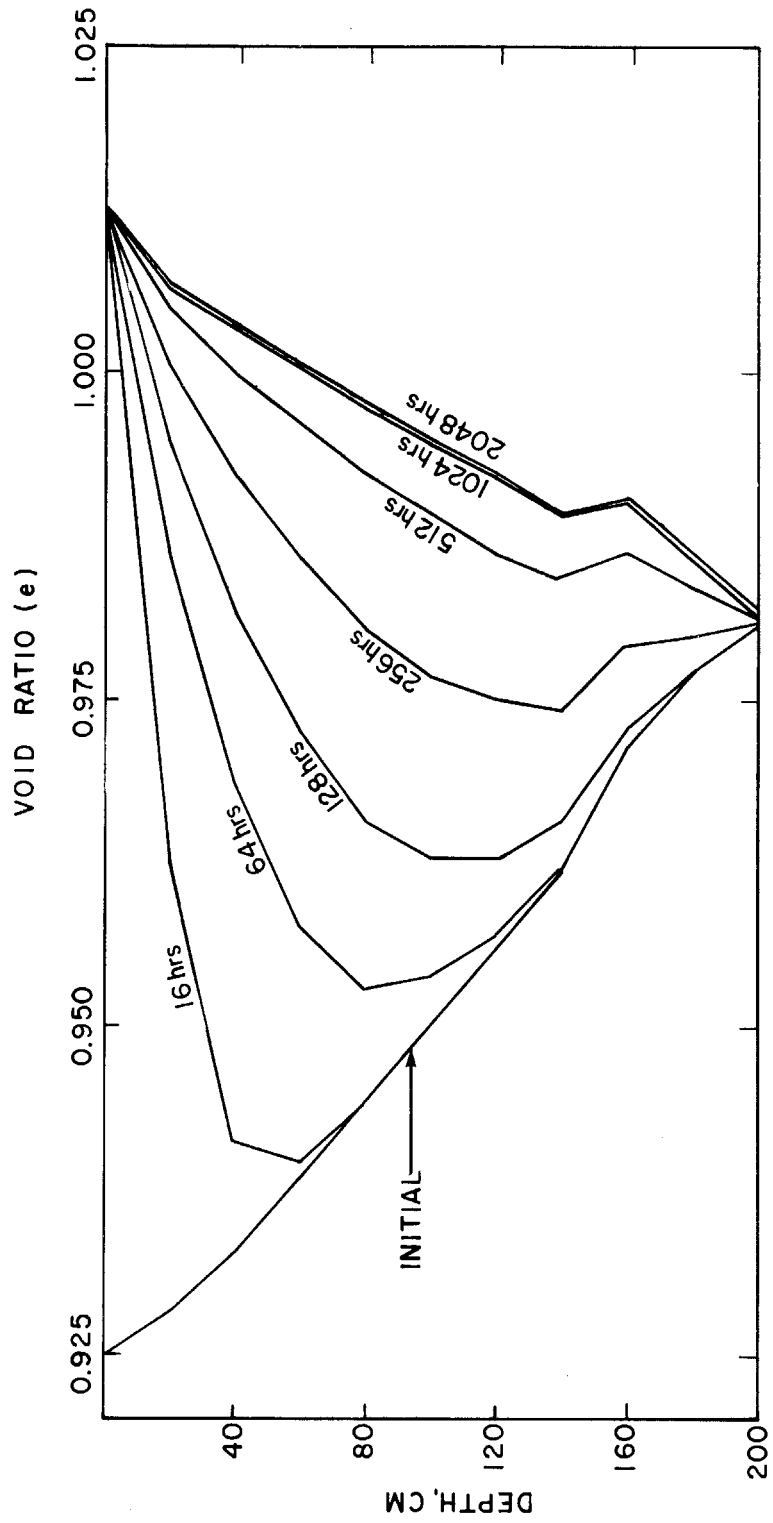
Fig. 4. Void ratio change throughout the soil layer for Example No. 1.
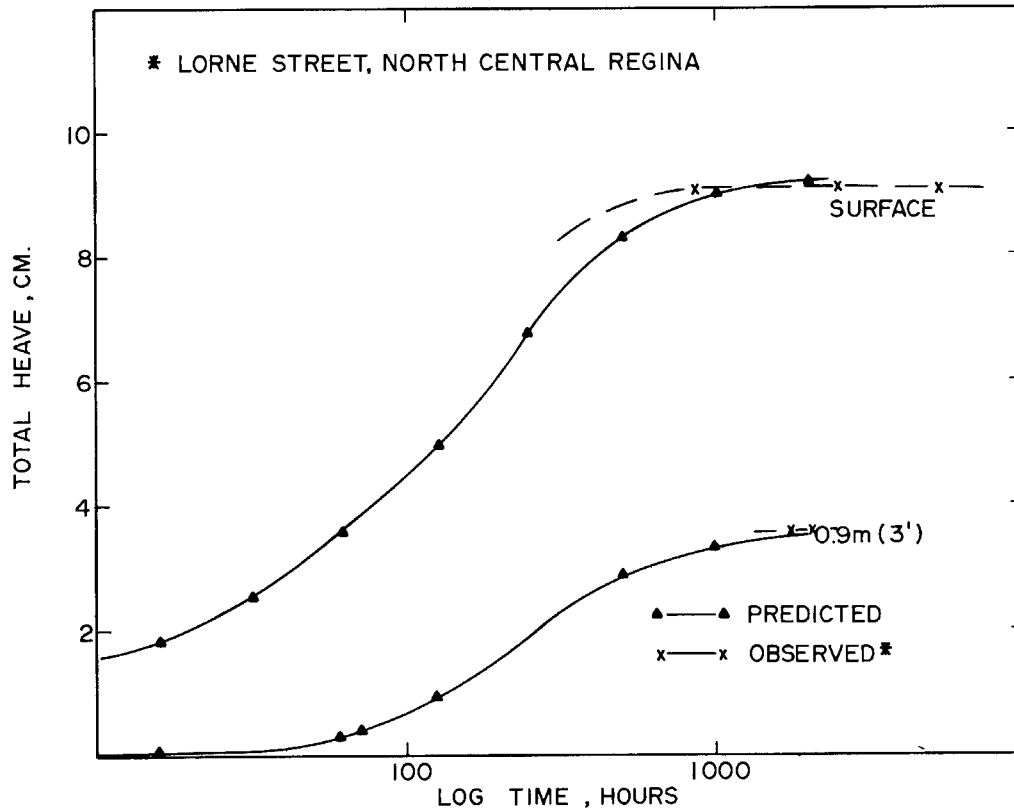
Fig. 5. Total heave log time.

Example No. 2 (Table 4)

Figure 6 shows the temperature isotherms within the clay layer as a result of an increase in temperature from 10°C (283.2°K) to 25°C (298.2° K). The imposed thermal gradient at the surface slowly dissipates to the bottom of the soil layer and should eventually equilibrate to the new boundary condition. In other words, the lower boundary of the soil layer is assumed to be insulated. It should be pointed out that the dissipation of the temperature within the two metres thick soil layer takes considerable time. Figure 6 shows that, even at the end of 2,048 hours (i.e., 86 days), the temperature remains unchanged approximately 90 cm below. Because of the relatively slow change in temperature within the soil layer, the pore-air pressure changes under non-isothermal conditions are insignificant. Figure 1 shows that pore-water pressurer under non-isothermal swelling conditions are the same as those

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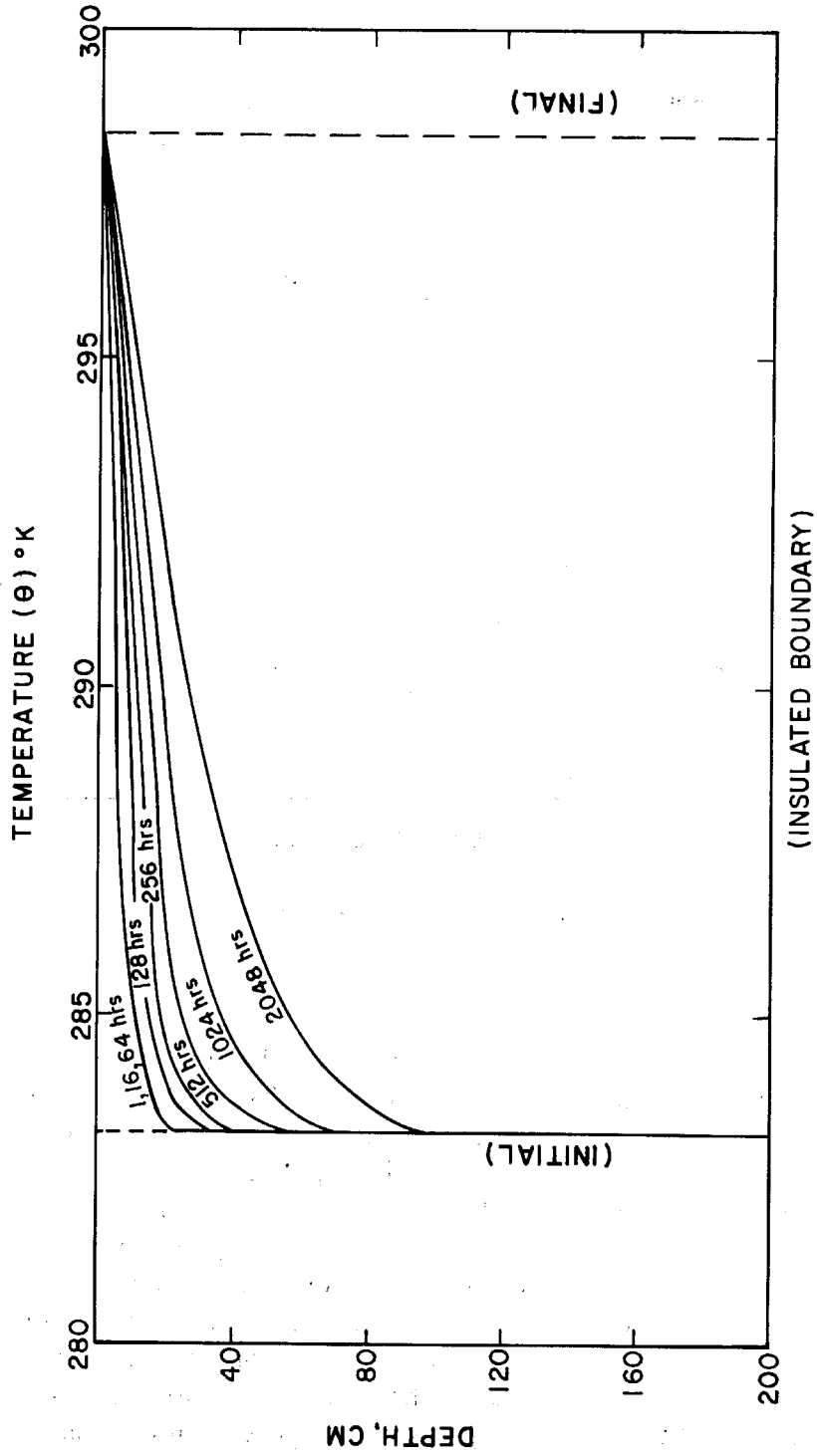
Fig. 6. Temperature isotherms for Example No. 2.
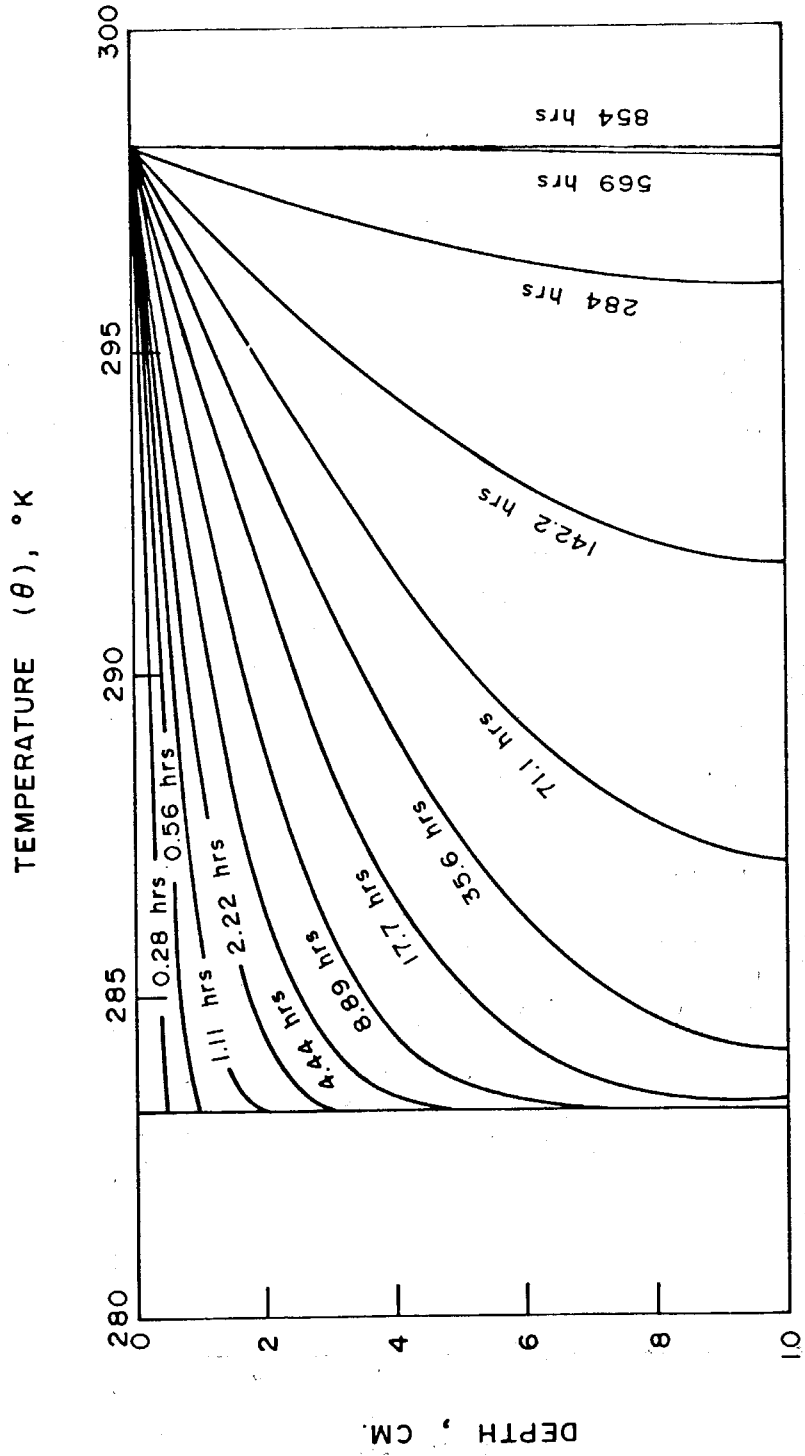
Fig. 7. Temperature isotherms for Example No. 4.
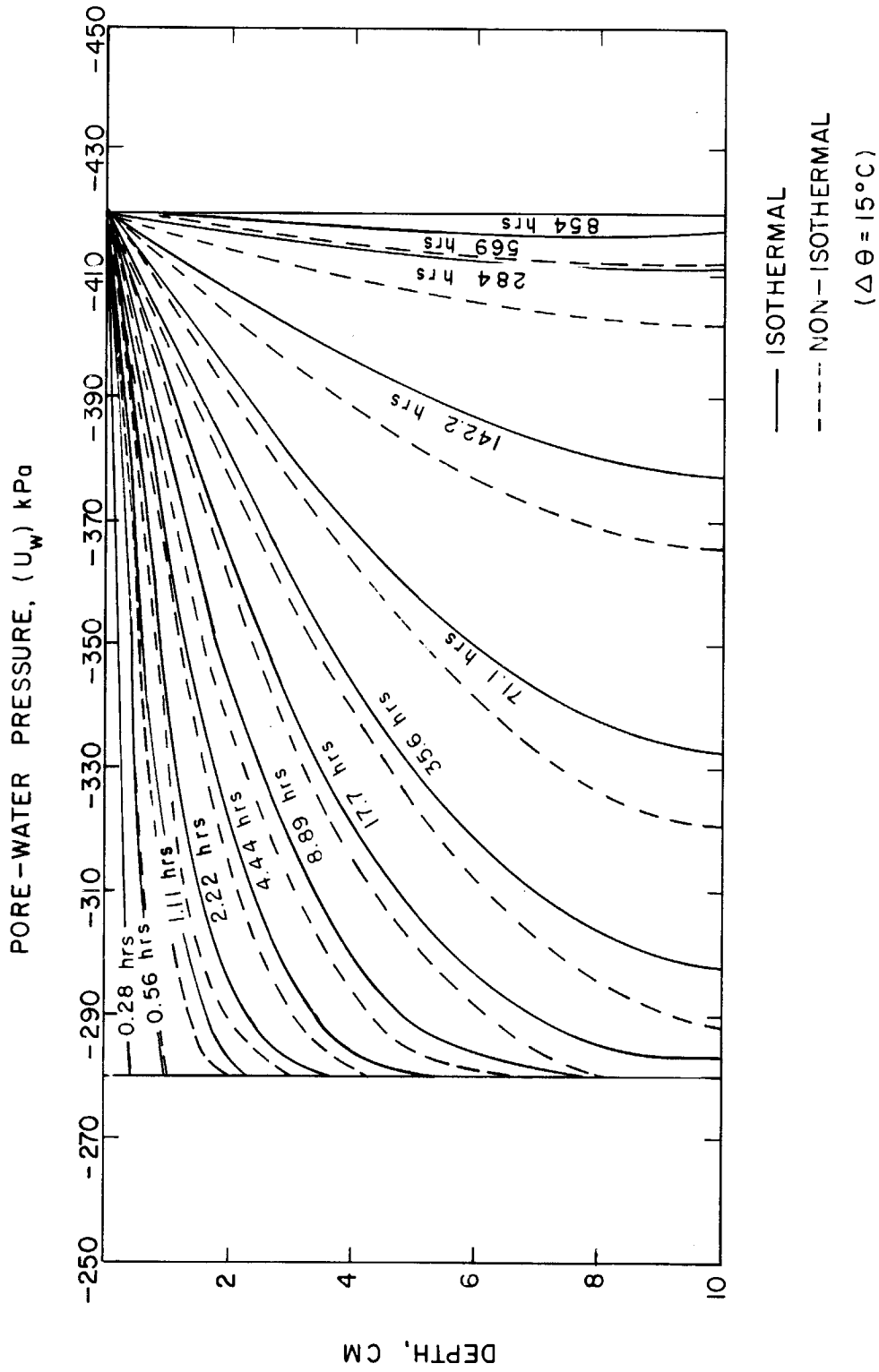
Fig. 8. Pore-water (liquid) pressure distribution (consolidation process) (Examples Nos 3 (a) and 4(a)).

ISOTHERMAL

NON-ISOTHERMAL

$(\Delta \Theta = 15°C)$

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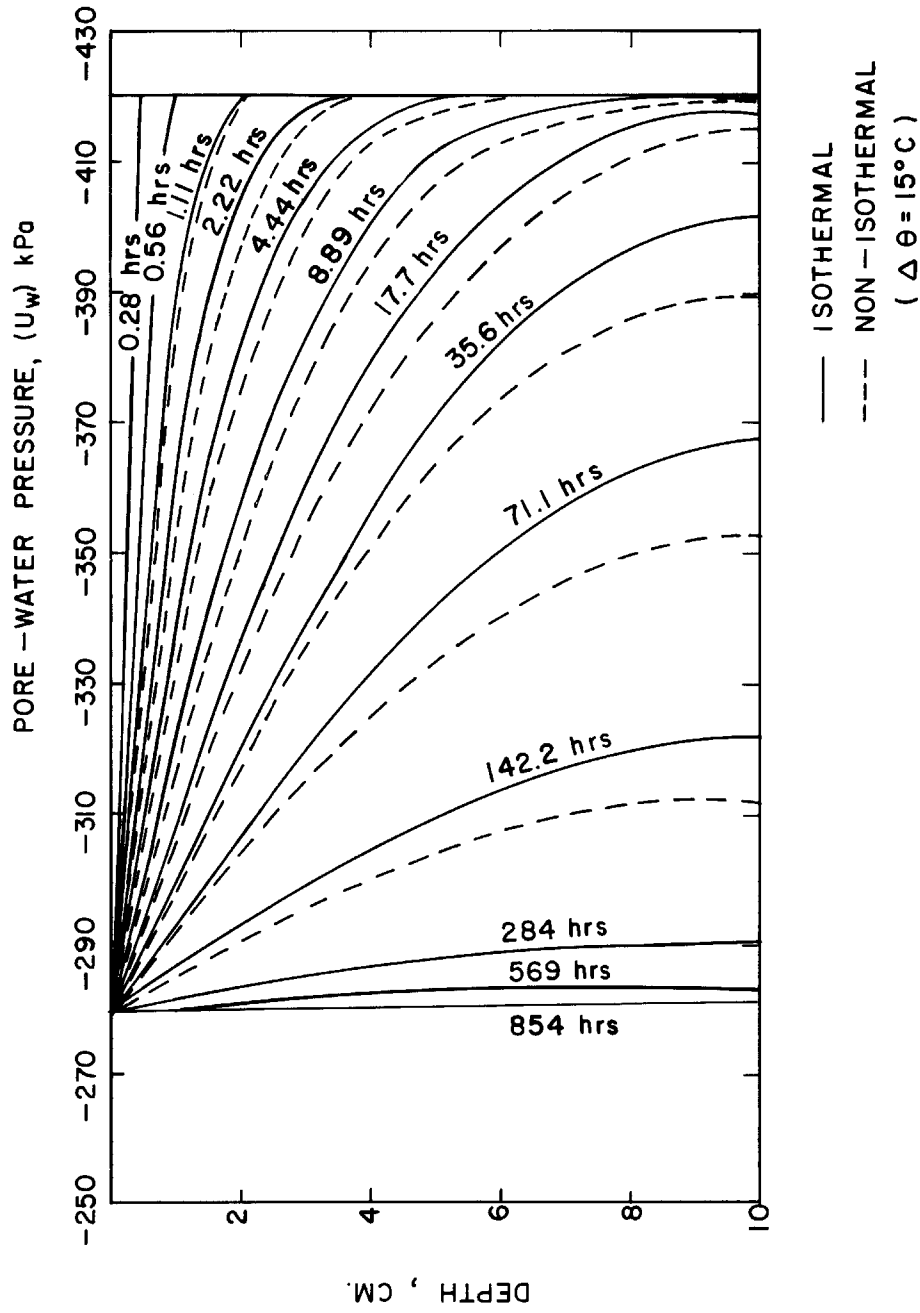
Fig. 9. Pore-water (liquid) pressure distribution (swelling process) (Examples Nos 3(b) and 4(b)).
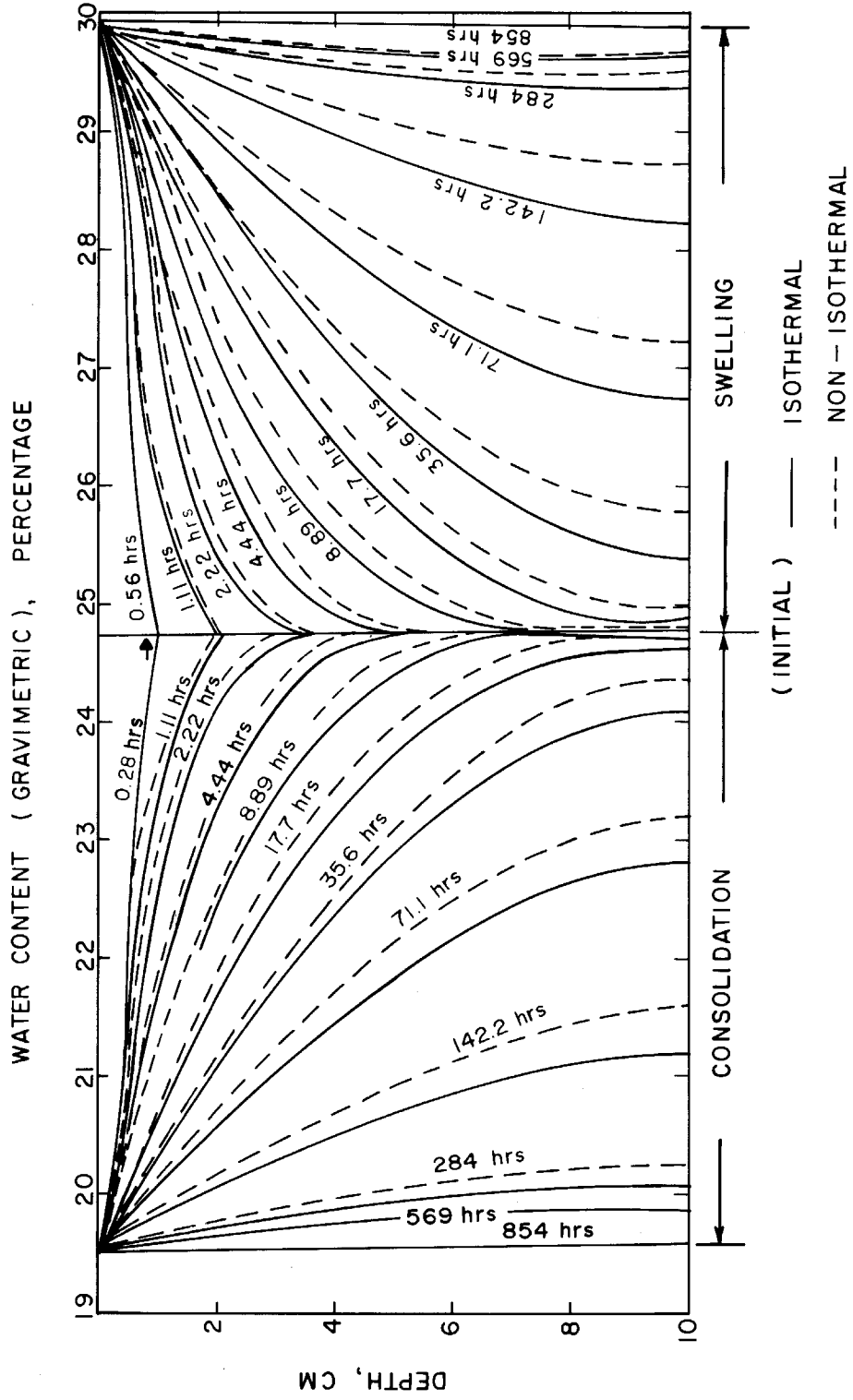
Fig. 10. Water content (Gravimetric) change throughout the soil layer (Examples Nos 3 and 4).
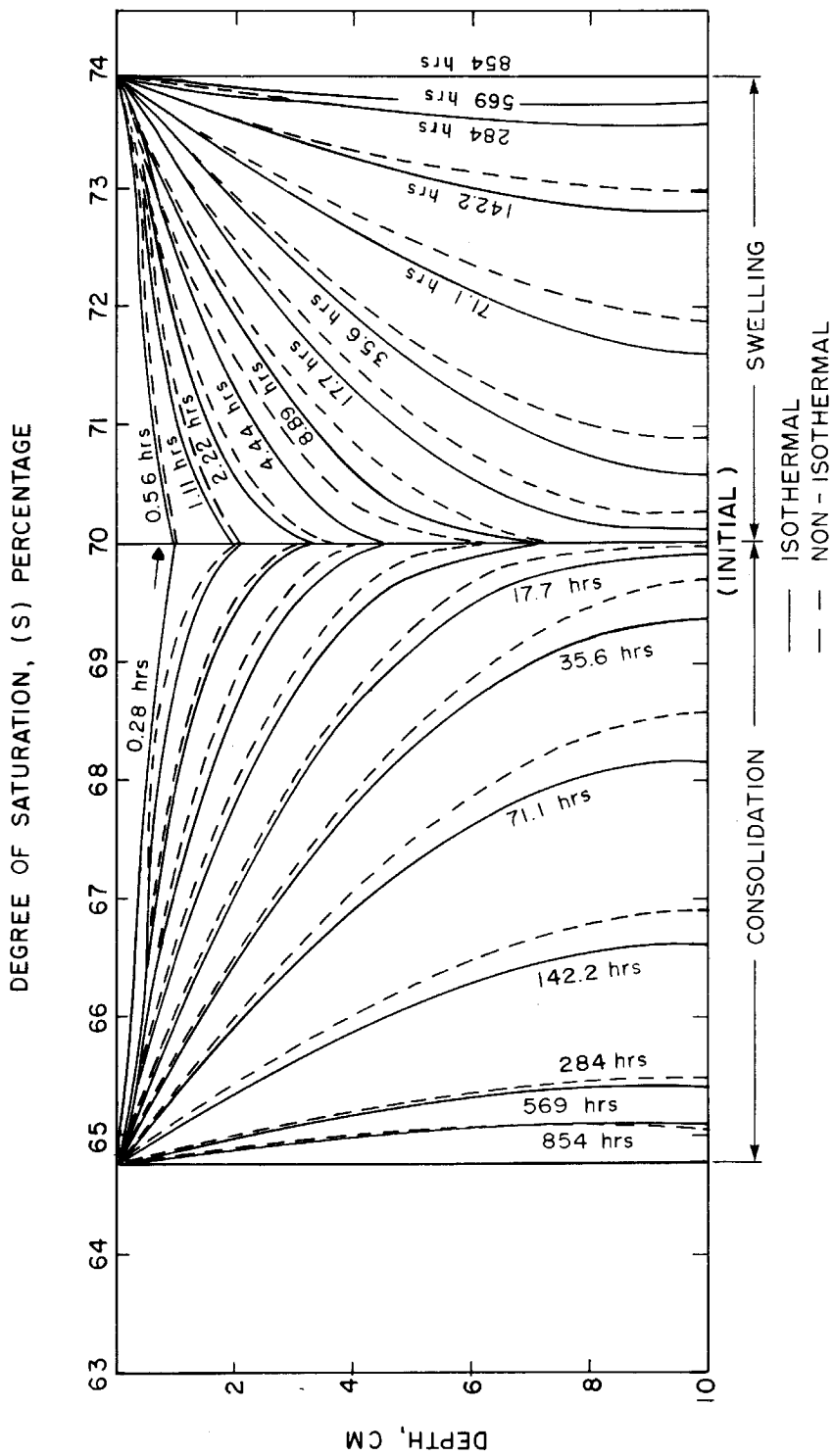
Fig. 11. Degree of saturation change throughout the soil layer (Examples Nos 3 and 4).
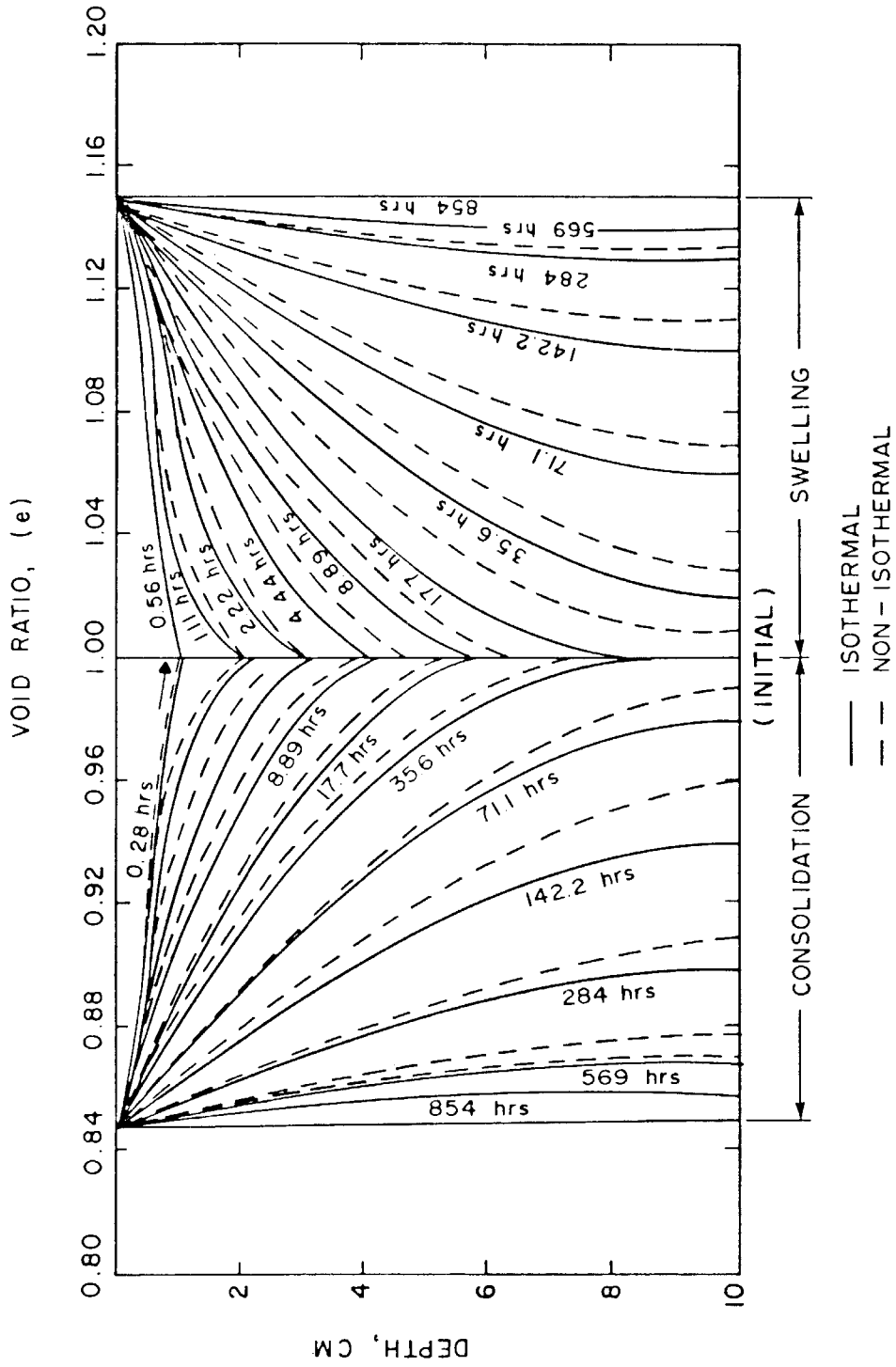
Fig. 12. Void ratio change throughout the soil layer (Examples Nos 3 and 4).

PORE-AIR PRESSURE, (U_a) kPa
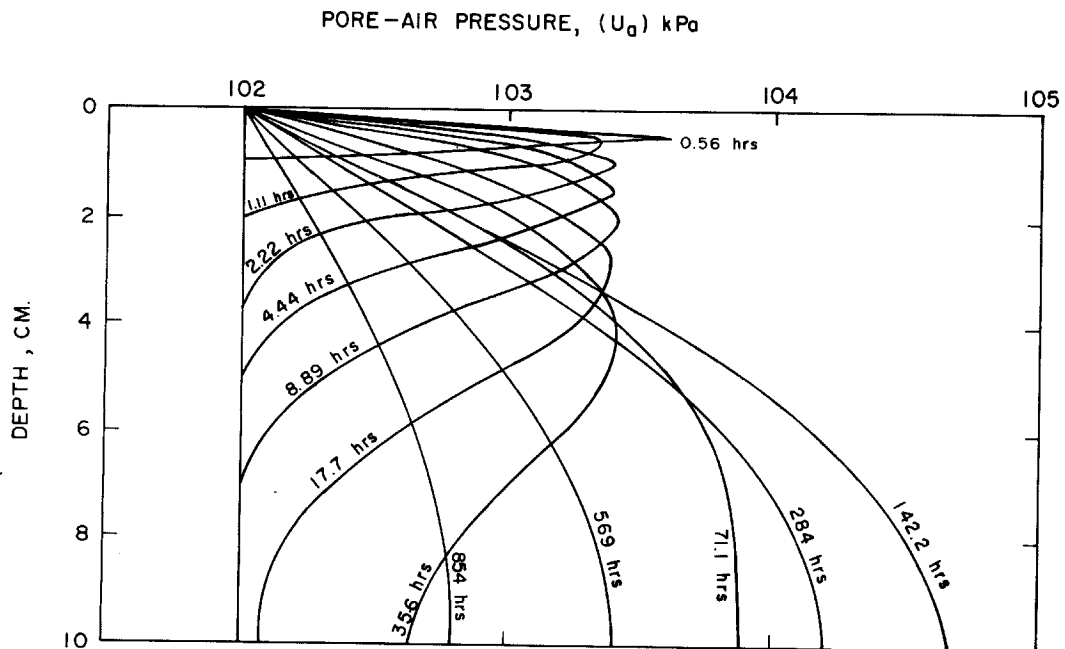
Fig. 13. Pore-air pressure distribution under non-isothermal condition.

for isothermal conditions. In order to explicitly bring out the effect of thermal and vapour pressure gradients, another series of examples were solved and are discussed later.

Examples Nos 3 and 4 (Tables 5 to 8)

The second series of examples was selected to demonstrate the effects of thermal and vapour pressure gradients. A ten centimetre thick compacted Regina clay is assumed. The family of curves shown in Figures 7 to 15 represent the solution of the transient flow equations.

Figure 7 shows the temperature isotherms within the clay layer as a result of an increase in the temperature from 10° to 25°. The figure illustrates how the imposed thermal gradient at the surface dissipates to the bottom of soil and eventually equilibrates at the new boundary condition. As in example No. 2, the lower boundary of the soil was assumed to be completely insulated. It should be pointed out here that a comparison of Figures 6 and 7 shows a

Fig. 14. Vapour-pressure distribution for Examples Nos 3 and 4.

significant difference in temperature distribution under the same thermal gradient (i.e., $15°C$) and thermal properties of the soil, but with different soil thickness assumed (i.e., 200 cm as opposed to 10 cm).

Figure 8 shows the pore-water pressure distribution for both isothermal and non-isothermal conditions under the consolidation process. The initial (equilibrium) pore-water pressure was -280 kPa, and the upper boundary was instantaneously changed to a value of -420 kPa. The temperature gradient was $15°C$. The dashed and solid lines show the difference between isothermal and non-isothermal conditions. Similarly, Figure 9 shows the numerically identical but reverse process to that presented in Figure 8.

Figure 10 shows the distribution of (gravimetric) water content throughout the soil layer, under the consolidation and swelling processes for both isothermal and non-isothermal conditions. The final water content throughout the soil layer reaches equilibrium at a value consistent with the change in pore-water and vapour pressures at the boundary.

Fig. 15. Partial pressure distribution for Examples Nos 3 and 4.

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Figures 11 and 12 show the related distribution of the degree of saturation (S) and the void ratio (e), during the isothermal and non-isothermal transient process.

Figure 13 shows the pore-air pressure distribution throughout the soil layer under non-isothermal conditions. Under isothermal conditions the pore-air pressure remains unchanged. However, for non-isot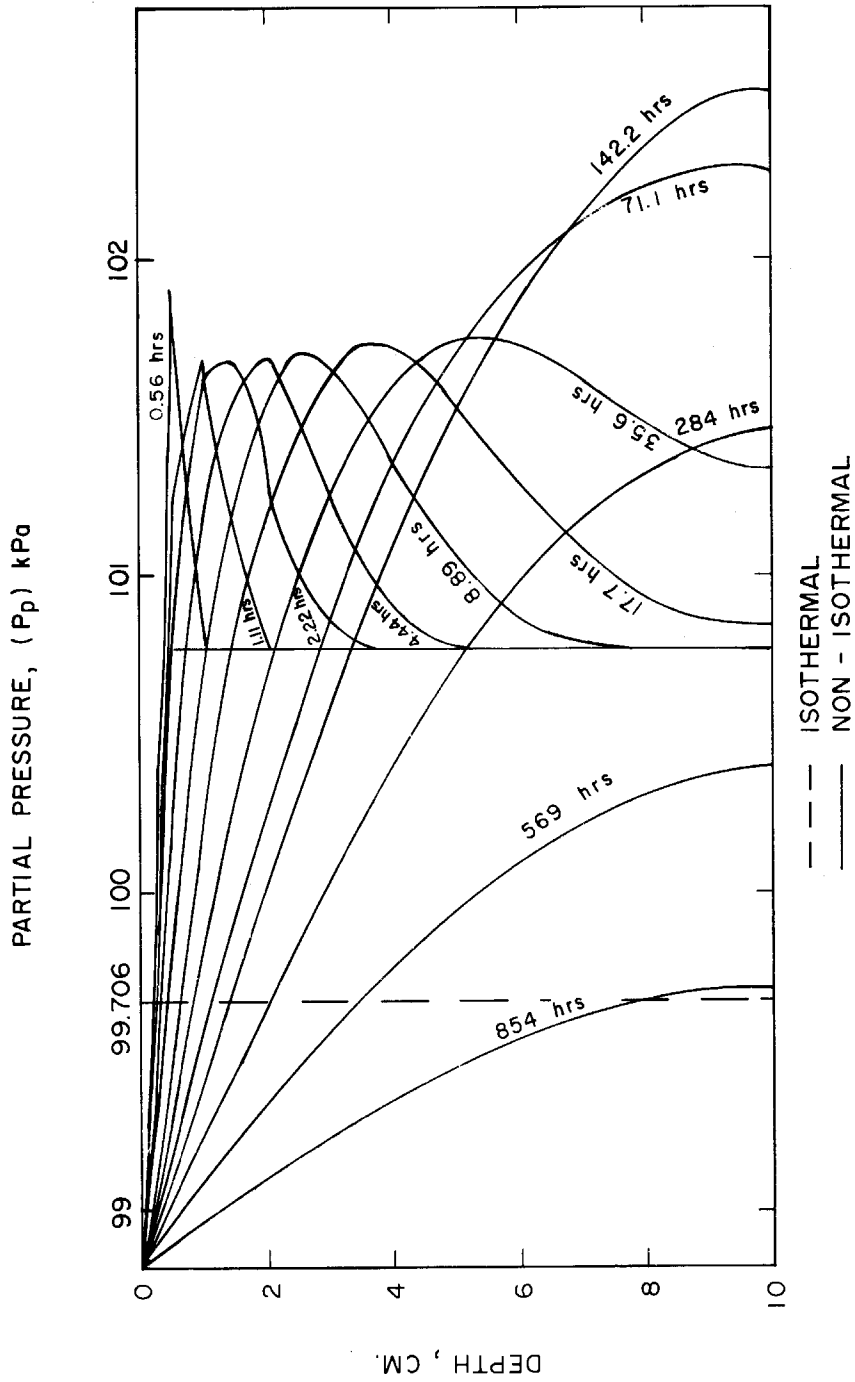hermal conditions the excess pore-air pressure generated as a result of the imposed thermal gradient varies with space and time as seen in Figure 13.

Figure 14 shows the vapour pressure distribution throughout the soil layer for examples Nos 3 and 4. For example No 3 (Isothermal Consolidation/Swelling) the pore-water pressure, which was initially at −280 kPa was instantaneously changed to a value of −420 kPa at the boundary, as shown in Figure 8. This change in the pore-water (liquid) pressure results in a consequent change in the equilibrium relative humidity from 99.6425 percent to 99.5393 percent (Table 5). This small change in equilibrium relative humidity results in essentially a constant vapour pressure (i.e., 2.294 kPa) throughout the soil layer during the transient process under isothermal conditions. The vapour pressure essentially remained constant when the pore-water pressure was reversed, as indicated in Figure 9, as the result of infiltration.

Under non-isothermal conditions (even though the initial and final pore-water (liquid) pressure remains the same) the equilibrium relative humidity changes as the result of temperature changes. Consequently, the vapour pressure which was initially at 1.188 kPa (corresponding to 10°C) was changed to 3.156 kPa (corresponding to 25°C) at the surface. This vapour pressure difference is eventually dissipated throughout the soil layer during the transient process, as shown in Figure 14.

Figure 15 shows the partial pressure (i.e., total pressure minus vapour pressure) distribution throughout the soil layer under isothermal and non-isothermal conditions. A constant partial pressure (i.e., 99.706 kPa) value is obtained throughout the soil layer under isothermal conditions. Under non-isothermal conditions the partial pressure which was initially at 100.812 kPa, was instantaneously changed to 98.844 kPa at the boundary. This partial pressure difference is eventually dissipated throughout the soil layer during the transient process, as seen in Figure 15.

The analysis of the above examples allows the distinguishing of the primary and secondary effects (i.e., significant and insignificant effects) as the result of complex environmental changes at the boundary. For example, pore-water pressure change as a result of infiltration or evaporation

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causes a primary or significant change in volume-weight properties of an unsaturated soil over a short period of time. On the other hand, the thermal gradients and relative humidity gradients imposed at the surface cause a secondary effect on volume-weight properties of the soil over a long period of time. This paper presents a rigorous theoretical analysis which considers both primary and secondary environmental effects. The practising engineer must be able to identify the primary effects of the complex environmental changes at the boundary and simplify the design procedure so as to predict reliable and realistic ground movements and related volume-weight properties for an unsaturated soil.

CONCLUSIONS

All the necessary physical relations are available to formulate a rigorous theoretical model to describe the transient flow process under combined hydraulic, relative humidity, and thermal gradients in an unsaturated soil. The heat flow equation is first solved using a forward finite difference technique. Then two partial differential equations (i.e., one for the water phase and the other for the air phase) are solved simultaneously. Families of curves show temperature, pore-water pressure, pore-air pressure, vapour pressure, partial pressure, moisture content, degree of saturation, and void ratio distribution throughout the soil layer as a result of combined boundary condition changes.

The distribution of moisture content and overall volume change enables the prediction of all volume-weight properties of the soil throughout the transient process. The model shows good promise for describing the behaviour of unsaturated soil systems under highly complex environmental changes. In addition, this model allows the assessment of the primary and secondary effects of complex environmental changes. The distinction between primary and secondary effects should assist practising engineers in producing realistic design procedures.

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SUBSIDENCE AND CHLORIDE CONTAMINATION AT NONG BO RESERVOIR, NORTHEAST THAILAND

JON L. RAU*, PRINYA NUTALAYA** and
MONTREE BOONSENER***

ABSTRACT

The storage of water in small reservoirs and tanks is a common practice in the arid northeastern part of Thailand. A small reservoir located 30 km southwest of Mahasarakham near the centre of the Khorat Basin is 1 sq km in area and has a storage capacity of 980,000 cubic metres. The reservoir and the drainage originating there were severely contaminated by sodium chloride stemming from the operation of salt wells, solar ponds, and brine disposal practices in the Borabu area of Mahasarakham Province. The level of salinity was intolerable for almost all species of aquatic life except for one imported variety of African fish. Moreover, the water quality was affected for more than 100 km downstream between 1970 and 1980. Eventually, the Thai government intervened to protect the drainage and the reservoir, sources of both food and water for thousands of villagers and farmers. Side effects of brine mining were subsidence and deforestation because more than 5,000,000 m^3 of salt were dissolved from just below the surface and wood was used to fire the boilers of some salt plants. The 2 km^2 area surrounding the reservoir is a scene of devastation today, more than 1 year after the cessation of salt mining and refining in 1980. Soils near the reservoir are infertile, salinities are still high and erosion is severe.

INTRODUCTION

The climate of northeastern Thailand is characterized by strongly seasonal rainfall and long dry periods. In May frequent rains begin and continue intermittently through September. During this period 89% of the annual average figure of 1,108 mm of rain falls. During the rest of the year the area is too dry to grow many crops without supplemental irrigation. Small reservoirs and wells are essential for domestic water and irrigation. Streams and ponds are also an important source of food such as fish and frogs. The purpose of this paper is to trace the development of a brine field surrounding a small but important reservoir near the village of Borabu in Mahasarakham Province.

SETTING

The salt contamination problem at Borabu occurs near the centre of a saucer shaped upland surface, the Khorat Plateau, extending over 200,000

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km^2 of northeastern Thailand as indicated in Figure 1. The contaminated reservoir, Nong Bo, shown in Figure 2, is only 2 km southeast of Borabu and drains into the Lam Sieo Yai (Creek) thence to the Mun River joining it about 13 km west of Rasi Salai. Figure 3 shows some brine wells and salt

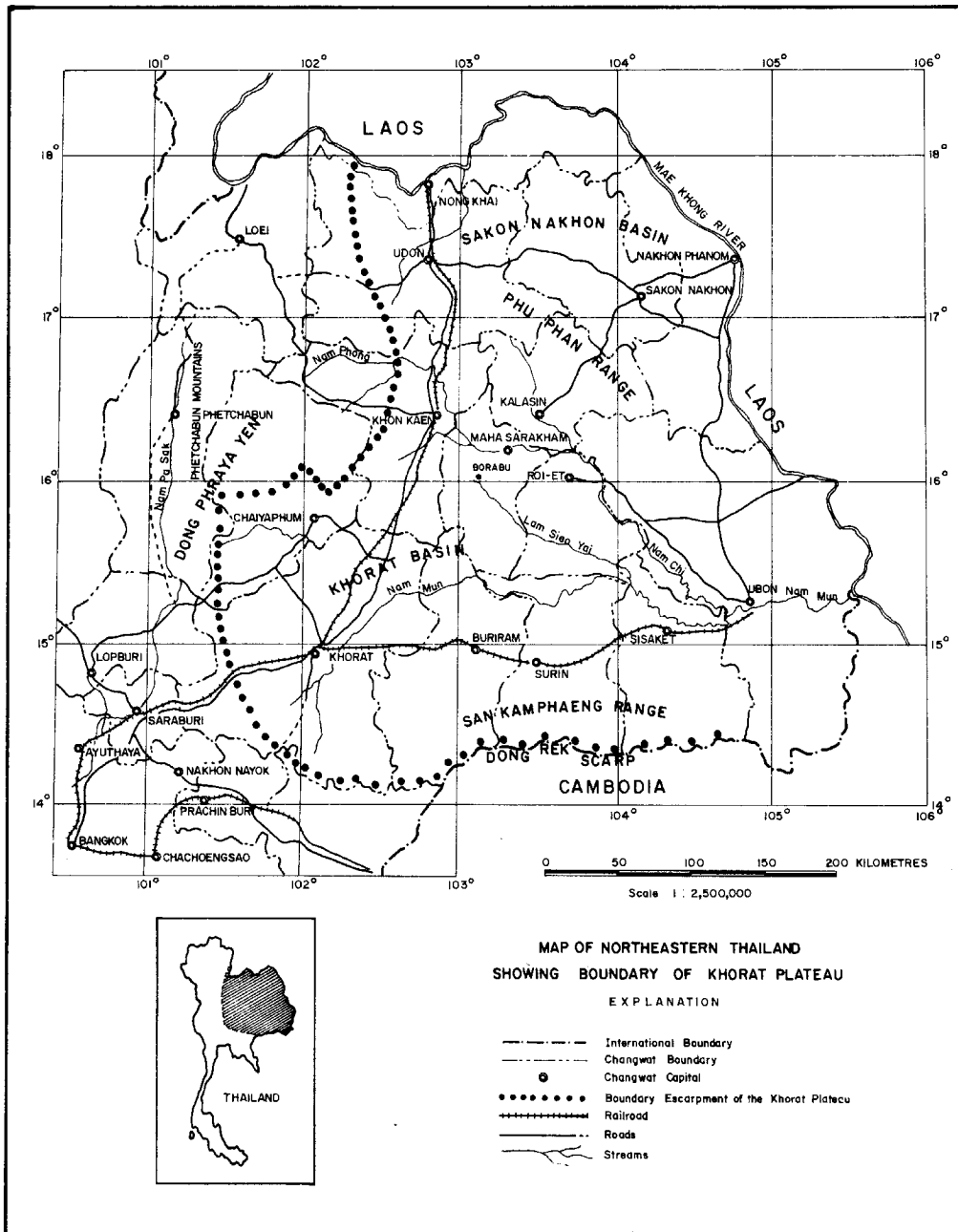Fig. 1. Map of northeastern Thailand showing the Khorat Plateau and the location of Borabu.

Fig. 2. Nong Bo reservoir and salt encrusted solar evaporation pans along its northern margin.

Fig. 3. Brine wells and salt ponds near the edge of Nong Bo reservoir.

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ponds near the edge of the Nong Bo reservoir. The reservoir is located near the boundary between two provinces, Khon Kaen and Mahasarakham, with a total population of more than 1.5 million people. Hence, Borabu is situated in the middle of a moderately populated and important agricultural area.

HISTORY

A small water reservoir, Nong Bo, built at the headwaters of Lam Sieo Yai (Creek), was designed to provide a village water supply and to supplement the needs of adjacent farmers during the low flow season. A shallow water well was drilled near the reservoir in 1967 and resulted in the accidental discovery of highly saline water. The owner, a village doctor, installed a boiler and used local wood to evaporate the water and recover the salt. The market price at that time was very high, about U.S.$70/tonne, because there was no other source of salt in the entire northeast. Salt is a valuable commodity there because it is used for food preservation during the long dry period extending from November to May when streams, ponds and lakes dry up cutting off sources of fresh fish and frogs. Refrigeration is normally not available to most villagers although it is reasonably commonplace in the cities and larger towns.

After a short time, perhaps a year, the word leaked out to the other villagers that salt recovery was possible by drilling a shallow well and by 1969 there were 34 separate salt recovery operations. Shortly thereafter the reservoir which was now completely rimmed by small solar ponds, salt wells and small salt plants using boilers, began to turn very saline. Eventually, almost all edible varieties of fish, except for one imported species from Africa, died and other forms of aquatic life such as frogs and salamanders also disappeared. By 1980 more than 47 separate salt plants were in operation (KHON KAEN UNIVERSITY, 1980). Figure 4 shows the remains of one operation. The effect of the salt operations, careless disposal of waste water from operations, improper storage of salt, leaking casings in salt wells, overflow from salt evaporation ponds and natural leakage of saline ground water all contributed to the high salinity in the reservoir. The location of the reservoir at the headwaters of the Lam Sieo Yai created a problem that was eventually responsible for high salinities extending downstream to the Mun River and thence to the Laotian border as shown in Figure 1. The villagers and farmers dependent on Lam Sieo Yai and Mun River water began to complain about the salt operations and the problems surrounding the Nong Bo Reservoir became very serious. As a result in 1978 the Thai Royal Irrigation Department, realizing that the reservoir problem was only going to become worse,

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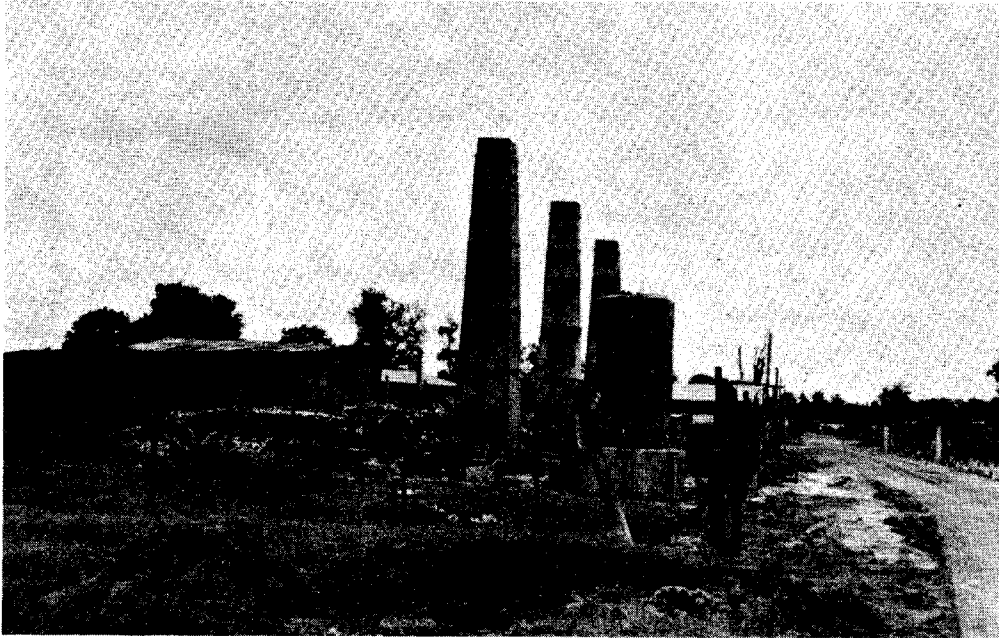
Fig. 4. The remains of a large boiler in a salt recovery operation near Nong Bo.

officially donated Nong Bo to the large city of Mahasarakham, 30 km to the northeast, a city which had no use for the reservoir water because it was too far away.

In early 1980 the effects of salt solution near the surface resulted in six separate areas of subsidence west of the reservoir and about 1,000 m from its centre. The areas of subsidence are small shallow depressions, averaging only a few m to about 50 m in diameter. Figure 5 shows a typical example near the west edge of Nong Bo reservoir. The pits created problems for local farmers when their water buffalo fell in and could not be pulled out because of the steep sides of the pits. During the rainy season these pits fill with from 6 to 8 m of water. In June of 1980 the Minister of Agriculture and Cooperatives of the Royal Thai government made an emergency visit to Borabu and to Mahasarakham to discuss the problem with the local people and to assure them that the Thai government was not going to let the salt operation endanger the livelihood of the thousands of Thai people depending on Nong Bo and the Lam Sieo Yai for their water supply and food. Subsequently, the King issued a Royal Decree stopping all pumping of salt water and its evaporation from solar ponds and boilers.

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Fig. 5. A subsidence pit near the west edge of the reservoir.

In mid 1980 the government constructed a dyke around the reservoir and improved the drainage by dredging a canal that would collect the run-off from solar ponds, wells and storage areas and divert it away from the reservoir. A pipe was installed beneath the dam so that the reservoir could be drained if necessary. In early 1981 the reservoir was partially drained and the source of the salt contamination was much reduced. Although most of the salt plants have been shut down, at least one or two are apparently still in operation. The once thriving salt village is now a ghost town and storage structures and ponds are in a state of disrepair. The landscape around the Nong Bo Reservoir is devastatingly barren and undergoing rapid erosion. Some of the concrete lined solar ponds are in good condition but render the land useless for agriculture. Salt content in the soil adjacent to the reservoir is too high for the cultivation of rice. It is not known how long it will take for the salt in the soil to be washed out by natural processes. Villagers returned to the lake to fish in June 1981 but the state of the recovery of aquatic life in Nong Bo is unknown at this time.

GEOLOGY

The rocks at the surface of the Khorat Plateau consist of a gently folded sequence of non-marine clastic sediments of Mesozoic age. The stratigraphy

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in the central part of the Khorat Plateau is not well known whilst the surrounding mountain ranges have provided several good sections which have been used to subdivide the Khorat Group into five formations (WARD and BUNNAG, 1964). No detailed mapping was done and no sections were measured for this investigation. The stratigraphy and regional relationships presented by Ward and Bunnag (1964) were accepted by the Thai Department of Mineral Resources. The authors accept the stratigraphic subdivisions proposed by Ward and Bunnag (1964) but question whether there is sufficient evidence to place a major unconformity between the Salt Formation and the underlying Khok Kruat Formation.

The rocks of the Khorat Plateau have been lumped together as the Khorat Group, a heterogeneous assemblage of classtic sediments ranging from shales to conglomerates totalling more than 4,800 m. The uppermost unit, the Salt Formation, is 610 m thick and is characterized by pale red to reddish brown sandstone, sandy shale and siltstone. Gypsum and anhydrite beds are sometimes present and locally the beds of salt are more than 246 m thick. The Salt Formation is Jurassic (?) and younger according to Ward and Bunnag (1964). The surface of the Borabu area is underlain by a deeply weathered loess cover over salt bearing shale and siltstone mapped as Salt Formation by Haworth, Chaingmai and Phiancharoen (1966).

STRUCTURE

The Salt Formation contains a bed of halite at a depth of less than 30 m, an elevation of 170 m above msl, in the vicinity of Nong Bo Reservoir. A bed of halite is also reported at a structurally lower elevation, -124 m below msl, about 27 km northeast of Borabu. Similarly, the salt bed is structurally lower, 50 m above msl, at Chaiyaphom southwest of Borabu. The structure is probably that of a simple dome with very low dips of less than $1°$ on its flanks. The dome is probably faulted between Borabu and Nong Bo. The evidence for this is the presence of the salt bed near the reservoir but its abrupt disappearance only one kilometre to the northeast at Borabu. Other evidence is presented in the discussion of the quality of the ground water. The presence of the dome is recognized by Haworth, Chiangmai and Phiancharoen (1966) but they suggest the difference in elevation of the salt bed could also be related to the unconformity beneath the Salt Formation.

METHOD

The reservoir, dyke and subsidence pits were mapped by students from the Department of Geology, Khon Kaen University at a scale of 1:500.

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Moreover, the drainage and land use was also studied in connection with the topographic mapping. The Thai Royal Irrigation Department collected water samples from Nong Bo from 1977 to 1979 and these were analyzed for specific conductance, pH, and sodium-adsorption ratio (SAR). Further, in 1980 samples were also collected from 5 stations downstream from the reservoir for a distance of 90 km to Kaset Wisai, and these were analyzed for specific conductance (micromhos/cm) and salinity (mg/1). The depth of the Lam Sieo Yai was also measured wherever samples were collected. The reservoir site was visited several times for studies of the development of subsidence features and these were mapped and soundings taken to determine the area and shape of each pond. Records of water wells drilled by the U.S. Geological Survey and water quality data from pre-salt well development were also examined from Borabu area. Photography was useful in documenting the changes in land use and the acceleration of erosion following the closure of the salt operations and the construction of the dyke enclosing the outer perimeter of the solar pond and reservoir area. No detailed geologic mapping was attempted because of a thick loess deposit in the area. Studies of topographic maps (1:250,000 and 1:50,000) were necessary to map the drainage of the Lam Sieo Yai but its exact course in the headwaters is still not well known. The topography is exceptionally flat and drainage for 2-4 km below Nong Bo is intermittent.

GROUND WATER

In spite of the presence of bedded salt in the Salt Formation 2 km southeast of Borabu from 50 to 80 percent of the wells drilled into it yield good water. Most wells yield from 20 to 30 gallons per minute (gpm) from joints and bedding planes. A few wells can yield up to 100 gpm. Some wells encounter salty water in low lying areas where shallow ground water is present in alluvial and colluvial material containing saline soils. This salt is apparently not bedded salt but secondary salt deposited near the surface in areas of ground water discharge similar to those which occur in playa lakes. Inasmuch as the Borabu area is near a surface water divide, the presence of saline water there cannot be attributed to the presence of secondary salt but rather to primary bedded salt. Only three well records have been published of the geology of the Borabu area (HAWORTH, CHIANGMAI and PHIANCHAROEN, 1964). The salt wells drilled near Nong Bo have no records of their geology because Thai law does not require water well drilling contractors to submit borehole records to the government.
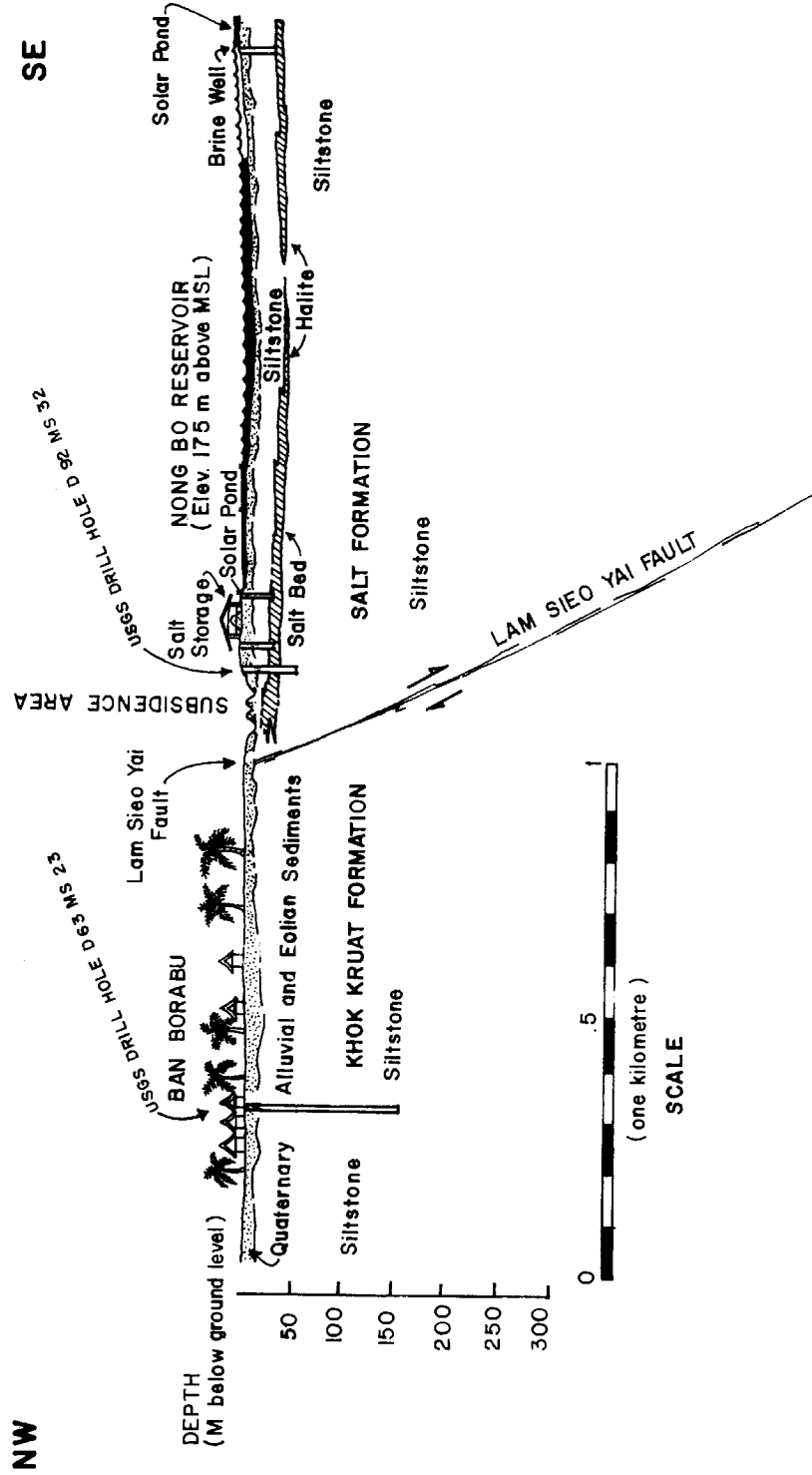
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Fig. 6. Diagrammatic cross section from Borabu to Nong Bo reservoir.

It is interesting to note that none of the three published well logs report the presence of salt at either Borabu or Nong Bo (HAWORTH, CHIANG-MAI and PHIANCHAROEN, 1964). The discovery of the salt at Nong Bo was accidental when a local doctor drilled a shallow well in search of potable water in 1967. Previously, a well had been drilled 2.5 km southeast of Borabu at Wat Ban Kho. Water quality analysis of this well revealed that it contained water with 241 ppm of chloride, 707 ppm of total dissolved solids and 470 ppm of hardness as $CaCO_3$. No halite was reported in the borehole record. The water in this well was obtained from a depth of 31 m to 46 m, the approximate depth of the salt bed at Nong Bo, only 1 km to the northwest. The water quality at Borabu, about 1.5 km northwest of the Nong Bo salt deposit, is even better (Figure 6). It is suggested that the abrupt change in water quality and geology between Nong Bo and Borabu warrants the consideration of a fault between the two areas. In the diagrammatic cross section of the Nong Bo Borabu area such a fault is indicated because it is preferable to an unconformity in explaining the abrupt appearance of the salt in the section at Nong Bo (Figure 6). The fault is interpreted as a normal fault parallel to the regional trend of the major folds west of Khon Kaen, or northeast-southwest. It is herein named the Lam Sieo Yai fault after the drainage originating at Nong Bo and flowing south toward the Mun River.

WATER QUALITY

Discharge from Nong Bo Reservoir

The level of water in the reservoir is controlled by a spillway and during the rainy season the volume of water in the reservoir remains unchanged at about 960,000 cubic metres. During the rainy season the average discharge from Nong Bo is about 180 m^3/min. Villagers remove a certain amount to satisfy the needs of the population and some water is used for irrigation. No data is available on water use from Nong Bo nor is the discharge during the dry season recorded.

Salinity

The waters of Nong Bo were analyzed during the period from 1977 to 1980. During this time specific conductance, measured in micromhos, fell below 10,000 only twice as indicated in Figure 7. Specific conductance was generally above 20,000 micromhos or approximately 18,000 parts per million (ppm) of total dissolved solids. Natural waters are characterized as very saline if they exceed 10,000 ppm of dissolved solids and briny if they contain more
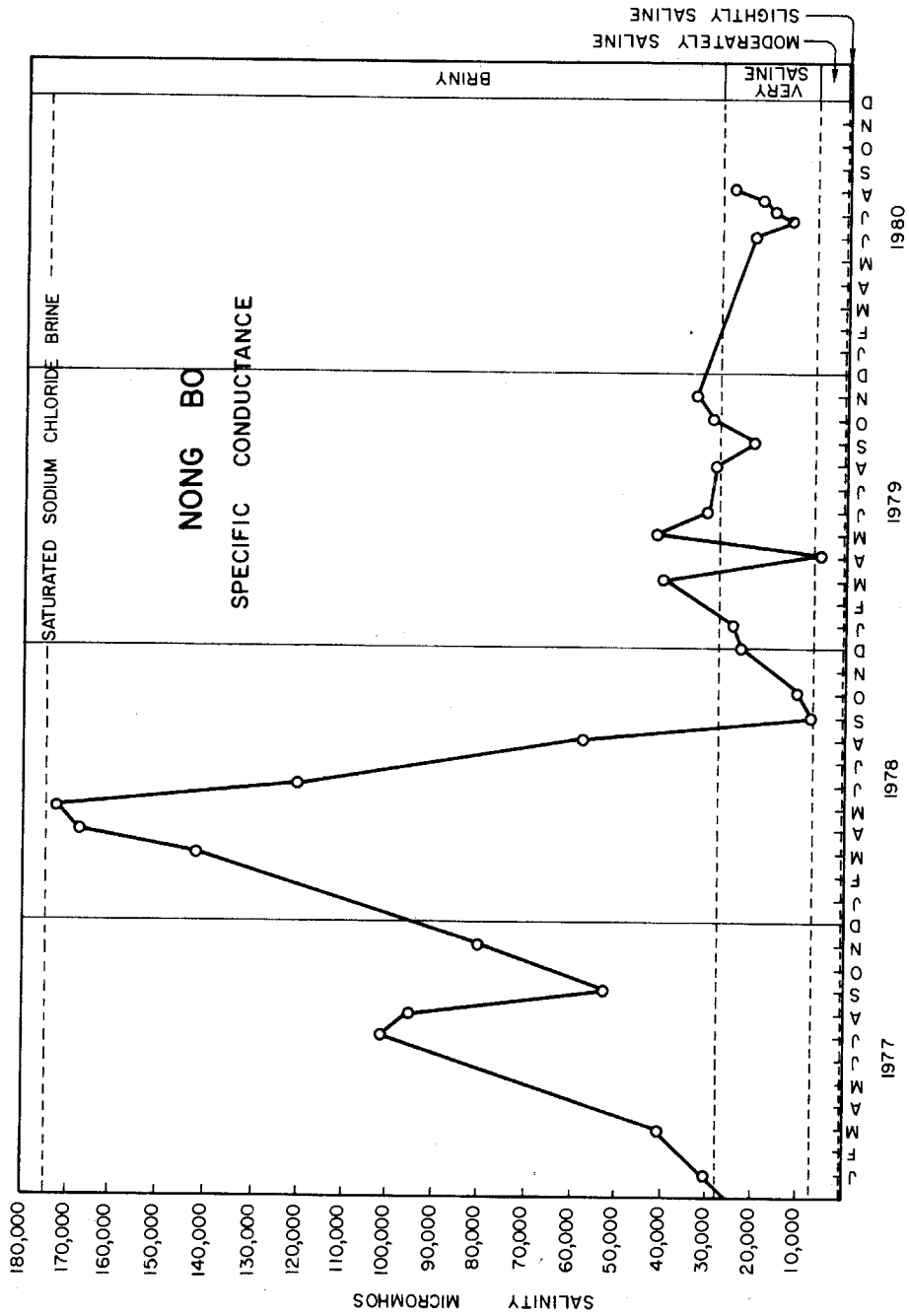
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Fig. 7. Specific conductance at Nong Bo (1977-1980).

than 35,000 ppm dissolved solids. The specific conductance of Nong Bo water indicated that its total solids were well over 30,000 ppm for most of 1977 and 1978 (Figure 7). The reservoir reached a maximum of 176,000 micromhos or about 285,000 ppm of dissolved solids in May and June of 1978. A saturated sodium chloride brine normally contains 300,000 ppm dissolved solids (HEM, 1970). It is clear that the waters of Nong Bo were nearly saturated during the dry period of 1978 when its waters contained a little less than the quantity of dissolved solids necessary for the precipitation of sodium chloride crystals. Nevertheless, in June of 1980 divers of the Thai Royal Irrigation Department observed crystals of halite on the floor of the reservoir.

An examination of Figure 7 reveals that between 1977 and 1980 the salinity of the water in Nong Bo Reservoir was briny 58 percent of the time, very saline 40 percent of the time and moderately saline only 2 percent of the time. At no time was the reservoir water quality good enough to be characterized as normal (less than 1,000 ppm total dissolved solids) or even slightly saline. One trend is especially apparent from the specific conductance data. The quality of water in Nong Bo steadily deteriorated during each of the dry seasons in the period 1977-1979. As shown in Figure 7 some suggestion of improvement in its quality is indicated beginning in 1980. Conversely, the quality normally improved after the beginning of the rainy season in June but the improvement was not enough to reduce the level of salinity in the reservoir below about 18,000 ppm of total dissloved solids most of which was because of sodium chloride content.

Sodium-Adsorption-Ratio (SAR)

The sodium-adsorption-ratio (SAR) is defined as

$$SAR = \frac{(Na^+)}{\sqrt{\frac{(Ca^{+2} + Mg^{+2})}{2}}}$$

where ion concentrations are expressed in millequivalents per litre (HEM, 1970). The SAR is useful in predicting the degree to which irrigation water loses sodium to the soil by replacing adsorbed calcium and magnesium. This replacement process damages the soil structure and indicates the degree to which a soil hazard may be created by using water of high salinity. The data for the SAR of Nong Bo Reservoir have been plotted in Figure 8 and the graph shows a close correlation between SAR and salinity. The SAR rises steadily during the dry season reaching a maximum in March or April
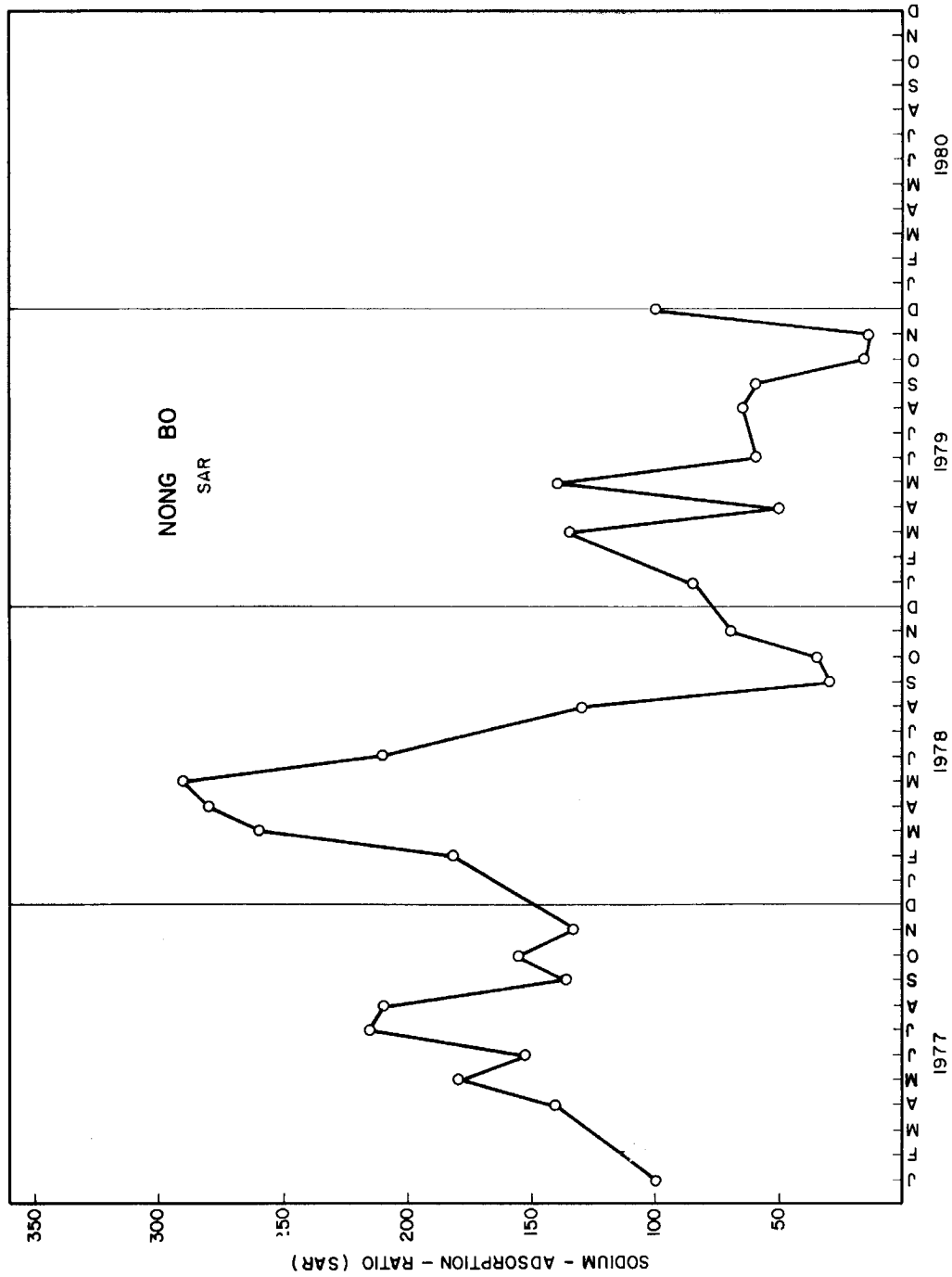
Fig. 8. Sodium-Adsorption-Ratio at Nong Bo (1977-1979).

of each year and then declines sharply with the advent of the frequent monsoon showers beginning in May. At the end of the rainy season in September or October the SAR begins to climb again. Hence, the SAR was too high for Nong Bo water to be used for irrigation water just when the reservoir would be most useful for this purpose as indicated by the data from the period of record (1977-1979).

pH

A glass electrode was used to determine the hydrogen ion activity, or pH, of water in Nong Bo and the Lam Sieo Yai. The pH of rainwater is in the vicinity of 5.6 as is that of ordinary laboratory distilled water (HEM, 1970). The pH of most natural water ranges from 6.5 to 8.5. Between 1977 and 1979 the pH of Nong Bo water exceeded 7.0 in all months except the period from May to July 1978, but never exceeded 8.8. In summary, the pH of Nong Bo water ranged from 7.0 to 8.0 about 78% of the time. Lowest recorded pH was 6.1 in June 1978 and highest recorded pH was 8.8 in December 1979.

Water Quality Downstream from Nong Bo

During the period from 12 June 1980 to 4 August 1980 the waters of Nong Bo and the Lam Sieo Yai downstream from the reservoir were sampled 5 times. At each of 6 stations data recorded included depth (m), specific conductance (micromhos), and salinity (ppm) as indicated in Figure 9. The data of the specific conductance have been plotted in Figure 10. With one exception, the highest values of specific conductance were recorded in Nong Bo reservoir where they ranged between 12,000 and 25,500 micromhos indicating that a very saline condition existed. A few kilometres downstream values in the Lam Sieo Yai at Ban Khok Li ranged from 2,900 to 27,200 micromhos of specific conductance indicating its waters were either very saline or moderately saline on 5 of the 6 sampling dates.

In general, the quality of the water in Lam Sieo Yai steadily improves downstream from Nong Bo. By the time the water reaches Kaset Wisai, about 90 km downstream from Nong Bo, it can be categorized as either slightly saline or normal with less than 1,000 ppm total dissolved solids. Previous studies have suggested that prior to the dates of the sampling period to which reference is made here the quality of the water in Lam Sieo Yai was even worse and threatened agriculture on both banks for a distance of more than 100 km downstream (KHON KAEN UNIVERSITY, 1980). During the sampling period of mid 1980 only the first 31 km downstream from Nong Bo were seriously affected by highly saline water discharged from the reservoir.

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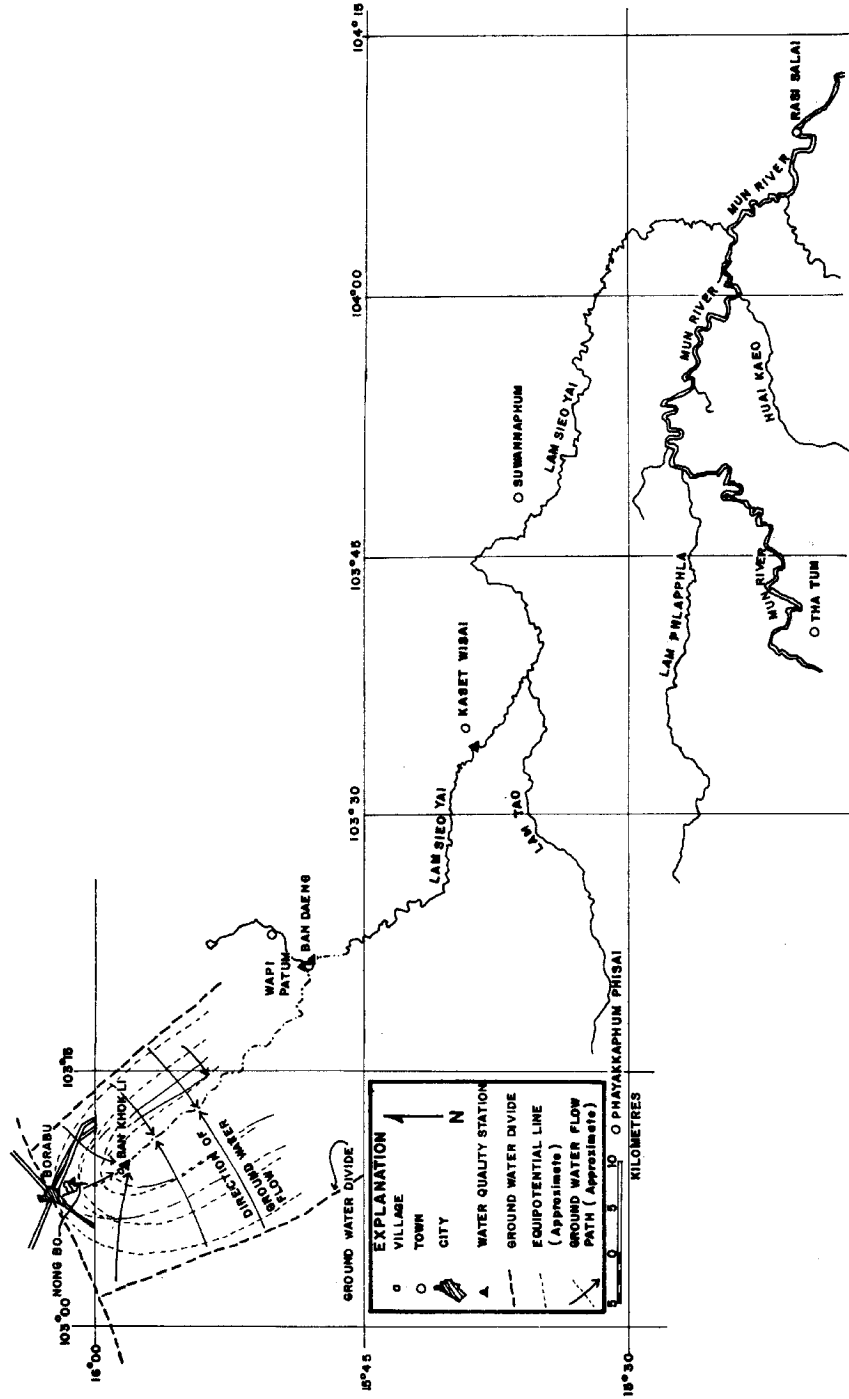
Fig. 9. Map of Lam Sieo Yai drainage showing water quality stations and ground water flow system just below Nong Bo.
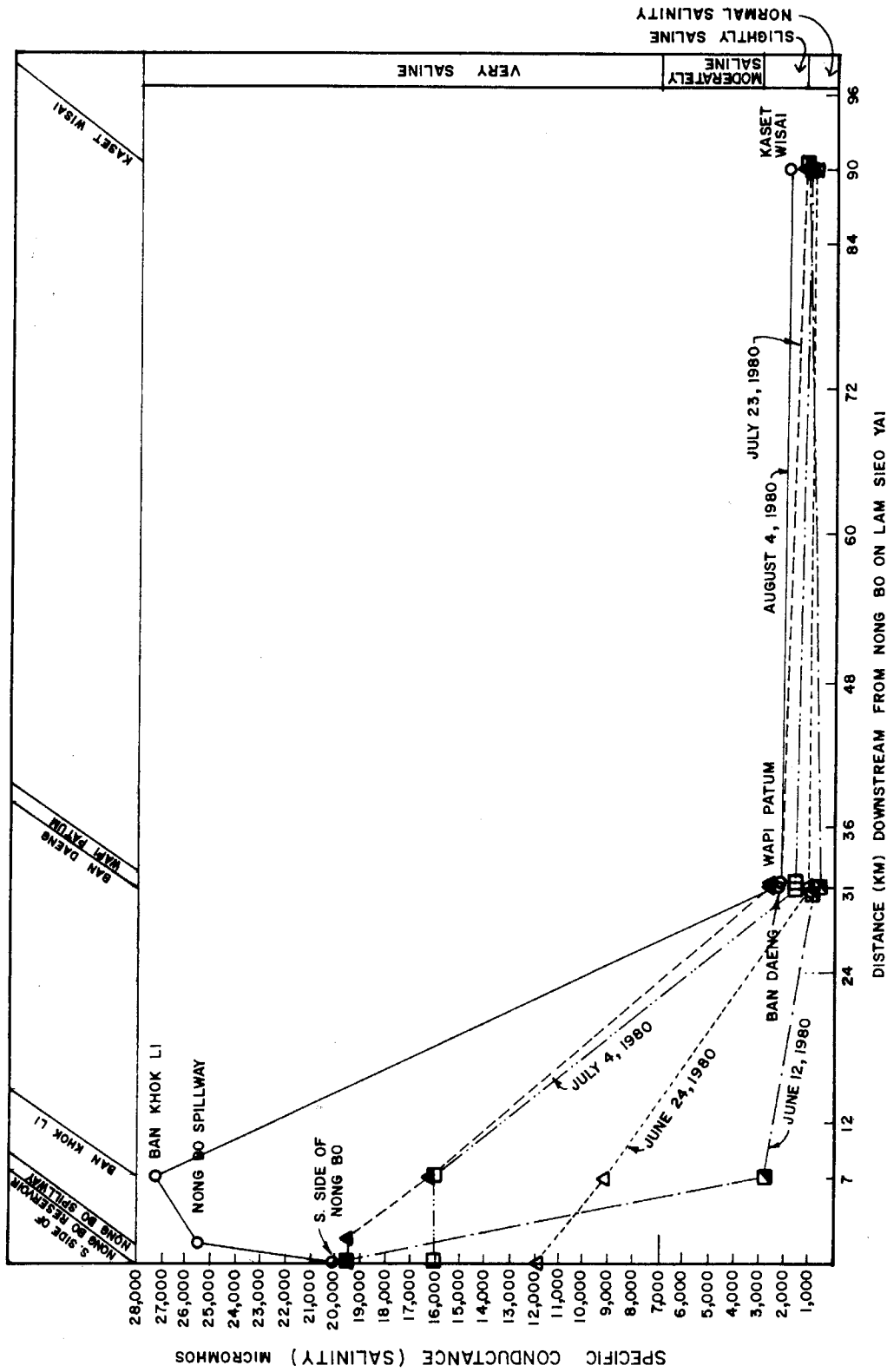
Fig. 10. Specific conductance (micromhos) below Nong Bo to Kaset Wisai (12 June, 1980–4 August, 1980).

Prior to 1980 the total area affected by saline water discharged from Nong Bo was reported to be 9,732 rai covering 5 districts and 38 villages (KHON KAEN UNIVERSITY, 1980). There is no question that the immediate area of Nong Bo was very seriously damaged by the salt operations and also that many villages in the Borabu and Nong Jig districts must also have been seriously affected.

SUBSIDENCE

A detailed study of the dimensions of the six surface su bsidence features was conducted by Khon Kaen University (1980). All of the subsidence areas are roughly circular and located about 1 kilometre west of the reservoir as indicated in Figures 6 and 11. They contain water and have caused problems for local farmers when water buffalo were trapped in them and were unable to climb back up the steep sides. The areas of subsidence range from 20 m^2 to
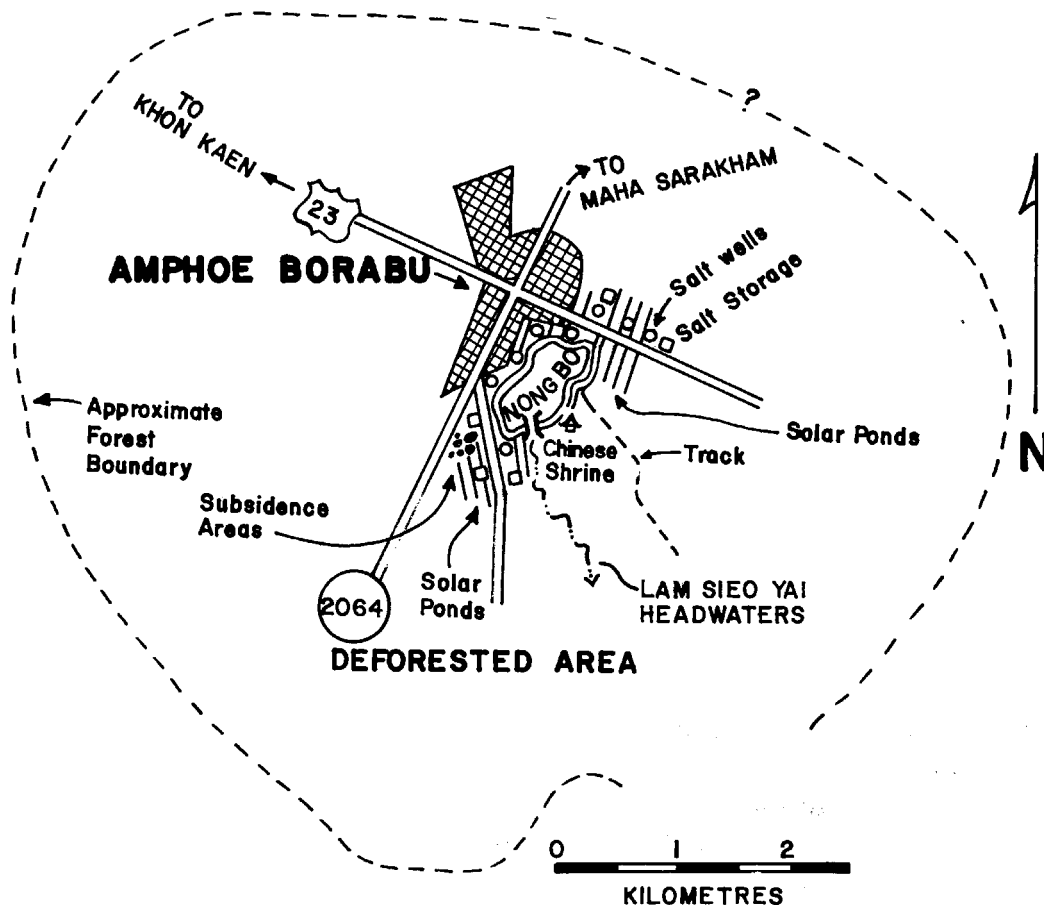
Fig. 11. Sketch map of Borabu showing the location of the city, the reservoir and the subsidence pits.

308 m² and the maximum depth ranges from 1.9 to 6m. The subsidence pits tend to be steeper toward the northeast suggesting that perhaps greater solution occurred on the northeast side of the salt deposit. The depth to the salt bed is unknown but is presumed to be about 25-30 m, the depth of most of the salt wells in the area. The thickness of the salt bed is unknown but can be computed to be at least 1 m thick over an area of 1 km² to account for the total amount of salt recovered during the 12 years of salt mining. The data regarding the subsidence pits follows.(KHON KAEN UNIVERSITY, 1980).

	Area (m²)	*Maximum depth* (m)
Pond 1	20	1.90
Pond 2	77	3.15
Pond 3	308	5.00
Pond 4	158	4.90
Pond 5	45	4.00
Pond 6	140	6.00

The total area affected by profound subsidence resulting in cone-shaped depressions at the surface is 748 m² or 0.00075 km². The total thickness of salt solution required to account for the development of the pits is difficult to compute because of their shape. Nevertheless, the data suggest that a maximum of 6 m of salt occurs west of Nong Bo reservoir and that the salt in this area probably ranges from 1 to 6 m in thickness, probably averaging about 2-3 m in thickness (Figure 6). Areas of less than 1 m of subsidence may exist over the reservoir area but would be undetectable by the methods used in this survey.

OTHER PROBLEMS

In addition to the subsidence and salinity problems there were several other interesting side effects resulting from the salt recovery operations. The wood used to fire the boilers was cut from the local forests and a one-half to one kilometre wide swath of open land now surrounds the reservoir area. A total area of 5-10 km² of forest land was cut to provide fuel during the period from 1969 to 1980. The reduction in vegetation has increased run-off and erosion. During monsoon rains, a large amount of soil is washed into the reservoir and local roads turn to quagmires. Soil erosion is especially severe on the west and north sides of the reservoir and the soil is too salty to grow rice.

The salt production rose steeply after 1969 and reached a maximum of 96,000 tonnes/year. This depressed the salt market in the northeast and the price fell sharply to about U.S. $7/tonne in 1980. In spite of this low price, salt production is still carried out in two or three places near the reservoir but it must compete with the readily available salt now harvested from the tidal flats at the head of the Gulf of Thailand near Samut Prakan.

SOURCE OF SALT CONTAMINATION

The Mun River and its tributaries drain a large area of the central Khorat Plateau which contains substantial rock salt and gypsum deposits. The rivers of this area can be expected to have a high salt content during their low flow periods. But during the monsoon season their quality should markedly improve. It is clear from the data presented here that even during the rainy season the salinities of Nong Bo and the Lam Sieo Yai were much too high. Further, the structure of the area, a gentle dome, brings the thickest of the salt beds near the surface at Nong Bo. The evidence of this is the apparent success of many wells in penetrating rock salt at a depth of 30 m or less southeast of Borabu. Salt extraction operations, boilers and ponds completely rimmed the reservoir by 1979. More than 100 individual salt evaporation ponds were in operation and most of these ponds were located within 100 m of the edge of the reservoir as indicated in Figure 11. The ponds are small, averaging about 100 m^2 each and were covered with a little water from the salt wells. In just a few days the water evaporated and the salt was scraped from the floor of the ponds and stored in sheds. Some of the ponds are lined with concrete while others are simply shallow pans lined with earthen embankments. The soil of the Borabu area is fine sand and silt and has a rather low permeability. However, it is not suitable for earthen embankments because of its low cohesion. In 1981, after the reservoir had been partially drained, many of the embankments were in obvious need of repair and monsoon rains had totally destroyed some of them. This problem must have plagued the operators during the entire period from 1969 to 1980. It is presumed that periodically embankments would either leak or fail permitting saline water to run into the reservoir.

The salt was stored in sheds, as indicated in Figure 12, some of which were in constant need of repair. There is no question that salt water periodically reached the reservoir from the storage sheds during the monsoon season. Ditches which drained the area all entered the reservoir area until 1980 when a shallow dyke was constructed around the perimeter of the reservoir. The

Fig. 12. Salt ponds, storage sheds and salt stockpiles in the midst of the deforested area on the southwest side of Nong Bo.

dyke was constructed of the same material used to build the embankments for the solar ponds and already shows the effects of erosion. Its height is approximately 1.5 m and width 2-3 metres. Large vertical holes, the result of piping, are gradually being widened during runoff periods and rainstorms of the 1981 monsoon season. The future of the dyke is probably limited to no more than two years unless funds are appropriated to maintain its height and to fill in the pipes that have developed on the western side of the reservoir.

Very little is known of the nature and construction of the salt wells but the failure of casing in similar wells is common after a period of from 6-10 years. Chemical weathering is rapid and cast iron and steel pipe decays rapidly in the hot, monsoon seasons of Southeast Asia. The source of much of the saline water in the reservoir was probably from leaking casings in salt wells.

The development of subsidence features on the west side of the reservoir suggests that the ground water flow system was mainly from the west and that the solution of rock salt was most effective about 1,000 m west of the centre of the present reservoir as shown in Figure 6. The geology of the subsurface is poorly known but the rock salt beds are at least 10-30 m below

the surface on the western edge of the present reservoir. The depth of the deepest subsidence pit is at least 6 m. Solution probably occurred over a wider area than that occupied by the subsidence pits because total production during the period from 1969 to 1980 was about 1,152,000 tonnes representing a volume of 5,760,000 m³. This is equivalent to the solution of a halite bed approximately 2.5 m thick over an area of 1,000 m². The assumption is made that the halite bed is pure and has a density of 2.16 (DEER, HOWIE and ·ZUSSMAN, 1964). It is clear that, should the solution be concentrated in a smaller area, more profound subsidence would have affected the surface than that currently observed at the west edge of the reservoir. Consequently, it is inferred that gentle subsidence must have affected a very significant area, one exceeding the limits of the present surface subsidence features.

THE FUTURE OF NONG BO RESERVOIR

The reservoir water is still saline although salt operations have all but ceased. During the 1981 monsoon the quality of water in the reservoir was expected to slowly recover but could take several years to reach its pre-salt well condition. The subsidence which has occurred is not recoverable and the land surface in the reservoir is probably lower over an area of at least 1,000 km². Saline springs may continue to discharge some salt water into the reservoir and its quality will deteriorate during each dry season but should improve substantially during each wet one. Fish and other aquatic life can be reintroduced into the reservoir. The storage of salt within one km of the reservoir should be prohibited. Paddy waters are likely to be enriched in salt in this area and should not be drained into the reservoir but be bypassed around it. Thus, it will be necessary to maintain the dyke that presently exists. Finally, a monitoring programme should be initiated for the entire length of the Mun River and the water quality should be determined on a regular basis. It will be especially important to determine the original or bench-mark quality of natural waters of the area so that future developments of rock salt can be monitored carefully. Other reservoirs along the Mun River should also be monitored carefully. At the end of the rainy season the waters should be used to irrigate dry season crops. Wells will be necessary to supplement village supplies because reservoir water will be too salty for the immediate future. Each season, it will be necessary to flush the reservoir, preferably near the beginning of the rainy season but care must be exercised so that a number of reservoirs along the Mun River are not flushed at the same time. Irrigation water must be carefully monitored so that excessive salt build up does not occur in paddy soils.

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EFFECT OF MECHANICAL PROPERTIES OF MEDIA ON CONTACT DEFORMATION

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SYNOPSIS

The stress distributions set up under plane strain conditions in a semi-infinite plate of elastic-viscoplastic celluloid compressed by a flat punch of similar material are shown using photo-viscoplasticity. For comparison results are also shown using a punch with elastic properties on the same semi-infinite plate. It was found that differences in deformation and stress patterns scarcely appeared at early stages of loading but became increasingly apparent with time. It is considered that punch rigidity was one cause of the differences observed, the second being the relative viscous effects of the punch and semi-infinite plate in the transient period between loading and the state of static equilibrium. The directions of principal stress were not unduly affected by the mechanical properties of the punch.

INTRODUCTION

In a previous paper (NISHITANI & ITO, 1979), the stress distribution and its time dependent variation for the case of a semi-infinite plate of an elastic-viscoplastic medium compressed by an elastic punch were analysed by using photo-viscoplasticity. In the paper, it was found that the bearing capacity of a semi-infinite plate was affected by the viscous effect of the plate with the lapse of time after loading. However, the stress distribution and its time dependent variation near the contact surface may also be seriously affected by the mechanical properties of media in contact and by the stress conditions on the contact surface. For example, the deformation and stress states between two elastic-viscoplastic media in contact are fairly different from those between elastic and elastic-viscoplastic media in contact, and the difference becomes remarkably large with the passage of time.

Since it is difficult to evaluate the effect of the mechanical properties of media in contact and the effect of the conditions on the contact surface in the ordinary theoretical analysis precisely, such problems have been analysed as those between rigid and rigid-perfectly plastic media under assumptions of idealized contact conditions (HILL, 1950; KACHANOV, 1971; ALEXANDER, 1955).

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In the present paper, by using photo-viscoplasticity (NISHITANI, 1976), the stress distribution and its time dependent variation near the contact surface in transient creep deformation are investigated experimentally on a semi-infinite plate of elastic-viscoplastic celluloid compressed by a flat punch of elastic-viscoplastic celluloid under plane strain conditions. The effect of the mechanical properties of media on contact deformation and their time dependency in transient creep deformation are discussed by comparing the results shown below with those obtained previously (NISHITANI & ITO, 1979).

EXPERIMENT AND EXPERIMENTAL RESULTS

The experimental apparatus used has already been described (NISHITANI & ITO, 1979). A semi-infinite ground celluloid plate was compressed in the centre by a flat punch of celluloid, acting much as a building, as shown in Figure 1. The specimens were constrained on both sides with thick glass plates, so that the deformation was under a plane strain condition. Square networks at 1 mm centres were incised on the surfaces of the specimens in order to trace the location of each element at each instant of transient creep deformation. The procedure and results of the calibration test for determining the mechani-
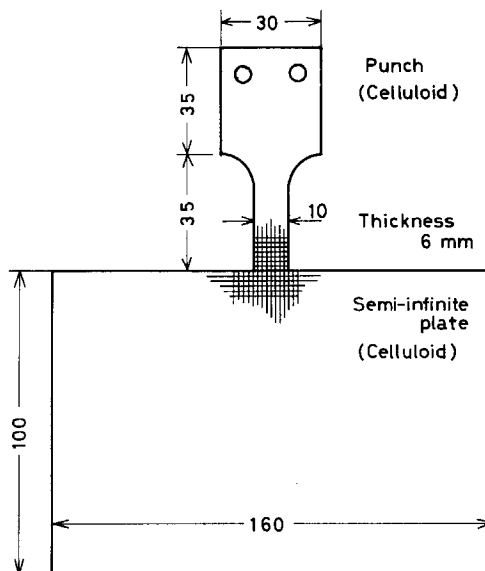
Fig. 1. Dimensions of test pieces.

cal properties of celluloid at 65°C were given previously (NISHITANI, 1976).

After the temperature had settled at 65°C in an oil bath, an instantaneous dead load of 588 N was applied. The mean contact pressure on the contact surface was equal to 9.8 MPa. The isochromatic fringe pattern at the instant of loading, t = 0, and the isochromatic and isoclinic fringe patterns as well as the pattern of deformed network at t = 2, 40, 160 and 300 minutes after loading were photographed.

In the present paper, Experiment 1 corresponded to the contact deformation between the elastic-viscoplastic celluloid semi-infinite plate and the elastic-viscoplastic celluloid flat punch. Experiment 2 corresponded to that between the elastic-viscoplastic celluloid semi-infinite plate and the elastic araldite punch discussed earlier (NISHITANI & ITO, 1979).

Isoclinic   Isochromatic

(a)
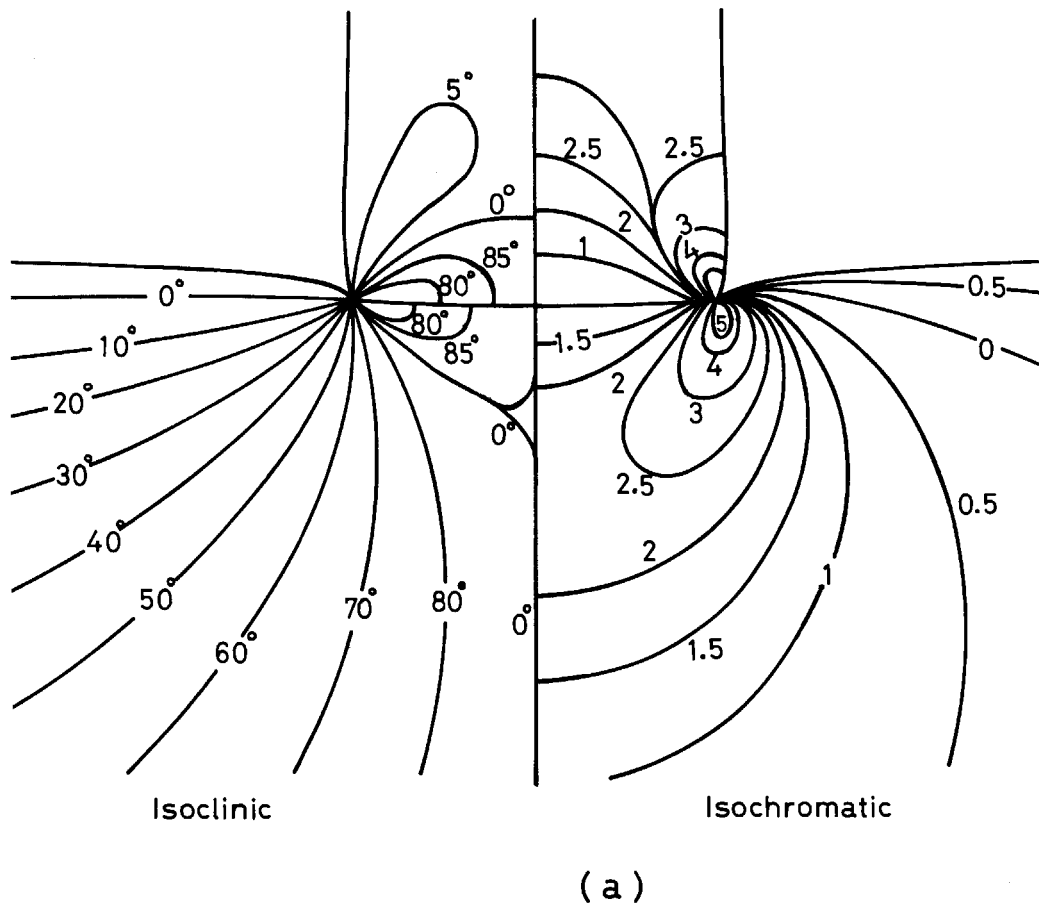
Fig. 2a.   Isoclinic and isochromatic fringe patterns at 300 minutes :
           Experiment 1.

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As examples of the results, Figure 2(a) shows the isoclinic and isochromatic fringe patterns at 300 minutes after loading for Experiment 1. Figure 2(b) shows those of Experiment 2 for comparison. The differences between the isochromatic fringe patterns between Experiments 1 and 2 scarcely appeared at the initial point of loading, t = 0, whereas significant differences appeared in the transient period from the instant of loading to the static equilibrium state at t = 300 minutes. As for the isoclinic fringe pattern, the differences were not so remarkable.

(b)
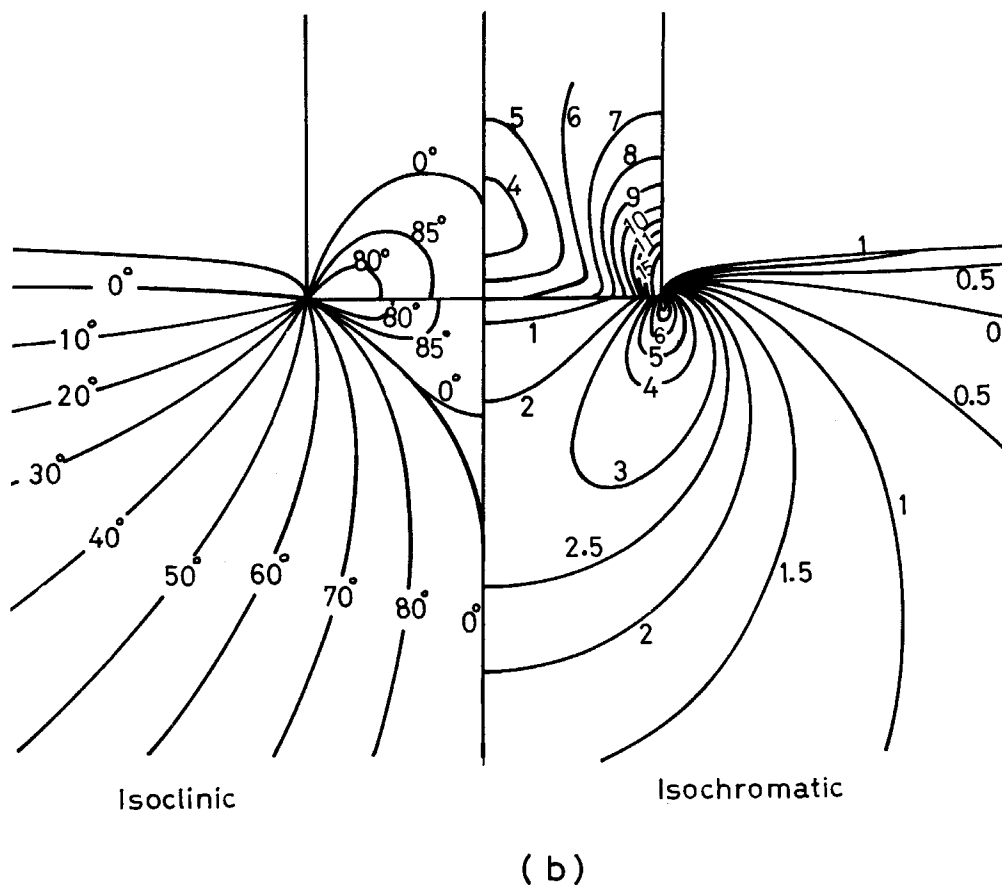
Fig. 2b. Isoclinic and isochromatic fringe patterns at 300 minutes :
Experiment 2.

The solid lines in Figure 3 show the extent of vertical deformation of the contact surface of the elastic-viscoplastic punch at 2 and 300 minutes for Experiment 1. The dashed lines in Figure 3 show those of the elastic punch in Experiment 2, for comparison. As shown in Figure 3, the amount of deformation of both ends of the punch for Experiment 1 at t = 2 is larger
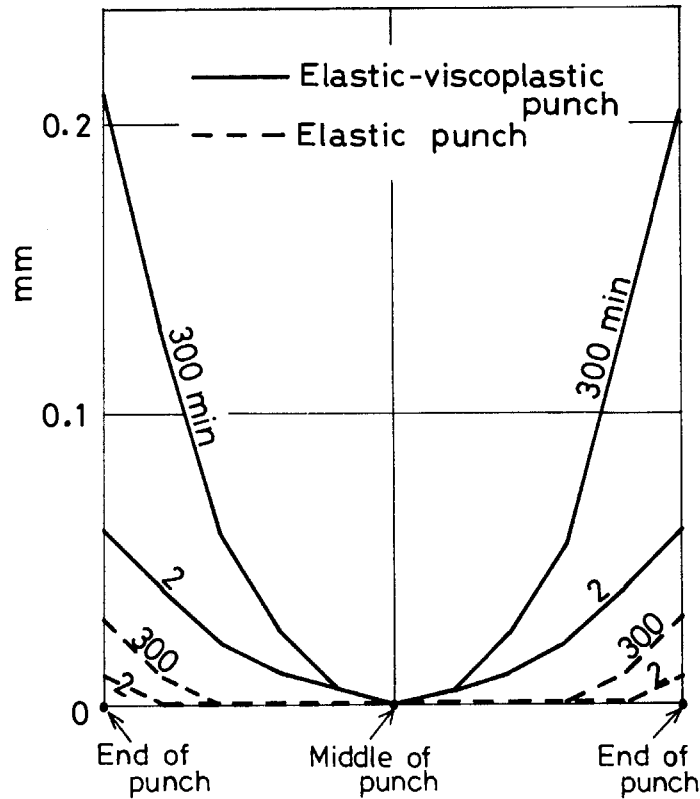
Fig. 3. Extent of vertical deformation of the contact surface of the punch at 2 and 300 minutesfor Experiments 1 and 2.

than that for Experiment 2, and the difference in the amount of deformation becomes significant with the lapse of time. The amount of deformation at each end of the punch on the contact surface is affected significantly by the different mechanical properties of the punch.

Figure 4 shows the relationships between the vertical displacement of the central element of the punch on the contact surface and time, t, for Experiments 1 and 2; the vertical displacement corresponds to the displacement between the initial state before loading and the current state as shown in the insert in Figure 4. The deformation behaviour near the contact surface and its time dependency are significantly influenced by the mechanical properties of the punch, particularly with reference to its rigidity.
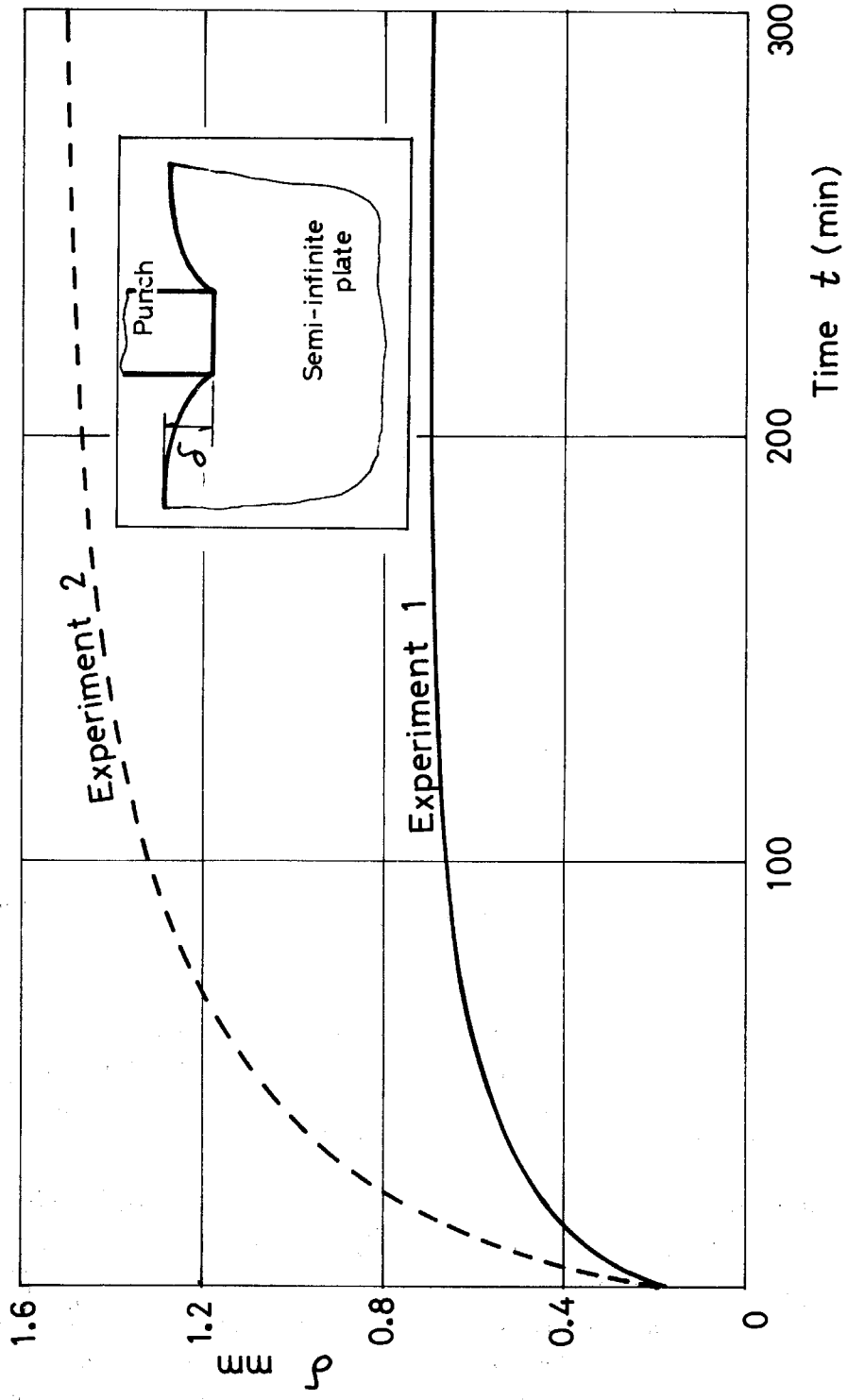
Fig. 4. Relationships between the vertical displacement of the central element of the punch on the contact surface and time t for Experiments 1 and 2.

STRESS DISTRIBUTION IN THE SEMI-INFINTE PLÂTE

The procedure for obtaining the distribution of the principal stress difference, $\Delta\sigma$, in the semi-infinite celluloid plate has been described (NISHI-TANI, 1976). Figure 5(a) shows the distribution of $\Delta\sigma$ at 300 minutes for Experiment 1, and Figure 5(b) shows the same condition for Experiment 2. Stress components in the semi-infinite plate were found by the shear difference method together with the distribution of $\Delta\sigma$ and the isoclinic pattern as shown in Figure 2. As examples of the results obtained, Figure 6 shows the stress components on the contact surface at 2 and 300 minutes for Experiments 1 and 2, respectively, where σ_x corresponds to the horizontal normal stress, σ_y corresponds to the vertical normal stress, and τ_{xy} is the shear stress.

The shaded region in Figure 5 corresponds to the plastic regions in the semi-infinite plate at 300 minutes for Experiments 1 and 2, in which the

Experiment 1 (a)   Experiment 2 (b)
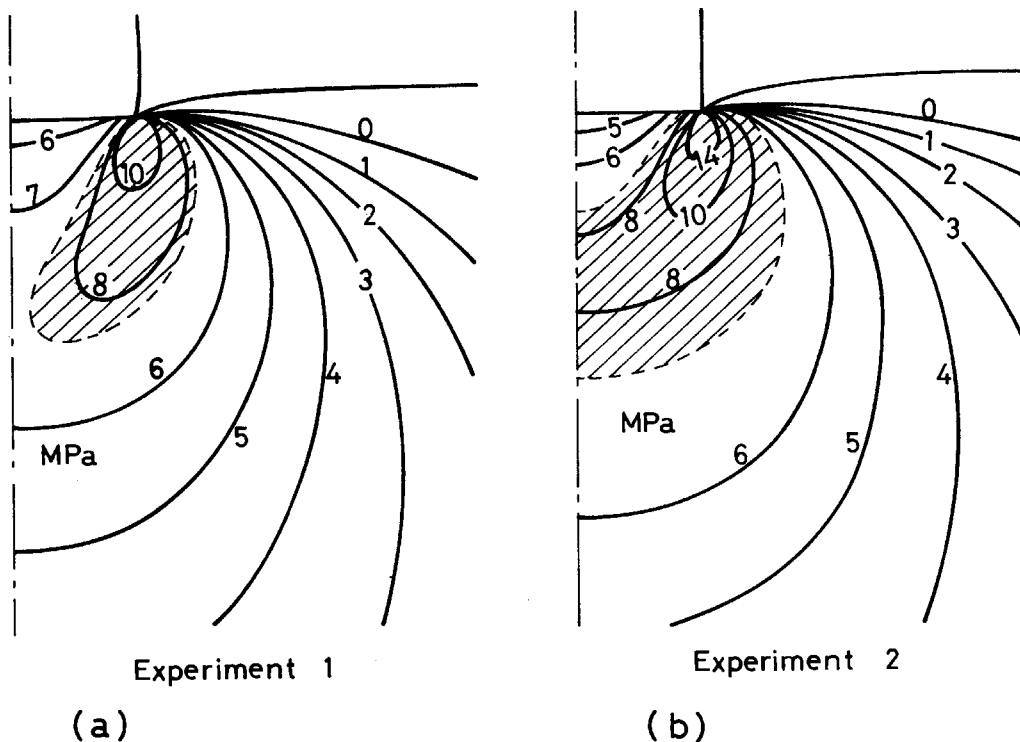
Fig. 5. Distributions of the principal stress difference, $\Delta\sigma$, (MPa) at 300 minutes.
(a) Experiment 1, (b) Experiment 2.

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yield stress is as defined previously (NISHITANI & ITO, 1979). Though the difference in the plastic regions for Experiments 1 and 2 scarcely appeared in the early stages after loading, significant differences appeared in the transient period leading to the static equilibrium state at 300 minutes. As shown in Figure 5, the plastic regions near each end of the contact surface for Experiment 1 are constrained individually by the elastic part and cannot show any large plastic deformation freely. However, the plastic regions near each end of the contact surface for Experiment 2 do join together. Such a state could indicate a large plastic deformation, and were the situation likened to that of a building a state of instability could arise. The unstable state was found to be affected significantly by the mechanical properties of the punch, and the instability increased for higher punch rigidity.

In Figure 6, the differences in the distributions of σ_y, denoted by the dashed curve for Experiments 1 and 2, was small at 2 minutes, but grew with time. The distribution of σ_y for Experiment 1 became fairly uniform except in
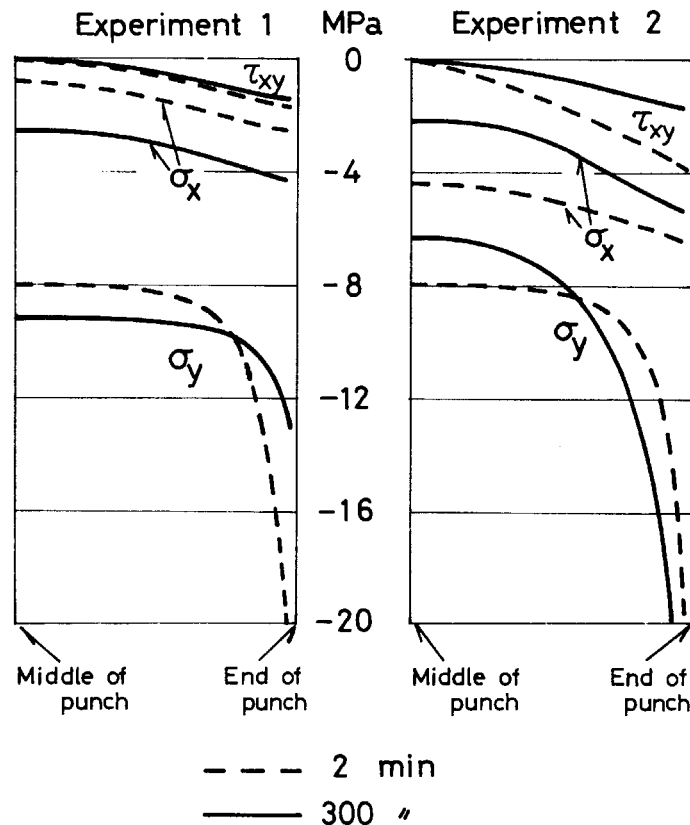
Fig. 6. Distributions of stress components on the contact surface at 2 and 300 minutes for Experiments 1 and 2.

the vicinities of the ends of the punch at 300 minutes as a result of the deformation of the punch, as shown in Figure 3.

DISCUSSION AND CONCLUDING REMARKS

Referring to Figures 3, 4 and 6, the deformations and contact pressures along the contact surfaces shown were considerably different because of the differences in the mechanical properties of the punches, or the differences in the rigidities of the punches. It could then be said that the first cause of the differences in deformation and stress of the semi-infinite plate lay in the differences in punch rigidities.

Though the differences in deformation and stress of a semi-infinite plate scarcely appeared at an early stage after loading, these grew significantly with time. Hence, the second reason for the differences was attributed to the viscous effects of the punch and semi-infinite plate in the transient period from the instant of loading to the state of static equilibrium.

As shown in Figures 4 and 5, the deformation and stress states in the semi-infinite plate became more unstable for larger values of punch rigidity. Actual structures have a high rigidity compared with the supporting subsoil.

As shown in the isoclinic fringe patterns of Figure 2, the directions of principal stresses were not too affected by the mechanical properties of the punch.

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STUDIES OF NEGATIVE SKIN FRICTION IN MODEL PILES

S. Narasimha Rao* and N. R. Krishnamurthy**

SYNOPSIS

Negative skin friction is a downward force acting on a pile as a result of relative downward displacement between the pile and the surrounding soil. The literature review given establishes that this force can be quite significant under certain circumstances. Some experimental work on the negative skin friction of model piles is thereafter discussed. Aluminium model pipe piles of 38 mm outer diameter and 1,050 mm length were tested. These piles were instrumented with electrical resistance strain gauges at depth intervals of 100 mm and tested for two soil conditions.

In the first series, a homogeneous soft clay layer was used and, in the second, a two layer situation with a layer of sand overlying a soft clay. The test results showed that drag forces increased with depth in the homogeneous clay deposit. It took nearly 60 hours for the mobilization of full drag force. For the two layer situation a significant amount of drag force was measured in the sand, while that in the clay deposit was less. The time required for the mobilization of the drag force in the latter case was considerably less.

INTRODUCTION

If a good bearing stratum does not exist near the ground surface or at a reasonably shallow depth, the load from the superstructure is quite often transmitted to a deeper dense or hard stratum by means of piles. When the piles are driven through the overlying compressible soil strata, the subsidence of the surrounding soil under surcharge loading can induce a downward drag force on the piles called *negative skin friction*. The amount of downward drag force developed depends on the relative displacement of the pile with respect to the soil and the type of soil involved. The piles have to carry this load in addition to the load transferred from the superstructure. In some cases, this force can constitute a significant proportion of the total design load of the pile.

BRIEF LITERATURE REVIEW

The realisation that a drag load can act on piles dates back to 1914 (ALDRICK, 1970), but investigations into this drag load attracted serious attention only in recent times. Some field and laboratory studies have been

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carried out by several investigators to elucidate the various aspects of the problem.

Using 50 mm diameter model piles, ELMASRY (1963) investigated the influence of the density of the surrounding soil and the thickness of the compressible layer on the negative skin friction of piles. From field test results, VAN WEELE (1964) showed that a displacement of about 10 mm was required to mobilize the maximum negative skin friction and the value of this was found to be more or less equal to the pull out resistance of the piles. JOHANNESSON (1965) and JOHANNESSON & BJERRUM (1965) measured the mobilization of this force on 55 m long steel piles in Norway. MAZURKIEWICZ (1968) used 50 mm diameter model piles and reported a drag force nearly equal to half of the positive skin friction. BOZOZUK & LABREQUE (1969) measured a drag force of nearly 109 tonnes as against a design load of 180 tonnes for 82 m long and 1.0 m diameter concrete piles. The results of BJERRUM et al. (1969) indicated that only a very small settlement was required to develop the maximum negative skin friction, which corresponded to 20 to 25% of the effective overburden pressure. ENDO et al. (1969) investigated the negative skin friction on 0.7 m diameter and 43 m long steel pipe piles. Settlements were caused by pumping of water from a deeper sand layer. The measured values were nearly equal to the undrained strength of the soil mobilized around the piles. KOERNER & MUKHOPADHYAY (1972) used model tests to investigate the negative skin friction in organic clayey silt. A higher frictional resistance was observed for wood than for concrete. WALKER & DARVALL (1973) reported that large drag down forces can be generated by small ground movement and the mobilization of negative skin friction is a function of ground settlement. The results of TORSTENSON (1973) indicated that 1 to 3 mm settlement was required for the full mobilization of this force and was independent of the diameter of the pile. This point needs further clarification.

From the above review, it is clear that several concentrated investigations have been made to measure the negative skin friction mobilized around piles embedded in compressible deposits. In this note, an attempt has been made to study this phenomenon for two soil strata conditions.

EXPERIMENTAL WORK

Model aluminium pipe piles of 38.1 mm diameter, 1.05 m length and 2 mm thickness were used. These piles were instrumented with electrical resistance strain gauges, pasted in the inner surface of the pipe at 100 mm centres. For

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the purpose of load and strain calibrations, a similar pipe of the same size and material was loaded in a loading frame. The loads were measured with a proving ring, and the deformations using dial gauges having a sensitivity of 0.001 mm.
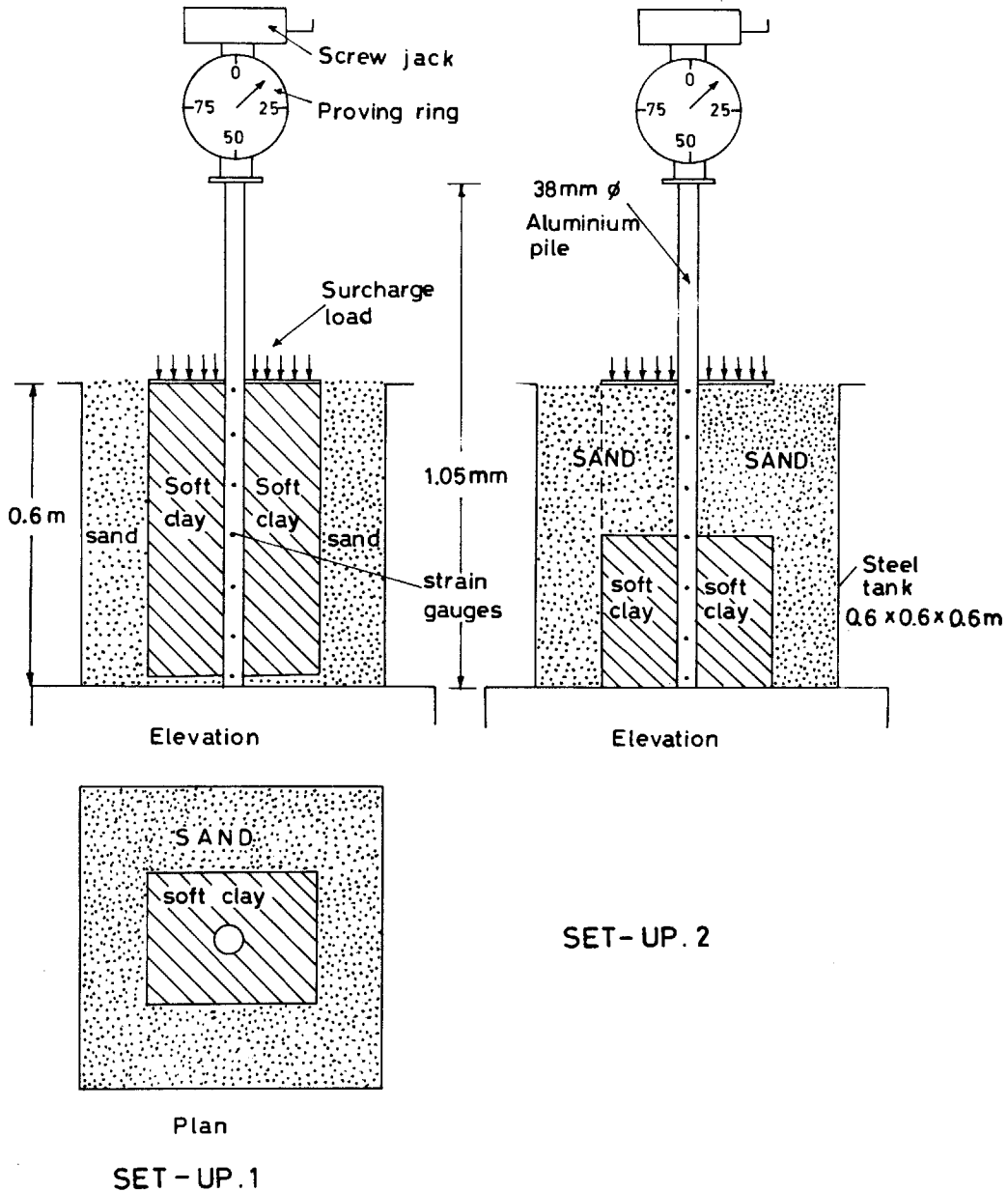
Fig. 1. Experimental set up.

Tests were conducted in a steel tank of size 0.6 m x 0.6 m x 0.6 m. The central portion of the area only, 0.35 m x 0.26 m, was filled with clay and the remaining portion was filled with clean sand. A 20 mm thick sand layer was provided at the bottom. The sand surrounding the clay packing was put in to expedite the consolidation of the clay layer. A schematic diagram of the experimental set up is shown in Figure 1.

In the first set up the negative drag force on a pile embedded in a homogeneous clay deposit was measured and in the second set up, the drag force on a pile in a two layer deposit with sand overlying clay was tried. The surcharge loading was placed on the template kept on the top of the clay or sand layer. The settlement of either the clay layer or the two layer system was measured by two dial gauges. The drag loads at different depths of the piles were measured through electrical resistance strain gauges connected to a strain measuring bridge and a switching unit. This switching unit and the measuring bridge can be used to measure strains from 75 locations simultaneously.

There were two soil types used in this investigation, one being a clayey soil and the other sand. The index properties and the compressibility characteristics of the clayey soil are given in Table 1. The sand used was uniformly graded and the angles of shearing resistance measured by direct shear tests conducted at different densities of the sand are given in Table 2. The initial placement moisture content of the clay was 95% and the density of sand placed was 1450 t/m³.

Table 1. Properties of clay.

Natural moisture content	=	100%
Liquid limit	=	115%
Plastic limit	=	40%
Plasticity Index	=	75%
Specific gravity of soil	=	2.6
Coefficient of consolidation, C_v (at p = 0.005 MPa)	=	5.63×10^{-6} m²/s
Compressibility index, C_c	=	0.75

Table 2. Properties of sand.

Maximum dry density	=	1753 t/m³
Minimum dry density	=	1410 t/m³

Results of direct shear test

Density t/m³	Angle of shearing resistance, ϕ
1410	37.8°
1600	39.1°
1660	40.6°

TEST RESULTS AND DISCUSSION

Negative Skin Friction in Homogeneous Clay Layer

The aluminium pipe pile, details of which were given in the previous section, was used with the pile resting on the bottom of the steel tank. A nominal surcharge pressure of 0.005 to 0.01 MPa was found sufficient to induce negative drag in the pile. The variation of drag forces with time is indicated in Figure 2. As expected, the gauge located in the lowest section of the pile registered the maximum force whereas the one located at the top registered the least. Nearly 60 hours were required to achieve full mobilization of drag force and, more or less the same length of time was needed at all the depths according to the gauges. The variation of ultimate drag force with depth is given in Figure 3 which demonstrates clearly that this force increases significantly with depth. The maximum value of the force
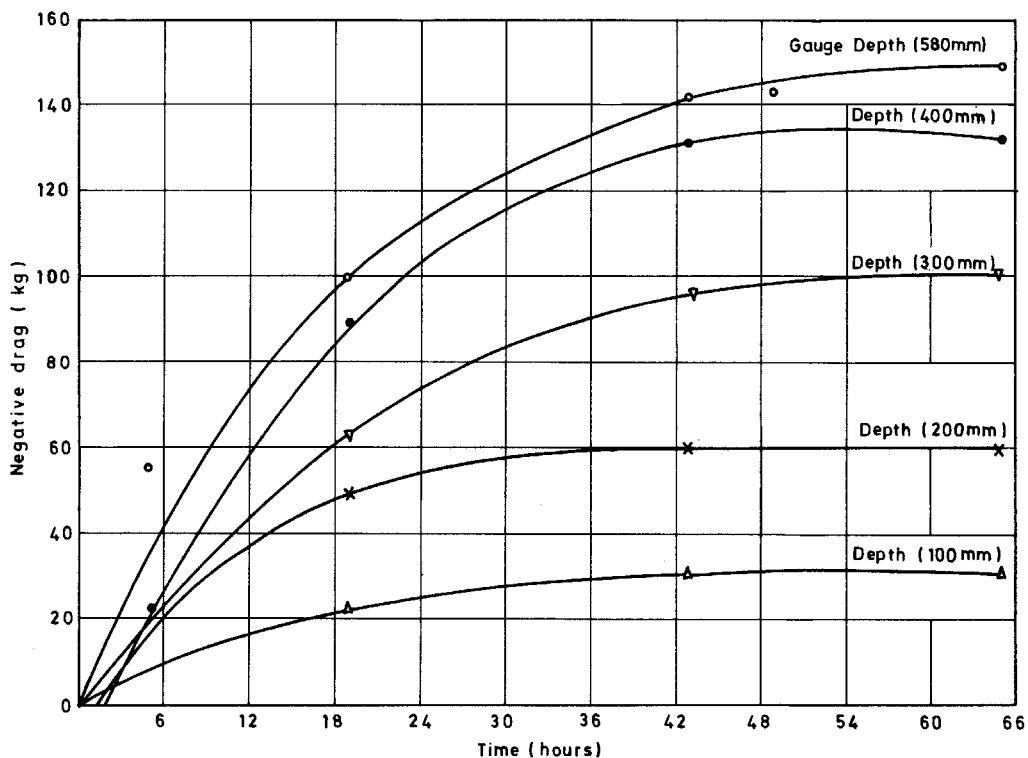
Fig. 2. Variation of drag force with time for homogeneous clay layer.

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measured was about 150 kg. In this set up, the ratio of negative skin friction to overburden pressure, calculated for the depths at which maximum negative skin friction was mobilized, was 0.33 and this compared reasonably with the previously reported values of 0.2 to 0.3 for clay.
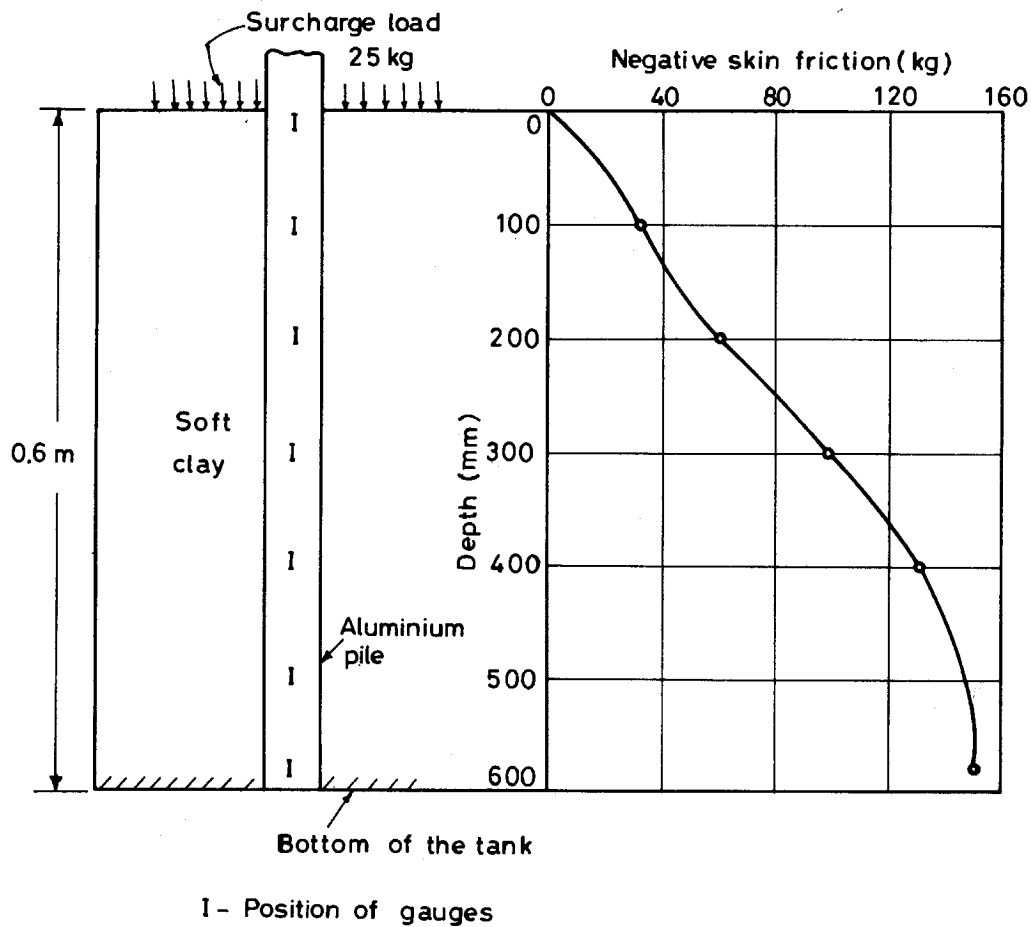
Fig. 3. Variation of ultimate drag force with depth for homogeneous clay layer.

Negative Skin Friction in Two Layered System

The soft clay used in the first series was similarly applied in this series. The tests were conducted for two different surcharge loading conditions. Once the consolidation in the bottom clay layer started, the negative skin friction was mobilized not only from the bottom clay layer but also from the top sand layer. The variation of negative skin friction with time is shown in

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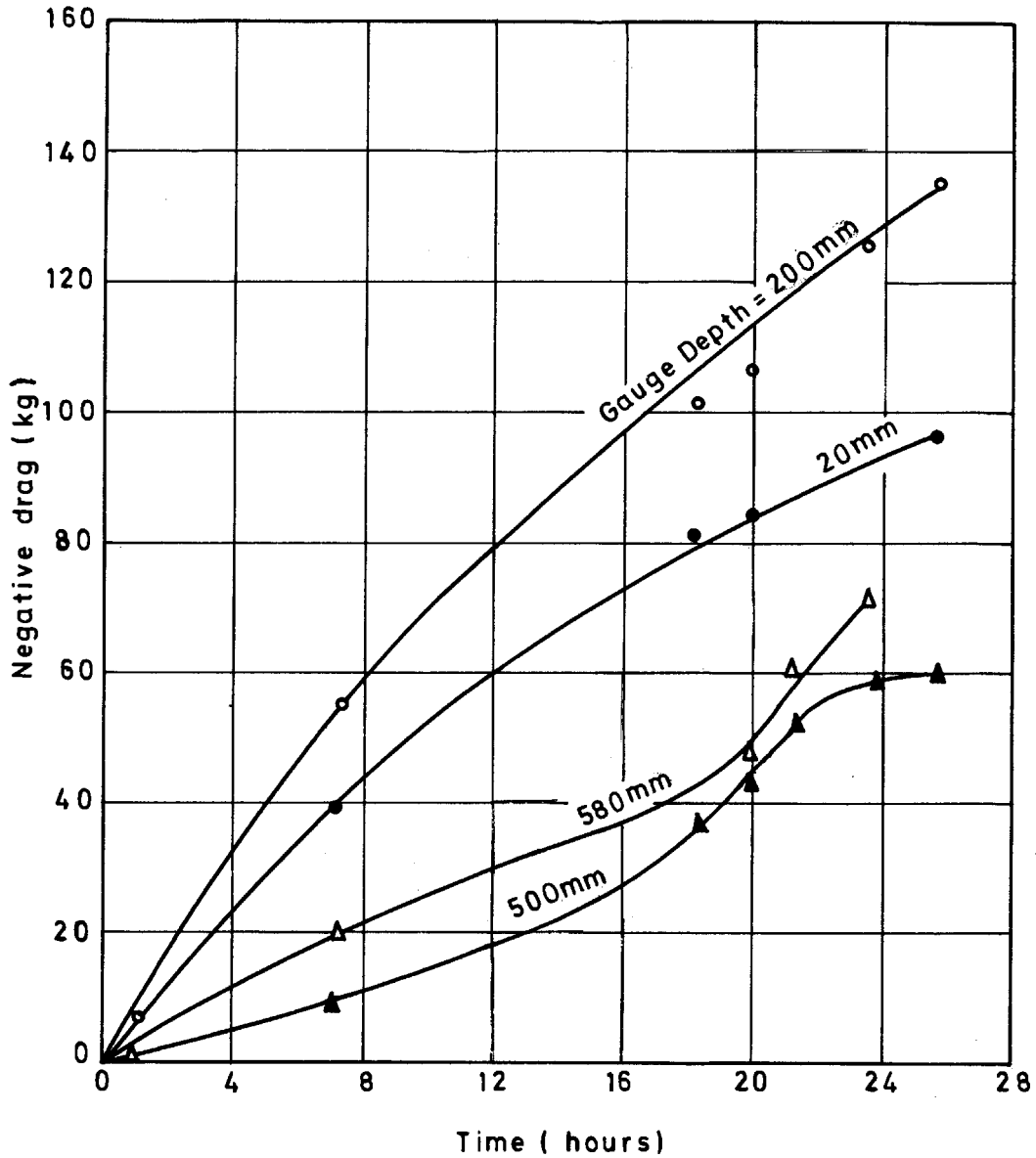
Fig. 4.   Variation of drag force with time for two layered system.

Figure 4.   The ultimate values of the drag forces were realised much faster in this series with the time taken being about 26 hours.   This rapid mobilization could be ascribed to faster consolidation of the thinner clay layer in this case. The rate of increase of drag force was much more in the overlying sandy layer than in the clay layer, the maximum force being recorded by the gauge located in the bottom of the sand layer.   This   was   because   of

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the higher frictional forces developed between the pile and the surrounding sand than between the pile and underlying clay layer. The variations of drag forces with depth for the two surcharge loading conditions are given in Figure 5.
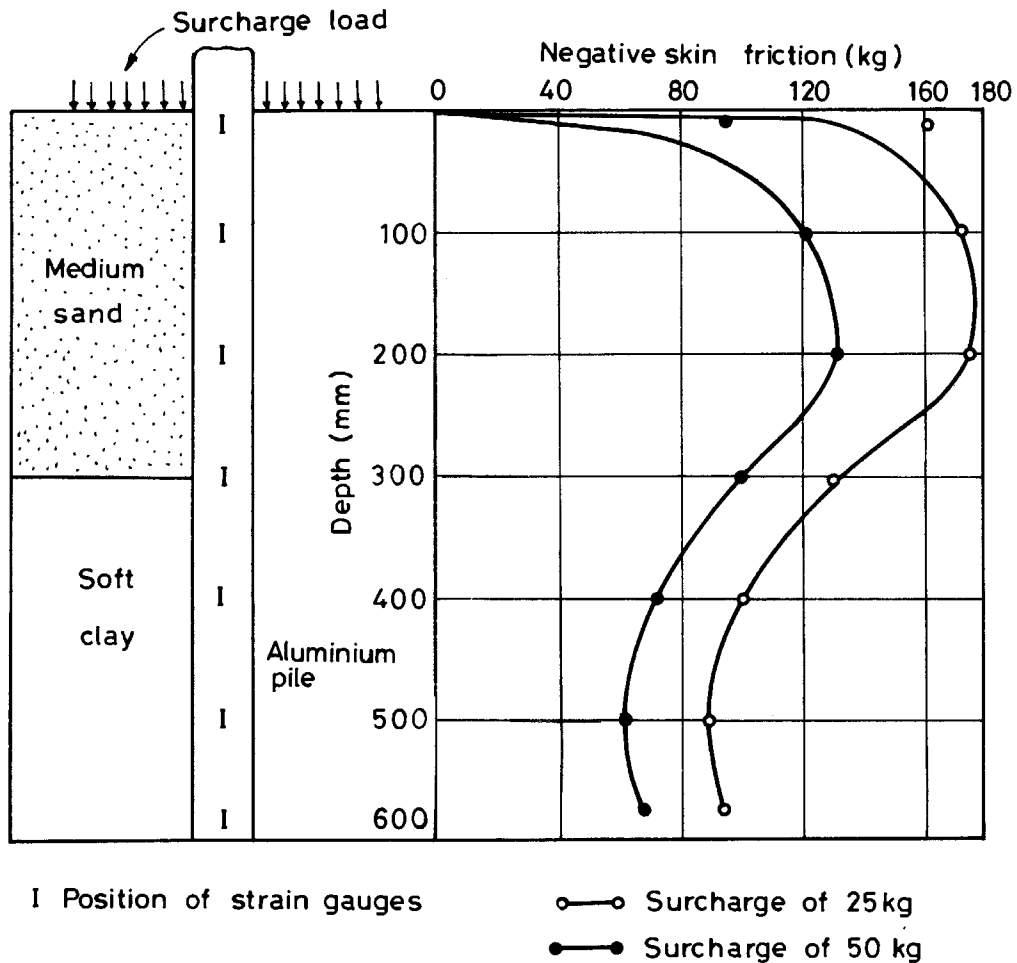
Fig. 5. Variation of ultimate drag force with depth for two layered system.

The settlements measured at the maximum drag forces were of the order of 1.9 mm and 3.4 mm for 25 kg and 50 kg surcharge loads respectively. To verify the conclusions arrived by WALKER & DARVALL (1973), experiments would have to be conducted using different types of soils inducing large variations in settlements.

CONCLUSIONS

For the model tests conducted in the homogeneous clay layer, the negative skin friction increased with depth. It took nearly 60 hours for the ultimate force to be mobilized.

Negative skin friction can develop in sand also, if it is underlain by a compressible deposit. The drag force measured in such sandy soils can be quite high and less time than for the case above is taken for mobilization of the force.

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DISCUSSION

PROPOSED HYPERBOLIC RELATIONSHIP BETWEEN SETTLEMENT AND TIME*

S. NARASIMHA RAO, K. KODANDERAMASWAMY AND J. R. SOMAYAJULU

Chin Fung Kee and **Tan Swan Beng***

It is noted with some surprise that the Authors of the above paper make no mention of the hyperbolic relationship between settlement and time as presented by Dr. Tan Swan Beng at the Fourth Asian Regional Conference held in Bangkok in July 1971 (Proc. Vol. 2, pp. 147-151). This was followed by a written submission in which Professor Chin Fung Kee presented a theoretical analysis in support of Dr. Tan's concept (Proc. Vol. 2, pp. 156-157). This hyperbolic relationship was later extended in a paper entitled "The Seepage Theory of Primary and Secondary Consolidation" presented at the Fourth Southeast Asian Conference on Soil Engineering in Kuala Lumpur in 1975. Some comment on the lack of reference to the above would be appreciated.

Author's Reply***

The authors are thankful to Professors Chin and Tan for their discussion pointing out the omission of the work of Tan (1971) in our paper. The work pointed out by the discussers follows an empirical approach which is similar to the one proposed by CHIN (1970) for estimating the ultimate load on piles. The one presented by the authors is also on similar lines and is an extension of the method suggested by KONDNER (1963) for characterizing stress-strain behaviour of soils. In fact, the authors followed it up by another paper (NARASIMHA RAO and KODANDARAMASWAMY, 1982) to estimate the ultimate compression in laboratory consolidation tests.

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Errata :

Analytical Solution of a Recharge Canal

by

PIYASENA, HOSKING and NUTALAYA

published in Geotechnical Engineering, December, 1981, Vol. 12, No. 2.

Page no.	as printed	to be corrected as
204	(head values ϕ_0 and ϕ respectively)	(head values ϕ_0 and ϕ, respectively)
207	Equation 8: $\zeta_1 = sn(\lambda\Omega, \bar{k})$	$\zeta_1 = sn(\bar{\lambda}\Omega, \bar{k})$
209	i.e. $l/b > 7$	i.e. $l/2b > 7$
211	$(l/b \sim 4)$	$(l/2b \sim 4)$
	$(l/b > 7)$	$(l/2b > 7)$

BOOK REVIEW

Salinity in Irrigation and Water Resources, edited by Dan Yaron, Marcel Decker Inc., New York, 1981, 432 pages.

This book deals with the problem of salinity in irrigation and water resources, which is widespread and affects roughly one third of all irrigated land. The areas involved include regions with humid climate as well as arid and semi-arid regions.

In order to understand the salinity problem, it is important to realize that all irrigation waters contain some amount of dissolved salts which tend to concentrate in the soil during extended periods of land irrigation. Factors such as quality of irrigation water, low soil permeability, improper land drainage conditions, low rainfall, and poor irrigation management contribute to the salt concentration in soils, and unless these salts are leached by rainfall and/or irrigation water the land, in the extreme, is totally lost with regard to agricultural production.

Any attempt to understand and successfully tackle the salinity problem must thus, by definition, be based on an effectively organised interdisciplinary approach. In this book distinct elements of the salinity problem are identified, and the techniques & terminology of the appropriate discipline, e.g., biology, agriculture, soil science, hydrology, economics, sociology, public administration, are applied to the analysis and discussion of each element. The treatment of these elements of the salinity problem are presented in a logical sequence by outstanding experts in each discipline.

The scope of the book spans over three main aspects of the salinity problem:
1. Basic Physical Relationships in salinity in irrigation.
2. Economic and social evaluation of salinity in irrigation.
3. Future Prospects.

Chapters 2, 3, 4, 5 and 6 focus on the study of basic physical relationships of salinity in irrigation.

An evaluation and classification of water quality for irrigation is presented in chapter 2. This excellent discussion of water quality aspects is recommended to geotechnical engineers interested in the study of pore-water chemistry. In chapter 3 are presented the relationships among the soil state variables (soil salinity and moisture, etc.) and the target variables (quantity & quality of crop yield). The theoretical background from which irrigation management decisions should be derived and the discussion of models for the estimation of

leaching, accumulation and distribution of salts under a variety of conditions form the scope of chapter 4.

Two major factors, namely osmotic potential of the soil solution and the influence of salinity on the uptake of both micro and macro nutrients by plant roots, underlying the interrelationships between fertilization and salinity, are discussed in chapter 6.

The economic & social evaluation of salinity in irrigation is dealt with in chapters 7 to 13. Chapter 7 presents an economic analysis of irrigation with saline water in the imperial valley of California, with reference to a farm unit and on the basis of a short-run time horizon.

Chapter 8 presents an analysis and empirical estimates of the substitution relationships, under a variety of conditions, between the quantity of salts in return flows and the income of the farm generating the return flows. A case study of economic impacts of management policies for controlling saline irrigation return flows within a regional framework is presented in chapter 9, and an extension of the scope of the economic analysis to a case of interaction between two regions is discussed in chapter 10.

In chapter 11 a study aimed at the analysis of optimal land and water use and allocation in the United States as a whole, in relation to nonpoint pollution and water environmental quality improvement, is described. Cost sharing and pricing for water quality, and legal and institutional approaches to salinity management are discussed in chapters 12 & 13 respectively.

Future prospects in relation to salinity problems in irrigation are discussed in chapters 14, 15 & 16. In chapter 14, the technology and typical features of the major desalination processes are reviewed, along with their limitations. Actual performance of seventeen desalting systems at eleven different sites are described.

In chapter 15, the strategy of water resource planning and utilization under conditions of scarcity and deterioration of quality is discussed. A broad perspective of the situation of the world's arid lands, the trends affecting them, and possible future developments, is presented in chapter 16. An important introductory overview of the problem of salinity in irrigation is presented.

This excellent reference work on salinity in irrigation and water resources is likely to be very useful to both research workers, engineers and planners associated with irrigation, management, and control of salinity conditions of land, particularly in arid and semi-arid regions of the world.

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CONFERENCE NEWS

GROUND IMPROVEMENT TECHNIQUES IN CIVIL ENGINEERING AND RESOURCES DEVELOPMENT

Report by

A. S. BALASUBRAMANIAM

As precursor to **Short Course and Symposium on Soil & Rock Improvement Techniques including Geotextiles, Reinforced Earth and Modern Piling Methods**

Arising from the ever increasing need to utilize construction sites with poor soil conditions and because many such soils can be made into useful construction materials, ground improvement techniques have become an important aspect of geotechnics. These techniques are also now used increasingly in underpinning and repair works related to historic cities and buildings. As reported by Smoltczyk, the preservation of old monuments, churches and castles always has been a concern and, as such, recent developments in piling, anchoring, grouting and jacking have been widely introduced in the field of underpinning to get away, to some extent, from manual labour which is usually too expensive and somtimes dangerous for large projects. In a recent state-of-the-art report, Sembenelli and Ueshita clearly stated the importance of environmental geotechnics by their opening remarks, stating that "engineering's horizon has dilated, from the principle of science and discovery, to the ethics of responsibility. In recent years, as if the time rhythm had accelerated, we realized that man is facing not a series of problems that can be defined and solved one after the other, but rather a number of interrelated facts so closely knitted that it proves to be nearly impossible to untie one of them without affecting the others, although they may be apparently remote. If information is not knowledge, we are now learning that knowledge is a different thing from wisdom".

In a recent study reported by the ASCE Committee on "Placement and Improvement of Soils", to identify significant long-range developments and to evaluate them in terms of their importance, feasibility and probable time of occurrence, a list of the developments judged to be innovative was presented as below.

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(1) Deep dynamic densification of cohesive soils.

(2) Membranes for osmotic dewatering in boreholes and trenches.

(3) In-situ fusion stabilization of soils to form impermeable barriers and reinforcing elements.

(4) Soil treated to change thermal conductivity.

(5) Embankments of soil encased and stacked in tough, durable membranes to eliminate the need for drying and compaction.

(6) Stabilization with bacteriological agents.

(7) Probes driven and exploded to release self dispersing grout.

(8) Settlement control by membranes to prevent drainage and consolidation.

(9) Densification by electrical shock.

(10) Soil property modification by radiation.

In addition, the ten most desirable developments were listed as follows.

(1) In-situ soil parameter evaluation.

(2) Utilization of solid wastes as construction materials.

(3) Travelling machines to form roadways by inplace processing of native soils.

(4) In-situ treatment to control expansive soils.

(5) Borehole techniques for evaluation of grouting.

(6) Slide stabilization with chemical grouts.

(7) Horizontal reinforcement nets for embankments on weak soils.

(8) In-situ installation of soil reinforcing members.

(9) Compaction equipment with variable characteristics controlled by feed-back from the soil.

(10) Densification of deposits on the ocean floor.

The Committee further stated that advances in the areas of soil improvement and placement would be very rapid in the next 50 years. However, innovations and advances would primarily be technological rather than conceptual.

In his state-of-the-art report on saving cities and old buildings, Smoltczyk said that, especially in over-populated areas such as central Europe, the continuous competition of traffic, industry and leisure activities for available sites had generated fierce controversies among people who try to develop either their own material standards or those of their community, and those who primarily feel obliged to preserve as much of the cultural scenery as

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possible no matter what the commercial consequences might be. It was a curious, but encouraging feature, that it was the most progressive section of the young which was ardently concerned with this type of conservatism. In the strengthening of foundations of old buildings, small diameter bored injection piles seemed to have had a great impact over the last few years.

Small diameter bored injection piles were generally known under such names as micro or mini piles, needle piles, root piles, mini shell piles, etc. A conventional type of bored pile was generally of the order of 300 mm or more in diameter while small diameter piles ranged in size from 120-250 mm. These piles were, however, of high bearing capacity and according to Koreck the high capacity resulted from the special method of construction involved during which concrete or mortar was forced into the soil. The pile could be constructed in all types of soil or rock and installed in any direction. The transfer of load to soil was mainly by skin friction and the piles could act both in compression and in tension, while being somewhat weak in resisting bending moments. The advantages claimed in using small diameter injection bored piles were:

(1) the equipment used for construction was easier to handle than that used in conventional bored and driven piling.

(2) the piles could be produced without vibration and noise.

(3) settlement of the piles was generally small.

Because of the above characteristics, small diameter bored injection piles were widely used in special repair jobs and in the preservation and restoration of buildings and structures. According to Koreck these piles had a wide range of applications as listed below.

(1) They could be used to strengthen existing foundations for the protection of buildings next to deep excavations and could supersede classical underpinning.

(2) In addition to remedial works for foundations, these piles could knit together, at the same time, dilapidated or bad construction.

(3) They could be used in increasing the number of storeys of an existing building when new columns could be founded on small diameter bored piles constructed from the inside of the building.

(4) For the strengthening of existing foundations to take more loads. For example, for chimneys the system had proved most suitable because of its ability to take up compression as well as tension loadings.

(5) For the protection of buildings sited next to deep excavations, instead of underpinning, injection piles could transfer the load into deeper layers. The additional amount of settlement was small. Sometimes in

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non-cohesive ground a grout had to be injected into the soil to bind together stones and gravels.

(6) Often ground anchors needed to be installed to carry horizontal loads; they were absolutely essential for deep excavations. Ground anchors were also necessary for shallow excavations when the piles were drilled with an angle of inclination from the outside of the building.

The Fondedile piling method is another system that could be used to arrest movements or settlement of structures quickly, with a minimum of disturbance to existing structures. The main feature of Fondedile patented root piles is that the piles can be formed through the existing foundation of the structure, thus providing a direct connection between the piles and existing foundations and so eliminating the need for any shoring of the structure during underpinning operations.

The use of mini-shell piles in the housing industry was recently reported in "Ground Engineering". The application of this novel technique in building low-rise structures on clays subjected to shrinkage and swelling and hence their potential for domestic housing projects was illustrated. The West shell piling system, which combines a driven shell and cast-in-situ core, would seem to have a wide range of applications in civil engineering for roads, bridges, industrial plants, and sewerage schemes for example. However, for housing developments, the mini-shell pile developed by the company can be driven with a small un-obstructive piling frame. For the mini-shell method West claim that cost savings are achieved in a number of ways; site preparation is kept to a minimum; the shells are driven quickly; there is saving of labour, time and reducing of delays for the main contractor; no removal of spoil is necessary; negligible trimming of the pile is necessary since it is only infilled to the required height; underground water has no effect on the construction programme.

Brown and Warner recently described, in an article in ASCE, the use of compaction grouting in correcting settlement of structures. In the past, when structures settled substantially because of faulty foundation soil, remedial methods were limited either to the replacement of the faulty soil or underpinning. In recent years, it was claimed that another alternative had been added i.e. the in-situ stabilization of the soil by compaction grouting. This was being found as frequently the most desirable alternative and had been successfully used on many different projects under a variety of soil and structure conditions. As described subsequently, knowledge of conventional grouting is fairly wide-spread, and its technology well documented. In contrast, available published information on compaction grouting would seem rather limited. The essential difference between compaction grouting and

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conventional grouting is that, whereas in compaction grouting the soil is displaced, in conventional grouting the grout penetrates the joints or fractures in rock or the pore spaces in soil.

Compaction grouting consists of introducing a mass of very thick consistency grout into the soil, thus both displacing as well as compacting the soil. Brown and Warner described compaction grouting as actually more effective in fine-grained soils which were formally considered ungroutable. Nearly all compaction grouting applications had resulted in improved soil conditions, but there seemed to have been cases where additional settlements had occurred because of :

(1) in-adequate pre-grouting investigation.

(2) the new, essentially empirical nature of the technique which had only been improving by experience.

(3) improper application or inadequate equipment.

(4) an inadequate programme as a result of cost or other factors.

According to Brown and Warner compaction grouting was most commonly used to stabilize the soil under residences and light commercial buildings. However, it had been extensively used to stabilize foundations of larger structures, including bridges and culverts and the ground under the tips of piles. The major advantages and also some disadvantages and limitations were pointed out as :

(1) minimum disturbance to the structure and the surrounding ground during repair; although it was commonly necessary to drill and grout holes both inside and outside the structure, yet it was possible to keep it in use during grouting operations. Large equipment, including the grout mixer and the pump could be placed several hundred feet from the working area.

(2) minimum risk during construction -- an excavation which may endanger the structure or adjacent structures was not required and hazards to workmen and occupants were largely eliminated.

(3) greater economy; although compaction grouting was expensive it was frequently the most economical method.

(4) support to all portions of a structure. Many structures had slab floors which had to be supported as well as the walls. This complicated underpinning but was easily accomplished by grouting. Grouting of underlying soils also minimized differential settlement between subsurface utility lines and structures.

(5) reduced need for extensive exploration. Although an investigation

should be made to determine the cause of a problem and approximate depth to adequate bearing soil or rock, it need not be nearly as thorough as that required for underpinning or removal and re-compaction.

(6) greater flexibility. If unanticipated conditions such as greater depth or some weak soil are encountered, they are more easily handled from a design and construction point of view when compared to other methods.

(7) ground water does not adversely affect injection or effectiveness.

(8) grouting can raise structures to grade, since the grout can literally jack up the ground and structures.

(9) grouting solves the problem at the source, the soil itself. The grout seeks out weak zones, and thus acts where the need is greatest. Although it is normally too expensive to stabilize ground not affecting structures, of necessity the ground immediately adjacent to structures is stabilized so it tends to be a more complete solution than underpinning.

Because of the obvious and sometimes rather spectacular successes of compaction grouting, there is sometimes a tendency to expect too much of it and Brown and Warner recognized that there were limitations as:

(1) relative ineffectiveness in stabilizing near-surface soils when the overlying restraint was small, such as immediately under a footing.

(2) prohibitive cost for some structures if the faulty soil was excessively deep — i.e. when depths exceed 6 m to 10 m.

(3) grouting adjacent to unsupported slopes may be ineffective. As a result of reduced grout pressures necessitated by a lack of lateral support, it was seldom possible to effectively densify soils nearer than about 3 m horizontally from the surface of a slope.

(4) difficulty in analyzing results. Grouting was not as clear cut or easy to analyse as underpinning or replacement of the problem soil.

(5) no value in decomposible material. It was not a permanent solution trash dumps or other soils with an appreciable organic content subject for to further decay.

(6) questionable effectiveness in saturated clays. More research needed to be done in this regard. However, because of pore pressure build up during injection, extremely close control was required and beneficial results were limited.

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(7) danger of filling underground pipes with grout. When settlement had occurred, sewer lines or other substructures might have ruptured and care had to be taken when grouting nearby.

When a quantity of grout was injected into a compressible layer between two zones of competent soil, Brown and Warner stated that the following might occur.

(1) The grout pressure caused a complex system of radial and tangential stresses in the soil, which were greatest near the grout mass and decreased outward. For a given soil, the soil stress and maximum distance of influence would vary with the injection pressure.

(2) For a given pressure, the diameter of the grout column would vary not only with different relative compactions, but also for different soil types and moisture contents.

(3) As the pressure increased, the grout column would increase in diameter, affecting compaction to a greater distance. Close to the grout column major displacement and shearing or plastic deformation of the soil would take place. At some distance, when the stress was low in relation to the soil strength, the deformation would be largely elastic. In between, a major zone of elastic and plastic deformation would occur.

(4) As the grout column increased in diameter, the compacted soil offered more and more resistance. Therefore, if pumping was continued, a pressure was ultimately reached at which the uplift pressure became large enough to cause upward movement of the ground surface. At this point, the pressure decreased, and continued compaction diminished.

(5) The total quantity injected would be a function of pumping rate: slower rates resulted in greater quantity.

(6) The degree of compaction achieved would be a function of the amount of grout injected.

Ground treatment by deep compaction has been in practice now for nearly three to four decades. In a recent symposium on ground treatment by deep compaction, Burland, McKenna and Tomlinson stated that the bulk of papers written in this field were by contractors specializing in these techniques and, not unnaturally, they concentrated on the successes obtained by their methods. The failures, or at least the lack of apparent successful applications, remained un-recorded. It was commented also that the result of this had been the growth of a certain mystique surrounding the techniques and claims had been made on their ability to "strengthen" ground, which could

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not always be substantiated when subjected to critical review. In reviewing three types of compaction methods, namely vibro-compaction of non-cohesive soils, stoned columns in cohesive soils and dynamic consolidation, Burland, McKenna and Tomlinson commented as follows.

(1) The effectiveness of vibro-compaction in free-draining non-cohesive soils appeared to be beyond question. The same could not be said of stone columns in soft cohesive soils.

(2) A major benefit of stone columns could be to strengthen soft ground against horizontal loading as applied by either seismic forces or under embankments by fill. Before employing stone columns it could be worth investigating the relative cost of employing gravel filled trenches beneath the toes of the embankment slopes.

(3) The technique of dynamic consolidation offered many attractions. The application of a compaction technique to non-saturated soils and loose granular soils presented few conceptual difficulties. Apparently, the application of intense impact energy to the ground surface induced liquefaction and this, coupled with the deformation of cracks, led to a large temporary increase in mass permeability giving rise to rapid settlement, dissipation in pore water pressure and gain in strength. As with stone columns, the application of dynamic consolidation to soft cohesive soils merited further investigation and field studies.

Janes recently described, in the Geotechnical Engineering Journal of ASCE, the successful densification of sand in a dry dock by using Terra Probe. The method of compaction was considered successful although it had limitations relating to depth, to the proximity of the original bottom of the unfilled site and to previously compacted areas. Janes suggested that the method could be accepted as a satisfactory tool for compaction of deep sand fills provided its limitations were recognized and provided uniformity was not an absolute requirement. Brown and Glenn, also in the Geotechnical Engineering Journal of ASCE, contributed a comparative study on vibro-flotation and Terra probe related to the extensive expansion programme carried out by the Newport New Shipbuilding and Dry Dock Company. This expansion involved reclaiming about 40 acres from the James River in U.S.A. and constructing a graving dock, steel production facilities and numerous support facilities. The river bottom at the site consisted of about 9 m of soft highly compressible soil. Various foundation systems were analyzed to select the most feasible method of supporting the proposed structures. Cost analysis seemed to have heavily favoured a shallow spread footing system involving dredge removal of the soft soil, replacement to grade with clean hydraulically placed granular fill followed by deep vibratory compaction.

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The Terra-Probe and vibroflotation methods were considered as the best available compaction methods for the project and Brown and Glenn reported on the merits and comparative performance of both systems.

Vibroflotation is a technique for densifying in-situ cohesionless soils with simultaneous vibration and saturation. It seems that the technique was developed in Germany in the 1930's and was first used in the U.S.A. in the 1940's. Brown commented that the technique could be used to densify granular hydraulic fills, coastal plain sediments and alluvial soils to increase the soil bearing capacity, reduce foundation settlements, and increase resistance to liquefaction. The experience gained from vibroflotation projects and test programmes indicated that several inter-related factors influence the densities achieved by vibroflotation. These factors included (i) the equipment, (ii) probe spacing and pattern, (iii) in-situ soil, and (iv) the vibroflot.

"Lime Columns — A New Foundation Method", was the subject matter of a recent study presented by Broms and Boman in the Geotechnical Engineering Journal of ASCE. Extensive study on the applications of lime column methods in cohesive soils had been made by Swedish and Japanese experts. Buildings in Sweden were usually supported on point bearing pre-cast concrete piles in soft clay. Rails or steel pipe piles were, as a rule, used only for light structures. Timber piles, usually untreated, were used in the Gothenberg area where up to 100 m thick deposits of soft clay are common. The lime column method offered a new foundation method in contrast to the traditional method. With the new method, columns up to 10 m long and 500 mm in diameter were manufactured in-situ by mixing soft clay with unslaked lime, using an auger shaped like a giant egg beater. Lime columns could be used to support light structures, and the total as well as the differential settlements could be reduced since the weight of the structure was transferred to deeper and less compressible strata. Other uses listed were as follows.

(1) They could be used to reduce negative skin friction below pile-supported buildings or below bridge abutments.

(2) They could prevent lateral displacement of soft ground under bridge abutments or under structures because of lateral creep.

(3) They could be used in soft clay to increase the stability of slopes and to reduce the lateral earth pressure on sheet piles and retaining walls.

(4) They could be used instead of sheet piles in deep excavations and to prevent failure by bottom heave.

(5) Lime stabilized clay was generally easy to excavate and could often be used as a backfill material.

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Electro-osmosis is another effective method for stabilizing fine grained soils. The process has been used for a long time now in dewatering foundations and also in stabilizing cut slopes and embankments. Electro-osmosis was recently investigated by Segall, O'Bannon and Mathias as a method of increasing the capacity of dredged material disposal sites by reducing the water content of the in-place material and by increasing the shear strength of the containment area embankment foundations.

Recently, Wood reported in Geotechnique that the past decade had seen significant advances in the development of installation plant and materials used in pre-fabricated drains. Wood added that this had also been associated with great economy in the labour required by comparison with conventional sand drains. The achievement of the high installation rate was linked with the recent revival of interest in band type prefabricated drains which could be mass produced independently of site preparations, subjected to close quality control and stockpiled ready for use. They also provided a greater probability of drain integrity than the conventional sand drains could afford, prior to the introduction of the Cementation sandwick drains. The first use of a band-type drain by Kjellman in 1937 post-dated, by only a few years, the first use of sand drains in 1934. There seemed to be about 50 varieties of pre-fabricated drains now available. With a number of major vertical drainage contracts recently completed in the U.K., some involving trials, the time appeared ripe to review the current situation with regard to the design and application of the process. The purpose of a recent symposium arranged by I.C.E. London was to collect together data and experience gained from some of this work in the hope of stimulating discussion amongst the members of the geotechnical community within which there remained conflicting opinions of the relative merits of different types of drain and installation systems, extending in some cases to a cynical view of the whole system.

The Committee of the ASCE Geotechnical Engineering Division on Grout Jacking, recently presented a state-of-the-art report on slab jacking. The Committee reported that pressure injection for the purpose of raising or stabilizing faulty concrete pavement had been practiced for more than 40 years. During this period a variety of different materials had been utilized including hot asphalt, various soil and soil-cement slurries, and a wide variety of cement-sand grouts. The Committee remarked that, over the years, a great deal of research and a large amount of actual work had been performed. However, because of limitations in the various systems and equipment, the full potential of the methods had only come to realization within the past decade or so. The technique, as presently practiced, was properly referred to as "slab jacking" when lifting or levelling was involved, or simply "pressure

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grouting" when void filling was the sole objective. Although the term "mud jacking" had been used extensively in the past arising from the practice of using the "mud jack" machine, the term was now considered inappropriate since modern practice involved many types of machinery and materials. Investigative efforts which peaked during the mid 1940's considered a number of different grout materials, although the base was usually reported as "clayey or silty loam soil". Added to the soil in varying amounts were a wide variety of materials including portland cement, hydrated lime, flyash, asphalt bitumen, rotary mud, casting plaster, limestone dust and calcium chloride. A large percentage of slab-jacking operations continued to be used in connection with highway maintenance.

Under-ground construction was an important consideration in the development of urban areas. Because of the disruptive effects of cut-and-cover excavations, it was becoming more common to employ tunnelling. These comments were made by Tan and Clough in their recent publication on ground control for shallow tunnels by soil grouting. Where tunnelling was shallow, there arose the potential problem of construction related ground settlements and their effects on overlying structures. Conventional measures to deal with this problem, such as compressed air or structural underpinning, were costly and time consuming. One of the most successful of the new methods for sandy or silty soils involves the injection of the soil around the tunnel heading with a chemical agent which is fluid upon penetrating the pores of the soil, but which hardens shortly thereafter. The soil is then transformed into a coherent, strengthened mass with a substantially reduced permeability. This serves to prevent soil runs, reduces the deformability of the mass stressed by the excavation tunnel and provides stand-up time for the soil at the heading and around the tail void created by shield passage. Grouting had the advantage that it could be performed well in advance of tunnelling from the ground surface. Tan and Clough claimed that in using silicate grouts, the injection technique had been found in the U.S.A. to be one-third to one-half of the cost of conventional measures used to protect critical structures.

Geotextiles is the title of a book recently edited by Mr. Rankilor, an authority on synthetic and other types of membranes and their applications in geotechnical engineering. In an appreciation of membranes in a civil engineering ground environment, he said that it was important to realize that such membranes, whether acting as filters, separators or reinforcing elements, were creating new composite materials hitherto naturally unavailable, and displaying exceptional properties which could be utilized by the engineer in the construction of virtually any type of civil engineering endeavour resulting in financial saving, increased efficiency and better performance. The synthesis of

strong, non-degradable textile materials had permitted the inclusion of modern membranes in long-term civil engineering structures, such as coastal defences, dams and major highways.

Recent research work by separate independent sources such as List and the Delft Hydraulics Laboratories showed that modern non-woven fabrics could perform as well as or better than woven materials, when subjected to dynamic filtration conditions. A brief list of one-way filter drains which come within the general group is:

(1) ground water-lowering trenches (both for agricultural and industrial works),

(2) drain filters on the down-stream of dams,

(3) blanket membranes under roadways in areas of rising ground water,

(4) filters for soak away,

(5) filters for settling tanks and waste lagoons.

The main uses of membranes in marine coastal reclamation projects, as listed by Mr. Rankilor, were :

(1) to protect the sides of a causeway or face of a marine defence from erosion by the sea.

(2) to act as a filter membrane on the inside of a permeable bund,

(3) to construct drainage elements in hydraulic fill areas to allow more rapid dissipation of included water,

(4) to provide basal support for boulder fills placed on soft seabed sediments.

The uses of permeable membranes in void construction were:

(1) water drainage schemes associated with road construction.

(2) as a sub-base / sub-grade separator in temporary road construction.

(3) as a separator at the sub-base / sub-grade interface or at another interface.

Further applications of other membranes were:

(1) railway ballast/sub-grade separator.

(2) dams, filters and impermeable barriers.

(3) roof garden drainage filters.

(4) artificial seaweed mats for marine sand precipitation and erosion prevention.

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(5) soil reinforcement by webbings in slopes or below foundation structures.

(6) wind fences for the precipitation of sand or snow or for the protection of crops.

(7) sun blinds or radiation insulators in green house temperature/humidity control systems.

(8) impermeable linings for refuse dumps to prevent untreated polluted water running into the ground.

(9) erosion control fences in steep hillsides.

(10) water scour prevention at pipe outflow points.

There appeared to be an enormous scope for the application of these synthetic membranes in many civil engineering as well as resources development projects. In a recent article in the Civil Engineering magazine of ASCE, Fowler and Haliburton said that practically every material known to man had been used in attempts to reinforce or separate embankments or roadways from soft, underlying foundation materials. In modern times both raw materials and manufactured products had been used. Fowler and Haliburton claimed that recently synthetic fabrics had been found to be more economical, more easily handled, stronger and longer lasting than many traditional materials. These synthetic fabrics resisted a large range of acids in basic soils and liquids natural and man made, as well as biological attack. The generic term "geotechnical fabric" was applied to a wide variety of artificial fibre textile products used in engineered construction of civil works. These products were also termed civil engineering fabric, geo-fabric, geotextile and filtered earth.

Coming to the traditional use of grouts in earth and rockfill dam construction, recently Houlsby reviewed the engineering of grout curtains in the geotechnical engineering journal of ASCE, making the following interesting points. Cement grout curtains at a dam, could, if desired, be constructed to achieve approximately the degree of water tightness most suitable for any type of dam and its foundation conditions. This avoided excessive expenditure and time on grouting but the production of a curtain of uncertain permeability. Grouting could never be a precise operation; experience played a dominant role in successful grouting operations. In reviewing the effect of grouting in earth and rockfill dams, ideally the foundation should pass seepage at a slightly lower "crack" pressure within it, than the "pore" pressures in the core immediately above the foundation. This slight difference in water pressures would tend to assist pore pressure dissipation from the core. Houlsby demonstrated that this was in contrast to the opposite situation where, if the foundation "crack" pressure exceeded "pore"

pressures immediately above, there could be a tendency for foundation see-page to enter the core and increase its pore pressure. In reviewing the effect of grouting in concrete dams, the curtains under them had some requirements in common with those under earth-core dams, if water was worth the cost of intensive grouting or if piping had to be prevented. Under gravity dams and to a lesser extent under arch and buttress dams, grouting was usually expected to reduce uplift pressures. The logic that traditionally water tightness of the curtain in concrete dams was made a little tighter than under an earth-core dam was questionable, given reliable and adequate drainage in a non-per-meable foundation when the quantity of seepage loss was immaterial.

Houlsby also dealt with the effect of grouting at sloping membrane dams. Concrete, steel or bitumen faced fill dams were frequently provided with one or more grout curtains at the up-stream toe. If the rock face was suitably protected downstream of the membrane, seepage emerging from the rock surface was of little consequence. Unless special conditions of possible foundation instability or piping applied, a relatively high quantity of seepage through the curtain could be permitted. Considerations concerning uplift or hydrostatic pressures rarely applied at this type of dam, and then there was a logical case for a relatively permeable curtain. Controlling standards during grouting works included requirements for the inclinations of curtains and holes; adequate interception of significant open joints was a target require-ment of high standard curtains and sometimes doubly inclined holes were necessary. Except in very low standard work, it was essential to make additional stages when drilling water was lost. The depth to which a curtain was taken was often a compromise. The aim was ideally to carry it down to reach depths where the requisite degree of tightness was available naturally. The achievement of durable permanent grouting was essential; grouting which was able to be decomposed, or dislodged or chemically leached by passing seepage was the product of faulty technology.

Developments in ground freezing techniques were also gaining greater popularity in tunnelling as well as in other underground works. The purpose of ground freezing was, of course, to make water bearing strata temporarily impermeable and to increase their compressive and shear strength by trans-ferring joint and interstitial water into ice. It therefore had a wide range of applications in the field of civil and mining engineering. There appeared to be essentially two types of freezing techniques. One was the conventional cooling method which used liquid coolants, circulated through a system of pipes. The alternative to this technique was brine freezing by direct gasification of liquid nitrogen. The ground freezing technique had been used in numerous projects in Japan as well as in Europe and North America. Aerni and Mettier from

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Electrowatt Engineering Services Ltd., presented an interesting article in Tunnels and Tunnelling on ground freezing as applied in a tunnel construction project in Switzerland. It was selected as the most suitable method of providing ground stability in the moraine section of an urban motorway tunnel under Zurich where clearance under some of the buildings was as little as 6 m. The ground consisted of interbedded marl and sandstone, an area of well compacted moraine with inclusions of non-cohesive silty sand material and the presence of an artesian pressure of up to 3 m. The extensive investigations carried out and the preliminary tests and previous experience led them to conclude, besides financial considerations, that the freezing method would be best for the difficult ground conditions. An important argument in its favour was that the use of this method would obviate the necessity for large-scale and costly underpinning of building foundations in the area which would be affected by the tunnel. The ground freezing technique had been used most successfully in several German projects; tunnelling under the river Main in Frankfurt; a main sewer construction in Dusseldorf; the pit Prosper 10 in Kirchhellen near Essen; a pumping station from the Rhein-Herne canal.

Shroff and Shah recently commented that, when considering a foundation problem, a civil engineer could examine the use of grouting as a possible solution. In their article on Resorcinolic Grout for injecting sandy foundations in the Geotechnical Engineering Journal of ASCE, they claimed that grouting was an invaluable tool today in engineering practice for injecting appropriate setting materials into soils, rocks and man-made structures so as to reduce permeability or increase strength or to achieve both improvements. Chemical grouts were developed for use in finely fissured rocks and fine sands to treat successfully foundations which were considered un-groutable using cement or cement-bentonite grouts. Silicates were suited for treating sands with a diameter ranging from 0.1 mm to 1 mm and the low viscosity chemical grouts could be employed for any material finer than 0.1 mm, which cement could not handle. Shroff and Shah had investigated the formation of gel by interaction of resorcinol and formaldehyde in aqueous media in the presence of acid. The aim was to find a strength injection mix which remained in liquid form for a required time to enable its injection into the sand and which possessed sufficient gel strength to resist any foundation stresses and washout forces after grouting.

Reinforced earth was another material that had now been used extensively in the construction of highways, expressways and railways. Reinforced earth was used for retaining walls and bridge abutments It was also used for other structures to serve a diversity of purposes in industrial, civil, defence and

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water works projects and was accepted on a worldwide basis. Tumay has stated that, in the last ten years, more than 1,500 reinforced earth structures representing over 1.2 million square metres of wall facing had been completed. Currently, construction of a new reinforced earth structure began every working day. Reinforced earth was first invented by French architect and engineer, Henri Vidal. It was an original composite building material formed by the association of soil and reinforcement. According to Tumay, reinforced earth's rapid development would not have been possible without a substantial and continuous research and test effort. Basically, this research was directed towards the analysis, and verification, both in-situ and in laboratory conditions, of the materials behaviour and its physical parameter conditions. The Laboratoire Central des Ponts et Chaussees in France, as well as other government and university laboratories, had contributed extensively to the research and testing. Tumay claimed that, in the last few years, the scope of research in reinforced earth had broadened in the following ways.

(1) More advanced analysis of the fundamental issues such as adhesion as a function of the different types of soil and reinforcing strips involved.

(2) Different dynamic behaviour characteristics of the material, the effect of extremely high or low temperatures, and, particularly, the problems of aging and of the durability of the buried materials.

(3) Improvements in dimensioning.

(4) Perfecting of the material both technologically and aesthetically.

(5) Determination of its various uses.

The potential application of reinforced earth in S.E. Asia and other parts of Asia needs to be considered most carefully since it has already been used extensively and successfully in developed countries.

In a recent article in Tunnels & Tunnelling, West and O'Reilly discussed the methods of treating ground especially for tunnelling and underground works. They discussed the three traditional methods of grouting namely, permeation grouting, fissure grouting and cavity grouting. In addition to these three methods there were some specialized techniques such as claquage, grouting in which fissures were actually formed and filled in cohesive soils by high-pressure grouting. The major types of grouts used seemed to fall into categories as cement-based grouts, silicates and emulsions and resins. Cement based grouts were used to grout soils ranging from fine to coarse sand and for fissure grouting in rocks. Silicates and emulsions were used for grouting sandy soils. The well-tried and effective Joosten and Guttman silicate techniques of grouting were suitable for soils ranging from water-bearing gravel to fine sand. Bitumen emulsion grouts could be used for

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fine sands. Resin grouts were particularly suited to the treatment of fine sands and coarse silts. Stage grouting, with is often practised, was also discussed. This consisted of first injecting the ground with a coarse grout such as a cement-based grout in order to fill the larger voids in the soil, and then following this with the injection of a fine grout such as a silicate or resin grout in order to fill any remaining fine voids.

The purpose of using grouts in cavity filling was to fill any large natural or man-made cavities that may be present on a tunnel line, whether they be above or below or at the proposed tunnel level. Suitable grouts used were mixtures of cement, pulverized fuel ash and water, or of cement, pulverized fuel ash, sand and water. In discussing the application of ground freezing techniques, West and O'Reilly mentioned that the technique had been used in shaft sinking and in tunnel driving. The method was applicable to most soils, from water-bearing gravel to clay, but the most common soils that had been successfully stabilized by freezing were water-bearing silts and fine sands which were unsuitable for ground water lowering or grouting and where compressed air had been ruled out because of other considerations.

One of the most exciting developments to take place for many years in the field of rock bolting was the introduction of a new cement anchor, known as Cemicron 2000; the system utilizes micro-encapsulated water which can be mixed dry with a variety of cement products and used in cartridge form to anchor roof and rock bolts. Water mixes with the cement only when the bolt is inserted into the cartridge. It seems that this invention can produce a full column cementitious anchor for underground roof bolting which would be cheaper than resin bolts. It was claimed that extensive tests carried out over the years had indicated that the product would provide consistent an chorage of high strength at high setting speeds more safely and more cheaply, as well as more conveniently, than alternative anchorage systems currently available. The product has a high potential in all mining countries and will be invaluable in tunnelling works as well as in general civil engineering applications.

Another recent major invention is the design of waterproof plastic liners for use in tunnelling works. A new plastic membrane, consisting of a three-layer laminate which is strong enough to sustain significant tearing forces under the weight of wet vibrated concrete which pushes the sheeting over and into irregularities on the rock surface, was supplied for use on lining tunnels in the South African Drakensberg pumped storage hydro-electric scheme. The membrane, known as hypershield is waterproof as opposed to water tight and can be drilled through in the course of erection. New materials, new methods of application and an increasing awareness of the need to consider water

proofing systems at pre-design stages are the three important factors to consider when assessing the latest developments in plastic technology for tunnels, said Peter de Antoni in his recent paper on "Latest developments in water proofing", published in Tunnels and Tunnelling.

Summarizing the foregoing discussion, there would seem to have been an enormous and ongoing development in ground improvement techniques in recent years related to civil engineering and resources development. There is thus a compelling need for specialists in these fields to constantly update and upgrade their knowledge and expertise.

The Asian Institute of Technology has, over the last few years, played an active role in the dissemination and discussion of the latest developments in geotechnology, by arranging regular annual meetings in December. The 1980 and the 1981 meetings held on dams, and offshore and coastal structures respectively enjoyed considerable success in updating and upgrading the knowledge of geotechnical and civil engineers, not only in Asia, but also from other countries further afield. The theme for the December 1982 meeting, consisting of short courses and a symposium, will be "Ground improvement techniques in civil engineering and resources development". The specific topics are to include modern piling methods in addition to micro and mini-piles; deep compaction methods; the time column technique; electro-osmosis; compaction grouting; geotextiles; rock bolts and rock anchors; vertical drainage; reinforced earth; grouting works in dams and tunnels; ground freezing techniques; etc. All interested academicians, researchers, consultants and contractors are most welcome to attend these meetings and take part in the technical lectures and discussion.

Second International Conference on Geotextiles, Las Vegas, U.S.A., August 1-6, 1982. All enquiries to: The Secretary General, Second International Conference on Geotextiles, IFAI, 350 Endicott Building, St. Paul, Minesotta 55101, U.S.A.

3rd International Conference on Behaviour of Offshore Structures, BOSS'82, Massachusetts, U.S.A., August 2-5, 1982. All enquiries to: Prof. Jerome J. Connor, Chairman BOSS'82 Organizing Committee, Room 1-280, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MASS 02139, U.S.A.

International Conference on Finite Element Methods, Beijing, China, August

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2-6, 1982. All enquiries to: The Conference Secretary, International Conference on Finite Element Methods, C/o Department of Civil Engineering, University of Hong Kong, Hong Kong.

11th International Congress on Sedimentology, Ontario, Canada, August 22-28, 1982. All enquiries to: IAS Congress 1982, Department of Geology, McMaster University, Hamilton, Ontario, Canada.

3rd Congress of the Asian and Pacific Regional Division (APD) of IAHR, Bandung, Indonesia, August 24-26, 1982. All enquiries to: Ms Soelstr Djenvoedin, Executive Secretary, 3rd Congress of APD of IAHR, Jalan Ir. H. Juanda No. 93, P.O. Box 51, Bandung, Indonesia.

23rd U.S. Symposium on Rock Mechanics, Berkeley, California, U.S.A., August 25-27, 1982. All enquiries to: Organizing Committee, 23rd Rock Mechanics Symposium, c/o Prof. R.E. Goodman, Dept. of Civil Engineering, 440 Davis Hall, University of California, Berkeley, CA 94720, U.S.A.

International Mining Conference and Exhibition, Birmingham, England, September 6-11, 1982. All enquiries to: Conference 1982 National Coal Board, The Lodge, South Parade, Doncaster, The Secretaries, ABMEC, P.O. Box 121, 301 Glossop Road, Sheffield S10 2HN, England.

International Symposium on Numerical Models in Geomechanics, Zurich, Switzerland, September 13-17, 1982. All enquiries to: Dr. R. Dungar, c/o Motor-Columbus, Consulting Engineers Inc, CH5401 Baden, Switzerland.

Symposium on Shoreline Protection, Southampton, U.K., September 14-15, 1982. All enquiries to: Institution of Civil Engineers, 1-7 Great George Street, Westminster, London SWIP 3AA, U.K.

31st Geomechanics Colloquy, Salzburg, Austria, October 7-8, 1982. All enquiries to: The Austrian Society for Geomechanics, 5020, Salzburg, Austria.

10th Czechoslovak Conference with International Participation, Foundations BRNO-1982, Brno, Czechoslovakia. All enquiries to: Josef Janovsky C.E., Geotest National Enterprise, tr kpt Jarose 28, 659 01 Brno, Czechoslovakia.

International Conference on Coastal Engineering, Capetown, Rep. of South Africa, November 14-19, 1982. All enquiries to: The Symposium Secretariat, S. 204 CSIR, P.O. Box 395, Pretoria 0001, Rep. of South Africa.

7th Southeast Asian Geotechnical Conference, Hong Kong, November 22-26, 1982. All enquiries to: VII SEAGC, c/o Hong Kong Institution of Engineers, P.O. Box 13987, Hong Kong.

Fourth International Congress of the International Association of Engineering Geology, New Delhi, India, December 1-6, 1982. All enquiries to: D.F. Van Dine, EGD Selection Subcommittee, Department of Geological Sciences, Queen's University, Kingston, Ontario K7L 3N6, Canada.

International Conference on Construction Practices and Instrumentation in Geotechnical Engineering, Surat, India, December 1982. All enquiries to: Prof. Mahesh Desai, Convenor, 1982 IGS Conference, S.V.R. College of Engineering and Technology, Surat 395007, India.

Conference on Advances in Piling and Ground Treatment for Foundations, London, U.K., March 2-4, 1982. All enquiries to: The Secretary, Conference on Advances in Piling and Ground Treatment for Foundations, Institution of Civil Engineers, 1-7 Great George Street, London SWIP 3AA, U.K.

5th International Congress on Rock Mechanics, Melbourne, Australia, April 10-15, 1983. All enquiries to: International Society for Rock Mechanics, Laboratorio National de Engenharia Civil, Avenida do Brazil, P-1799 Lisboa, Cedex, Portugal.

8th European Conference on Soil Mechanics and Foundation Engineering, Helsinki, Finland, May 23-26, 1983. All enquiries to: Mr. Hans Rathmayer, Finnish Geotechnical Society, c/o VTT/Geotechnical Laboratory, 02150 Espoo 15, Finland.

4th International Conference on Applications of Statistics and Probability in Soil and Structural Engineering, Florence, Italy, June 13-17, 1983. All enquiries to: Prof. Giuliano Augusti, Chairman, ICASP-4, Istituto di Engeyneria Civile, Via di S. Marta, 3, I 50139 Firenze, Italy.

7th ISSMFE Pan American Conference, Vancouver, Canada, June 19-25, 1983. All enquiries to: The Secretary, 7th ISSMFE Pan American Conference 6060 Marine Drive, West Vancouver, B.C., Canada.

Symposium on Engineering Geological Problems of River Valleys, Poland, June(tentatively), 1983. All enquiries to: Dr. hab. E. Falkowski, Institute of Hydrogeology and Engineering Geology, Warsaw University, al. Zwirkii Wirgury 93, 02-089 Warsawa, Poland.

International Conference on Permafrost, Fairbanks, Alaska, U.S.A., July 18-22, 1983. All enquiries to: Louis DeGoes, Polar Research Board, National Academy of Sciences, 2101 Constitution Ave NW, Washington, D.C. 20418, U.S.A.

7th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Haifa, Israel, August, 1983. All enquiries to: The Secretary, Organizing Committee, Seventh Asian Regional Conference, Soil Engineering Building,

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Israel Institute of Technology, Haifa, Israel.

International Symposium on Engineering Geology and Underground Construction, Lisbon, Portugal, September, 1983. All enquiries to: Sociedade Portuguesa de Tecnica, c/o L.N.E.C., Av. Brasil, 101, 1799 Lisbon Codex, Portugal.

20th Congress of the International Association for Hydraulic Research, Moscow, U.S.S.R., September, 1983. All enquiries to: Committee on Water Research (COWR), 51 Boulevard de Montmorency, F-75016 Paris, France.

8th African Regional Conference on Soil Mechanics and Foundation Engineering, Salisbury, Zimbabwe, September, 1983. All enquiries to: Prof. W.R. Mackechnie, Chairman, Organizing Committee, 8th African SMFE, Faculty of Engineering, University of Zimbabwe, P.O. Box MP155, Mount Pleasant, Salisbury, Zimbabwe.

International Symposium on Rock Bolting, Sweden, September, 1983. All enquiries to: Kurssekretariatet, University of Lulea, S-951 87 Lulea, Sweden.

8th World Conference on Earthquake Engineering, San Francisco, California, U.S.A., July 21-28, 1984. All enquiries to: R.B. Matthisen, Chair-8WCEE, Earthquake Engineering, Research Institute, 2620 Telegraph Avenue, Berkeley, California 94704, U.S.A.

27th International Geological Congress, Moscow, U.S.S.R., August 4-14, 1984. All enquiries to: N. Bogdanov, Secretary General, 27th IGC Secretariat, Lithosphere Institute, 22 Staromonetry per., 109180 Moscow, U.S.S.R.

CONFERENCE REPORT

SYMPOSIUM AND SHORT COURSE

ON

GEOTECHNICAL ASPECTS OF OFFSHORE AND COASTAL STRUCTURES

Report by

Prof. A. S. Balasubramaniam

A full range of geotechnical and civil engineering aspects of the design and construction of offshore and coastal structures was discussed at the Asian Institute of Technology during the December 1981 Symposium and Short Course on Coastal and Offshore Structures. The Symposium which was convened on the 14th of December and continued till the 18th was formally opened by H.E. Dr. Thanat Khoman, the Deputy Prime Minister of Thailand. Professor Robert Banks, President of AIT, delivered the welcome address in which he outlined the growth of the Institute in the recent five years. Since 1975, the Institute has been fortunate to have established a Regional Computer Center with the assistance of IBM and also the government of the U.S.A. Prof. Banks also outlined the establishment of academic programmes in the broad fields of resources, rural and industrial development. In particular, mention was made of the new academic divisions — Energy Technology, Human Settlements Development, Agricultural Engineering and Computer Applications. Preparations were already in hand for the establishment of a Remote Sensing Training Center to be incorporated in the Regional Computer Center. According to Prof. Banks, the establishment of the new academic divisions and centres has greatly contributed to the development of the traditional civil engineering field of studies in which most of the symposium participants were involved.

In his opening address Dr. Thanat Khoman touched upon the current trend of research into renewable energy resources in Thailand. According to Dr. Thanat, oil and gas would continue as the most important source of energy throughout the remainder of this century and perhaps well into the next. Encouragingly, over the past few years, an increasing amount of exploration and drilling work for oil and gas had occurred in the Association of

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Southeast Asian Nations (ASEAN), comprising Indonesia, Malaysia, Philippines, Singapore and Thailand. He, therefore, anticipated that there would be a steady increase in offshore activities in this region. When speaking about Malaysia and Philippines, Dr. Thanat mentioned that in Malaysia alone, the revenue from its oil industry had increased as much as 50%, oil thereby becoming the country's biggest export besides rubber. The Philippines had also recently come into the picture of oil producing countries. For Thailand, the natural gas project in the Gulf of Thailand alone involved several offshore platforms and nearly 500 km of pipe laying of which more than half was offshore.

Prior to the Symposium, there was a Short Course on Offshore and Coastal Structures. Over 150 participants from many countries in Asia and elsewhere attended the Short Course. The lecturers of the Short Course were Prof. C.P. Wroth, Dr. H. Mori, Mr. Ove Eide, Dr. E. Dibiagio, Dr. H.G. Poulos, Prof. P.W. Rowe, Dr. J. Ringis, Dr. A. Tomiolo, Mr. P.D. French, and Mr. H. Eilenberg. The Short Course lectures concentrated on fundamentals of soil mechanics, site investigation and in-situ tests, instrumentation, piled foundations, centrifugal model tests and case histories on offshore and coastal structures.

Prof. Wroth opened the Short Course lectures by outlining the engineering problems related to production platforms for oil and gas, pipe lines and terminals. He also identified the design problems in the case of piled foundations and gravity foundations as those arising from large amplitude cyclic loading, lateral loading, scour, tension piles and under water slope stability. The relevant soil properties which had to be known included undrained shear strength, overconsolidation ratio, the elastic stiffness and the in-situ stresses. In his first lecture on fundamentals of soil mechanics, Prof. Wroth clearly illustrated the differences and similarities between offshore and onshore soil mechanics. He emphasized that the type of soil encountered in offshore and onshore works were much the same though the voids in offshore samples contained salt water, dissolved gas and were subjected to higher ambient pore pressures and wave loading effects. Offshore soil mechanics generally had to be based on relatively disturbed samples and limited data from laboratory and field tests. Prof. Wroth also mentioned that the fundamental behaviour exhibited by soils onshore and offshore were essentially the same with respect to the principle of effective stress, the Mohr-Coulomb failure criterion and the stress-strain theories which should incorporate dilatancy and inelastic behaviour. Under the section dealing with fundamentals of soil mechanics, Prof. Wroth advanced the knowledge of the short course participants by clearly demonstrating the special features of offshore

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soil mechanics. He provided many useful correlations: natural void ratio-overburden pressure, liquidity index-overburden pressure, overconsolidation ratio-depth of soil profile and strength to overburden ratio-overconsolidation ratio. In summary, he mentioned that :

(i) The best use had to be made of limited soil tests data, in which the measured natural water content plays an important role Remoulded undrained strength and compressibility coefficients had to be estimated from the liquidity index while the overconsolidation ratio could be related to the liquidity index, overburden pressure and compressibility and swell indices.

(ii) The available index tests could be improved by the use of fall cone tests to derive parameters similar to liquid and plastic limits.

A second lecture by Prof. Wroth concerned the stress-strain behaviour of soils under monotonic loading. He referred to the two basic types of calculations used in geotechnical engineering as the limit analysis and the deformation analysis. The stability of slopes was quoted as an example in the category of limit analysis, where the only soil properties used are unit weight and the shear strength, with boundary conditions such as the geometry of the soil profile and the position of the water table. He then illustrated the two types of stability problems arising under short term conditions (total stress analysis with undrained strength) and long term conditions (effective stress analysis with effective cohesion and angle of internal friction). How the soil was treated as a rigid perfectly plastic material in the limit analysis was illustrated. In contrast to the problem associated with limit analysis, the important features of deformation analysis required the soil to be treated as a two-phase material with total stresses satisfying the equilibrium conditions and effective stresses dictating the evaluation of the material properties and the deformation. In addition non-linear response and irrecoverable (permanent) strains together with dilatancy characteristics were further important aspects of soil behaviour related to deformation analysis. The influence of consolidation history, time effects (during secondary consolidation, creep aging and cementation), anisotropic behaviour and the rotation of the axes of principal stresses were also of importance in studying the stress-strain behaviour of soils prior to and at failure.

Prof. Wroth also concentrated on the various types of mathematical stress-strain models which are broadly classified as linear elastic, linear elastic and perfectly plastic, non-linear elastic and finally linear elastic and work hardening plastic types of behaviour. The usefulness of linear elastic models in deriving close formed solutions, applying super-position principles

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and to incorporate such factors as consolidation, anisotropy were well demonstrated. In describing the linear elastic and perfectly plastic models, Prof. Wroth questioned the criterion of the change in behaviour from one model to the other and the role of the total and effective stress concepts. The hyperbolic stress-strain law and other non-linear elastic models were pointed out to require special tests and were found to be only applicable to a limited range of stress paths. The potential of the linear elastic-work hardening plastic models in which he had been thoroughly involved over the last two decades for the development of the Cam-Clay models and their modifications were then described in great detail. Prof. Wroth accepted the complexity of these models and their out weighing superiority in having the generality to be applicable to a wide range of stress paths and phenomena such as consolidation, swelling, failure. Further, these models were based on the principle of effective stresses.

Under the section on site investigation, Dr. H. Mori gave two lectures, the first one being on the requirements of soil sampling for laboratory testing. The absolute requirements for perfect laboratory testing were very simple as illustrated by D.C. Procter: samples should be representative and undisturbed. Samples were considered to be representative when they were large enough to contain a sufficient quantity of the salient fabric of the ground to behave in a manner representative of an entire stratum. To be undisturbed, samples must not only be mechanically undamaged but also be in a condition that the in-situ effective stresses and degree of saturation were unaltered. Down-the-hole samples were generally obtained from offshore foundation borings by percussion with wireline techniques. For shallow sampling, various free falling gravity open and fixed pistons amplers seemed to give the best quality samples in clays. In the North Sea projects, rotating and vibrating corers had been tried with limited success to obtain samples of denser and coarser soil up to a depth of 10 m. For a feasibility study, one borehole drilled per platform location was the norm. A piled platform could have a triangular borehole pattern. Generally, the borings should cover a larger area than the platform base width to account for positioning inaccuracies. It appeared that additional boring could be required if adequate samples were not recovered from the initial boreholes. Schjetne and Brylawski stated that Hvorslev recommended one borehole per 1,000 m² in uniform soil conditions. In addition, for gravity structures one borehole was usually drilled to a depth equivalent to at least the width of the gravity base. This borehole was usually on the periphery of the proposed structures to minimize drilling mud and cuttings under the structure. The other three or four shallow boreholes were drilled to depths of 30 to 50 m. For piled structures at least one borehole extended to the pile tip depth plus the width of the pile group. During down-

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the-hole sampling samples were taken at one metre intervals up to about 30 m depth and thereafter the intervals could be increased to 3 m.

Dr. Mori described in detail the drilling equipment and procedures used in taking offshore soil samples. Shallow sampling by grabs and gravity corers could be conducted from smaller vessels while drilling of boreholes required larger vessels. A second lecture on site investigation with particular emphasis on in-situ tests was also given by Dr. Mori. In this lecture, he described in detail the application of a self-boring pressuremeter as developed by him and currently being studied extensively in Japan. Soil disturbance caused by boring and the stress relief of the soil around a borehole left unsupported could influence the parameters obtained from the Menard pressuremeter. The self boring pressuremeter had been used on a variety of soils such as soft clay, loose and dense sand in the northern coast of Tokyo Bay. The instrument contained a self-boring mechanism and penetrated the ground statically, digging a hole simultaneously. The undrained shear strength of soft clay obtained from the self boring pressuremeter correlated well with that obtained from undrained triaxial tests on undisturbed samples reconsolidated and tested under in-situ stress conditions. The shear modulus obtained from the self boring pressuremeter correlated well with those obtained from triaxial tests, but were 2 to 7 times higher than the values measured with the Menard Pressuremeter.

A third lecture on site investigation with emphasis on geophysical methods was given by Dr. John Ringis. Generally, a bathymetric survey was carried out over an area of about 1 km² centred on the proposed platform site using an echo sounder and side scan sonar. The side scan sonar system could be used to map under water surface features such as rock outcrops, boulders, wrecks, sand waves. Seismic reflection survey measurements were often made with the bathymetric soundings and side scan sonar survey.

The final lecture on site investigation, in the short course, was given by Mr. O. Eide who spoke authoritatively from his extensive experience on North Sea projects. Mr. Eide opened his lecture with special reference to the geology of the superficial sediments in the northern North Sea. The distribution of the superficial sediments were mainly sand and gravel on the submarine banks, while depressions in the sea floor were filled with normally conso- lidated clay, silt and sand, and the areas between the banks and the depres- sions had stiff and overconsolidated soils under a thin top layer of fine sand. Before carrying out a detailed planning of the site investigation, all available information on the upper soil strata was studied. According to Mr. Eide, this could even include the results of geophysical downhole logging and

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exploration seismic surveys. In the vicinity of the optimum production location it was customary to cover a large area by seismic survey with high power sparker profiling. Detailed information was, however, obtained by using a multi-electrode sparker. In addition to the seismic survey, high precision echo sounder and side-scan sonar were used to obtain a bathymetric map of the area. Subsequently, a sub-marine survey was conducted to produce detailed maps of the seabed topography.

In his description on the type of soil boring programmes carried out in North Sea projects, Mr. Eide mentioned that a detailed programme included soil sampling and cone penetration tests. The maximum depth of deep borings depended on the type of structure involved. Several positioning systems were used in the North Sea. Seismic surveys were started using continuous wave radio systems with fixed on-shore stations. During the shallow seismic survey 3 or 4 acoustic transponders were placed on the seabed and signals were sent to transducers on vessels. The accurate location of the drill vessel could be determined by satellite navigation. About 4 to 6 anchors were generally used to keep the drill ships in position. The major part of the expenditure for offshore site investigations arose from the cost of hiring a vessel. Small ships of 30-50 m length were used for seismic surveys, gravity coring and vibro-coring while larger ships of 70-100 m length with 4-6 anchors were used as drilling ships. The maximum drilling depth achieved in the North Sea project was about 240 m below seabed while the maximum water depth is about 180 m.

Scrapers were towed along the seabed and the seabed materials collected in a bag, behind the scraper. This method was used for collecting disturbed coarse material. Good-size grabs could be used to obtain samples containing gravel and pebbles. Open or piston gravity samplers were lowered into the seabed on a wire line and penetrated into the seabed by means of a dead weight. The sampling tubes ranged from 50-150 mm diameter and some of them were fitted with plastic liners. With soft clay a penetration depth of 5-6 m could be achieved. Vibro-corers and other remote controlled drilling rigs were used for coarser material and stiff clays. Deep borings had been taken to 100-150 m depth, but the soil sampling was not continuous. Penetration into the seabed was achieved by rotary drilling combined with direct mud flushing. Percussion wire-line sampling with thin-wall cylinders were driven into the soil by blows from hammers. The sample quality was generally poor when compared with that obtained by push sampling.

The most widely used in-situ test in the North Sea project was the cone penetrometer test. The maximum capacity of a 10 cm^2 cone penetrometer

was about 15 tonnes. A good field laboratory on-board where samples could be extruded, classified and tested was necessary. The classification test devices for undrained shear strength were pocket penetrometer, torvane, fall cone and laboratory vane. Unconfined compression tests and unconsolidated undrained tests were also carried out to a limited extent. The onshore laboratory testing consisted of geological tests such as organic content, thin section preparation, mineralogical investigation, cation exchange capacity, pore water analysis, foraminiferal biostratigraphy and C 14-age determination.

Various types of sophisticated laboratory tests were conducted. These were oedometer consolidation tests, permeability tests, triaxial tests, simple shear tests. The samples were tested under-cyclic loading as well as under single loading conditions.

A series of excellent lectures on instrumentation was given by Dr. E. Dibiagio. His first two lectures concentrated on the general principles of geotechnical instrumentation and their applications to a wide range of geotechnical projects. His final lecture was on the instrumentation of gravity platforms. Although, instrumentation of offshore structures was not a new endeavour, the introduction of gravity-type production platforms and storage facilities had certainly caused a marked increase in the amount of geotechnical and structural instrumentation employed offshore. It had always been the common practice in the construction industry to rely significantly on instrumentation programmes in order to verify the validity of design assumptions, to monitor the safety of projects during construction, and to obtain improved information for future design. The need for instrumentation of offshore platforms arose from the simple fact that it was not possible at present to predict their behaviour with the same degree of confidence that held for normal engineering structures. Dr. Dibiagio continued by emphasizing that, even though many offshore structures were being built, yet they were nevertheless a new type of structure and thereby differed considerably from the conventional buildings that had been constructed on land for centuries.

According to Dr. Dibiagio, there were at least 3 principal reasons for instrumenting gravity platforms. First of all there was a need to monitor the actual installation of the platform to insure that neither the structure nor the underlying soil, which must eventually support the platform, was damaged during the installation. Secondly, it was recognized in the industry that knowledge of the behaviour of these structures was limited and a systematic collection of performance data was essential for improvements in design procedures, so that better and more economical platforms could be built in

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the future. Thirdly, there were special design details on many gravity structures that should be investigated by means of full scale measurements.

The type and the amount of instrumentation needed on a platform depended on details in construction peculiar to the structure to be installed and used. Dr. Dibiagio mentioned that the instrumentation systems on Condeep structures could be divided into two groups; those intended specifically for measurements during the installation of the platform and those designed for the collection of data during the operational life of the platform. The information sought by instrumentation during the installation of platforms was:

(1) wave data by means of a buoy anchored some distance from the platform.

(2) bottom clearance by means of echo-sounders installed under the base of the structure.

(3) draught which was determined from sea water pressure measurements near the base.

(4) the ballast water level in cells and shafts by means of pressure transducers or other level measuring devices.

(5) bending moments and axial forces in dowels using strain gauges.

(6) the water pressure in the skirt compartments beneath a platform during penetration and contact grouting by means of differential pressure transducers.

(7) tilt measurements using a biaxial inclinometer.

(8) dome contact pressures using earth pressure transducers mounted flush with the underside of the domes.

(9) the strain in the reinforcing steel placed in lower domes and the bases of cell walls, with the aid of strain gauges in the reinforcement.

(10) short term settlement by means of pressure measurements in a closed hydraulic system or some other suitable system.

The data acquisition system used with the above instrumentation was a digital system with an on-line computer and a complete back up analogue system. Outputs from the digital systems were in the form of compact tabular summaries or graphs such as cross sections, histograms or time-history plots on CRT graphical displays equipped with hard copy units.

Dr. Dibiagio mentioned that typical instrumentation for performance measurements could be of the following :

(1) a complete system for oceanographic and meteorological measurements.

(2) basic contact stresses by means of earth pressure transducers mounted on the lower domes.

(3) structural strain at the base of the shafts, giving the moments transferred to the foundation by wave action.

(4) linear accelerations and displacements at the base, at mid-height of the shaft and at deck level.

(5) long term horizontal and vertical displacements by means of a flexible telescope casing installed underneath the base of the structure.

(6) pore pressures in the foundation soil by means of piezometers installed beneath the platforms.

The practical problems associated with the installation of the instruments were then discussed. A large number of instruments had to be mounted in the structure during the early stages of construction, and many of the sensors and a major portion of the platform cabling were submerged in salt water under relatively high pressure for perhaps as long as two years before the platform was installed and the instruments put into use. In addition, it was remarked that many of the sensors had to be installed in such a way that service or replacement was impossible. A Condeep construction site, for example, was a very complex work area with a great number of workers performing many different operations at the same time in a very congested space. Therefore, the instrumentation work had to be integrated into a very complicated construction sequence and the systems had to be installed in part by part simultaneously with concreting, welding, burning and other construction activities potentially hazardous to delicate instrumentation system components.

Dr. Dibiagio also stressed the philosophy of selecting types of instruments. Instruments were designed to provide a high degree of reliability through careful selection of components, planning, back up systems and strict supervision and control during the installation phase. The majority of the instruments installed by NGI used a vibrating - wire strain gauge as the sensing element. Piezo-resistive sensors were used in some instruments and servo accelerometers were used for the measurements of dynamic motions.

Surface strains whether on steel or concrete were measured by means of vibrating wire surface mounted strain gauges. They were used, for example, for the measurement of axial loading and bending moments in dowels. Internal strains in concrete were sometimes measured with embedment gauges of the vibrating wire type. Most of the internal strain measurements in concrete were carried out with the aid of instrumented reinforcing steel. Measurements

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of strain in reinforcing steel were used to determine the loading on the lower domes of the structure and to monitor the moments transmitted to the base of the structure through the shafts. Contact earth pressure between the base and the sea floor was measured by vibrating wire pressure transducers cast flush with the lower domes. Draught of the structure during towing and installation, as well as ballast water level in the cells, pore pressure and tidal variations were determined by means of pressure measurements. All these pressure transducers had two separate sensors built into them. Water pressure in the skirt compartments was measured by differential pressure transducers. Linear and angular-servo accelerometers were used to measure the dynamic acceleration of the structure.

During the Short Course, six hours of lectures were given by Dr. H.G. Poulos on Piled Foundations. Dr. Poulos comprehensively dealt with the behaviour of single and groups of piles under vertical and lateral loading as well as under cyclic loading. He remarked that the design of a piled foundation involved a number of stages including the following:

(1) the selection of the type of piles and the method of installation.

(2) the determination of the size of the piles and the number of piles in order to ensure an adequate factor of safety against failure of the supporting soil and of the pile material.

(3) the estimation of the settlement of the foundation and the differential settlement between adjacent foundations, under working loads, to check that such settlements and differential settlements could be tolerated by the structure.

(4) the consideration of the effects of any lateral loads which may be transferred from the structure to the foundation, and in particular, the estimation of the lateral deflection of the foundation caused by such loads.

(5) the evaluation of pile performance from pile loading tests and extrapolation from these tests to predict the performance of the actual foundation.

Since piles were almost invariably used in groups, it was also essential to consider the effects of group action. In his presentation Dr. Poulos first concentrated on the behaviour of single piles and then their group action. The ultimate load capacity of single piles was usually obtained by static analysis, for driven piles by dynamic analysis also, and further by test loading for all types of piles. The load carrying capacity of single piles in cohesionless and cohesive soils by static and dynamic analysis was discussed in detail.

The behaviour of piles bearing on a rock stratum was also discussed. Such factors as uplift of piles, negative skin friction and the buckling of piles also received attention.

Following the above, Dr. Poulos dealt with the settlement characteristics of single piles and groups of piles. The older methods of estimating the settlement of a single pile relied on the conventional one-dimensional consolidation theory and were unlikely to give accurate predictions since the main source of settlement of a single pile was predominantly as a result of shear rather than consolidation. In his opinion, the most reliable means of estimating the settlement of a single pile was probably to carry out a pile loading test, and a convenient means of making preliminary settlement predictions was to use a finite element analysis or an analysis based on elastic theory. In an earlier publication he had demonstrated a close agreement between solution from the elastic analysis and the finite element analysis for a pile in a homogeneous soil layer.

Dr. Poulos then went on to discuss the settlement of pile groups through what he defined as "interaction factor", a factor defined as the ratio of the increase in settlement of a pile (because of an adjacent pile) to that of the same pile under its own load. He then presented typical interaction curves for floating piles in a deep layer of uniform soil as well as those bearing on a very stiff layer. A method of analysis to take into account the effect of underlying compressible strata was also indicated. In obtaining a theoretical prediction of the settlement of the piles and pile groups by the method proposed, it was first necessary to idealize the problem by simplifying the soil profile, and possibly the pile, so that the simplified system became amenable to analysis.

The theoretical solutions presented could be used to prepare design charts to assist in the selection of the necessary number and spacing of piles to support a given load with a specified maximum settlement and factor of safety against failure. The procedure described above was then illustrated by a simple example. Both model and field tests in layers of clays and dense sand were considered wherein good agreement was noted between the observed behaviour and that predicted by his method.

In his second presentation Dr. Poulos discussed the behaviour of single piles and pile groups under lateral loading. Several examples of piles subjected to lateral loads in coastal and offshore works were cited and, in all cases, the design of such piles had to satisfy two criteria: an adequate factor of safety against ultimate failure and, secondly, an acceptable deflection at working loads. The calculation of the ultimate lateral load of piles and pile groups

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was breifly considered, but the calculation of deflections, since it was generally the deflection criterion which governed the design of piles under lateral loads, was considered in some detail. The behaviour of single piles was dealt with and then extended into analysis of pile groups. Thereafter the methods for the analysis of the deflection of pile groups subjected to combined lateral and vertical loads were described. As part of the lecture series on piled foundations, Dr. A. Tomiolo gave a comprehensive lecture on the important aspect of pile testing. Currently, piles of all types were tested in S.E. Asia on major civil engineering projects and the testing of piles formed an important aspect in all major foundation designs. In his lecture, Dr. Tomiolo discussed the various types of pile testing methods and how the results could be interpreted and presented to derive useful design values for foundation design in coastal and offshore structures.

Two excellent lectures were given by Prof. P.W. Rowe on the 'Use of Large Centrifugal Models for Offshore and Nearshore Works'. Prof. Rowe opened his lecture by mentioning that the 700-ton centrifuge at the Simon Engineering Laboratories, University of Manchester, had been used for testing authentic models of offshore structure proto-types for 10 years. Over that period, facilities had been developed to simulate :

(1)   sea bed strata to 90 m depth,
(2)   platforms to 100 m diameter,
(3)   piles of 2 m diameter, singly or in realistic groups, and
(4)   caissons of 12 m diameter.

The loading equipment allowed

(1)   any combination of static and cyclic components in compression or tension, simulating field loads in the range of 10 to 100,000 t,
(2)   any combination of horizontal and vertical loading over a practical range of eccentricity,
(3)   any phase difference, 0-360° between horizontal and vertical cyclic loading with frequencies available in the range of 0.01 to 10 Hz,
(4)   percussive driving of piles.

The control and data acquisition systems included

(1)   servo-controlled loads or displacements,
(2)   input wave loading signals controlled by micro-computer,
(3)   acquisition of data from load cells, displacement and pore pressure transducers by automatic print out and/or storage for in-house computing.

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A few of these project investigations which had had a role in the design and/or certification of actual offshore and nearshore structures were illustrated. Included also were some of the findings from general model testing.

One of the most impressive projects was the work carried out in relation to the Oosterschelde Storm Surge Barrier. The closures varied between 1 to 1.8 km in length, where the estuary was 22 to 32 m in depth. When the steel gates are closed, the barrier has to withstand a peak horizontal load of 13,500 t per peir, at 45 m spacings. As a result of wave action, the load is cyclic, varying approximately as 7,500 ± 6,000 t. The foundation soils consist mainly of sands whose cone penetration resistance varied in the range of less than 10 MN/m^2 near the surface to greater than 30 MN/m^2 below a depth of 25 m. The cardinal questions were: What weight, area and depth of pier should be used? What design would provide the minimum deflection and were there any unexpected modes of failure? An available "field model" caisson 15 m wide by 27.7 m long weighing 1,375 t was tested on the sand surface by Rykswaterstaat Deltadienst and L.G.M. Delft, (a) on the natural sand bed and (b) on the sand after densification by vibroflotation. Centrifugal models were run to predict the performance of these field tests to see if the sand would liquefy.

The centrifugal model tests contributed to the major design decision to dredge a channel 12 m deep and to sink piers, precast onshore, onto sand densified at the base of the channel. The sea bed between piers was located with granular material up to the underside of the gates, forming a sill. The question remained as to the optimum area and weight of the precast piers. Some 50 models were run simulating alternative prototype designs.

Prof. Rowe then presented centrifugal model tests which had been run to simulate offshore gravity oil production platforms in the Ekofisk, Thistle and Frigg Fields in the North Sea where the submerged weight could reach 200,000t. In contrast to the Oosterschelde piers, the horizontal force was almost totally cyclic and could reach ± 80,000 t. Prof. Rowe elaborated that a feature of the North Sea areas was that often one found a layer of softer clay at some 10 m depth. The presence of this layer, the degree of weathering of clay under cyclic loading and the likelihood of the liquefaction of even dense sand were some of the questions which the models had clarified.

Although piled steel jacket structures had long been in use, and were the most common form of offshore platform design, there were still remarkably few field test data to guide field performance predictions. The code guidance was well advanced in respect of limit loads but interest in deflections arising from cyclic loading and associated weakening and softening of clays provided a role for physical models.

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During the short course several other case histories on offshore and coastal engineering works were illustrated by Mr. Ove Eide, Mr. P.W. French and Mr. H. Eilenberg. Mr. Eide spoke on the alternate types of foundation design for the new Memorial Bridge in Thailand. Mr. French described the geotechnical aspects related to the design and construction of a 100 million dollar dry dock project sited in Bangkok clays. The other impressive case history, presented by Mr. Eilenberg, related to the foundations for a cable-stayed bridge across the Chao Phraya river in Bangkok.

Following the five day short course from 7-11 December 1981, an International Symposium on Offshore and Coastal Structures was held from 14-18 December 1981. The first technical lecture of the Symposium was on 'Avoiding Jack-up Rig Foundation Failures' given by Mr. Bramlette McClelland, a former Terzaghi Lecturer. On July 16, 1981, the mobile jack-up drilling unit, Rio Colorado I was being positioned at a site off the coast of Tierra del Fuego, Argentina, when one of its legs suddenly plunged deeper into the sub-bottom, causing the rig to tilt 15 degrees. Subsequent reports indicated physical damage to be minor, and the rig was returned to service. Not so fortunate was the Triton 11, a three-legged jack-up unit that was being installed in the High Island Area off Texas on January 8, 1980. While its foundation was being preloaded, one leg plunged rapidly, causing the other two legs to buckle. Damage has been reported at U.S. $4.8 million. With these opening remarks, Mr. McClelland spoke eloquently on many aspects such as footing configuration, foundation design loads, location and pre-loading, foundation failure modes, interpretation of failure records, site investigations and predictions of footing performance. In conclusion, he remarked that "jack-up mobile drilling units, especially of the footing-supported type, had a poor safety record in comparison to other structures". About one third of the recorded jack-up rig accident experience was associated with foundation problems experienced while moving on or off location. It had been a standard operating practice of the industry, with few exceptions, to move these giant structures onto a new site without specific knowledge of sub-bottom soil types and properties, relying instead on adjustable and extensible foundations, combined with preloading, to adapt to any condition encountered. Further, neither regulatory agencies nor insurance companies had required more than occasional or token departure from this characteristic risk-taking posture.

Following Mr. McClelland, Dr. Elmo Dibiagio gave a comprehensive lecture on the "Methods of Instrumenting Gravity Base Structures for Implementing Structural and Geotechnical Monitoring Programs". This lecture was followed by one from Prof. Peter Rowe, who spoke authoritatively from

his extensive experience on centrifugal model testing in offshore and near-shore works.

Coastal structures are frequently situated in places with erratic and very compressible soil strata, such as delta regions, or in places previously filled up with uncontrolled masses of varying quality. Therefore, there was generally a considerable need for soil improvement to avoid detrimental differential settlements of structures to be erected. A stimulating lecture was given by Prof. Sven Hansbo on "Soil Improvement Techniques in Design of Coastal Structures". Topics covered were soil improvement by means of vertical drains, heavy tamping and explosives, and the less frequent methods such as electro-osmosis, lime columns and sand piles.

In discussing the usefulness of vertical drains, Prof. Hansbo remarked that, in areas with clay deposits or with highly organic sub-soil, long-term settlement usually created great problems in foundation engineering. One way of avoiding this was to use vertical drains in combination with an overload or a vacuum. In relation to vertical drains some important aspects of consolidation theory and its applicability in practice were stressed. Further, problems of drain installation and of checking the rate of consolidation were discussed. When the soil consisted of loose silt or other more pervious materials, heavy tamping (or dynamic consolidation) could be a suitable solution to soil stabilization. In saturated silt and sand, the impact of the falling weight caused excess pore water pressures leading to liquefaction and vertical flow channels. The drainage paths, created by the flow channels and the fissures accelerated the dissipation of excess pore water pressure. In addition, the excess pore pressure had a peak below the point of compaction and, therefore, three-dimensional pore water out-flow took place leading to a fast dissipation of the excess pore pressure. These occurrences explained why it was possible to compact and consolidate even saturated silt in a short time by heavy tamping.

In discussing the principle of the blasting technique, Prof. Hansbo mentioned that the technique could be successful in cases where the soil consisted of loose saturated sand. The best results were obtained when the grain size distribution of the soil fell within a certain limit, the same range being also suitable for densification by the vibro-flotation technique. The basic principle of compaction by blasting was that the detonation of explosives would induce a shock wave in the soil of such a magnitude that it caused a state of liquefaction followed by pore water escape and re-arrangement of grains in a denser packing. The success of the method depended on the ability of the shock wave, induced by blasting, to break down the soil structure. The presence of air or gas in the soil would reduce the efficiency of the method and, therefore, the best result was achieved below the water table.

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Following Prof. Hansbo, Mr. Ove Eide gave a comprehensive lecture on the design and installation of concrete gravity platforms. Building a huge gravity structure with a caisson area of the order of 10,000 m^2 in protected water nearshore, and then towing it out and setting it on the sea floor in 70 to 150 m deep waters of the North Sea was, in many respects, a fascinating undertaking. From a geotechnical engineer's viewpoint it means that the foundation work for a huge structure had to carried out in the course of a few days. The site and soil conditions, installation fundamentals, installation manual, ballasting system, skirt compartment excavation systems, skirt compartment over and under pressure systems, control measurements, data handling and the installation sequences were then described as well as positioning technique, touch down, skirt penetration and base seating during the installation sequences. Many details were provided on underbase grouting, and the various design calculations presented for the installation phase. Included were several design charts and tables which could be most valuable to those who engaged in future gravity platforms works. Mr. Eide was proud to mention that more than thirteen concrete gravity structures had been installed in the North Sea and, in all cases, the Norwegian Geotechnical Institute had been the contractor's geotechnical consultants. In conclusion he mentioned that during the installation of the thirteen platforms no serious problems or delays had occurred. Towing vessels had positioned the platforms with the required tolerances. The dowels used had proved useful in the positioning of the platforms and they were necessary if the skirts were not strong enough to take the touch down forces. Valuable experience had been gained over the years and the design for further platforms with repect to their installation could now be made with greater confidence.

A wide range of grouting applications and techniques had now developed in relation to the installation, strengthening, protection and repair of offshore platforms and pipe-lines. Dr. G.S. Littlejohn, an expert in grouting technology, gave a very fascinating lecture on "The Grouting of Platforms and Pipelines Offshore". Examples quoted included underbase grouting of concrete gravity structures, pile annulus grouting of steel jackets, strengthening of steel legs by grouting, aggregate concrete infilling, sleeve protection of pipes using flexible formwork, saddle weight coating of gas lines, free span support of pipelines and their crossings, and repairs to noded joints. In conclusion Dr. Littlejohn remarked that "in spite of rigorous performance specifications and, bearing in mind the often critical nature of the work, it is encouraging to note that traditional prejudices in grout design have been overcome by the necessity for rapid practical solutions, and thus the use of seawater/cement mixes is widespread and, where appropriate,high alumina cement is exploited".

He also pointed out that "there is, however, a growing requirement for a basic understanding of the behaviour of grouted systems with particular reference to grout properties and boundary conditions as successful performance offshore continues to outpace a detailed knowledge of. load transfer mechanisms".

Four interesting lectures were then given on piled foundations. The first on "Open-end Steel Pipe Piles in Offshore and Nearshore Works" was given by Prof. H. Kishida. The second lecture on the topic of "Prediction of capacity of long piles in clay — a status report" was presented by Mr. John A. Focht, Jr. Prediction of the capacity and performance of deeply penetrating pipe piles is a key element in the design of offshore structures for petroleum production. In was emphasized that, because of the economic impact of pile penetration on foundation costs, the prediction of axial pile capacity required the best technology and judgement. This lecture was a status report on recent developments on procedures for capacity predictions for long piles in clay, with emphasis on piles in normally or near-normally consolidated formations. Both the theoretical developments and their relevance to actual applications in the offshore industry were examined and these were supplemented with the results of four studies on specific components of the capacity prediction process as published recently by staff of McClelland Engineers Inc. Mr. Focht stated that each of the four studies had improved their insight into the performance of axially loaded long piles in clay. He commented that further major improvement in the quality of pile capacity predictions by refining the empirical correlations or effective stress procedures yielding peak stress on a pile soil element appeared to be unlikely, and claimed that the predictive model proposed offered the opportunity to evaluate the effect of loading sequence on pile capacity and performance. In addition, the model propounded was claimed to provide a better understanding of pile performance at working load levels in comparison to performance at loads approaching failure.

A third lecture on the "Displacements and Pore Pressure Development during Pile Driving" was given by Prof. R. Massarsch, who demonstrated that, at present, excessively sized driven piles were used in S.E. Asia for major port developments and other coastal engineering projects. During the course of these lectures, the participants also enjoyed a short presentation by Prof. G. Zeitlen on "Offshore Piling Works and Test Loading", with reference to a project currently in progress close to Israel.

The last lecture dealing with piled foundations was given by Prof. C.P. Wroth under the title "Effective Stress Analysis of Piled Foundations". Prof.

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Wroth gave a very stimulating and thoughtful lecture on this interesting topic which fitted admirably with the preceding presentations of Prof. Kishida, Mr. J. Focht and Prof. Massarsch.

Arising from a shortage of land available for industrial development, there had been recently a rapid development in reclaimed land in the coastal region of many countries in Asia and elsewhere. Prof. H. Aboshi gave a very interresting lecture on the geotechnical aspects related to the "Construction of a Huge Sewage Treatment Station on an Artificial Island". He described the soil stabilization works carried out using pre-compression and sand drains to accelerate consolidation.

The symptoms of scour are commonly associated with cohesionless soils, water movements and the occurrence of an object which disturbs the original balance of forces in the sea-bed. "Experience of different Scour Protection Technologies for Offshore Structures and Offshore Pipelines" was the title of a lecture presented by Prof. B. Maidl, who dealt authoritatively with the installation and performance evaluations of four different scour protection methods, namely:

(1) precast concrete slabs with hinges to the edges of the platform base,
(2) the development of sand-bag clusters in nylon netting,
(3) the use of nylon mats filled with concrete grout, and
(4) the use of artificial seaweed.

It was emphasized that the various scour protection measures had advantages and disadvantages which engineers usually assessed in terms of cost/benefit yields. Rock armour, sand-bags and protective mats could be expensive. However, this in turn could depend on the size of the scheme, the water depth and the severity of the scour-inflicted damage.

Another important feature of the symposium was the comprehensive country reports presented on works in Asia. The general reporters were Mr. S.G. Elliott, Mr. R. Radhakrishnan, Mr. P.D. French, Prof. Kim Sang Kyu, Mr. Hanna Kopenen, Mr. A. Burdal and Mr. Benjamin Chi. These reporters described respectively the major coastal and offshore projects of Hong Kong, Singapore, Thailand, Korea, Vietnam, Indonesia and the Republic of China. Most of the reports were related to coastal engineering and reclamation works and gave a valuable insight into the various geotechnical problems related to coastal and offshore projects in the Asian region.

Mr. O. Kjölseth presented an informative lecture on sea-bed investigations performed for route selection and design of sub-marine pipelines for offshore

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Norway. He described in detail the extent of pipeline surveys performed to date on the Norwegian Shelf, and reviewed the soil composition and sea-bed features of the shelf in the North Sea. Vessel selection, navigation systems (The Motorola Mini Ranger III system, The Cubic Argo DM -54 Pulse-8 system), acoustic instrumentation (Echo-sounders, Side Scan Sonars) and sub-bottom profilers were discussed as were the use of remote-controlled vehicles for submersible inspection in pipeline surveying.

Three lectures given by Prof. E. Lackner, Dr. J. Ringis and Prof. R. Massarsch followed. Prof. Lackner described construction of 'The Quaywall of the Container Terminal in Bremerhavn", while Dr. Ringis gave a comprehensive lecture on the subject "Offshore Geophysical Techniques in Engineering Investigation". Prof. R. Massarsch discussed the use of lime column methods in soil improvement works and the evaluation of dynamic properties of soil.

A number of interesting short presentations were made by Mr. Biserna, Mr. K. Tobe, Dr. C. Galateri, Mr. R. Dahlberg, Dr. J. Boswell, Prof. R. Massarsch, Mr. Eilenberg, Prof. S. Matsuo and Mr. P. Risseeuw. These covered a wide range of topics such as a new boring machine (Mr. Tobe), marine embankments (Dr. Galateri), piles for offshore works (Mr. Biserna), dynamic pile testing (Mr. Dahlberg and Mr. Van Koten), cable-stayed bridge (Mr. Eilenberg), risk analysis (Prof. Matsuo), foundations for light-houses (Prof. Massarsch), scour around offshore structures (Mr. Dahlberg), and grouted piles (Dr. Boswell).

Three additional lectures were given on the last day of the symposium by Dr. E. Dibiagio, Prof. P. Habib and Mr. J. Matich. The lecture by Dr. Dibiagio concerned the instrumentation for installation and long-term performance of gravity based offshore structures.

Up to the present, marine anchors had been studied essentially by seamen. Only very recently had soil engineers become really interested in this problem. Prof. P. Habib gave an interesting lecture on "In-situ and model tests of high capacity anchors". He remarked that "the seaman will not always know the nature of the sea bottom where he will cast anchor, but even if he knew, he would probably not dispose a range of anchors specific for each soil. On the other hand, the geotechnician has little knowledge of wind or tide. His first reaction is to investigate and define the sea bed soil to draw best advantage from its properties. These are, therefore, two worlds unacquainted with each other which have had no opportunity of meeting before the new and difficult problems of soil drilling in the high sea". Anchor characteristics were then

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reviewed and their applications discussed in detail based on extensive study using in-situ and model tests.

The final main lecture of the symposium was on "Offshore Geotechnical Problems Unique to Canada". The lecture was delivered by Mr. Matich, who spoke on the many geotechnical problems offshore engineers were currently solving in Canada.

The numerous lectures presented during the short course and the symposium on a wide range of interesting and relevant topics clearly demonstrated the challenging geotechnical problems which had been solved in the last decade and which had yet to be solved in the next. The session chairmen Mr. J.S. Younger, Dr. Chai Muktabhant, Prof. Yudhbir, Dr. A. Tomiolo, Dr. H. Ohta, Mr. S.G. Elliott, Dr. F. Prinzl, Mr. P.D. French, Dr. M. Wieland, Dr. Prinya Nutalaya, Prof. Vichien Tengamnuay, Dr. H. Orth, Dr. F. Christoph, Dr. Y.S. Lau, Mr. Nibon Rananand and Prof. Pisidhi Karasudhi earned congratulation for their able chairmanship of the various technical lectures.

The short course which was formally opened on the 7th December by Prof. J.H. Jones and subsequently followed by the Symposium till 18th December proved a most fruitful period of deliberation on this important field which affects the development of oil and gas not only in S.E. Asia, but also in many other countries. The symposium and the short course were jointly sponsored by the Southeast Asian Geotechnical Society and the Asian Institute of Technology in collaboration with several other organizations and individuals who have contributed significantly to the advancement of geotechnical engineering.

Casagrande: One of the Great Civil Engineers

Professor Casagrande died on 6th September 1981, aged 79, and with his death our profession lost one of the great civil engineers of this century. He was renowned as a university teacher, as a geotechnical consultant, and for his research work in soil mechanics.

Casagrande was a close associate of Karl Terzaghi as early as 1926 and since then, for more than 50 years, he devoted himself to advancing the science and practice of soil mechanics. On the research side of his activities, he wrote about 70 papers and reports, covering almost every aspect of geotechnical engineering. Many of the papers are contributions of the highest order. As a teacher, Casagrande was unsurpassed. His graduate course at Harvard, which he started in 1932-1933, became the model for others throughout the world and the class lists contain an impressive number of names later to become distiguished in soil mechanics. Rutledge, Shannon, Peck, Carillo, Stanley Wilson, Seed and Sherard are just a few.

In the consulting field, Casagrande was engaged internationally on a wide variety of projects to which he contributed his particular combination of scientific insight, originality and engineering common sense. Problems in building foundations, air fields and slopes all came his way, but during the past 20 years, he tended to concentrate on large earth and rockfill dams, perhaps the most difficult but rewarding aspect of geotechnical engineering, in which he displayed masterly skills.

Casagrande's list of honours is naturally a long one: he was the first recipient of the Karl Terzaghi Award, the first Rankine lecturer, received honorary doctorates from the universities of Vienna and Mexico, and was president of the Boston Society of Civil Engineers to mention but some. It was he who conceived, and personally carried into execution, the idea of organizing the First International Conference and this, of course, led to the formation of the International Society of Soil Mechanics and Foundation Engineering. We owe an incalculable debt to the life's work of this modest but very eminent engineer.

(from Prof. A. W. Skempton — Excerpts from New
Civil Engineer)

Dr. R. H. G. Parry Becomes Secretary General of ISSMFE

Following the term of office of Prof. J.B. Burland as Secretary General, Dr. R.H.G. Parry from Cambridge University recently became the Secretary General of the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). On behalf of the members of SEAGS, we wish to congratulate Dr. Parry on his recent appointment and look forward to having his strong support for the many activities of SEAGS. Thanks are also extended to Prof. J. B. Burland for his support to SEAGS on many matters concerning ISSMFE and other related professional activities.

Seventh Southeast Asian Geotechnical Conference

The Southeast Asian Geotechnical Society (formerly the Southeast Asian Society of Soil Engineering) announces that the Seventh Southeast Asian Geotechnical Conference will be held in Hong Kong from 22nd-26th November 1982 under the joint auspices of the society and the Hong Kong Institution of Engineers. Successful previous conferences in the series were held in Bangkok (1967), Singapore (1970), Hong Kong (1972), Kuala Lumpur (1975), Bangkok (1977) and Taipei (1980). The closing date for submission of abstracts of papers, preferably dealing with practical problems in geotechnical engineering including engineering geology and rock mechanics and of relevance to the Southeast Asian region, was 31st December 1981.

Bulletin No. 1 giving further information was made available in mid-1981 from VII SEAGC, c/o Hong Kong Institution of Engineers, P.O. Box 13987, Hong Kong.

Seventh Asian Regional Conference on Soil Mechanics and Foundation Engineering

Preliminary plans are now underway to hold the Seventh Asian Regional Conference on Soil Mechanics and Foundation Engineering in Haifa, Israel in April 1983. The conference will be hosted by the Israel Society of Soil Mechanics and Foundation Engineering. Initial thinking on the themes of the conference are as follows.

(1) Unsaturated soil behaviour -- Expansive soils and desert soils.
(2) Geotechnical aspects of pavement design and construction.
(3) Water front and offshore structures.
(4) Specialty Techniques -- Slurry walls and piles, anchoring, reinforced earth.

Further information on the conference will be announced in due course.

Fourth Conference of the Road Engineering Association of Asia & Australasia

Preparations are underway to hold the 4th Conference of the Road Engineering Association of Asia & Australasia in Jakarta, Indonesia in August/September 1983. Interested persons are requested to contact :

> Organizing Committee,
> 4th REAAA Conference,
> P.O. Box 106 KBY,
> Jakarta-Selatan,
> Jakarta,
> Indonesia

Eighth European Conference on Soil Mechanics and Foundation Engineering

The Finnish Geotechnical Society (SGF), incorporating the National Society of the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) takes pleasure in extending an invitation to the Eighth European Conference on Soil Mechanics and Foundation Engineering which will take place in Helsinki in May 1983.

The theme of the conference will be 'Improvement of Ground'. The main sessions are entitled as follows.

(1) Improvement of Cohesive Soils

(2) Improvement of Cohesionless Soils

The other nine Specialty Sessions are listed below.

(1) In-situ testing for design and control of ground improvement.

(2) Soil grouting

(3) Deep compaction

(4) Laboratory testing for design and control of ground improvement

(5) Soil re-inforcement

(6) Speeding up consolidation

(7) Improvement of special soils

(8) Soil improvement under sea level

(9) Soil stabilization

The main sessions and the specialty sessions are also supplemented with special lectures on as follows.

(1) The geotechnical design of foundations on improved sub-soil.

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(2) Techno-economic trend of sub-soil improvement methods.

(3) The influence of sub-soil improvement methods in a compact urban milieu.

The technical programme also includes a design and work performance session devoted to :

ground improvement from the view-point of :-

-- Site investigation,

-- Designer, and

-- Contractor

Further information on the conference can be obtained from :-

Secretary General
VIII ECSMFE,
c/o VTT/GEO,
SF-02150 Espoo 15,
Finland

Performance of Dams and Reserviors

The 1982 Autumn Conference on Performance of Dams and Reservoirs will take place from the 9th - 11th September, at Keele University, U.K. The conference is proposed to be an informal technical meeting comprising talks, papers, discussions and visits on the general topic of performance of dams and reservoirs. The technical programme includes the following.

(1) Talk by Dr. A.D.M. Penman, General Reporter for Question No. 55 at 1982 Large Dams Congress.

(2) Spillways and Flood Estimation by K.T. Bass.

(3) Slope Protection by R.M. Shuttler.

(4) Remedial works to puddle clay core by W.J.F. Ray and T. Bulmer.

(5) Instrumentation developments by M.S. Bourne.

(6) Surveillance of an Authority's Reservoirs by F.G. Johnson.

(7) Dam practice—good and bad.

(8) Dam deterioration — a British perspective.

Further information can be obtained from :-

N. J. Tyler,
Secretary, BNCOLD,
Institution of Civil Engineers,
1-7 Great George Street,
London, SWIP 3AA,
U.K.

11th Australian Road Research Roard Conference

The 11th Australian Road Research Board (ARRB) Conference will be held at the University of Melbourne, August 23-27, 1982. Papers are encouraged on all aspects of roads, in particular: road transport, transport data, travel behaviour, freight movement, transport systems management, loads on roads, drainage and drainage maintenance, pavement performance evaluation, road planning and financing, road user information and guidance, road safety, environmental and energy issues.

Prominent speakers include Mr. R. Bridle, Controller of Research and Development and Director, Transport and Road Research Laboratory, U.K; Dr. K. Rumar, Professor and Research Director, Head of Road User and Vehicle Division, National Swedish Road and Traffic Research Institute (VIT); Dr. B.F. McCullough, Director, Center for Transportation Research, University of Texas at Austin, U.S.A. and Mr. C.K. Orski, Vice President, The German Marshal Plan of the United States.

Further information on the conference can be obtained from :-

> Australian Road Research Board,
> P.O. Box 156 (Bag 4),
> Nunawading,
> Victoria 3131,
> Australia

Development of New Energy Sources Delayed by Reliance of Fossil Fuels

The slowing down of development of alternative sources of energy by the excessive reliance placed on fossil fuels during the past century was described as unfortunate by His Excellency Mr. Chia-Kan Yen, former President of the Republic of China. He was delivering the Closing Address at the Third Road Engineering Association of Asia and Australasia Conference in Taipei.   Lt. General Fu-Sheng Chan and Dr. Za-Chieh Moh are to be highly congratulated on the pleasing outcome to their efforts in organizing a very successful road engineering conference.

Mr. Yen said that it was not until the 1973 oil crisis that the future prospects of energy began to receive world-wide attention as a matter of growing concern. If the history of development of alternative sources of energy was any guide for the future, he was optimistic that before the phasing out of the petroleum age, the world would see the successful emergence of another age— an age of alternative sources of energy.

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"However, let us not overlook the extreme importance of two questions that should be of grave concern to the world. One is the need for research and development of alternative sources of energy so as to accelerate the phasing in of the new age. The other is the need for promotion of energy-saving so as to delay the phasing out of the petroleum age", Mr. Yen added.

According to recent polls conducted in a number of countries, more than 70% of the people polled identified unemployment and inflation as economic issues of most concern to the public. Those who regarded energy as the most important issue were in the absolute minority. In terms of the present structure of world energy consumption in 1979, petroleum accounted for 44.8%, or the lion's share of the total, estimated at 7 billion tons of oil equivalent. Coal accounted for 28.4%; natural gas, 18.6%; hydro-power, 5.9% and nuclear power, 2.3%. Furthermore, the consumption of oil relative to total energy consumption exceeded 70% in a number of industrial countries and upwards of 90% of that oil might be imported. "In other words, the energy issue we are all facing really boils down to an oil issue", Mr. Yen said.

General Committee Members of SEAGS

The Governing Committee for the period 1980-1982 is as follows:

> Dr. E.W. Brand (President)
> Dr. W.H. Ting
> Mr. S.G. Elliott
> Dr. Chai Muktabhant
> Prof. J.J. Hung
> Mr. J.S. Younger (Editor)
> Dr. Za-Chieh Moh (Founder President)
> Dr. Tan-Swan Beng (Immediate Past-President)
> Mr. Nibon Rananand
> Prof. Chin Fung Kee (Past-President)
> Prof. Peter Lumb (Past-President)
> Prof. Seng Lip Lee
> Prof. A.S. Balasubramaniam (Secretary-Treasurer)

Membership Fees

Invoices for SEAGS Membership fees for 1981 have already been mailed to all members followed with a second reminder. Some members have still not paid up their dues. All members who have not yet paid their dues up to 1981 are kindly requested to do so as early as possible.

SEAGS Newsletter

All correspondence related to the SEAGS Newsletter should be addressed to:

Prof. A.S. Balasubramaniam,
Division of Geotechnical and 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
	millimetre	mm
	micrometre	μm
Area	square kilometre	km^2
	square metre	m^2
	square millimetre	mm^2
Volume	cubic metre	m^3
	cubic millimetre	mm^3
Mass	tonne	t
	kilogramme	kg
	gramme	g
Density ρ (mass density)	tonne per cubic metre	t/m^3
	kilogramme per cubic metre	kg/m^3
Unit weight γ (weight density)	kilonewton per cubic metre	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-1
	1/kilopascal	kPa-1
Coefficient of consolidation or swelling c_v	square metre per second	m^2/s
	square metre per year	m^2/year
Hydraulic conductivity k (formerly coefficient of permeability)	metre per second	m/s

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