# GEOTECHNICAL
# ENGINEERING

# CONTENTS

# LOAD-DEFORMATION CHARACTERISTICS OF MODEL ANCHORS UNDER HORIZONTAL PULL IN SAND

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## SYNOPSIS

The load-deformation characteristics of vertical anchor plates buried in sand and subjected to horizontal pull were studied through model tests. Four different sizes of anchor plates ranging from 25 cm to 10 cm embedded at various depths were tested. Two different states of packing of sand were used, and anchors were subjected to horizontal pull only. The test results were analysed to study the influence of important factors on anchorage capacity and also displacements at failure. The experimental results were compared with those of other investigators. Anchorage capacities were computed by several methods and were compared with the experimental observations.

## INTRODUCTION

In the Civil Engineering design of structures like bulkheads, suspension bridges, antenna towers, etc., a designer is often faced with the problem of designing foundations that should be able to resist the pullout forces acting on them. These foundations are referred to as "anchors" or "anchorages". Anchorages may be subjected to vertical pullout forces, as in the case of foundations for transmission towers, or to inclined forces, as in the case of anchorages for suspension bridge cables. In anchored bulk-heads, the anchorages are usually subjected to horizontal pullout forces, and in the present study, only anchorages subjected to horizontal pull have been considered

An accurate evaluation of the pullout capacity of an anchor is still a complex problem. The common design practice is based on more or less empirical rules which have been given by various investigators from time to time and included in various codes. Attempts have been made in the past to develop analytical expressions for pullout capacities of anchors subjected to vertical pullout forces (MATSUO, 1967; MEYERHOF & ADAMS, 1968; HEALY, 1971; VESIC, 1971 and HANNA, 1972). However, the anchorage capacity of vertical plates under horizontal pull has not attracted much attention from investigators apart from the work reported by MEYERHOF (1973) and NEELY et al (1973).

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*Discussion* on this paper is open until 1 September, 1978.

Experimental data on horizontal anchors has also been reported by HUECKEL et al (1961, 1965) and DAS (1975). However, uncertainties still exist as to how the anchor tends to move, the degree to which the surface frictional forces may develop, the mass of soil that may be involved in any movement of the anchor, and the true pattern of the potential rupture surface that may develop in the surrounding soil.

To develop a better understanding of the load-deformation characteristics of anchors in sand, the present model study was carried out. The various factors controlling the load-deformation characteristics were investigated. The experimental results are compared with various theories, and conclusions regarding the load-deformation characteristics of anchors are made.

## THEORETICAL ESTIMATES

A review of available literature reveals that, in general, continuous anchors are primarily divided into two categories, shallow and deep (RANJAN, 1974). For shallow anchors, the anchor capacity is given by the difference between the total passive and active earth pressure (TERZAGHI, 1943). For deep anchors, the general belief is that the anchor capacity is equal to the bearing capacity of a footing placed at a depth equal to the depth up to the centre of the anchor.

DOUGLAS (1964) evolved a theory of displacement and rotation of a thin, rigid vertical plate buried in an elastic medium when acted upon by a moment and a horizontal load applied to its upper edge. This mathematical theory is based on Mindlin's equation and is confirmed by model tests. The use of this theory for footings buried in soil was discussed by Douglas with regard to both immediate and total final settlement. He also discussed the movement of anchor plates as a specific case of his general theory. The theory is limited to soil in the elastic state.

NEELY et al (1973) developed theoretical solutions for anchor plates using both limit analysis and plastic theory. They used the equivalent free surface concept (MEYERHOF, 1951) for developing the theoretical solutions. However, the theory proposed by MEYERHOF (1973) is more rational, simple and versatile. The theory covers the case of axially pulled anchors with pull varying from vertical to horizontal. Strip and square anchors in both cohesionless and cohesive soils were considered. According to MEYERHOF (1973) the ultimate load, $Q_u$, of a shallow anchor may be expressed by:

$$Q_u = (c\,K_c\,\frac{D_o}{B} + \gamma\,D_o^2\,K_b/2B) + W Cos\,\alpha \quad\ldots\ldots\ldots (1)$$

where $Q_u$ = ultimate load
     $A$ = area of the anchor base
     $B$ = width of anchor base
     $D_o$ = maximum depth of anchor base
  $K_c, K_b$ = uplift coefficients
     $W$ = weight of anchor and soil mass vertically above the anchor base
     $\alpha$ = load inclination with the vertical
     $c$ = unit cohesion
     $\gamma$ = unit weight of soil

The values of the coefficients $K_b$ & $K_c$ have been given for different values of angle of internal friction, $\phi$, and inclination of load, $\alpha$, for shallow strip anchors.

At critical depths, when local shear failure occurs, the anchor capacity is given by:

$$Q_u = c \,(N_{cu} + \gamma D N_{qu}) \,A + W_a Cos\,\alpha \quad \ldots\ldots\ldots\ (2)$$

where $Q_u$ = ultimate load
     $c$ = unit cohesion
 $N_{cu}, N_{qu}$ = uplift coefficients for strip anchors at great depth
     $D$ = average depth of anchor base
     $W_a$ = anchor weight
     $A$ = area of anchor base
     $\gamma$ = unit weight of soil
     $\alpha$ = load inclination with the vertical

The values of $N_{qu}$ have also been given, whereas $N_{cu}$ can be calculated from the general relationship:

$$N_{cu} = (N_{qu} - 1) \,cot\,\phi \quad \ldots\ldots\ldots\ldots\ (3)$$

Coefficients for square anchors have also been given.

### EXPERIMENTAL PROGRAM

The present experiments were carried out in a box 8 cm wide, with wooden frame and perspex sides. To maintain the rigidity of the perspex walls during load application, angle-iron stiffners were suitably fixed with bolts and butterfly nuts. The model anchor plates used were 8 cm wide and 2 cm thick, with heights of plate, h, of 2.5, 5.0, 7.5, & 10.0 cm (Fig. 1).
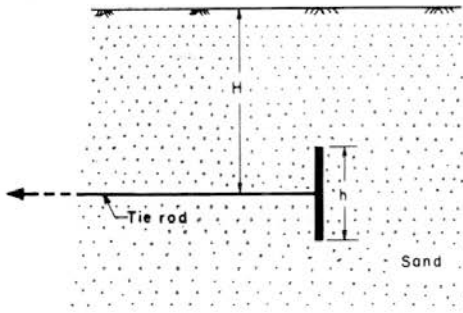
Fig. 1. Geometric parameters of anchor.

The soil used in the investigation was a locally available poorly graded clean dry sand. The properties of the sand are given in Table 1. The sand was compacted in 2.5 cm thick layers by the rainfall method to attain two different relative densities, namely a dense state (relative density, $D_R = 70\%$) and a medium state (relative density, $D_R = 50\%$). A grid using colored sand was marked on the observation face for observing the rupture surface during the test (KAUSHAL, 1973).

### Table 1. Soil properties.

| | |
|---|---|
| Soil type, SP — poorly graded sand with little or no fines | |
| Uniformity coefficient | = 2.08 |
| Effective size, $d_{10}$ | = 0.12 mm |
| Specific gravity of solids, $G$ | = 2.59 |
| Maximum void ratio, $e_{max}$ | = 0.92 |
| Minimum void ratio, $e_{min}$ | = 0.58 |
| Relative density under test conditions: | |
| a) dense state, $D_R$ | = 70% |
| b) loose state, $D_R$ | = 50% |
| Angle of internal friction: | |
| a) dense state, $\phi$ | = 40° |
| b) loose state, $\phi$ | = 33.8° |

The horizontal load on the anchor plate was applied by hooking a wire to the outside end of the tie rod. The wire passed over a pulley fitted with ball bearings. To the other end of the wire, a hanger was attached on which weights were added. The height of the pulley was adjusted so that the top of the pulley was always in line with the tie rod. The load on the anchor plate was applied by adding weights to the hanger. Horizontal displacements of the anchor plate under load were measured by means of a dial gauge of least count 0.01 mm. Figure 2 shows the experimental set-up.

At the beginning of each test, the required height of sand was deposited in the box. The anchor plate with the tie rod in position and supported by threads was then placed on the top of the layer. Sand placement in layers was then continued until the plate was buried at the required depth.
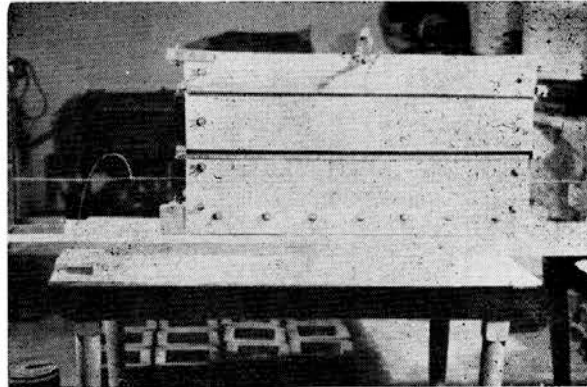
**Fig. 2. Experimental set-up.**

The load on the anchor plate was applied by adding weights to the anchor. Adequate time was given for complete displacement to occur under a given load when the horizontal displacement reading was taken. The next increment of load was then applied. The process was repeated and the load-displacement curve was plotted. Figure 3 shows the load-displacement plot obtained in the test using the 2.5 cm size plate in medium sand.
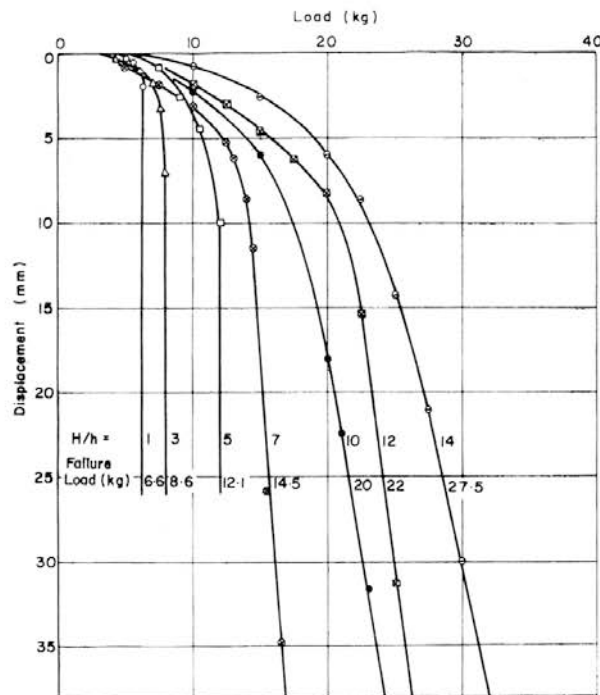


**Fig. 3. Load-displacement curves. Plate size of 2.5 cm, $D_R = 50\%$.**

Failure patterns during the tests were observed. Figure 4 shows a typical failure pattern. The pullout load was obtained from the load-displacement
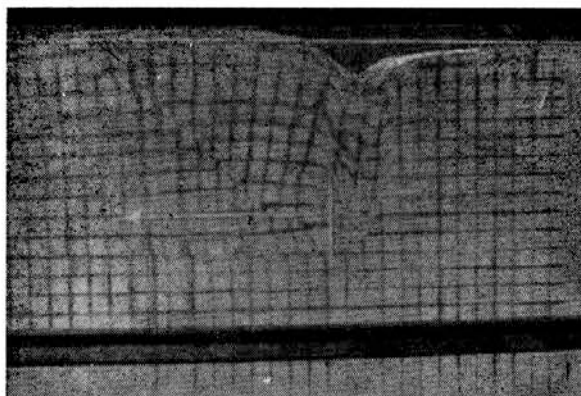
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**Fig. 4. Failure pattern for shallow anchor.** $D_R = 70\%$, $h = 7.5$ cm, $H = 15$ cm.

plot. Pullout load is defined as the load at which the load-displacement curve passes into a steep straight tangent. These have been indicated in Fig. 3. The anchorage capacities, after accounting for the pullout resistance of the connecting rods, are given in Table 2.

**Table 2. Experimental results.**

| $h_{(cm)}$ | 2.5 | | 5.0 | | 7.5 | | 10.0 | |
|---|---|---|---|---|---|---|---|---|
| $\dfrac{H}{h}$ | $P$ (Kg) | $\Delta_u$ (mm) | $P$ (Kg) | $\Delta_u$ (mm) | $P$ (Kg) | $\Delta_u$ (mm) | $P$ (Kg) | $\Delta_u$ (mm) |
| Dense sand = ($D_R$ = 70%) | | | | | | | | |
| 1. | | | | | 12.2 | 5.0 | 17.9 | 8.0 |
| 2. | 4.7 | 2.5 | 9.9 | 5.0 | 29.5 | 12.5 | 46.3 | 20.0 |
| 5. | 12.7 | 10.0 | 42.1 | 15.0 | 96.0 | 25.0 | 161.3 | 40.0 |
| 7. | 17.4 | 12.5 | 62.3 | 25.0 | 125.7 | 32.5 | | |
| 10. | 24.5 | 17.5 | 86.3 | 27.5 | 155.5 | 40.0 | | |
| Medium Sand ($D_R$ = 50%) | | | | | - | - | - | - |
| 1. | 2.4 | 2.0 | 6.0 | 4.0 | 9.5 | 10.0 | 13.8 | 15.0 |
| 3. | 6.0 | 7.5 | 18.5 | 12.5 | 33.8 | 22.5 | 75.7 | 40.0 |
| 5. | 9.1 | 10.0 | 41.1 | 17.5 | 91.0 | 30.0 | 136.5 | 45.0 |
| 7. | 11.3 | 12.0 | 61.0 | 25.0 | 110.5 | 37.5 | | |
| 10. | 16.6 | 17.5 | 68.0 | 32.0 | 125.5 | 40.0 | | |

$P$ = Anchorage capacity (pullout load) of 8 cm wide plates after deducting the resistance offered by the connecting rods.

$\Delta_u$ = Displacement corresponding to the failure load.

## TEST RESULTS AND ANALYSIS

*Anchor Failure Mechanism*

The development of failure surfaces during each stage of loading in a test were carefully examined. The anchor failure mechanism along with the development of failure surface is discussed elsewhere (RANJAN & KAUSHAL, 1975). The observations indicate that the anchor failure mechanism is a

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progressive phenomenon. In shallow anchors the failure surfaces originate from top and bottom edges of the anchor and reach the soil surface. There is a rotational movement from front to rear in the case of deep anchors.

*Anchorage Capacity*

Figure 5 shows the plot between the anchorage capacity and the embedment ratio for different plate sizes in medium sand. The observations indicate that for a given anchor plate size the anchorage capacity increases with the increase in embedment ratio. This possibly is on account of the fact that with larger embedment ratios a larger body of soil is involved which offers greater resistance. The results also indicate that after reaching a certain value of embedment ratio the anchorage capacity remains practically constant. The probable reason for this constant value is that beyond a particular embedment ratio the plate behaves as a deep anchor where the failure is rotational. Once the anchor becomes deep the anchorage capacity remains constant and does not increase with increase in embedment ratio. A similar trend in results was obtained in the case of anchors in dense sand.
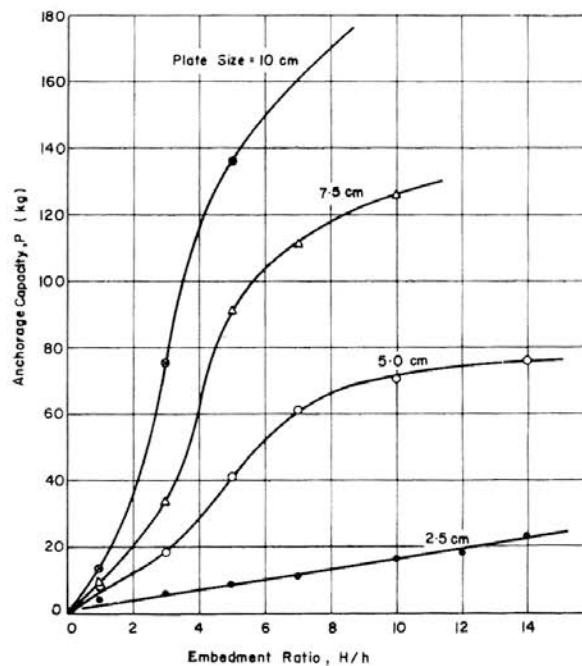


Fig. 5. Anchorage capacity vs embedment ratio, $D_R = 50\%$.

*Displacement at Failure Load*

Figure 6 shows a plot between displacement at failure load and the plate size for different embedment ratios in medium sand. Figure 6 indicates that for
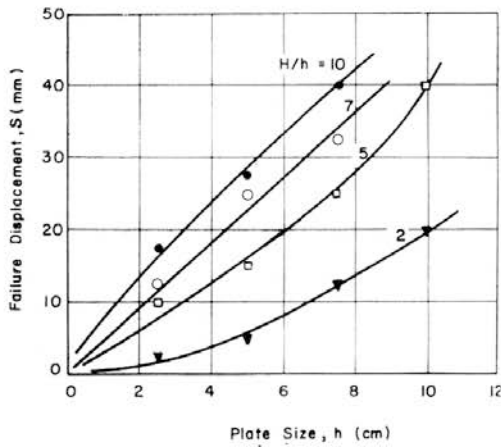
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Fig. 6. **Failure displacement vs plate size,** $D_R = 70\%$.

any particular embedment ratio the failure displacement increases with the increase in the size of the plate. The rate of increase is small and practically linear for anchors at shallow depths. Whereas at deeper depths of embedment the rate of increase in failure displacement for increase in plate size is relatively more. The reason for the relatively large displacement at failure is probably due to the larger soil mass involved in the case of large plates and deeper soil mass involved in anchors at deeper depths. A similar trend was observed in the case of anchors in medium sand.

The displacements at failure of 2.5 cm plate are compared with those of displacements of other plates for the same depths of embedment (Fig. 7). The displacement ratio R is defined as the ratio between the displacement at failure for any size plate and the displacement at failure for a 2.5 cm plate, embedment being the same in the two cases. Figure 7 shows the relation between the displacement ratio and the plate size in dense and medium sand. The figure indicates that for the plate sizes tested the displacement ratio varies in a very small range. It remains practically constant and equal to 1.0 for plate sizes up to about 5 cm and increases slightly for large size plates.
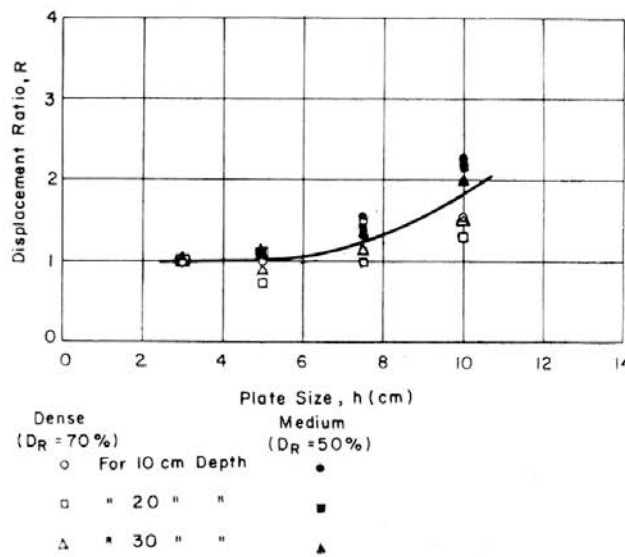


Fig. 7. **Displacement ratio vs plate size.**

*Comparison with Conventional Earth Pressure Theory*

Theoretical concepts of TERZAGHI (1943), TENG (1965) and PARCHER & MEANS (1967) are based on the conventional earth pressure theories according to which the anchor pull, $p_u$, is given by:

$$p_u = \tfrac{1}{2}\,\gamma H_1^2\,(K_p - K_a) \quad \ldots \ldots \ldots \ldots \ldots \quad (4)$$

where  $p_u$ = anchor pull per unit length of anchor plate
$\quad\quad\ \gamma$ = unit weight of soil
$\quad\quad H_1$ = depth of anchor plate
$\quad K_p, K_a$ = earth pressure coefficients

The values of $K_p$ and $K_a$ as per Rankine's theory (TERZAGHI & PECK, 1967) were calculated and compared to the experimental results as shown in Table 3. The experimental results are about 2 to 3 times greater than the theoretical values, indicating that the estimates based on Rankine's theory are conservative.

The results based on the equation proposed by DONOVAN (1961), Eq. 5, are likely to be still more conservative since he has neglected the passive resistance of the soil above the top of the anchor plate.

$$p_u = \frac{\gamma}{2}\,(H_1^2 - H_2^2)\,\frac{(1 + sin\ \phi)}{(1 - sin\ \phi)} \quad \ldots \ldots \ldots \ldots \quad (5)$$

where  $p_u$ = anchor pull per unit length of anchor plate
$\quad\quad\ \gamma$ = unit weight of soil
$\quad H_1, H_2$ = depth of lower and upper edge of anchor plate
$\quad\quad\ \phi$ = angle of internal friction

This discrepancy is probably due to the large $\phi$ versus pressure dependency. From the test results the coefficients of earth resistance have been computed for Hueckel's tests and the present experiment. The computations indicate larger values as compared to Rankine's coefficient.

*Comparison with* HUECKEL'S *(1957) Results*

HUECKEL (1957) carried out tests in sand on square plates of 7.5 cm, 10 cm, and 20 cm size. The soil conditions are similar to those of the medium state of packing under the present study. To compare the results with those of Hueckel, additional tests on 8 cm, 15 cm, and 20 cm plates were carried out. The basic difference between Hueckel's tests and the present tests being that

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Table 3. Comparison of results with Rankine's theory.

| h (cm) | 2.5 | | | 5.0 | | | 7.5 | | | 10.0 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $\frac{H}{h}$ | $P$ (Kg) | p expt (kg/cm) | p theo (kg/cm) | $P$ (Kg) | p expt (kg/cm) | p theo (kg/cm) | $P$ (kg) | p expt (kg/cm) | p theo (kg/cm) | $P$ (kg) | p expt (kg/cm) | $P$ theo (kg/cm) |
| **Dense Sand ($D_R = 70\%$)** | | | | | | | | | | | | |
| 1 | | | | | | | 12.2 | 1.50 | 0.444 | 17.9 | 2.24 | 0.79 |
| 2 | 4.7 | 0.59 | 0.137 | 9.9 | 1.24 | 0.548 | 29.5 | 3.7 | 1.235 | 46.3 | 5.80 | 2.20 |
| 5 | 12.7 | 1.60 | 0.604 | 42.1 | 5.26 | 2.66 | 96.0 | 12.0 | 5.96 | 161.3 | 20.16 | 10.60 |
| 7 | 17.4 | 2.17 | 1.235 | 62.3 | 7.80 | 4.94 | 125.7 | 15.7 | 11.1 | | | |
| 10 | 24.5 | 3.06 | 2.42 | 86.3 | 10.80 | 9.68 | 155.7 | 19.4 | 21.8 | | | |
| **Medium Sand ($D_R = 50\%$)** | | | | | | | | | | | | |
| 1 | 2.4 | 0.30 | 0.035 | 6.0 | 0.75 | 1.140 | 9.5 | 1.2 | 0.32 | 13.8 | 1.70 | 0.563 |
| 3 | 6.0 | 0.75 | 0.192 | 18.5 | 2.41 | 0.765 | 33.8 | 4.2 | 1.73 | 75.7 | 9.50 | 3.06 |
| 5 | 9.1 | 1.13 | 0.473 | 41.1 | 5.12 | 1.89 | 91.1 | 11.2 | 4.25 | 136.5 | 17.0 | 7.55 |
| 7 | 11.3 | 1.47 | 0.88 | 61.0 | 7.60 | 3.52 | 110.5 | 14.0 | 7.90 | | | |
| 10 | 16.6 | 2.07 | 1.73 | 68.0 | 9.0 | 6.90 | 125.5 | 15.7 | 15.5 | | | |

$P$ = Total anchorage capacity (Kg) of plate with height $h$ and width 8 cm.

p expt = Experimental anchorage capacity per unit width of anchor plate.

p theo = Theoretical anchor capacity per unit length according to Rankine's theory.

they were three 'dimensional whereas the present [tests are two dimensional. A comparison of results is made in Table 4. The results are in agreement and indicate an average shape factor of 1.2.

**Table 4. Comparison of results with Hueckel's results.**

| Sl. No. | Hueckel (1957) | | | Present Study | | | | |
|---|---|---|---|---|---|---|---|---|
| | Square Plate Size (cm) | $\dfrac{H_1}{h}$ | Capacity $P$ (Kg) | Corresponding $\dfrac{H}{h}$ | Capacity $P$ (Kg) | Strip Plate Size (cm) | Capacity for Equivalent Area (Kg) | Shape Factor |
| 1. | 7.5 × 7.5 | 2 | 16.42 | 1.5 | 14.5 | 8.0 × 7.5 | 13.6 | 1.210 |
| 2. | 10.0 × 10.0 | 2 | 39.70 | 1.5 | 23.0 | 8.0 × 10.0 | 28.7 | 1.370 |
| 3. | 15.0 × 15.0 | 2 | 122.60 | 1.5 | 54.0 | 8.0 × 15.0 | 101.0 | 1.215 |
| 4. | 20.0 × 20.0 | 2 | 256.30 | 1.5 | 90.0 | 8.0 × 20.0 | 225.0 | 1.140 |

Average = 1.2

*Comparison of Results with* MEYERHOF'S *Theory*

Table 5 shows the comparison of experimental results with the theory proposed by MEYERHOF (1973). Table 5 indicates that the experimental pull out loads in general are higher than the theoretically estimated values.

**Table 5. Comparison with Meyerhof's theory.**

| h (cm) | 2.5 | | 5.0 | | 7.5 | | 10.0 | |
|---|---|---|---|---|---|---|---|---|
| $\dfrac{H}{h}$ | P *expt (kg) | P **theo (kg) | P *expt (kg) | P **theo (kg) | P *expt (kg) | P **theo (kg) | P *expt (kg) | P **theo (kg) |
| Dense Sand ($D_R$ = 70%) | | | | | | | | |
| 1. | | | | | 12.2 | 4.03 | 17.9 | 7.20 |
| 2. | 4.7 | 1.25 | 9.9 | 5.0 | 29.5 | 11.20 | 46.3 | 20.00 |
| 5. | 12.7 | 6.08 | 42.1 | 24.20 | 96.0 | 53.00 | 161.3 | 97.00 |
| 7. | 17.4 | 11.2 | 62.3 | 44.60 | 125.7 | 101.0 | | |
| 10. | 24.5 | 16.0 | 86.3 | 64.0 | 155.5 | 144.0 | | |
| Medium sand ($D_R$ = 50%) | | | | | | | | |
| 1. | 2.4 | 0.27 | 6.0 | 1.05 | 9.5 | 2.4 | 13.8 | 4.26 |
| 3. | 6.0 | 1.45 | 18.5 | 5.8 | 33.8 | 12.9 | 75.7 | 23.2 |
| 5. | 9.1 | 3.48 | 41.1 | 14.3 | 91.0 | 32.3 | 136.5 | 57.5 |
| 7. | 11.3 | 7.70 | 61.0 | 30.8 | 110.5 | 69.5 | | |
| 10. | 16.6 | 11.00 | 68.0 | 44.0 | 125.5 | 99.0 | | |

\* Experimental failure load.
\*\* Theoretical failure load according to Meyerhof's theory.

The conservative theoretical estimates are probably due to the fact that the theoretical computations are primarily based upon the angle of shearing resistance. Since the Mohr Coulomb strength envelope for sand is curved, the angles of mobilized shearing resistance are therefore likely to be different at

various points in any individual failure zone and also different at corresponding points in the failure zones for different sized plates. Thus scale effects are quite important and are likely to influence the results.

NEELY et al (1973) studied the scale effects in anchors. He compared tests on model scale and full scale (18 times larger) in identical sand and reported a 16% reduction in the dimensionless force coefficient, $M_{\gamma q}$ (Eq. 6)

$$M_{\gamma q} = \frac{p}{\gamma h^2} \quad \dots\dots\dots\dots\dots\dots\dots \quad (6)$$

where $M_{\gamma q}$ = force coefficient
$p$ = failure load per unit width of anchor
$\gamma$ = unit weight of sand
$h$ = height of anchor plate

NEELY et al (1973) also reported that for estimation of reduction in magnitude a detailed knowledge of the shear strength of the sand from plane strain tests at a wide range of pressure is required.

*Displacement at Failure*

Figure 8 shows the variation of displacement at failure, $\Delta_u$ expressed as a percentage of height of anchor $h$ versus the depth ratio $H/h$. The figure indicates that with increasing $H/h$ ratio, the failure displacement expressed as per cent of $h$ increases, the rate of increase in failure displacement being smaller at higher values of $H/h$. This possibly is due to the different behaviour of the anchor at deep depths. The results reported by other investigators
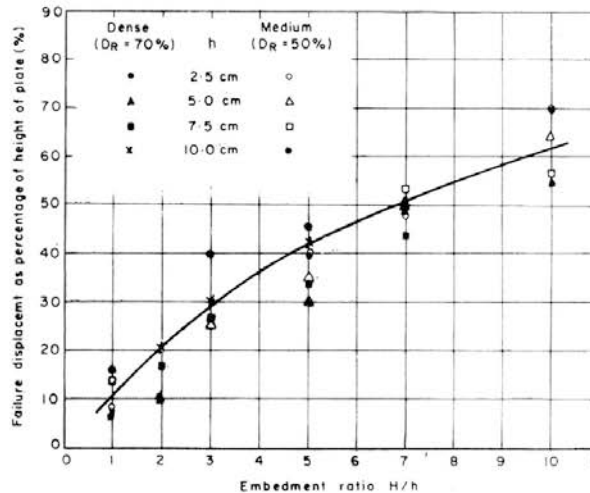


**Fig. 8. Variation of displacement at failure load as % of plate height vs embedment ratio.**

(NEELY et al, 1973) are for square or rectangular anchors and for shallow depths only.

## CONCLUSIONS

On the basis of the present study the following conclusions are drawn:

(1) The anchorage capacity increases with the increase in the plate size and also with the increase in the embedment ratio. The magnitudes are, however, correspondingly less in medium sand as compared to dense sand. The trend of experimental results indicate that the anchorage capacity tends to approach a constant value.

(2) The displacements at failure load increases with the increase in the size of the anchor plate and also with the increase in the depth of embedment.

(3) The displacement ratio '$R$' defined as the ratio between the displacement at failure load of a plate and the displacement at failure load of a 2.5 cm plate, both plates having been placed at the same depth, increases with the increase in the size of plate.

(4) A comparison of experimental results with the theory proposed by Meyerhof indicates that the theoretical estimates in general are lower than the experimental observations. This difference may be on account of variation in estimated angle of shearing resistance used in the analysis.

(5) Comparing the results with the three-dimensional results of HUECKEL (1957), a shape factor of 1.2 has been obtained for the square anchors.

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# DIGITAL COMPUTER SIMULATION MODEL OF AN AQUIFER — A CASE STUDY

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## SYNOPSIS

The Nakhorn Luang aquifer, which is one of the eight aquifers under Metropolitan Bangkok, Thailand, and which is vitally important because it probably supplies more fresh water than any other aquifer, is the case studied in this paper. The study is based on the water level data recorded during the periods of 1968/1969, 1973/1974 and late 1974. The governing equation has been reduced to a steady-state equation and the overall identification of the transmissibility distribution is obtained using a numerical procedure. Resulting values of the transmissibility are used to estimate the discharge rates during the said periods and the increasing trend in the pumping rate from the Nakhorn Luang aquifer is also shown.

## INTRODUCTION

Demand on the fresh water resources caused by the ever increasing world population necessitates exploiting the available ground water resources. The city of Bangkok, being one of the fastest growing capitals of the world with a metropolitan population in excess of three million, has been extracting water from deep aquifers at an ever increasing rate since 1954. In recent years the lack of proper ground water resources management has caused great concern among the city authorities. Undesirable results of excessive pumping, namely salt water encroachment, subsidence of the city, permanent depletion and uneconomic pumping operations are widely feared (BRAND & ARBHABHIRAMA, 1973). Insufficient data on the properties and behaviour of the aquifers make the proper engineering appraisal of the situation a difficult task. It is therefore important that relevant data should be collected and analyzed in order to determine the response of the Bangkok aquifers to "hydrogeologic" stresses.

With a view to making a suitable digital computer simulation model feasible and with the given limitation set by the present data availability, this paper concentrates on identifying the most important parameter in every groundwater study, the transmissibility distribution. There are eight aquifers which

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*Discussion* on this paper is open until 1 September, 1978.

have been identified under Metropolitan Bangkok, namely Bangkok, Prapadaeng, Nakhorn Luang, Nonthaburi, Samkok, Phya Thai, Thon Buri and Paknam aquifers. The aquifer considered for the case study here is the Nakhorn Luang aquifer which is of prime importance because it is the aquifer which supplies the majority of wells in Bangkok at the present time. One of the major drawbacks of every aquifer evaluation is the difficulty of securing enough and accurate values for the recharge and withdrawal rates, the coefficients of transmissibility and of storage, etc., and of defining the physical boundary surrounding the aquifer basin. The same is felt here. Thus, this study depends much on limited available data, on the efficiency of "subjective" approximation and on some trial and error adjustments of parameters guided by mathematical equations.

## NAKHORN LUANG AQUIFER

As seen in Fig. 1, Metropolitan Bangkok and its aquifers are situated in the central part of Thailand, particularly on the Chao Phraya Plain which is
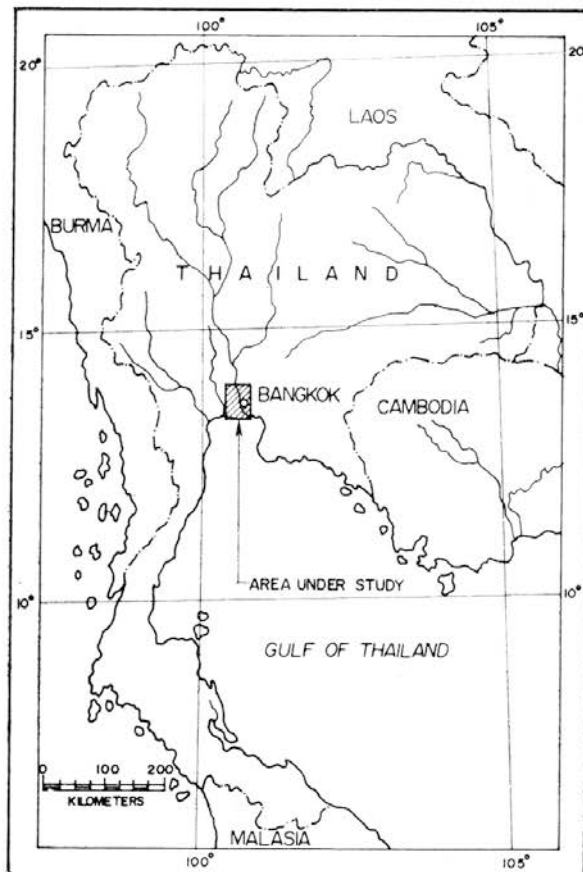


**Fig. 1. Index map of Thailand.**

traversed by the river Chao Phraya. The plain consists of a broad deep basin filled with sedimentary soil deposits which form alternate layers of sand gravel and clay and is about 1 to 1.5 meters above the mean sea level. The profile of the bedrock's face is still not fully determined. However, it is thought to be approximately at a depth of about 550 meters. Known for its unusual complexities, the aquifer system, judging from the light of the available geological, hydrological and geophysical records, has eight aquifers separated by a relatively impervious strata of clay. The North-South hydrologic section of the Chao-Phraya delta is shown in Fig. 2.
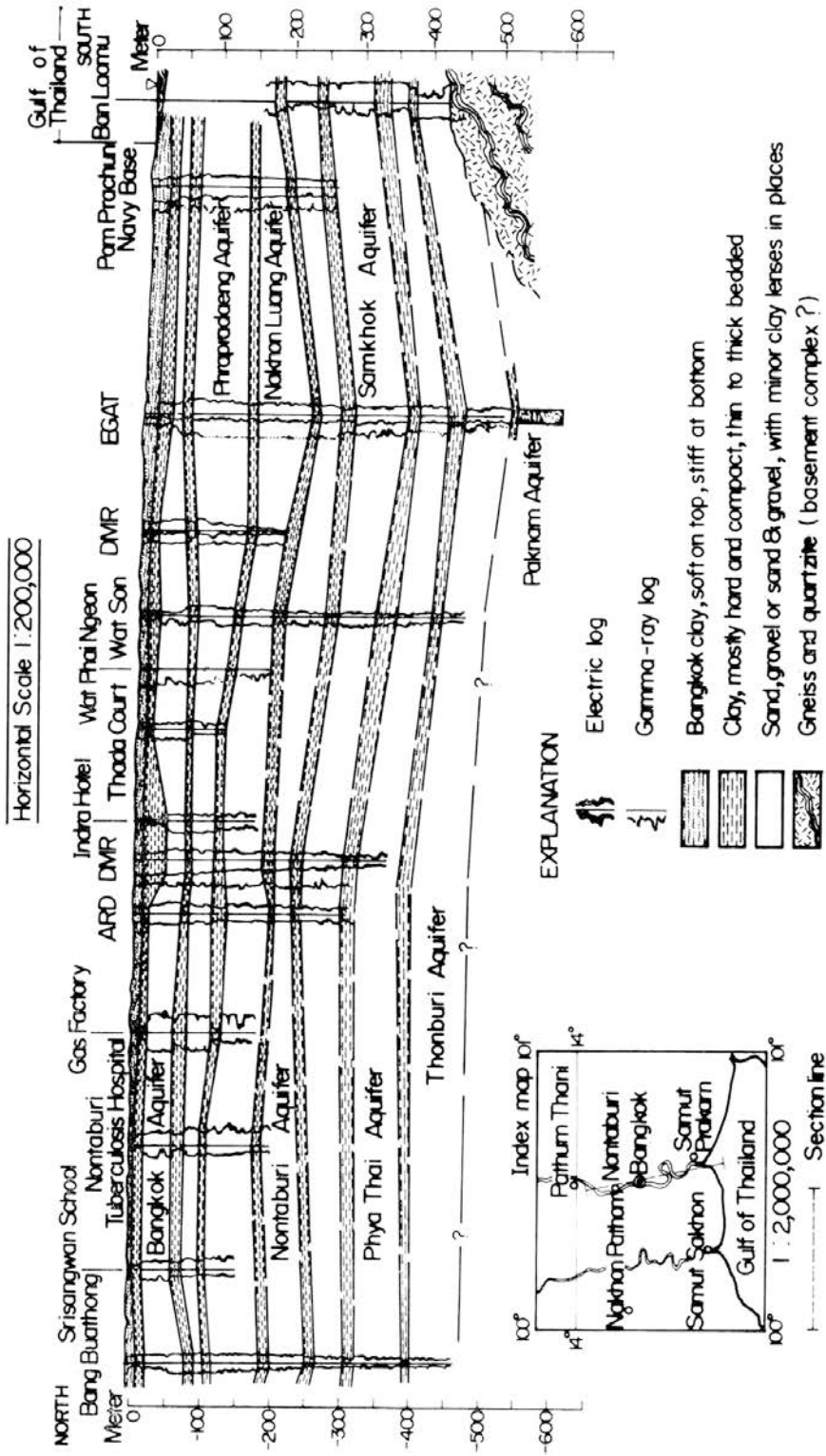
80

Fig. 2. Hydrogeologic north-south of the lower Chao Phraya delta showing principal aquifers.

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Nakhorn Luang aquifer which supplies the majority of the wells in the area at present is of vital importance to Bangkok. Practically all municipal water wells tap water from this aquifer which yields drinking water of excellent quality except in places in the southern region where salt contamination begins as a result of over-pumping. The aquifer has a thickness of 30 meters near the coast and about 100 meters in the Singburi area. In the southern-most part, the aquifer is at a depth greater than 120 meters, but decreases in depth as it goes north and is within 60 meters from the ground surface at In-buri. It is reported (BRAND & ARBHABHIRAMA, 1973) that the Nakhorn Luang aquifer could be roughly subdivided into two sub-aquifers separated from each other by a leaky confined stratum of sandy clay. The aquifer contains a range of grain sizes varying from fine sand to coarse gravel, and small pockets of clay.

As the Nakhorn Luang Aquifer plays a great role in the city's water supply, this aquifer has been subjected to some pumping tests. The results of these tests can be considered to be approximate values only, because of the im-possibility in controlling the pumping from production wells within the tests' vicinity. The following results are the aquifer properties as deduced from the tests. It can be seen later that it is sufficient for the iteration if the relative values of the transmissibility are known, and the absolute values are necessary only for the discharge computations.

Permeability ---------------------------- 170 - 270 ft/day
Transmissibility ----------------------- 11 - 18 $\times$ 10$^4$ gal/day/ft
Storage coefficient ---------------------- 1 $\times$ 10$^{-4}$

On account of the high permeabilities, properly drilled and developed wells in the Bangkok area give discharges in the range of 100 cu.m/hr - 400 cu.m/hr (370 - 1470 gals per minute). Because of the same reason, well interference is not a big problem. Generally, the governing portion of the wells is found under the heart of Bangkok where cones of depression occur. However, well withdrawals within the outskirts of Bangkok may also be very significant, especially those operated by big industries.

Four piezometric maps marked 1958/1959, 1958/1969, 1973/1974 and late 1974 that had been prepared by the Ground-water Division, Department of Mineral Resources, Bangkok (PHIANCHAROEN, 1972) were made available for the present study (Figs. 3a & 3b).

### ITERATION MODEL

The actual phenomena of ground water flow may be so large and compli-cated that to analyze it directly with all its aspects considered may be very
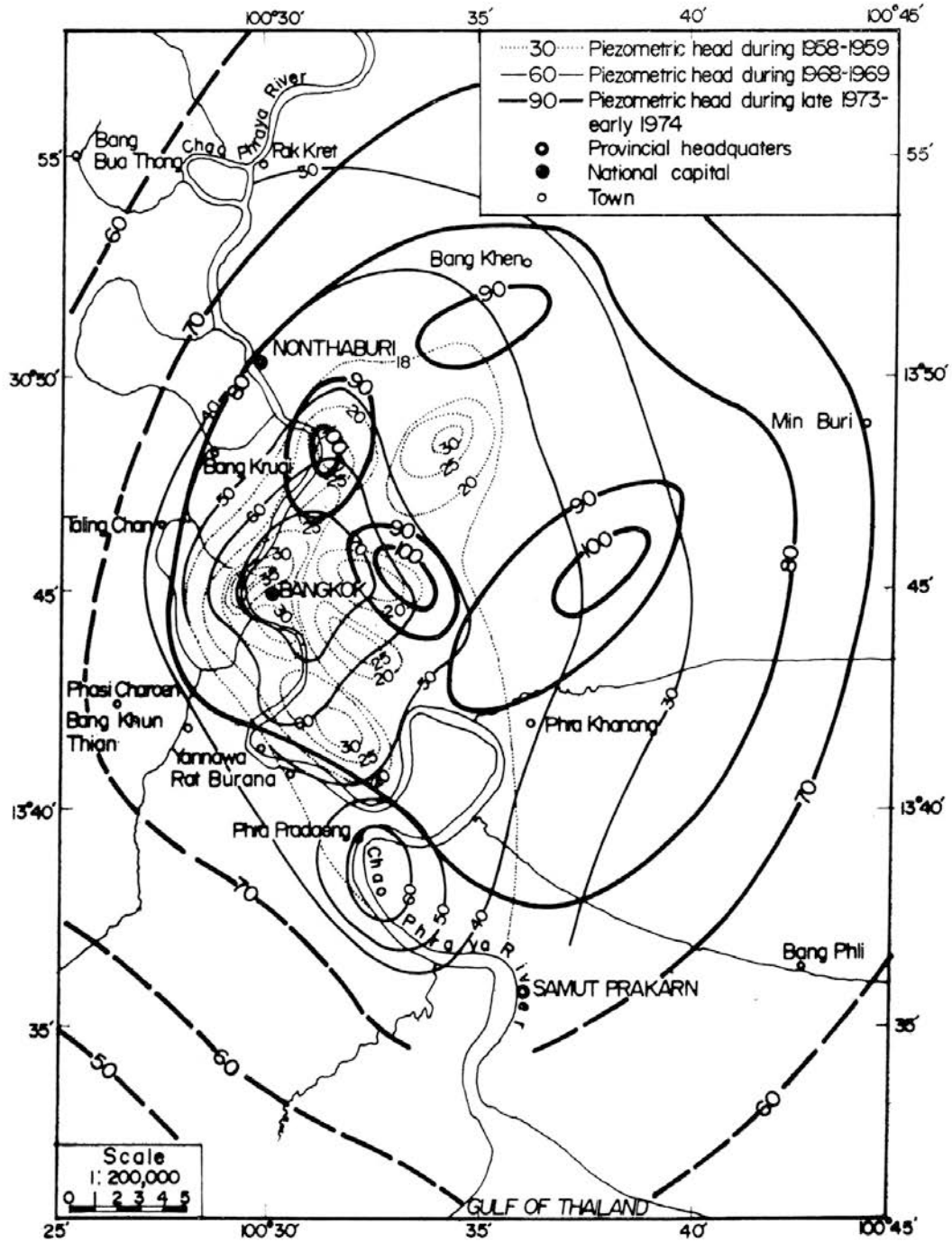
**Fig. 3a. Piezometric map of Nakhorn Luang aquifer in feet below land surface 1958/59, 1968/69 and 1973/74.**
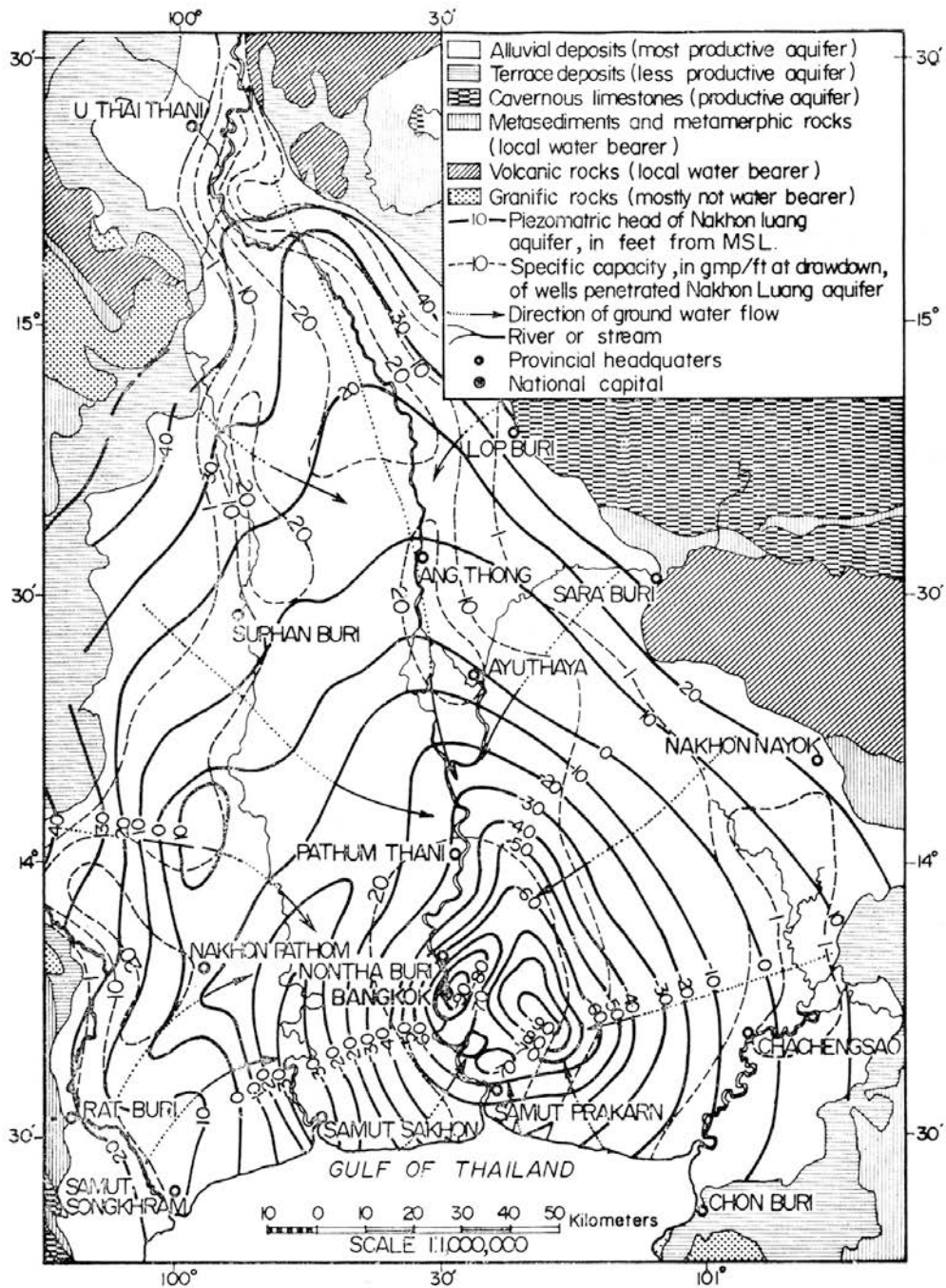
**Fig. 3b. Piezometric map of Nakhorn Luang aquifer in feet from MSL, late 1974.**

tedious. For this reason, there is a need to simplify the case, disregarding nominal details and considering only some significant characteristics, to be integrated into a simple system which is amenable for the iterative solution.

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The general equation describing the unsteady flow through a porous medium is:

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right) = S\frac{\partial h}{\partial t} + Q(x,y,t) \quad \ldots\ldots\ldots (1)$$

where $T_x$ and $T_y$ are the transmissibility coefficients in the $x$ and $y$ directions, $S$, the storage coefficient and $Q(x, y, t)$ is the net effect of recharge and discharge from the aquifer. It may include the man-made aquifer stresses like pumping, artificial recharge and the fluxes from vertical leakage and infiltration.

In order to describe the flow from the above equation it is most important to define first the transmissibility and storage coefficients, and the sink or source function $Q(x, y, t)$ plus the boundary conditions. An inventory of all the wells in Bangkok and its suburbs is necessary to define $Q(x, y, t)$ as a function of spatial coordinates and time. However, the equation may be simplified by neglecting the terms in the right hand side of the equation as suggested by STALLMAN (1956). Stallman reasoned that:

*"The velocity of flow through the aquifer at any position on a flow line equals the sum of the values of $S\frac{\partial h}{\partial t} + Q(x, y, t)$ along the flow line from its origin to the position being considered. If the values of $S\frac{\partial h}{\partial t} + Q(x, y, t)$ are reasonably uniform in space, the velocity past any position far from a groundwater divide is usually large compared with the value of $S\frac{\partial h}{\partial t} + Q(x, y, t)$ at that position. This conclusion is equally applicable to the small finite areas in the flow field. The configuration of the water table is dependent on both the rate at which water flows through the aquifer and the rate of addition to that flow. If the velocity of flow in the zone of saturation is very large compared with $S\frac{\partial h}{\partial t} + Q(x, y, t)$, it is apparent, then $S\frac{\partial h}{\partial t} + Q(x, y, t) = 0$ can be used as an approximation."*

Thus:

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right) = 0 \quad \ldots\ldots\ldots\ldots\ldots (2)$$

and:

$$S\frac{\partial h}{\partial t} + Q(x, y, t) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

If the net accretion rates of an aquifer and the water level changes with respect to a coordinate system and time are known, the transmissibility values and the storage coefficients in their regional trend can be estimated by way of the above equations.

Equation 2 can be written in the form:

$$T_x \, \frac{\partial^2 h}{\partial x^2} + \frac{\partial T_x}{\partial x} \cdot \frac{\partial h}{\partial x} + T_y \cdot \frac{\partial^2 h}{\partial y^2} + \frac{\partial T_y}{\partial y} \cdot \frac{\partial h}{\partial y} = 0$$

and conveniently transformed into their equivalent finite difference form using a square grid:

$$T_x \cdot \frac{h_{i-1, j} - 2h_{ij} + h_{i+1, j}}{\Delta x2} + \frac{\partial T_x}{\partial x} \cdot \frac{h_{i+1, j} - h_{i-1, j}}{2\Delta x} +$$

$$T_y \cdot \frac{h_{i, j-1} - 2h_{ij} + h_{i, j+1}}{\Delta y^2} + \frac{\partial T_y}{\partial y} \cdot \frac{h_{i, j+1} - h_{i, j-1}}{2\Delta y} = 0 \quad \ldots (4)$$

For the square grid $\Delta x = \Delta y$ and using $\dfrac{\partial T_x}{\partial x}$ and $\dfrac{\partial T_y}{\partial y}$ in their forward difference form for the coefficients of $h_{i+1, j}$, $h_{i, j+1}$ and backward difference form for $h_{i-1, j}$, $h_{i, j-1}$, the above equation simplifies to :

$$h_{ij} = \frac{T_{i-1, j} \, h_{i-1, j} + T_{i, j-1} \, h_{i, j-1} + T_{i+1, j} \, h_{i+1, j} + T_{i, j+1} \, h_{i, j+1}}{T_{i+1, j} + T_{i-1, j} + T_{i, j-1} + T_{i, j+1}} \quad \ldots (5)$$

where values of $T$ are the average between the nodes given by the subscripts and the nodes i.j. When the media is homogeneous the equation simplifies to:

$$h_{i, j} = \frac{h_{i-1, j} + h_{i, j-1} + h_{i+1, j} + h_{i, j+1}}{4} \quad \ldots \ldots \ldots (6)$$

Once the transmissibility and drawdown values are generated at all nodes, the total discharge across any equipotential line can be computed using Darcy's law. Since the transmissibility values are computed using the steady state Eq. 2, it is to be expected that the discharges across equipotential lines are generally equal. Thus, this computation would more or less verify the accuracy of the former.

The grid sizes were chosen in such a way that they would not be so small as to make computational cost prohibitive and hydraulic gradient inconsiderable, and not so large as to make the convergence of the solution difficult. The external boundary of the model was assumed to be of no flow by way of placing equal heads between two successive nodes nearest to the boundary. Out of the four available piezometric maps, the map of 1958/1959 was too small to fit a convenient grid system for the other three. Therefore an area of

60 km × 60 km with a square grid system of 1 km was considered in the iterations to determine the transmissibility values and thus the discharges.

With a given piezometric map, the outer boundary and the inner boundaries which were the cones of depression, were identified. Assuming approximate values of transmissibility (the initial assumption was usually that of the homogeneous case) the potential at each node was generated through repeated iteration. The results were then compared to the actual piezometric map. If a discrepancy between the computed and the given exist, a trial and error adjustment of transmissibility values was made. The same process was repeatedly done until a reasonable correspondence between the actual and computed contours resulted.

## TRANSMISSIBILITY DISTRIBUTION

The piezometric map of 1973/1974 was chosen for the adjustment of the transmissibility values using the iteration model described in the above section.

Initially, the equipotential lines 70 ft. and 90 ft. were set as the outer and inner boundary conditions of the region which was believed to satisfy the assumption:

$$S \frac{\partial h}{\partial t} + Q(x, y, t) = 0.$$

With the assumption of homogeneity at first, the potential values of all nodes were generated. They in turn defined the drawdown values (Fig. 4a). Noting the discrepancy between the actual and computed, it was decided to vary the transmissibility values. By assuming first a value which was more or less practically reasonable for the location, the rest were adjusted based upon the first guess. Contours were calculated using the new transmissibility values and compared with the actual. The cycle was repeated until a reasonably possible correspondence between the two contours were obtained. Figure 4b shows the computed and actual contours after a certain degree of practically reasonable similarity was achieved. The resulting transmissibility distribution was concluded to be representing the regional trend of transmissibility variation of Nakhorn Luang aquifer. It must be pointed out that the values of the transmissibility distribution outside the region bounded by the 70 ft. contour are not reliable. At the cones of depression within the 90 ft. closed contours, the steady state Eq. 2 is no more valid and thus the values of transmissibility are indeterminate using the iteration model. On the other hand, the values outside the 70 ft. contour may suffer some errors because of the effect of the external boundary which was assumed to be of no flow. Besides there was no basis to make detailed changes in these regions.
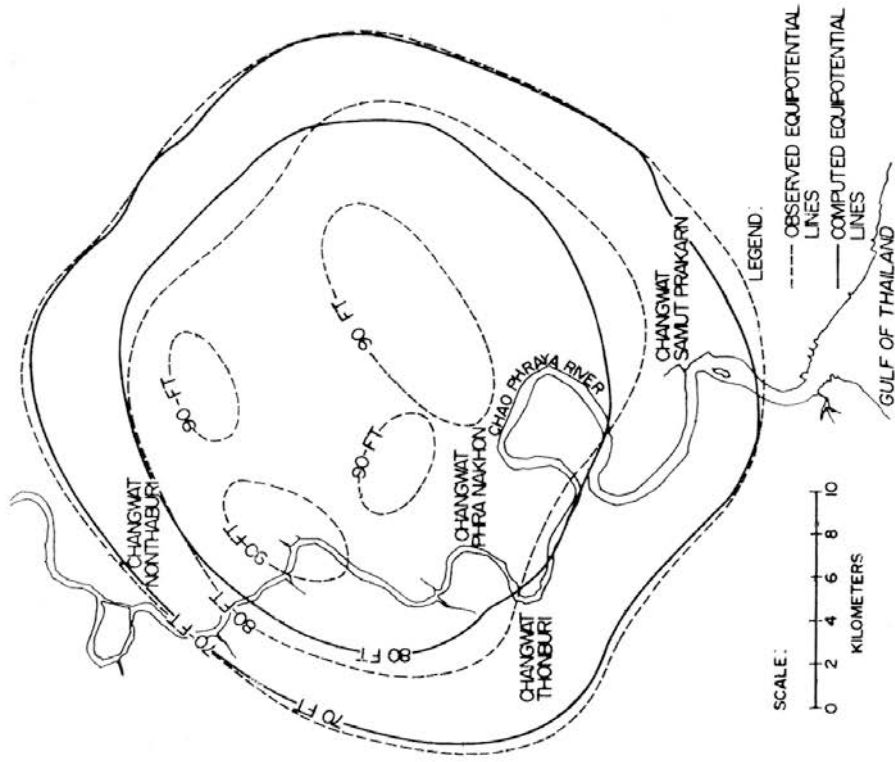
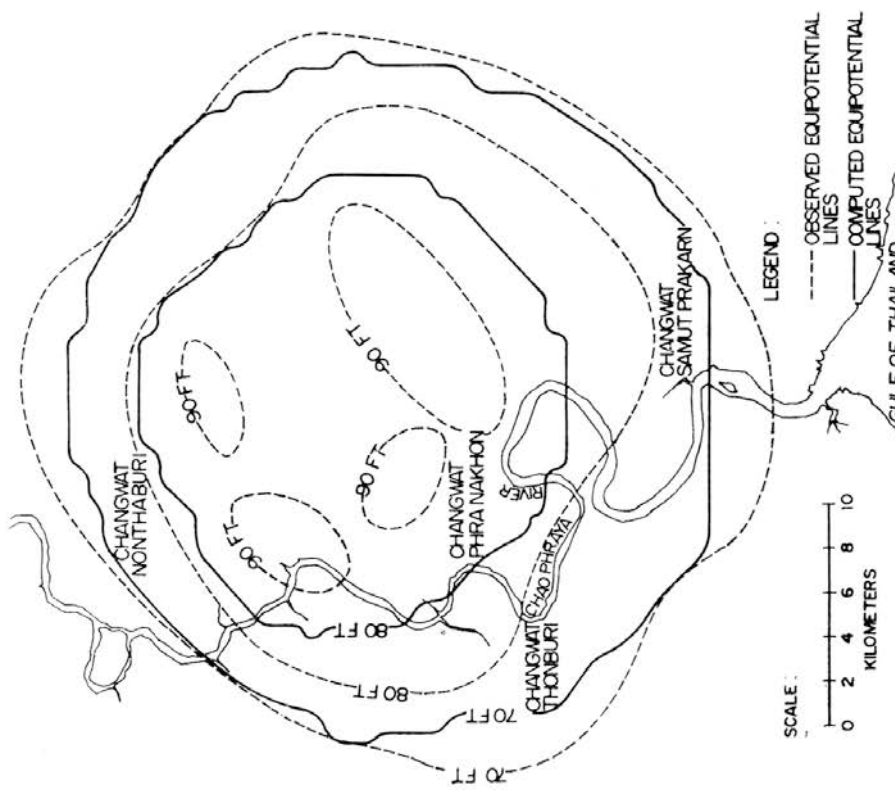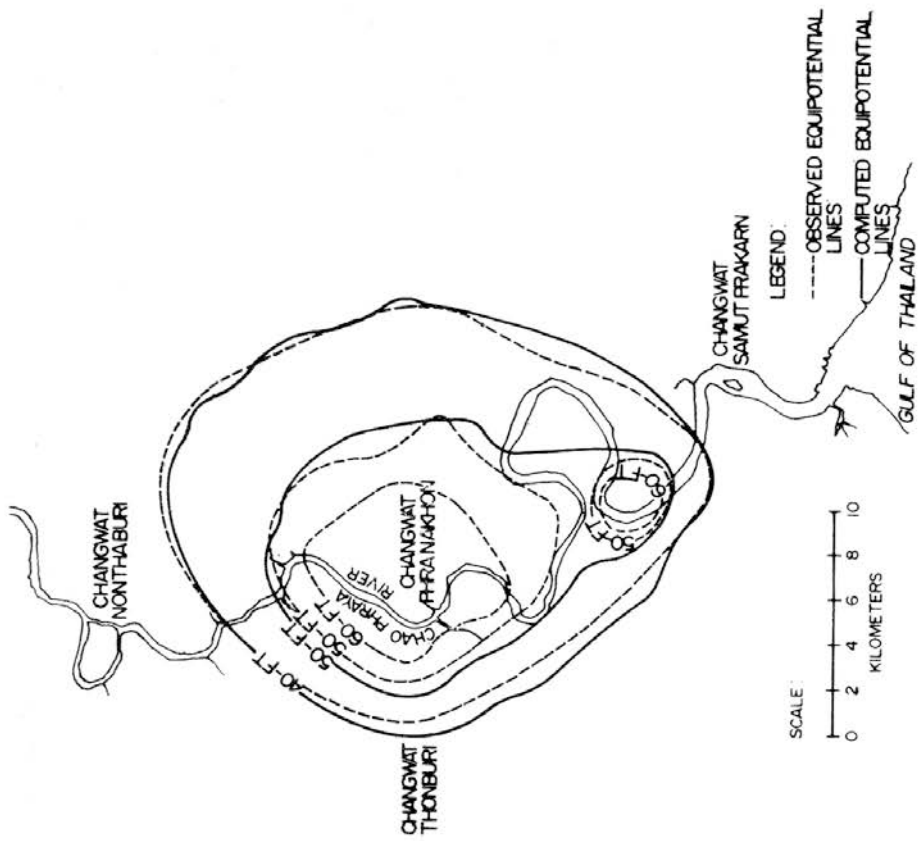Fig. 4a. Piezometric map (1973/74) showing observed and computed equipotential lines (homogeneous scheme).



Fig. 4b. Piezometric map (1973/74) showing observed and computed equipotential lines (inhomgeneous scheme).

Fig. 4d. Piezometric map (late 1974) showing observed and computed equipotential lines (inhomogeneous scheme).



Fig. 4c. Piezometric map (1968/69) showing observed and computed equipotential lines (inhomogeneous scheme).
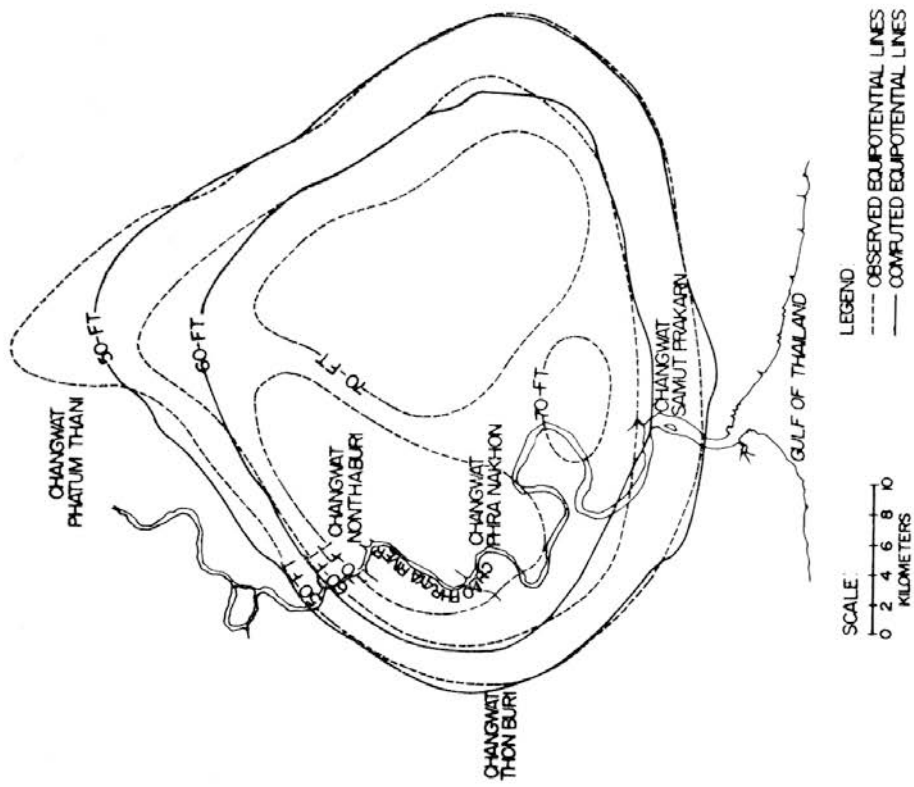
**SCALE**

```
|---|---|---|---|---|
0   2   4   6   8   10
     KILOMETERS
```

---- EQUI-POTENTIAL LINE
——— EQUI-TRANSMISSIBILITY LINE

LEGEND:
1 MEANS 4-5 x 10 E4 GAL/DAY/FT
2 MEANS 6-7 x 10 E4 GAL/DAY/FT
3 MEANS 8-9 x 10 E4 GAL/DAY/FT
4 MEANS 10-11 x 10 E4 GAL/DAY/FT
5 MEANS 12-13 x 10 E4 GAL/DAY/FT
6 MEANS 14-13 x 10 E4 GAL/DAY/FT
7 MEANS 16-17 x 10 E4 GAL/DAY/FT
8 MEANS 18-19 x 10 E4 GAL/DAY/FT
9 MEANS GREATER THAN 19 x 10 E4
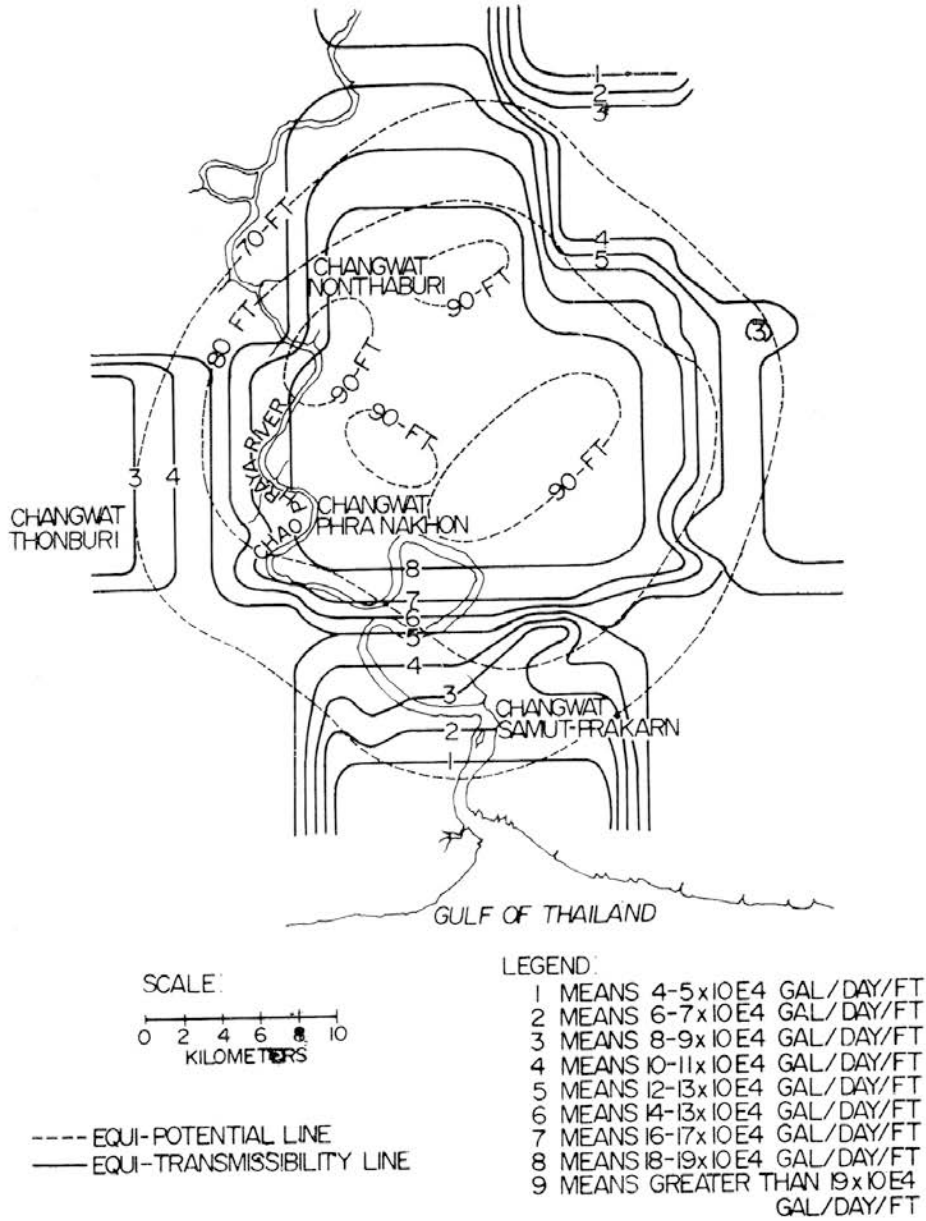                        GAL/DAY/FT

Fig. 5. Transmissibility map of the Nakhorn Luang aquifer.

The set of transmissibility values which had been found by way of the simulated 1973/74 map was applied to that of the 1968/1969 with the boundary conditions set at the 40 ft. and 60 ft. contours. It was found that there was a reasonable similarity between the actual and computed maps as illustrated in Fig. 4c.

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A late 1974 map was the most recent piezometric map available for the present investigation. The same set of transmissibility values was used for the overlapping regions. Many boundary conditions enclosing the domain where the initial assumption was believed to hold, were tried. After several trials the boundaries were set at the 50 ft. and 70 ft. contours. This outer boundary covered a bigger area than the one with the simulation of the 1973/1974, thus it was necessary to make slight changes to the transmissibility values of the non-overlapping regions. Comparison of the computed and observed contours are shown in Fig. 4d. The resulting transmissibility distribution is shown in Fig. 5.

## PUMPING RATES

The total discharges across various equipotential lines were computed for each of the three cases discussed earlier. As the calculations are based on the steady state equation, the discharge values are expected to be the same across different equipotential lines. Within practically reasonable limits, the calculations gave equal discharges. The error was most present for the 1968/69 case, which covered a far smaller region than the other maps. The total discharge during the 1968/1969 period was found to be equal to $20 \times 10^6$ gal/day. For the 1973/74 map, the magnitude of the total discharge near the 90 ft. contour was smaller than the outer ones, which is accredited to its proximity to the region where $Q(x, y, t)$ is not negligible. So the total discharge at the 70 ft. contour equal to $26 \times 10^6$ gal/day is assumed to be more practical because it is far from the region where the withdrawal rates significantly affect the shape of piezometric configuration.

For the late 1974 map, the discharges across the equipotential lines vary from 31 to $35 \times 10^6$ gals per day. The differences are due to the fact that at the region enclosed by the outer boundary where a portion of Pathum Thani province, Thailand, is located, changes in transmissibility did not give correspondence between the actual and computed. Therefore, the initial assumption may not be applicable at the above location. As the above mentioned area is small compared to the entire area, the average of computed discharges equal to $32.5 \times 10^6$ gals per day would well serve as a proper approximation for the discharge during the late 1974 period, and would be adequate enough for some practical purposes such as the measures for the groundwater management of Metropolitan Bangkok.

Figure 6 illustrates the trend with time of the pumping rate from the Nakorn Luang Aquifer. It must be pointed out that the actual magnitudes of the discharges depend on the absolute value of transmissibility values assumed.
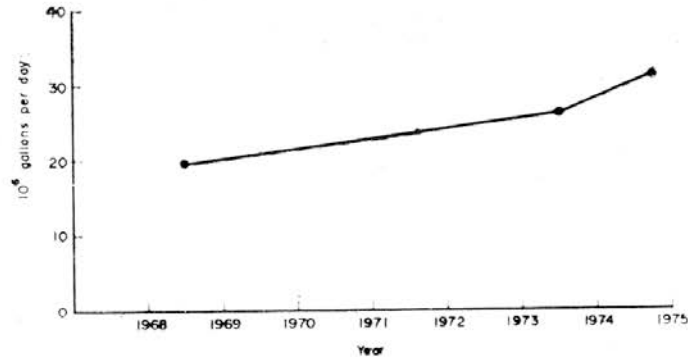
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Fig. 6. Variation of discharge rate with time.

If further accurate pumping tests reveal different values, corrections can be made to the discharge values by multiplying them with the ratio of the new value to the assumed value. However, the trend would remain the same.

## SUMMARY AND DISCUSSION

This study deals with the overall identification of the transmissibility distribution and the determination of the total water withdrawal from the Nakorn Luang Aquifer under the Metropolitan Bangkok during the piezometric observation periods of 1968/1969, 1973/1974 and late 1974.

The first was undertaken by means of the digital computer simulation of the 1973-1974 piezometric map with the help of the governing equation, reduced to its finite difference form and finally into a computer program. The move to achieve this end was simply done by the trial and error procedure. Transmissibility values were assumed and changed, based upon the comparison of the computed with the observed contours. This phase was repeated a sufficient number of times until the two achieved some reasonable correspondence.

After the transmissibility variation was determined with the map of 1973/1974, it was applied to the other two maps, 1968/1969 and late 1974 and subjected to some slight changes. Finally, it was seen that the computed and observed contours were practically similar and therefore the transmissibility distribution was concluded to be reasonably correct at the specified locations.

The deduced transmissibility variation agrees with the physical nature of the Nakhorn Luang Aquifer as shown by the section of the aquifer correlated from electric and Gamma ray logs (Fig. 2). At places near the shore line particularly the outlet of the Chao Phraya River to the sea, the determined

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transmissibility is about 4 to 5 times lower than that under the central Bangkok. The section of the aquifer in Fig. 2 also shows that the thickness near the Gulf of Thailand is smaller than that under central Bangkok. Obviously, the change in the aquifer thickness therefore appreciably affects the regional transmissibility distribution of the Nakhorn Luang aquifer.

Because of the relatively low transmissibility at the sea shore line, the salt water encroachment is comparatively lower despite its being extensively exploited, a fact that makes the Nakhorn Luang aquifer a blessing to the people of Bangkok.

The total discharge computed during the three periods show that the time rate of change of the total withdrawal from 1968/1969 to 1973/1974 is about 1.5 million gals per day per year and for the period 1973/1974 to late 1974 is 3.25 million gals per day per year. Signs of depletion has been noted in the Nakhorn Luang Aquifer as shown in the annual decrease of peizometric surface with an amount of 4 meters in the zone of heaviest pumping. At four more different sites, the total head decline from 1959 to 1968 was in the order of 15 meters (BRAND & ARBHABHIRAMA, 1973). The change of the piezometric surface from the period 1973/1974 to late 1974 is very remarkable. At Changwat Nonthaburi and other places along Chao Phraya River, there is strong evidence of recovery if the piezometric maps of the said periods were correctly made. On the other hand, the depletion tends to progress east of the Chao Phraya River. Whether this phenomenon is caused by the response of the Nakhorn Luang aquifer to the potential of the adjacent aquifer which, in turn, might be directly affected by the Chao Phraya river's infiltration, or by the change in position of pumping (caused by the abandoning of wells), remains to be seen through further field investigations.

Transmissibility values found in the mentioned method are most likely valid only to the regions where $S\frac{\partial h}{\partial t} + Q\ (x,\ y,\ t)$ is negligible and/or is reasonably distributed uniformly in space. The study indicates that the steady state Eq. 2 is valid mainly in the regions not too far from and not too close to the cones of depression. Deviations of the computed contours in the upper region of Fig. 4d and close to the cones of depression in Figs. 4b, 4c and 4d seem to support the above statement. Thus it may be concluded that the steady state Eq. 2 is not applicable in the regions where the velocity of flow (left hand side of Eq. 1) is small or where the term $S\frac{\partial h}{\partial t} +$ $Q\ (x, y, t)$ is large. This is in agreement with the basis for the assumptions of STALLMAN (1956). Comparison of the apparent transmissibility distribu-

tion obtained in this study with those resulting from future field investigations would help to confirm the above conclusions.

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# STABILIZATION OF HEAVY CLAY WITH POTASSIUM CHLORIDE

SAM FRYDMAN*, ISRAELA RAVINA** and TUVIA EHRENREICH+

## SYNOPSIS

Some basic considerations of clay mineralogy suggest that potassium chloride, which is readily available in Israel from the Dead Sea, may significantly improve the engineering properties of heavy clay.

This paper describes the results of a laboratory investigation into the effect of potassium chloride on the engineering properties of an Israeli heavy clay. The investigation indicated that the addition of potassium chloride results in a significant decrease in the activity of the clay, and apparently in a change in its mineral structure. The overall stabilizing effect appears to be a result of this mineralogical change together with the increase of electrolyte concentration.

## INTRODUCTION

Construction on a clay subgrade often requires alteration of the engineering properties of the upper region of the profile. Such alteration may be achieved by replacement of the soil, or by changing its properties by one of many possible methods of soil stabilization. Soil stabilization may be employed for several reasons:

(1) To create a relatively cheap pavement subbase, base or surface course. This use of stabilization is becoming more important since the availability of naturally occurring pavement materials is diminishing, and with the increasing cost of fuel the price of haulage of such materials over large distances is becoming prohibitive.

(2) To create an inactive impervious soil carpet immediately below the road, or foundation structure.

(3) To construct a working mat and/or access roads in order to enable work to proceed in wet periods when mobility of construction equipment on the natural soil is not possible.

Considerable research has been done on the subject of soil stabilization and excellent summaries of the present day state of knowledge are available

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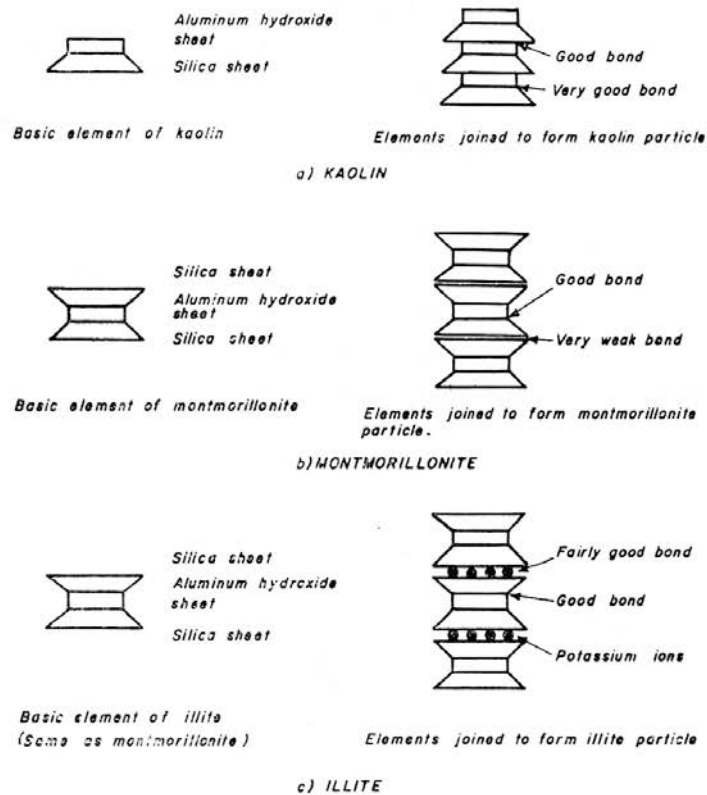*Discussion* on this paper is open until 1 September, 1978.

(INGLES & METCALF, 1972; WINTERKORN, 1975). The work described in this paper deals with the method of stabilization known as chemical stabilization—the alteration of the properties of the soil achieved by addition of an additive causing a chemical reaction within the soil-additive system. The most common methods of chemical stabilization employed for the stabilization of clay soils in pavement work are cement and lime stabilization; these processes produce a stabilized layer of significant strength and stability which may be incorporated into the pavement structure. The high strengths obtained from cement and lime stabilization may not always be required, however, and there is justification for seeking a cheaper additive which may be used to satisfactorily alter the soil properties in such cases.

This paper describes an investigation into the effect of the addition of potassium chloride (KCl) to a heavy Israeli clay on the engineering properties of the clay. KCl, which has not been widely investigated as a potential stabilizer is readily available in Israel at the Dead Sea. Furthermore, some basic considerations of clay mineralogy suggest that this additive may significantly improve the properties of heavy clay, and it is on this basis that the investigation was performed.

## CLAY MINERALOGY CONSIDERATIONS

The clay minerals, which are basically hydrated aluminium silicates in a crystalline form, may be divided into three major groups—the kaolinite group, the montmorillonite group and the illite group. All of these minerals are constructed essentially from two basic building blocks—the silica tetrahedron and the aluminium hydroxide octahedron. A description of the structure of the clay minerals has been presented by GRIM (1953) and briefly by SCOTT (1963), and need not be repeated here. For purposes of the following discussion, however, Fig. 1 is included to show, schematically the structure of particles of three groups of clay minerals. It is significant that the bonds between the elementary sheets of the montmorillonite particle are extremely weak, and water molecules can enter between these layers. It is for this reason that montmorillonite soils show high activity, absorbing large quantities of water and swelling when wet, and alternately exhibiting large shrinkage when dried. The illite particle consists of essentially the same structural elements as montmorillonite but it contains a relatively large number of potassium ions between the adjacent elementary sheets. The potassium ions bond the sheets more firmly than is the case in montmorillonite, and therefore illite swells less in the presence of water than does montmorillonite.

Aluminum hydroxide
sheet
Silica sheet

Good bond
Very good bond

*Basic element of kaolin*

*Elements joined to form kaolin particle*

*a) KAOLIN*

Silica sheet
Aluminum hydroxide
sheet
Silica sheet

Good bond
Very weak bond

*Basic element of montmorillonite*

*Elements joined to form montmorillonite particle.*

*b) MONTMORILLONITE*

Silica sheet
Aluminum hydroxide
sheet
Silica sheet

Fairly good bond
Good bond
Potassium ions

*Basic element of illite
(Same as montmorillonite)*

*Elements joined to form illite particle*

*c) ILLITE*

**Fig. 1. Schematic representation of clay mineral structure.**

The possibility of illite forming from pre-existing montmorillonite was discussed by KELLER (1964), who presented a field example of a soil profile north of Uravan, Colorado, where the originally montmorillonite clay had been converted to illite, presumable by the addition of potassium contained in evaporites which have risen through salt-cored anticlines in the vicinity.

The prospect of being able to convert the highly problematic montmorillonite to relatively inactive illite by the addition of potassium is a most attractive one. Farmers have been aware of the fact for many years that potassium ions added to soils are fixed to the soil particles; however, the form of the change which occurs in the clay mineral is not clear. A limited amount of experimental data has been published relating to the influence of addition of potassium on the engineering properties of clays.

KATTI & BARVE (1962) reported the effect of addition of various salts on the consistency limits of Indian black cotton soil. For additions of between 2% - 10% by weight of $CaCl_2$, $BaCl_2$, $MgCl_2$, NaCl and KCl, it was found that the KCl led to a lowering of the activity of the montmorillonite clay far in excess of that achieved with the other additives.

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WINTERKORN & MOORMAN (1941) performed a variety of tests, including consistency limits, consolidation, permeability and shear strength on samples of heavy Putman clay which had been made monoionic by thorough washing of the natural clay with salt solutions. The samples were made monionic with the following exchangeable cations: $H^+$, $Na^+$, $K^+$, $Mg^{++}$, $Ca^{++}$, $Al^{++}$. All the tests performed indicated that the $K^+$ soil was the most stable and least active of the specimens tested. It has been pointed out by FRYDMAN et al (1971) that according to double layer theories, clays having the monovalent $K^+$ as the exchangeable cation would be expected to be more active than those with divalent (eg $Ca^{++}$) ions; the results of the above mentioned investigations would therefore tend to indicate some form of crystallographic changes in $K^+$ montmorillonite, although no discussion to this effect was undertaken by the authors of the respective papers.

## THE MATERIALS TESTED

Israeli clays are mostly derived from sedimentary rock (dolomite, limestone, marl or calcareous shale), and are predominantly $Ca^{++}$ rich montmorillonites, with high plasticity. The properties of these clays have been discussed by KASSIFF et al (1969). The clay studied in this investigation was from Ramat David, near Haifa, and has been found to have the following approximate mineral composition: montimorillonite 40%, quartz 20%, calcite 10%, feldspar 5%, kaolinite 5-10%.

The proportions of the various exchangeable cations are given in Table 1, and it is seen that the clay is a calcium clay.

**Table 1. Exchangeable cations, Ramat David Clay.**

| Ion | Meq per 100 gram of soil |
|---|---|
| $K^+$ | 0.9 |
| $Na^{++}$ | 2.5 |
| $Ca^{++}$ | 28.8 |
| $Mg^{++}$ | 7.9 |

The consistency limits and mechanical analysis are given in Table 2. The clay is defined as CH according to the Unified Classification System, plotting just above the A-line.

The KCl was obtained from the Dead Sea Chemical Works where it is obtained as a by-product of the potash industry.

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**Table 2. Properties of Ramat David Clay.**

| Property | Value |
|---|---|
| Liquid Limit (%) | 65 |
| Plastic Limit (%) | 22 |
| Plasticity Index (%) | 43 |
| Shrinkage Limit (%) | 9 |
| Per cent sand size | 3 |
| Per cent silt size | 22 |
| Per cent clay size (less than 0.005 mm) | 75 |
| Specific gravity | 2.74 |

## TEST RESULTS

*Consistency Limits*

The consistency limits of the clay stabilized with between 0-9% KCl were tested at varying mixing moisture contents and after varying curing times. Curing was carried out by covering the mixture with damp sacks. The results of these tests are given in Table 3.

Hydrometer tests were performed on a number of the stabilized materials and the resulting mechanical analyses are also included in the table. It is seen that curing time and mixing moisture content do not appear to have a major influence on the stabilization mechanism. Addition of 2.2% KCl had a small effect on the properties of the clay, while additions of 5.5% and 8.8% significantly altered the properties. In the case of the addition of 8.8% KCl, there appears to be a preparation effect. From Fig. 2, it seen that the classification of the soil is altered from CH at 0% and 2.2% additions to ML with 5.5% and 8.8% additions. The mechanical analysis tests indicate about 75% clay sized particles with no stabilizer, about 66% with 2.2% KCl, about 50% with 5.5% KCl and about 10-15% with 8.8% KCl; with 5.5% or more KCl, the soil is predominantly of silt size.

*California Bearing Ratio (CBR)*

The *CBR* of subgrade soil, which is commonly used in the design of flexible pavements in Israel is specified following soaking of a compacted soil cylinder during a 4 day period. Perhaps of significance not less than that of the *CBR* value itself in the case of swelling clays, is the value of the per cent swell mea-

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sured during soaking under a vertical confining pressure equivalent to the pressure which the pavement layers will apply (0.025 kg/cm$^2$ is commonly employed).

Table 3. Consistency limits and mechanical analysis.

| Per cent KCl | Mixing Conditions | Curing Time between Mixing & Test | Consistency Limits % | | | Mechanical Analysis | | |
|---|---|---|---|---|---|---|---|---|
| | | | L.L. | P.L. | P.I. | Sand % >.074 mm | Silt % .074-.005 mm | Clay % <.005 mm |
| 0 | Air dried soil (10% moisture content) | 24 hours | 65 | 22 | 43 | 3 | 22 | 75 |
| 2.2 | Air dried soil | 24 hours | 64 | 25 | 39 | 2 | 34 | 64 |
| | Air dried soil | 48 hours | 65 | 28 | 37 | | | |
| | Moisture content = 13.3% | 0 | 65 | 28 | 37 | 2 | 32 | 66 |
| | Moisture content = 13.3% | 24 hours | 65 | 27 | 38 | 2 | 28 | 70 |
| 5.5 | Air dried soil | 24 hours | 49 | 33 | 16 | | | |
| | Moisture content = 13.7% | 0 | 50 | 33 | 17 | 2 | 46 | 52 |
| | Moisture content = 13.7% | 24 hours | 48 | 32 | 16 | 2 | 53 | 45 |
| 8.8 | Air dried soil | 48 hours | 49 | 33 | 16 | 3 | 80 | 17 |
| | Moisture content = 15.3% | 0 | 44 | 30 | 14 | 3 | 87 | 10 |
| | Moisture content = 15.3% | 24 hours | 40 | 32 | 8 | 3 | 86 | 11 |
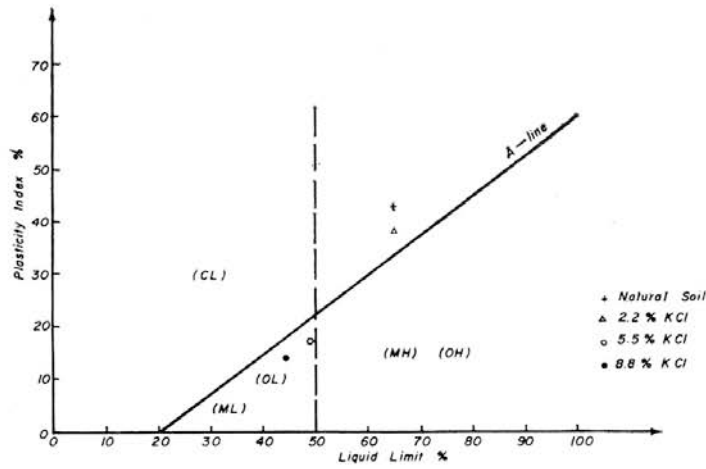
**Fig. 2. Consistency limits of natural and stabilized soil.**

Figs. 3-5 show the results of the *CBR* tests. It is seen that an addition of 5.5% KCl increases the maximum density of the soil from about 1.66 t/m³ to 1.74 t/m³, and the maximum *CBR* from 2% to 17.5%, and decreases the swell during soaking as shown in Fig. 4. At 20% moisture content (approximately the optimum moisture content for the natural soil) the percentage swell under an overburden pressure of 0.025 kg/cm² decreased from 10.5% in the natural soil to 1% in the soil stabilized with 5.5% KCl. No improvement in the soil properties as indicated by the *CBR* test resulted from increasing the KCl content from 5.5% to 8.8%.

The stabilizing effect of KCl noted above could be a result simply of increased electrolyte concentration and its effect on the double layer. In order to check this, a parallel series of *CBR* tests were performed on the clay stabilized with $CaCl_2$ and NaCl; according to double layer theory it would be expected that the $Ca^{++}$ soil would be stronger and more stable than the $K^+$ soil, and that this latter should behave similarly to the $Na^+$ soil. The results of the *CBR* tests performed on $CaCl_2$ and NaCl stabilized clay are shown in Figs. 6-8.

It should be noted that for the same concentration by dry weight of $CaCl_2$, NaCl and KCl, the concentration, in terms of milliequivalents of the first two additives is higher than that of KCl, and consequently they would be expected, on the basis of double layer theory, to have a more pronounced stabilizing effect. Comparing the results in figs. 3-5 and 6-8, it is seen KCl has a much more beneficial effect on the properties of the clay than do $CaCl_2$ or NaCl.
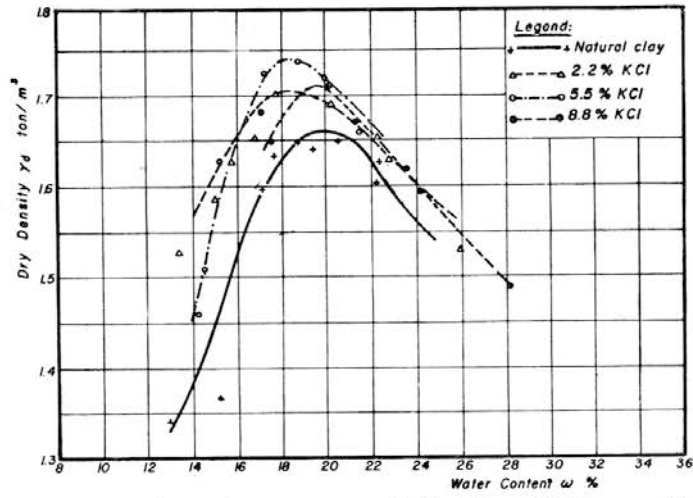
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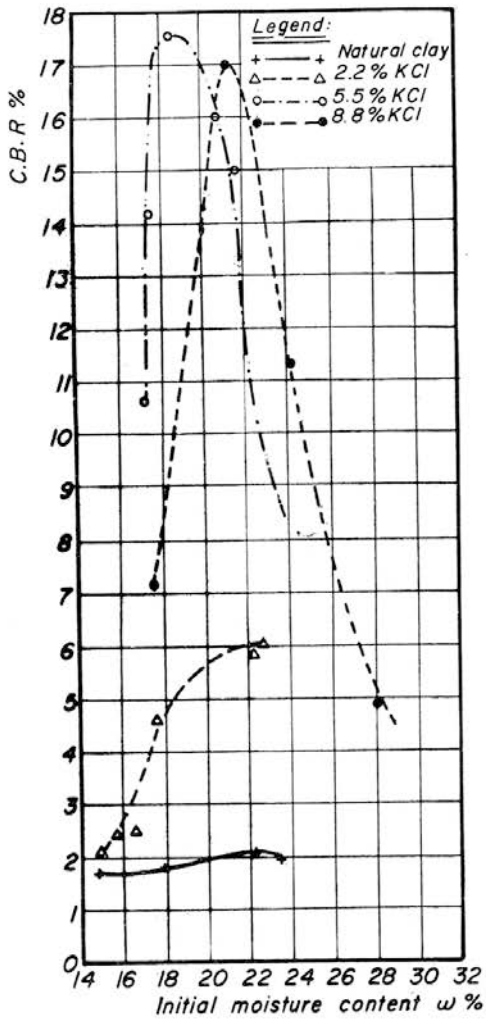
Fig. 3. Density-moisture content relationship, *CBR* compaction.
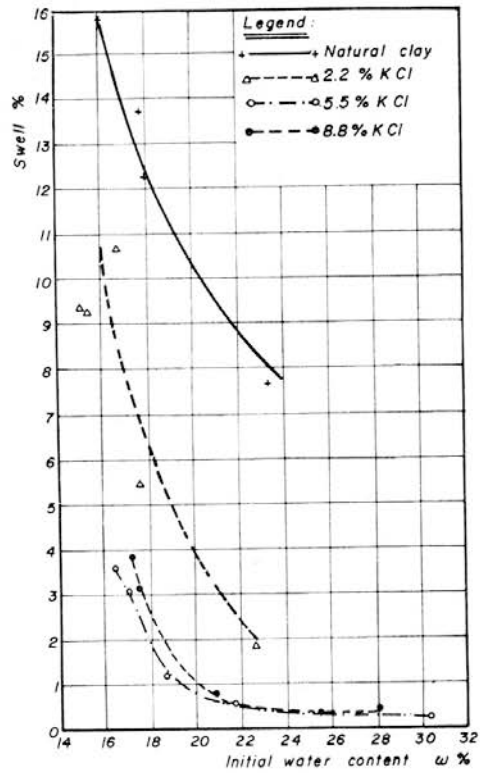


Fig. 4. *CBR* versus preparation moisture content.



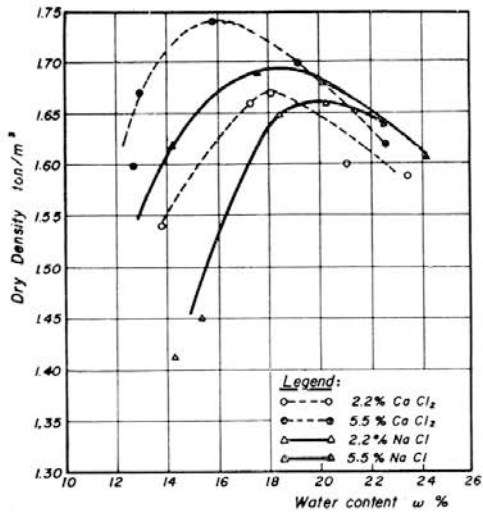Fig. 5. Per cent swell versus preparation moisture content, *CBR* specimens.

Fig. 6. Density-moisture content relationship
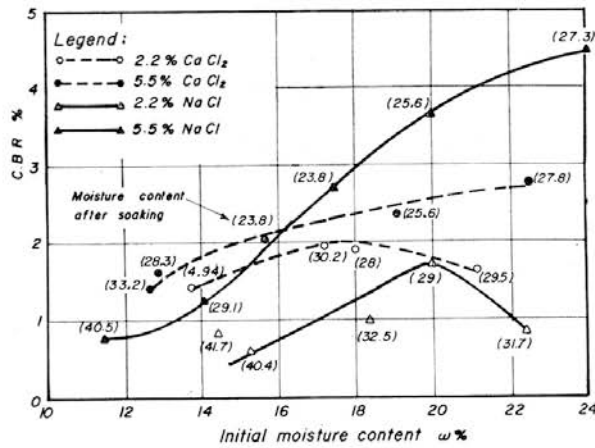with addition of NaCl, CaCl$_2$.



Fig. 7. *CBR* versus preparation moisture content
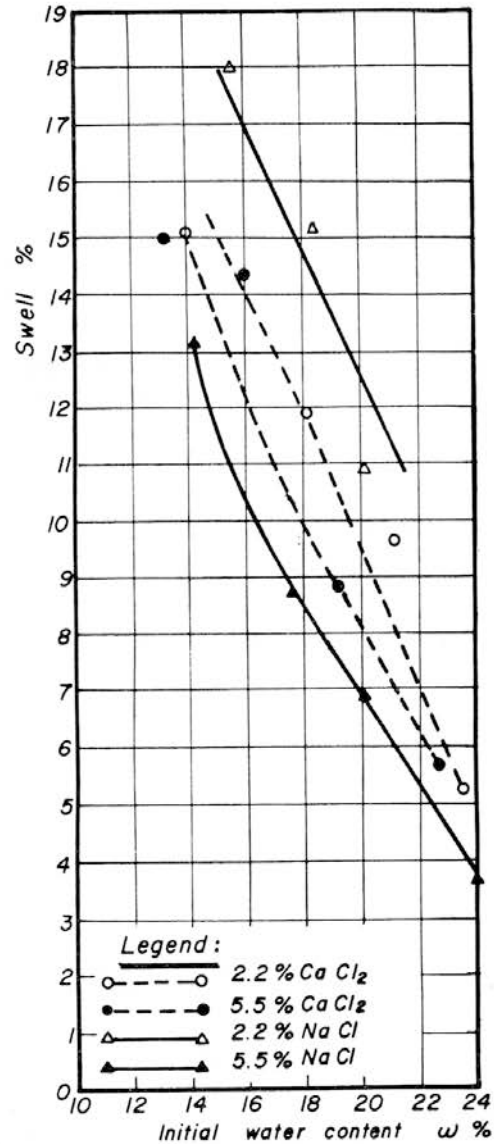with addition of NaCl, CaCl$_2$.



Fig. 8. Per cent swell versus preparation
moisture content, *CBR* specimens
with addition of NaCl, CaCl$_2$.

*Triaxial Strength of Compacted Specimens.*

Quick triaxial compression tests were performed on specimens of natural
clay and clay stabilized with 5.5% KCl, compacted to optimum moisture
content and maximum density. The Mohr failure circles are shown in
Figs. 9 and 10, and some stress-strain curves are shown in Fig. 11. It is
seen that the strength parameters change from $\phi = 24°$, $c = 6.4$ kg/cm$^2$ for
the natural clay to $\phi = 30°$, $c = 4.5$ kg/cm$^2$ for the stabilized clay – KCl increases
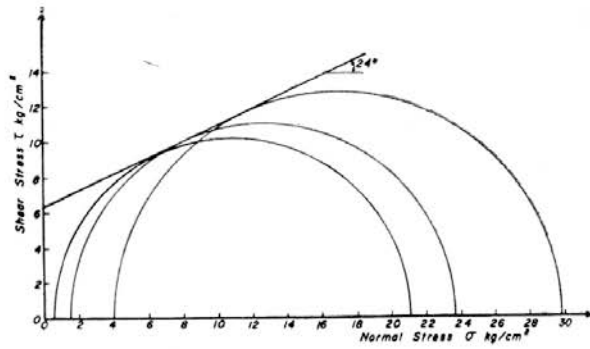
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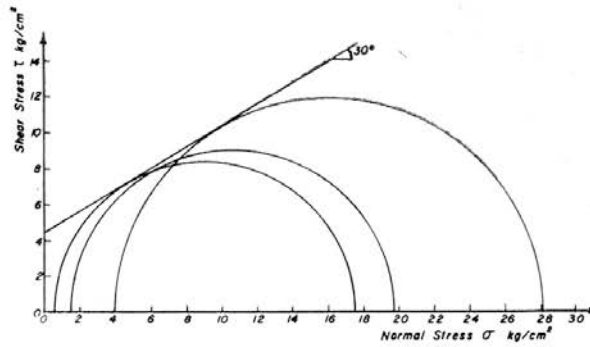Fig. 9. Mohr failure circles and envelope, compacted
natural clay.



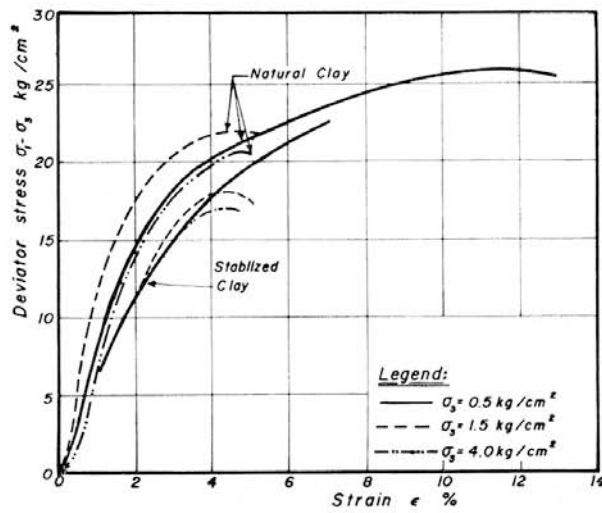Fig. 10. Mohr failure circles and envelope, compacted
stabilized clay.



Fig. 11. Stress-strain curves from triaxial test on compacted natural
and stabilized Clay.

the frictional property and decreases the cohesion property of the clay. This is in keeping with the change in the particles from clay size to silt size noted above. The natural soil is also seen to be stiffer than the stabilized soil – this reflects the developments of a more friable structure on addition fo KCl.

*Swelling Pressure*

Swelling pressure tests were performed in the oedometer on specimens of natural and clay stabilized with 5.5% KCl. Specimens were compacted both dynamically and statically to optimum moisture and maximum density. The swelling pressures obtained were as follows:

natural clay, dynamic compaction 8.4 kg/cm$^2$
natural clay, static compaction 16.2 kg/cm$^2$
stabilized clay, dynamic compaction 1.5 kg/cm$^2$
stabilized clay, static compaction 3.6 kg/cm$^2$

The addition of KCl decreased the swelling pressure of the natural clay by a factor of about 5. The effect of method of compaction on the swelling pressure has also been reported by SEED et al (1962) for natural Pittsburg Sandy clays; it is believed that this may be due to a difference in the distribution of pore sizes developed in the clay structure due to the different methods of compaction. This reasoning has been used previously to explain the difference between swelling pressure of compacted and undisturbed specimens of heavy clay (TOVEY et al, 1973).

Following the swelling pressure tests a number of specimens were taken for examination in the scanning electron microscope. The specimens were freeze-dried in order to preserve as much as possible their true structure. (TOVEY et al, 1973). Micrographs of 2 of the specimens are shown in Figs. 12 and 13. It is clearly seen from the micrographs that the stabilized soil particles are significantly larger than the natural soil particles.
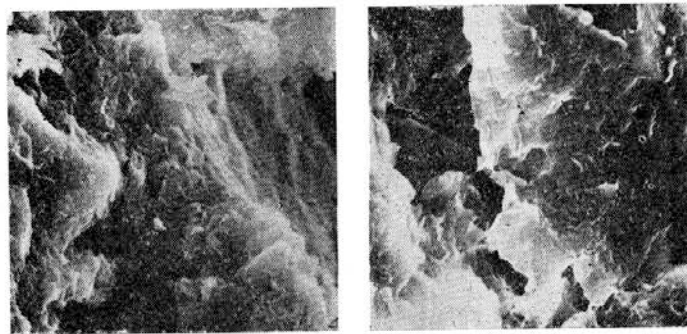


**Fig. 12. Scanning electron micrographs, natural clay following swelling pressure test.**
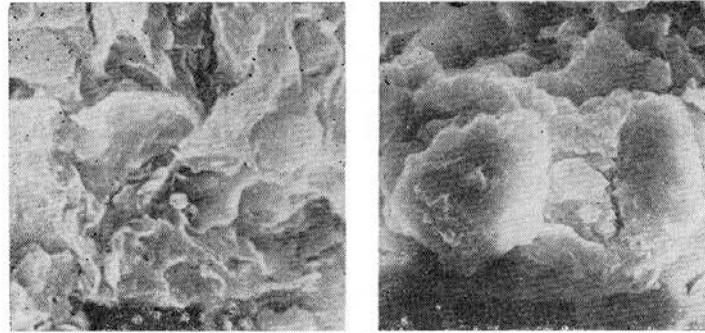
**Fig. 13.** Scanning electron micrograph, stabilized clay following swelling pressure test.

*X-ray Diffraction*

In order to study the effect of the addition of KCl on the mineralogical composition of the clay, x-ray diffraction tests were performed on natural clay, and on clay stabilized with 5.5% KCl. The tests were performed following washing out of all excess salt, since the presence of excess salt interferes with the diffraction analysis; the soil was washed with distilled water until the electical conductance of the effluent reached a constant value close to that of the water — the washing process took about 2 weeks to complete.

Figs. 14 and 15 show x-ray diffraction patterns obtained from the natural and stabilized soils. The montmorillonite peak evident in Fig. 14 is absent in Fig. 15, suggesting a change in the mineral structure. X-ray diffraction tests were also performed on specimens treated with ethylene glycol and following heating. In all cases the montmorillonite peak was absent in the stabilized soils.
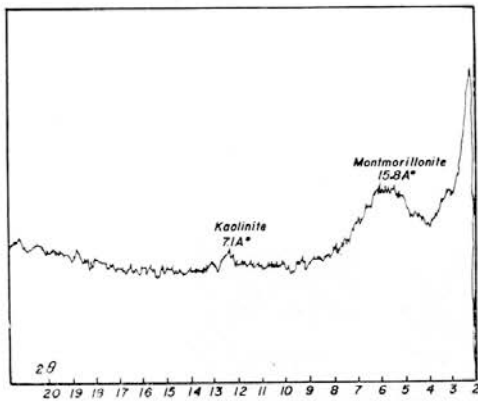


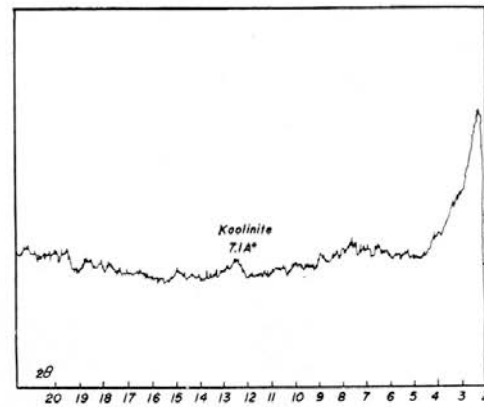**Fig. 14.** X-ray diffraction pattern-natural Clay.

**Fig. 15.** X-ray diffraction pattern-stabilized Clay.

*Effect of Washing on Engineering Properties*

As is evident from the test results presented, the addition of KCl leads to an improvement of the engineering properties of the clay. In particular, the

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activity of the clay is decreased. In order to study the degree of reversibility of this improvement, consistency limit tests were performed on stabilized soil which had been washed with distilled water for two weeks (the same samples (which were used for x-ray diffraction testing). The results obtained were $LL = 74\%$, $PL = 26\%$. The washed, stabilized soil is therefore more highly plastic than the natural soil. The reversible trend was confirmed in *CBR* tests which were performed on two washed specimens. One specimen was compacted at 15.6% moisture content — the specimen swelled 10.9% during soaking under a confining pressure of $0.025$ kg/cm$^2$, and had a soaked *CBR* of 1.1%; the second specimen was compacted at 21% moisture content, swelled 9.3% and had a soaked *CBR* of 2.5%. The compacted densities were similar to those obtained from the natural clay.

In considering these test results, it must be remembered that the "natural" soil contains an excess of salts whereas the washed, stabilized soil has had all excess salt removed. It is relevant, then, to compare the properties of the washed stabilized soil with a sample of washed natural soil, from which all excess salt has been removed. Consistency limit tests were performed on a sample of natural soil washed in a manner similar to the stabilized soil, and the results obtained were $LL = 84\%$, $PL = 26\%$.

As the washed natural soil would be expected, on the basic of double layer theory alone, to be less active than the washed stabilized soil, these results further suggest a change in the mineral composition of the clay.

### CONCLUSIONS

The addition of KCl to a heavy clay has been found to significantly decrease the activity of the clay—this effect is reflected in decreased liquid limit and plasticity index, in decreased swelling behaviour, in a change of particle size from predominantly clay size to predominantly silt size, and in a change in the strength properties of the soil making it a less cohesive and more frictional material. A comparison to stabilization with other salts (NaCl and $CaCl_2$) indicates that the stabilizing effect is not due solely to increasing the electrolyte concentration; x-ray diffraction studies further indicate a change in the mineral structure of the clay. On the other hand the reversal of the clay to a highly active soil following leaching of all excess salt indicates that the major effect of the KCl is a salt concentration effect; this appears to be accompanied by a change in the mineral structure which adds to the stabilizing effect.

From an engineering point of view, the use of KCl as a stabilizer appears potentially promising in locations where it is readily and cheaply available,

and where the major purpose of the process is to decrease the activity of the soil. The reversibility of the stabilization found in the laboratory on washing out excess salt raises a question as to the permanency of the process under field conditions where the soil may be subject to many cycles of wetting and drying. This aspect of the problem deserves further investigation, although it is felt that even the most extreme field conditions would not remove most of the excess salt from the soil.

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# DISCUSSION OF PAPERS

## FOUNDATIONS IN COASTAL MARGINS*

S. Vongvisessomjai and S. Thinaphong

### H.Y. Fang**

The authors have presented useful information on the foundation problems in coastal margins, however, most of the information presented focuses on the hydraulic aspects. Since foundation problems in coastal margins require knowledge from other disciplines, the writer wishes to add additional experience and data from the geotechnical engineering point of view with emphasis on coastal landslides and liquefaction behaviour of marine deposits.

Because of the classifying action of wave attack on the shore material and of the seaward transportation, most marine deposits have a relatively narrow particle size range as shown in Table 1 (Winterkorn and Fang, 1970). These relatively uniform deposits make them susceptible to quake liquefaction (Winterkorn and Fang, 1970, 1971). Also, their void ratio normally exceeds their critical void ratio *(CVR)*, and therefore, are in a potentially liquid state (Winterkorn, 1953). They may be changed into actual macromeritic liquids, not only in the specific shear zones but throughout the whole granular system. Relatively small energies, such as machine noises, vibrations, or minor earthquakes acting on the deposits may trigger the slope failure. Some marine deposits also contain high percentages of clay (35-60%) and silt (40-60%), and the sensitivity value ranges from 1.6 to 26.0, therefore, indicating very strongly that sediments are in a potential macromeritic liquid state and can become actual liquids after destruction of their interparticle bonds. Any man-made disturbance during the construction period will reduce the strength and bearing capacity significantly.

Since the liquifaction behaviour of the marine deposits is an important factor for analysis and design of foundation structures and landslide control, it is necessary to examine or assess the soil behaviour toward water and to indicate whether the soil has expansive or nonexpansive characteristics. In order to evaluate this phenomenon, a simple procedure as suggested by Winterkorn and Fang (1975) can be used. The procedure is based on the relative

**Table 1.** Granulometric constants of beach sand samples (after WINTERKORN & FANG, 1970).

| Location | | $D_{10}$ | $D_{60}$ | u.c. |
|---|---|---|---|---|
| Alaska | Adak | 0.25 | 0.48 | 1.9 |
| | Attu | 0.40 | 1.03 | 2.6 |
| | Kodiak | 0.25 | 0.30 | 1.2 |
| | Unalaska, Broad Bay | 0.64 | 1.18 | 1.9 |
| | Wide Bay | 0.15 | 0.33 | 2.2 |
| Canal Zone | Pima Beach | 0.18 | 0.42 | 2.3 |
| | Venado Beach | 0.38 | 1.15 | 3.0 |
| Cuba | Windmill Beach | 0.63 | 1.15 | 1.8 |
| | Conejo Bay | 0.30 | 0.48 | 1.6 |
| Florida | Daytona Beach | 0.15 | 0.20 | 1.3 |
| Guam | Blue Beach | 0.48 | 1.45 | 3.0 |
| | Purple Beach | 0.20 | 0.68 | 3.4 |
| | Tarague Beach | 0.20 | 0.65 | 3.3 |
| | Tumon Beach | 0.20 | 0.55 | 2.8 |
| Hawaii | Hilo Island | 0.07 | 0.23 | 3.2 |
| | Oahu, Barbers Point | 0.41 | 0.75 | 1.8 |
| | Pearl Harbor | 0.47 | 0.90 | 1.9 |
| Marshall Islands | Kwajalein | 0.60 | 1.50 | 2.5 |
| Massachusetts | | 0.15 | 0.24 | 1.6 |
| Midway Islands | | 0.20 | 0.44 | 2.2 |
| New Jersey | Spring Lake | 0.30 | 0.60 | 2.0 |
| Panama | Taboga Beach | 1.43 | 2.81 | 2.0 |
| Puerto Rico | mouth of Rio Grande, De Loiza | 0.25 | 0.38 | 1.5 |
| | Navy Beach | 0.25 | 0.43 | 1.7 |
| | Palo Seco | 0.15 | 0.30 | 2.0 |
| | Playa, Grande | 0.80 | 1.03 | 1.3 |
| | Port Maldonado | 0.35 | 0.39 | 1.4 |
| Trinidad | Macqueripe Bay | 0.18 | 0.38 | 2.1 |
| Virgin Islands | Frenchman's Bay | 0.53 | 0.90 | 1.7 |
| | | | Average value | 2.3 |

| u.c. | lowest: | 1.2 |
|---|---|---|
| | highest: | 3.4 |
| | median: | 2.3 coincides with average. |

Note: All samples are taken at low-water mark, $D_{10}$ = effective size, $D_{60}$ = 60% size, u.c. = uniformity coefficient = $D_{60}/D_{10}$, $D_{10}$ and $D_{60}$ in mm.

position of soil parameters such as liquid limit *(LL)*, plastic limit *(PL)*, field moisture equivalent *(FME)* and centrifuge moisture equivalent *(CME)*. An *FME* greater than *LL* indicates the danger of autogenous liquefaction of a soil in the presence of free water. If both *FME* and *CME* exceed 30 and if *FME* is greater than *CME*, the soil is probably expansive after release of a load. Two marine clays from the gulfs of Mexico and Maine have been studied using this procedure (FANG and OWEN, 1977).

In coastal margins, especially in tropical-earthquake areas, other factors such as residual soil, weathering rock, progressive erosion also will contribute

to coastal foundation problems (FANG, 1977). Finally, foundations in coastal margins are not a simple matter. It requires knowledge from other disciplines, therefore, a joint effort from hydrologists, geologists, geotechnical and structural engineers is needed to tackle this problem.

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## Authors' Reply

Prof. Fang is to be complimented for providing a clear lead to the valuable sources of geotechnical research and experiences related to the emplacement of structures in the coastal margins. It is of essense that due considerations be taken of quake lique faction, residual soil, weathering rock and progressive erosion in the analysis and design of the structure foundations lying low along the tropical earthquake coastlines. The hydraulic aspect alone can only give the intensity of acting hydrodynamic forces; while the capacity of such force as well as any others to affect the foundations need the full knowledge of geotechnical conditions. Thus the writer's conclusion deserves a reemphasis: "a joint effort from coastal engineers, hydrologists, geologists, geotechnical and structural engineers is needed to tackle this problem".

# CONFERENCE NEWS

**The Fifth Southeast Asian Conference on Soil Engineering** sponsored by our Society, AIT and CIDA during 2nd to 4th July this year was a great success. The conference was opened by H.E. Mr. Amphorn Chanvijit, Deputy Prime Minister of Thailand and all participants were welcomed by Dr. H. Shi-igai, Vice-President & Provost of Asian Institute of Technology. The Presidential address of SEASSE was given by Prof. Peter Lumb. Prof. Edward W. Brand, Chairman of the Organizing Committee chaired the opening session.

Prof. Chin Fung Kee, our past-president of the Society was one of the four distinguished guest lecturers. The other three being Dr. A.D.M. Penman, Prof. N.R. Morgenstern and Prof. Ian B. Donald.

The conference was attended by over 300 participants of which more than 200 were from 23 countries outside Thailand. There were four Technical Sessions on Pile Foundations, Earth Structures and Slope Stability, and, Soil Properties and Improvement. The Chairmen of the Technical Sessions being Dr. A.S. Balasubramaniam, Dr. Tan Swan Beng, Dr. Chai Muktabhant and Mr. Nibon Rananand.

Prof. Za-Chieh Moh, the founder President of SEASSE and the Vice-President of ISSMFE for Asia was a distinguished guest of the Society during the conference and was honoured by the presentation of a token gift from our Society during the conference dinner for his outstanding contribution in the professional development of Geotechnical Engineering in Southeast Asia.

About 40 papers were included in Volume 1 of the Proceedings which may be ordered at a price of US$ 40 from the Geotechnical & Transportation Engi-, neering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok Thailand. The second volume of the Proceedings will contain the guest lectures and will be published shortly.

**The International Symposium on Soft Clay** followed the Fifth Southeast Asian Conference on Soil Engineering during 5th & 6th July 1977. This Symposium, also sponsored by our Society, AIT and CIDA, was the first of its kind held on Soft Clays and was also an immense success. The Symposium was attended by over 350 participants of which more than 250 are from 23 countries outside Thailand.

The Symposium was opened by Prof. Za-Chieh Moh, our founder President of SEASSE and the Vice-President of ISSMFE for Asia. The main feature of the Symposium was the presentation of twelve state-of-the-art reports prepared by distinguished engineers from all parts of the world. The Chairman of the Technical Sessions being Dr. R.P. Brenner, Dr. A.D.M. Penman, Dr. Nimitchai Snitbhan, Prof. N.R. Morgenstern, Prof. Peter Lumb and Mr. S. Holmberg. The contributed papers of the Symposium have also been printed in a volume and can be obtained for US$ 50 from Geotechnical & Transportation Engineering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand. The state-of-the-art-reports will subsequently be printed as a separate volume and can be purchased separately at a later date.

# BOOK REVIEWS

**Earthquake Resistant Design, A Manual for Engineers and Architects** by D.J. Dowrick, John Wiley & Sons, London 374 p., US $ 27.50.

In the past two decades the literature on earthquake engineering has been growing at a rapid pace, but only a few books have been written for practicing engineers to help them absorb and understand new methods of analysis and progress in research. D.J. Dowrick's book helps alleviate this deficiency and is therefore a welcome addition to the engineers bookshelf.

The book is divided into eight chapters, entitled: (1) Consequences of Earthquakes, (2) Seismic Activity of a Region - Seismic Risk, (3) Site Response to Earthquakes, (4) Determination of Structural Form, (5) Structural Response to Earthquakes, (6) Structural Design and Detailing for Earthquake Resistance, (7) Earthquake Resistance of Services, and (8) Architectural Detailing for Earthquake Resistance. The text was originally written for engineers and architects employed by Ove Arup & Partners and is easily readable. In the reviewer's opinion, however, the designation "Manual" might be misleading with respect to the first five chapters. If the reader is looking for ready-made methods they are not available, nor are illustrative examples given. The author rather sketches the various approaches existing today for arriving at the various design parameters and techniques of analysis. Although comments and design recommendations are based on a thorough synthesis of the pertinent literature, the reader is referred to the references for detailed procedures.

The last three chapters of the book are mainly of interest to structural designers as they provide quite comprehensive information and guidance on detailing procedures, in particular for reinforced concrete, but also for prestressed concrete, masonry work, steel, and timber work. The chapter on services is particularly welcome as only in recent years seismic design of services and utilities has grown to an independent specialized field in earthquake engineering. As the author bases his recommendations on experiences and existing codes from various regions of the world, an international approach is maintained throughout.

The book is a suitable text for any practicing engineer dealing with aspects of earthquakes in his work and it will certainly stimulate the reader to further explore the specialized literature of earthquake engineering.

*R.P. Brenner*