GEOTECHNICAL CHARACTERISTICS OF SOILS IN THE TAIPEI BASIN

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Geotechnical Characteristics of Soils in the Taipei Basin

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SYNOPSIS: This paper summarizes some geotechnical data gathered from various projects carried out in the Taipei Basin. The purpose is to highlight geotechnical problems and to reflect areas of future studies. The general subsoil profile and zoning of subsoils are presented. Distribution of gravel and gas emission problem are reported. Preliminary correlations for the subsoils properties are established. Discussions are focused on the major geotechnical problems in the Basin; which covers groundwater drawdown and recovery, ground subsidence and possibility of soil liquefaction. Field monitoring data were compiled to indicate the behaviors of deep excavation including the penetration depth of retaining wall, bottom heave, wall deflection and ground settlement. Comparisons of piles resting on the gravel layer with clean base and soft slime base are made to show the significance of installation process of piles.

1. INTRODUCTION

The geotechnical characteristics of soils in the Taipei Basin have been studied for more than 25 years. Research papers by Hung (1966), Moh and Ou (1979) and Wu (1979) etc reported respectively the physical properties, engineering characteristics and dynamic parameters of the soil deposits in the Basin. In the past, studies were focused primarily on the Taipei City area where most of the developments were located. Due to the recent rapid growth of population and commercial activities, the Taipei City has expanded to cover the entire Basin. More geotechnical data and new findings become available from construction sites in the previous outskirts of Taipei and large scale project sites like the MRT Systems, new Expressway Systems and Railway Underground Projects. A comprehensive collection of geotechnical data has been conducted by Moh and Associates in 1987. Results of study based on these data, have been published in MAA (1987), Huang et al (1987), Cheng (1987) and Wu (1988).

The purposes of this Report are attempted to:

(a) update existing geotechnical information,
(b) highlight geotechnical problems, and
(c) reflect areas of future studies.

2. GEOLOGY AND SEDIMENTATION

The Taipei Basin is located at the northern corner of the island of Taiwan. It lies between latitude N24°58' to N25°09' and longitude E121°25' to E121°37', occupying a total area of 243 sq km below elevation of +20m above mean sea level. The Basin is geographically bounded by three major rivers : the Keelung River, the Taihan River and the Hsintien River as shown in Fig.1. Surrounding the Basin is the Tatun Vocano Group on the north, the Linkuo Terrace on the west and foothills on the east and the south. Geologically speaking, the Taipei Basin is a tectonic basin covered with more than 200 m of Quaternary sedimentary deposits overlying the Tertiary bedrocks. The Quaternary deposits can be further classified into three major formations, i.e. the Hsinchuang Formation, Chingmei Formation and the Sungshan Formation.

The Hsinchuang Formation immediately overlies the bedrocks and has a thickness of about 80 m. It consists of alternative layers of yellow or gray sandy clay and gravel.

The Chingmei Formation which overlies the Hsinchuang Formation is basically a gravel layer with large amount of coarse sand. This formation is an alluvial fan centered at Chingmei and disperses towards the northwest (Cheng, 1987). The thickness of this formation is generally 90 to 130 m. Because of its high permeability, this gravelly formation serves as the main acquifier of the Taipei Basin.

Overlying the Chingmei Formation is the Sungshan Formation which is the most recent deposit of 40-70 m thick. It consists of alternate layers of silty clay and silts interstratified with fine sand layers containing high silt content. The Sungshan Formation is of prime engineering concern because of its high compressibility and low shear strength.

According to studies by Lin (1957), Wang et al (1978) and Shih et al (1981), the geological history of the Taipei basin can be hypothetically divided into five stages:

(1) Tectonic movements of Hsinchuang, Kangiao and Taipei Thrust Faults - The active faults formed hilly ground with the ancient river flowed at north-east direction.

(2) Sedimentation of the Linkou gravels - The gravelly material transported by the ancient river from Hsientien were deposited at Linkou which was then a cliff shore or the lower side of the faults.

(3) Tectonic movements of Sanchiao Fault - The Taipei Basin was formed by Block fault. At this stage, the outlet of the ancient Hsientien River was blocked and the Taipei Basin became a closed lake covering Taishan and Hsienchuang. The Hsienchuang Formation was then formed.

(4) Sedimentation of Chingmei Formation - The ancient Hsintien River continued to deposit gravel materials in the Basin forming an alluvial fan centered at Chingmei and extending to the northwest. At later stage, the fan extended to Hsinchuang and overlied the Hsinchuang Formation.

(5) Sedimentation of Sungshan Formation - At about five to six thousand years ago, the Taipei Basin was flooded by sea water, probably because of a rise in sea level. River deposits carried by the Hsientien River, Keelung River, and Taihan River settled in the seawater lake and formed the Sungshan Formation. Later the bed sediments were exposed after the withdrawal of seawater through Kwantao. Alluvial sedimentation from the three Rivers continued and subsequently reached the present elevation.

3. SOIL PROFILE AND ZONING

3.1 Soil Profile

Distinct stratification of the subsoils in the Sungshan Formation were found. Hung (1966) first proposed that the subsoils can be subdivided into six sublayers in the following sequence:

Sublayer VI	: the uppermost layer; grayish black silt (ML)
Sublayer V	: gray silty fine sand (SM)
Sublayer IV	: gray silty clay (CL-ML)
Sublayer III	: yellowish gray non-plastic silty fine sand
Sublayer II	: gray silty clay (CL-ML)
Sublayer I	: medium dense to dense silty sand (SM)

The six sublayers classification is generally applicable to the town center area of Taipei City. The thickness of each sublayer may vary from area to area in the Basin. MAA (1987) reported the distribution of the typical sublayers as presented in Table 1.

3.2 Zoning of Subsoils

Based on the geological origin and sedimentary environment, the Sungshan Formation can be divided into three major zones according to the tributary of the rivers, namely: Tamshui River Zone, Hsintien River Zone and Keelung River Zone. For each zone, the subsoils can be further divided into 2 to 3 subzones according to the soil properties. The zoning of subsoils in the Basin is given in Fig. 2 and the distribution of each subzone is shown in Fig. 3. Table 2 summarizes the average physical properties of each sublayer in the various subzones. For subzones T3 and H1, insufficient data are available to draw any conclusions. It is generally found that thick deposits in subzone T3 within the top 10 m is loose silty sand. In some occasions, andesite rock boulders were found in the silty sand layer. Subzone H1 is located at Mucha area which is an alluvial plain of the Chingmei Creek, a tributary stream of the Hsientien River. Clay deposit of 20-30 m thick overlying bedrock was discovered at Yungchien Primary School and Chingmei Girls Secondary School. At the edge of the plain where it is close to the valley, shallow but steep sloping bedrock are generally found.

3.3 Distribution of Gravel

Underlying the Sungshan Formation is the Chingmei Formation. The Chingmei Formation consists of mainly gravels and sands. The size of gravels may be up to a diameter of 30 cm. The maximum thickness of this layer is 140 m at the New Park which is located approximately at the center of the Basin (Wang et al, 1978). The distribution of the gravel layer is important to foundation design because it has been used to support many piled structures. However, boreholes are generally sunk only a few meters or not at all into this layer in order to save time and cost for investigation. Many engineers in the past treated this layer with indifference because they believed that the thick gravel layer is down there anyway! Recent report by Fu et al (1990) points out that the gravel layer needs study as well. A few interesting conclusions have been drawn:

(a) At east of Taipei city (Hoping E. Rd. Sect. 3) three layers of gravels have been found. The uppermost layer is only a few meter below the ground surface, at about the same elevation of layer V of the Sungshan Formation. This layer is relatively thin. The second gravel layer exists at El.-20 m to -30 m. Again its thickness is less than 10 m. Underlain the second gravel layer is a 20 m to 30 m thick clay overlying the third gravel layer. (Fig.4a)

(b) A shallow thin layer of gravel is also found at the Yungho city. This layer is about 4-8 m thick existing between the Hsientien River and The City boundary of Yungho and Chungho. (Fig.4b)

(c) A shallow gravel layer of about 15 m thick covers the area between Hsintien and Kungkuan. (Fig.5a)

(d) Only a very thin layer of Chiengmei Formation is found at Nankang along the Keelung River area. (Fig.5b)

From the results of Fu's study, it is realized that the Chingmei Formation may not be a single layer of gravel material and it may not be as thick as what it was believed. Apparently, more attentions and studies are needed to give a clear picture of its distribution.

4. PHYSICAL AND CHEMICAL PROPERTIES

The physical properties of the subsoils are summarized in Table 3. The unique characteristics of this soil as compared to many well-known soils in the world is the large amount of silt blending together with either sand or clay in sublayers.

The chemical properties of the subsoils were studied e.g. MAA (1988). Generally, the very low salt contents and the neutral properties, i.e. pH around 7, of the soil have not caused great engineering concerns. However, the Writers wish to point out that the chemical properties of soils should not be overlooked in site investigation, particularly in Shihlin and Peitou area (Subzone T3) because of its proximity to the sulfur-rich hot springs and streams of Yangmingshan.

4.1 Gas Problems

In the past, gas emission has seldom been reported during borehole drilling in Taipei. Gas problems caused attentions only in the last two years when detailed site investigation were carried out for the MRT project (Tanal, 1990). Figure 6 plots the six site locations where gas was encountered during drilling. Incidents at three sites were verbally reported by drillers and other three sites were documented (MAA, 1988; MAA, 1989). Gas emissions in boreholes were reported usually when penetrating into soil layers containing organic matter or when the casing was being withdrawn after a borehole was completed. In one of the cases, gas erupted at a high pressure that small gravels and sand with water were ejected to about 15 m high above the ground. The eruption continued for almost a whole day. Gas sample was tested at one site and the results indicated that the gas was odourless, flammable methane (CH_4).

From these studies, a few points should be noted:

(a) all the sites where gas was encountered were at or near to existing rivers, or old river channels.

(b) organic matters were found in soil layers at depths about 20 m to 30 m below ground surface.

(c) all cases indicated that gas emission was localized.

(d) the gas type appears to be methane (natural gas).

5. ENGINEERING CHARACTERISTICS

Based on an extensive collection of subsoils data, MAA (1987) summarized the engineering properties of each sublayers in the three major subzones, namely T2, K1 and K2 as given in Table 3. It is seen that the compressible, low shear strength and low permeability cohesive layers (Sublayers VI, IV, II) are sandwiched by the less compressible, higher shear strength and higher permeability cohesionless layers (Sublayers V, III, I). Moh et al (1989) established some empirical correlations for the cohesive and cohesionless deposits in the Taipei Basin. In their study, each deposit is considered as one type of soil disregarding its subzone and sublayer. Due to inherent variations in the Taipei soil deposit, the established engineering correlations are more valuable to be used for projects in the planning and preliminary design stages. In the following sections, some of the engineering correlation results are presented.

5.1 Engineering Correlations for Cohesive Soils

(a) Stress history - Figure 7 presents the variation of the overconsolidation ratio (OCR) in relation with soil depth, Z, in which OCR is defined as the ratio between the maximum past pressure ($\bar{\sigma}vm$) and the effective overburden pressure ($\bar{\sigma}vo$). An equational expression to describe OCR profile is

$$OCR = \frac{Z}{Z-3}, \text{ for } Z > 3, Z \text{ in m} \quad ..... (1)$$

Data in Fig. 7 are test results obtained from projects located in many different subzones and sublayers around the Basin. The soil depth 50 m in Fig. 7 means the soil encountered at that depth, it does not imply a cohesive soil layer of 50 m thick. Fig. 7 indicates that the cohesive deposit in the Basin is slightly over-consolidated at shallow depth and it can be considered as normally consolidated at depths below 20 m.

(b) Virgin compression index - A correlation between the virgin compression index C_c in the normally consolidated range and the natural water content w_n was established as follows:

$$C_c = 0.015\,(w_n-8) \quad ..... (2)$$

The C_c values used for correlation were obtained from conventional oedometer test results with correction for sample disturbance according to the method suggested by Terzaghi and Peck (1967). In order to illustrate the data scatter, regions of 95% reliability of Eq.(2) are shown in Fig. 8. Also plotted in Fig. 8 are the correlations for other soil deposits. Among them, the correlation for Chicago Clay proposed by Peck and Reed (1954) appears to be very close to that of the Taipei silts.

Since the natural water content is closely related to the initial void ratio (e_o) for saturated soils, the correlation between C_c and e_o can also be established:

$$C_c = 0.54\,(e_o-0.23) \quad ..... (3)$$

(c) Coefficient of consolidation - The value of coefficient of consolidation c_v of the cohesive soils in Taipei is approximately constant in the normally consolidated range. The relationship between the c_v values as determined by Casagrande's log-time method and the liquid limit (w_L) of soils is as follows:

$$c_v = 0.033\text{x}10^{(-0.025w_L)} \quad ... (4)$$

where c_v is in cm²/sec, and w_L in percent.

Comparing Eq.(4) with the empirical correlations of U.S. NAVFAC (1982), Figure 9 shows the fairly good agreement between the two suggestions.

(d) Undrained strength profile - During the last decade, the saturated unconsolidated undrained triaxial test (SUU test) has become one of the very popular testing methods in Taipei in determining the undrained strength of soil samples retrieved below groundwater table. The SUU test is carried out by artificially saturating the soil specimens before the confining pressure is applied. Figure 10 shows the variation of the strength ratio $s_u/\bar{\sigma}vo$, for which s_u is the undrained strength and $\bar{\sigma}vo$ is the effective overburden pressure, with respect to soil depth. It indicates that the strength ratio decreases with increasing depth for the upper deposits and remains almost constant for the lower deposits. This trend is in accordance with the earlier discussion of the OCR profile (Fig. 7). Based on the data shown in Fig. 10, a simple correlation is proposed.

$$s_u/\bar{\sigma}vo = 0.21\,\left(\frac{Z}{Z-3}\right)^{0.9}, \; Z > 3, \; Z \text{ in m} \quad ... (5)$$

or as given in Eq.(1)

$$s_u/\bar{\sigma}vo = 0.21\,(OCR)^{0.9} \quad ..... (6)$$

It should be noted that s_u determined by SUU test is only about 75% of that determined by UUU test for soil samples not artificially saturated before applying the confining pressure. In such a case, Eq.(5) can be rewritten as

$$s_u/\bar{\sigma}vo = 0.28 \left(\frac{Z}{Z-3}\right)^{0.9} \quad .... \ (7)$$

for s_u obtained from UUU test.

(e) Anisotropic consolidation and mode of loading effects - Liu et al (1990) studied the effect of anisotropic consolidation and mode of loading on the shear strength of the normally consolidated Taipei silts. Four types of triaxial undrained test were carried out, namely $\overline{CIU}$, $\overline{CAUC}$, $\overline{CIUE}$ and $\overline{CAUE}$ in which $\overline{CIU}$ is isotropically consolidated undrained test, $\overline{CAU}$ is anisotropically consolidated undrained test, C is for compression test and E for extension test. Figure 11a plots the results of undrained shear strength ratio $s_u/\bar{\sigma}vc$ with the plasticity index, I_p of the soil. Correlation factor for each type of test data is also given. Similar to Fig. 11a, Fig. 11b plots the results of effective angle of shearing resistance $\bar{\phi}$ with I_p. The test results indicate that soils with higher I_p, lower $s_u/\bar{\sigma}vc$ and $\bar{\phi}$ are obtained. Within the range of I_p from 10% to 20%, the effects of anisotropic consolidation are as below:

$$\frac{(s_u/\bar{\sigma}_{vc})_{\overline{CAUC}}}{(s_u/\bar{\sigma}_{vc})_{\overline{CIUC}}} = 0.76 \sim 0.86 \quad .... \ (8)$$

$$\frac{(s_u/\bar{\sigma}_{vc})_{\overline{CAUE}}}{(s_u/\bar{\sigma}_{vc})_{\overline{CIUE}}} = 0.74 \sim 0.82 \quad .... \ (9)$$

$$\frac{(\bar{\phi})_{\overline{CAUC}}}{(\bar{\phi})_{\overline{CIUC}}} = 0.97 \sim 0.92 \quad ...... \ (10)$$

$$\frac{(\bar{\phi})_{\overline{CAUE}}}{(\bar{\phi})_{\overline{CIUE}}} = 1.07 \sim 1.14 \quad ...... \ (11)$$

and the effects of mode of loading are given in Eq.(12) to (15):

$$\frac{(s_u/\bar{\sigma}_{vc})_{\overline{CIUE}}}{(s_u/\bar{\sigma}_{vc})_{\overline{CIUC}}} = 0.72 \sim 0.78 \quad ...... \ (12)$$

$$\frac{(s_u/\bar{\sigma}_{vc})_{\overline{CAUE}}}{(s_u/\bar{\sigma}_{vc})_{\overline{CAUC}}} = 0.70 \sim 0.73 \quad .... \ (13)$$

$$\frac{(\bar{\phi})_{\overline{CIUE}}}{(\bar{\phi})_{\overline{CIUC}}} = 1.10 \sim 0.97 \quad .... \ (14)$$

$$\frac{(\bar{\phi})_{\overline{CAUE}}}{(\bar{\phi})_{\overline{CAUC}}} = 1.22 \quad .... \ (15)$$

5.2 Engineering Correlations for Cohesionless Soils

(a) Permeability - In Taipei, laboratory permeability tests are commonly performed in triaxial cells in which saturated specimens are consolidated under various confining pressures. Moh et al (1989) compiled laboratory results of 55 silty sand samples (SM) with D_{10} size varying from 0.002 to 0.006 cm and 39 poorly graded sand with silt samples (SP-SM) with D_{10} size varying from 0.006 to 0.01 cm. Based on these test results, Eq.(16) is suggested for preliminary estimation of permeability of granular soils in Taipei

$$k = 19D_{10}^2 \quad .... \ (16)$$

where D_{10} is in cm, and k in cm/sec. Statistical analysis indicates that there is no significant difference between the empirical correlations of the SM and the SP-SM soils. Figure 12 shows the variations of permeability with void ratios for the SM and SP-SM samples respectively. It is noted that for most soil samples tested with a void ratio ranging from 0.5 to 1.0, the permeability of the SM materials varies from 10^{-5} to 10^{-3} cm/sec and the permeability of the SP-SM materials ranges from 10^{-4} to 10^{-3} cm/sec.

(b) Angle of shearing resistance - Based on results of direct shear tests conducted on undisturbed cohesionless samples and the corrected blow counts obtained from in-situ standard penetration tests. Moh et al (1989) established a correlation as follows:

$$\bar{\phi} = 28 + 1.3\sqrt{N_c} \quad ..... \ (17)$$

where $N_c = 0.77\log(195/\bar{\sigma}vo)N$;
$\bar{\sigma}vo$ in tons/m^2

Figure 13 shows the correlation for the granular soils in Taipei and its regions of 95% reliability.

5.3 Engineering Correlations of Soils in Subzones

Attempts were also made to set up correlations of general geotechnical properties for sublayers of each subzone of the Taipei soils (Cheng, 1987). Cheng's study is preliminary but encouraging. Further studies with additional data are necessary for establishment of better correlations for the engineering properties of the sublayers.

6. MAJOR GEOTECHNICAL PROBLEMS

6.1 Groundwater Conditions and Ground Subsidence

It is well recognized that the initial groundwater conditions in the Basin were hydrostatic with a groundwater level close to ground surface. Due to the rapid growing of

Taipei City, pumping of groundwater from the Chingmei gravel has been started prior to year 1950. Ever since, subsidence of the ground becomes a major geotechnical problem. Settlement of benchmark was first discovered in the city in year 1955. Since 1960, ground subsidence was so serious that pumping of groundwater was officially restricted in 1969, but it has never been completely ceased. Because of the presence of the less permeable sublayer IV, the water pressures in the overlying soils remain hydrostatic related to a perched water table near to ground surface.

Figure 14 compares the annual pumping rate of groundwater from the Basin with the drawdown of a deep well and the associated subsidence of a benchmark at city center area. It is seen that the pumping rate has progressively reduced since year 1970, and ground subsidence slowed significantly after late 1970s. Nevertheless, ground subsidence has covered the entire Basin, where in some areas, the ground has subsided for more than 2 m (Fig. 15). As a result of the large ground sudsidence, many engineering facilities have been affected, namely:

(a) Reduction of the function of flood protection dikes: height of dikes needs to be raised.

(b) Reduction of the effectiveness of drainage systems; more pumping stations need to be built.

(c) Increase in area subjected to floodings.

(d) Causing negative skin friction to piles.

(e) Damages to structures such as bridges, buildings and airports due to differential settlements.

6.2 Recovery of Groundwater

As shown in Fig. 14, the groundwater in the Basin has recovered somewhat since 1975. Comparison of Fig. 16(a) and Fig. 16(b) indicates that the water head in the gravel layer recovered 20 m from year 1975 to 1985. Two sites in the city (Fig. 17 and Fig 18) with long term groundwater monitoring data prove that the groundwater pressures in the sublayers indeed have partly recovered. In association with the recovery of groundwater in the subsoils, some adverse effects should be considered:

(a) possibility of slight soil heave,

(b) increase in the potential of liquefaction in sandy soils,

(c) increase in difficulties in construction of deep excavation and tunnelling work,

(d) increase of the buoyant force to buildings with basement , or reduction in the capacity of tension piles.

It is not known at this moment what will happen to the overall groundwater conditions of the Basin after the construction of the forthcoming MRT Lines and Railway Underground Extension Projects where many kilometers of diaphragm walls will be installed down to the deep layers, some will even cut into the gravels. With all these cut off walls in ground, the natural flow patterns of the groundwater are expected to be effected. Currently, more than four hundred piezometers are being monitored every month along the proposed MRT lines. This work has been started in year 1987 and will be carried on till year 1992. It is hoped that with the longterm groundwater monitoring data, better understandings of the groundwater conditions in the Basin can be achieved in future.

6.3 Potential of Liquefaction

The Taipei Basin is subject to numerous earthquakes every year. The sandy sublayers III and V will have highest possibility of soil liquefaction if the sublayers are liquefiable. However, with the present state of knowledge, it is difficult to make conclusive statement on the risk of soil liquefaction in the Basin. Ishihara (1989) classified the sublayers III and V as old deposits of sand containing silt of low to non-plastic, and would be expected to display moderate to medium high potential for liquefaction. Wu (1979) studied the events of soil liquefaction in Taiwan in the period of year 1904 to 1964, none of the cases reported occurred in the Taipei Basin, however. Since 1964, due to the pumping effect as stated in the previous section, the sublayer III would have been densified slightly and thus reduced its potential risk of liquefaction.

Local geotechnical engineers commonly evaluate soil liquefaction potential by Seed's empirical approach (Seed & Idriss 1971; Seed, 1979). This approach is based on the SPT N values of the soils to indicate their cyclic shear strength. Ishihara (1989) pointed out that for the Taipei sandy soil which contain considerable amount of fines (silt and clay), the N values obtained at field should be increased to take into account the effect of fines. In other words, by using the "raw" SPT N value for evaluation, it tends to give a lower factor of safety because at the same relative density (or N value) the cyclic shear strength will increase with increasing fines content. Ishihara further suggested that undisturbed sandy samples be tested in laboratory to determine their cyclic shear strength. At present, only limited cyclic shear strength tests have been conducted on the Taipei subsoils (Wu, 1987; MAA, 1989). More researches along this line are certainly required.

Again, the recent recovery of groundwater level in the Basin will cause significant effects on the evaluation of soil liquefaction potential. Rise of groundwater level decreases the effective stress of subsoils if all other conditions are similar; in other words, recovery of groundwater level tends to increase the potential of soil liquefaction. This would mean sites previously evaluated as "safe" against liquefaction may no longer be safe anymore in future after the recovery of groundwater. An example is presented in Table 4 to illustrate this phenomenon.

6.4 Behaviors of Deep Excavations

Compensated foundations or so-called floating foundations have been extensively used for buildings in Taipei. Many highrise buildings in order to have sufficient 'floating' need a deep basement. Other reasons for which designers prefer to use deep basement are civil defence requirement, parking space shortage and maximizing land use. Basement excavations up to 20 - 30 m are not uncommon anymore. To secure deep excavations against failure in the soft Taipei deposits, concrete diaphragm walls with instrumentation are popularly used. In this report, monitoring data of some 27 strut-braced excavation sites have been compiled and the following items are addressed:

(1) Penetration depth of retaining walls
(2) Bottom heave
(3) Wall deflection and ground settlement

Bearing in mind that the behaviors of deep excavations are not only site specific, they are highly related to individual design and construction workmanships. The results presented in this Report should be considered preliminary only and brief discussions will be given. Most deep excavation sites are located in subzones T2 and K1 where for the former alternating layers of silty sand and clay are found and for the latter, it consists mainly of soft silty clay. Observation data for other areas are scarce and inconclusive.

(a) Penetration depth of retaining walls - Figure 19 shows the distribution of data points regardless of area and type of retaining wall. All data fall in between D=1.3 H and D=2.2 H where D is the penetration depth and H is the excavation depth. In K1 area, where deep /soft clay deposits exist, longer penetration depth of retaining wall are generally required, the depth falls between 1.6 and 2.2H. In T2 area, the penetration depth of retaining wall falls between 1.3H and 2.2H. It should be pointed out that with the data studied, maximum excavation depth is about 22 m, and the total penetration depth is still within the Sungshan formation.

(b) Bottom heave - Based on the available information (Fig. 20) the magnitude of bottom heave is less than 10 cm and no definite relationship between the amount of heave and excavation depth can be established. It is generally believed that bottom heave is due to elastic rebound, swelling and plastic flow. Higher bottom heave is expected for deeper excavation. In the Taipei soils, problems of bottom heave may be more complex because of the layered soils overlying gravel formation. Deeper excavation may result in less cohesive soils left over at the site which are subject to upheaval.

(c) The relationship between maximum wall deflection and excavation depth - Excavation causes the retaining wall to deflect or move towards the site. Maximum wall deflection (DHmax) normally occur at depth slightly below the excavated level. In the design of diaphragm walls, local engineers usually specify a definite deflection value of 5 to 10 cm. The design value will be used later as a means of safety control at site. Data in Fig. 21 indicate that maximum wall deflection monitored from most sites fall between a range of $0.00025H^2$ to $0.0005H^2$, i.e. 10 to 20 cm for an excavation of 20 m (H=20 m).

(d) The relationship between maximum wall deflection and ground settlement - Ground settlement will occur behind a retaining wall if the wall deflects due to excavation. The volume of ground settlement will be equal to the volume of wall deflection if the soil mass in the rear of wall is mobilized undrained. Figure 22 plots the relationship between normalized maximum wall deflection (DHmax) and normalized maximum ground settlement (DVmax). It can be seen that majority of data fall between DVmax=0.25 to 1.0 DHmax. Some data shown fall over the range of 1.0 DHmax may due to local failure, wall leakage, traffic, surcharge etc. It is noted that Mana & Clough (1981) found excavation in soft clay with sheetpiles or soldier pile walls, the maximum ground settlement DVmax=0.5 to 1.0 DHmax. The slightly lower ground settlement induced by excavation in the Taipei soils may be attributed to the higher rigidity of diaphragm walls used.

(e) Settlement profile induced by deep excavations - In deep excavation projects, it is necessary to evaluate the safety of adjacent properties as may be affected due to ground settlement. In this regard, the area of ground settlement and its magnitude should be assessed. Figures 23 and 24 compile the settlement data from excavation sites at subzones K1 and T2 respectively. Because of the thick and soft clayey deposit in K1 area, the ground settlement at rear of wall is generally larger than that in the T2 area, for same excavation depth. For preliminary assessment, the envelope of ground settlement profile can be expressed as follows:

δ/H (%) = 0.8-0.2 D/H for K1 area .... (18)
δ/H (%) = 0.4-0.08 D/H for T2 area .... (19)

where δ is ground settlement
H is depth of excavation
D is distance from wall

6.5 Bearing Capacity of Piles

Cast-in-situ bored piles, which are called reverse circulation piles in Taiwan, have been successfully used for supporting many bridges, flyovers and highrise buildings. For installation of this type of pile, a hole is first bored by a drill bit. To prevent collapse of hole, the hole is always filled with clay slurry. The slurry is also used to carry away the soil cuttings by suction through the bit. New slurry is pumped into the hole to maintain a constant head of fluid. After the desired depth is reached, a reinforcement casing is lowered and tremie concrete is placed to completely replace the slurry. The Chingmei Gravel layer has always been used as the bearing stratum for the bored piles. Because of the deep depth of the gravel layer from the ground surface, very long piles have to be designed. To avoid the problem of pile slenderness, pile diameters ranging from 1.0 to 2.5 m are commonly used. In normal cases, a pile is designed to take both skin friction and end bearing. In the past, what confused local pile designers most was the problem of "how much end bearing can be designed for a pile resting on the Chingmei Gravel?" Taking "undisturbed" sample from the gravel stratum is virtually impossible and hence no rational design parameters can be used. As a rule of thumb, it is suggested to consider the gravel layer as a cohesionless sand with STP N value ranging from 50 to 70, realizing that the actual in situ SPT N value of the gravel layer is often more than 100 blows. By using this assumption, one will often find that the pile capacity is not controlled by the soil capacity (skin friction and end bearing) but by the concrete strength of the pile. However, it should be noted that for large diameter piles, e.g. 2.0 to 2.5 m, the pile capacity may be controlled by soil capacity, especially, the end bearing from the gravel layer.

The above discussions are based on an assumption that the piles have clean bases and are resting on the gravel material. However in reality, this is not always true. Many local piling contractors prefer to drill pile holes without adding bentonite as slurry, but using natural clay at site. Due to the high silt content in the Taipei soils, slurry prepared with natural clay is liable to fast segregation. If the hole is not cleaned carefully, a layer of slime may accumulate at the bottom of the hole which may not be completely replaced by tremie concreting. As a result, the end bearing capacity of a pile will be greatly reduced, even though it was originally designed to be supported by the firm gravels.

In summary, there are several major geotechnical problems concerning large capacity pile foundation in Taipei:

(1) Unknown end bearing capacity of piles resting on the Chingmei Gravels;

(2) In Subzone K1, for example, where several layers of gravels exist, do they contribute enough end bearing to piles?

(3) The effect of slime at the bottom of bored holes on piles;

(4) Pile capacities with tip socketed in sandstone - shale formation.

Recently, the Writers have collected some data from projects in Taipei which MAA are involved. The information will be useful for pile design and it is hoped that the studies will shed light on the problems mentioned.

(a) End bearing capacity of piles resting on gravels - Figure 25 shows the results of two pile tests at a site located in Subzone T2. Pile A was cased down to the gravel. It had a clean base and the casing was greased outside to reduce friction. Pile B was partially cased. It was found that there was a layer of slime about 25 cm thick at the bottom of the borehole. Instruments were installed in the piles to monitor loading at the pile tip and the tip settlement. In Fig. 25, range of load-settlement relationship is presented for Pile A to account for some skin friction acting on the greased casing. Nevertheless it clearly shows that the ultimate capacity of the gravel is about 700 ton at a settlement equal to 10% of the pile diameter, i.e. 10 cm. Taking a factor of safety of 2.5, the allowable bearing capacity of the pile is 280 ton corresponding to a tip settlement of about 1 cm. Pile B which has a 'soft end' exhibits a much lower ultimate capacity of 360 ton. It is only about half of that for pile A.

(b) End bearing capacity of piles in Subzone K1 - Figure 26 shows the test results of four piles at a site in Subzone K1. The subsoil profile at this site can be referred to Fig. 4(a) where there exists an uppermost gravel layer near ground surface. The four piles were bored down to the upper gravel of Chingmei Formation. This upper gravel is only about 5-9 m thick at the site overlying a clay layer of 15 m thick. The clay had SPT N value of 13 to 34. The main concern of the tests was to find out whether the relatively thin gravel layer is firm enough to support the piles. Different processes for cleaning the slimes in the hole were used . For Pile F and Pile C, cleaning was carried out just before concreting. For Pile D and Pile E, cleaning of slime was executed for 5 hr and 0.5 hr, respectively, before lowering of the steel casing. Concrete was tremied with 2 to 3.5 hr lapsed time.

Test results in Fig. 26 indicate that for piles F and C, acceptable bearing capacities can be achieved. Pile E settled 5 cm at a final loading of 1,500 ton. Pile D settled 18 cm at a final loading of 1,700 ton. Results of pile integrity test performed on Pile D showed that nonhomogeneous concrete was in existence at the lowest 1.5 m.

Another interesting result is also shown in Fig. 26. For the four piles tested all the load-settlement curves cross at a point indicating 850 ton of load and a total settlement of 7 mm. This is possibly due to full mobilization of skin friction of the piles at a settlement of 7 mm. It is also known to the Writers, pile tests carried out in other subzones, largest amount of skin friction of piles was monitored at a relative movement of 7 mm.

(c) Capacity of piles socketed in sandstone formation - The project site is located at the Panchiao city. The subsoils at the site consist of 5 m backfill, 15 m of medium dense silty sand, 10 m of medium silty clay and 5-10 m of weathered sandstone and shale. Piles of 1.6 m diameter were designed to socketed into fresh sandstone of about 3 m. The fresh sandstone is poorly cemented and fractured. The SPT N value is generally larger than 100 blows in this layer, however. Three piles were selected for loading test and the results are given in Fig. 27. Piles G and J which have clean base show acceptable capacity. These two piles settled only about 10 mm when loaded to 1,300 ton.

Pile K was detected having a 'soft end' by integrity test. This pile had a much lower capacity and it failed to support a load of 1,175 ton. Large settlement occurred when the loading exceeded 1,000 ton.

From the above review of the pile tests, it can be concluded that piles with no 'soft end' can gain great capacity from the bearing stratum. To achieve a slime-free base of a pile, installation workmanship is of paramount importance.

7. CONCLUDING REMARKS

This Report summarizes some of the geotechnical characteristics of deposits in the Taipei Basin. Due to the variable environmental conditions and complex sedimentation natures, the subsoil are known to be more heterogeneous than what was previously realized.

Geotechnical mapping of soils in the Basin by classifying the soils into several subzones appears to have had a successful start. Collection of data should be continued to increase the data base. Many other geotechnical problems not mentioned in the Report are also of importance. For example, studies on soil improvements, chemical groutings, tunnelling and underpinning etc. are still extremely scarce. More studies in these areas are urgently needed to meet the requirements of the forthcoming MRT projects.

ACKNOWLEDGMENTS

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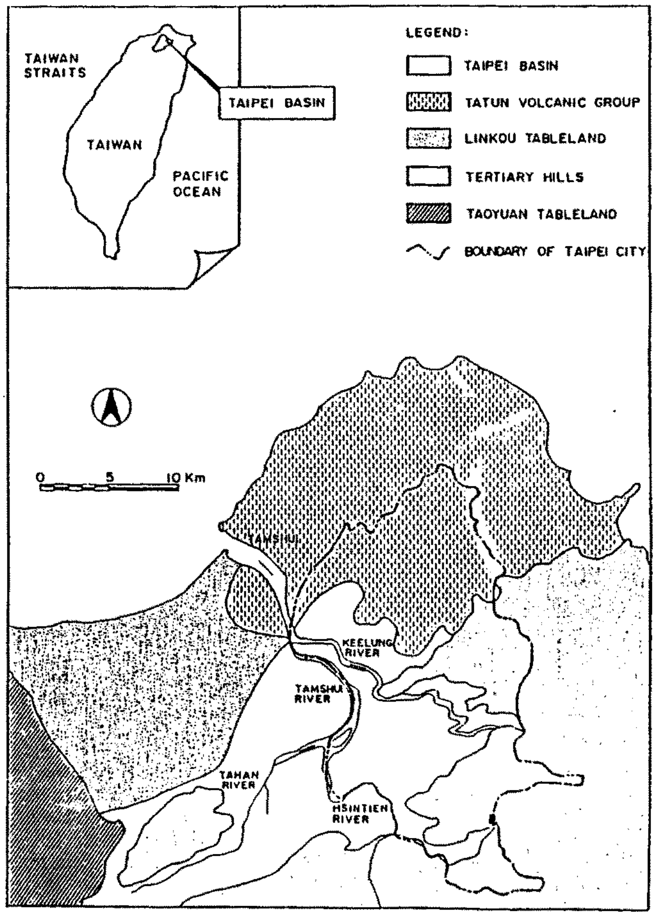

FIG. 1 GEOLOGICAL CONDITIONS OF THE TAIPEI BASIN AND ITS SURROUNDING AREAS

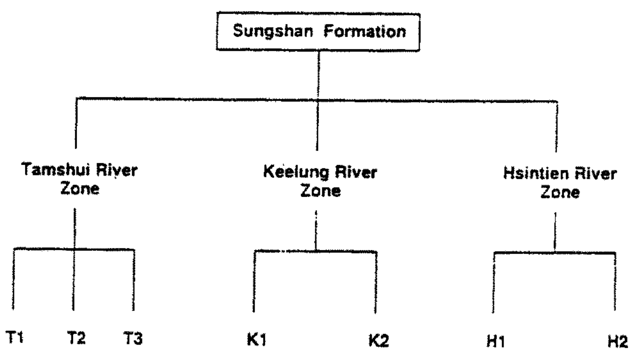

Fig. 2 Zoning of Subsoils of Sungshan Formation

Table 1 Distribution of Sublayers in the Taipei Basin

Sublayer	Soil Description	Thickness m	Distribution
VI	yellowish brown or gray silty clay (CL-ML)	0-6	6 m at Hsinyi; slightly thinner along Tamshui River; not found at Kwantao Plain and Hsiehchi; top of this sublayer inclines towards NW direction.
V	gray silty fine sand	0-20	15-20 m at Kwantao Plain and Hsiehchi; not found at Hsinyi. Mingchuan E. Rd., Naihu; top of this sublayer varies from El.+2 m to El.-4 m.
IV	gray silty clay	5-30	25 m at Hsinyi; 15 m east of Fushing S. Rd.; <10 m at Chengchung, Chien Chen Districts and Kwantao; 25-30 m at Shihlin, Peitou; <5 m along Tamshui River; top of this sublayer inclines towards NW direction with elevation varies from -5 m to -15 m.
III	gray medium dense sand interstratified with silt or silty caly seams	0-15	10-15 m at Lungshan. Yenping, Chengchung Districts; <4 m at east of Fushing S. Rd. and south of Mingchuan E. Rd.; not found at Hsinyi; 2-4 m at Shihlin, Peitou.
II	gray silty clay	2-15	10-15 m at east of Shinshan S. Rd. to Chunghsiao E. Rd. Sect. 5<10 m at west of Shinshan S. Rd. 2 m at Peitou, Shihlin; not known at Kwantao.
I	medium dense to dense silty sand or sand gravel	0-5	thicker at west of Chungshan, Ta-an Districts; thinner or not found at Neihu, Sungshan, Nankang; not known at Peitou, Shihlin.

Table 2 Summary of Physical Properties of Subsoils

Subzone	Sublayer	Thickness m	Dry Unit Weight, t/m³	Water Content, %	Specific Gravity	Liquid Limit, %	Plasticity Index, %	Particle Size Distribution, ※ %			
								Gravel	Sand	Silt	Clay
T1	VI	3.7	1.68	26.6	2.68	33.3	11.2	0	34	31	26
	V	11.6	1.85	18.7	2.71	–	NP	11	63	16	5
	IV	15.1	1.70	30.6	2.72	26.9	8.3	1	22	43	26
	III	9.4	1.66	21.7	2.71	–	NP	1	54	18	8
	II	12.9	1.51	28.5	2.73	31.2	9.4	ND	ND	ND	ND
	I	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
T2	VI	4.5	1.45	31.2	2.72	35.8	12.9	0	10	58	32
	V	10.1	1.54	26.3	2.68	–	NP	1	75	19	4
	IV	9.0	1.43	32.1	2.72	34.3	12.0	0	8	61	31
	III	10.6	1.61	23.9	2.69	–	NP	0	60	34	7
	II	8.4	1.55	27.2	2.72	30.3	9.2	0	9	67	25
	I	4.5	1.70	20.3	2.68	–	NP	1	62	29	7
K1	VI	4.2	1.43	32.1	2.71	34.1	11.1	0	11	61	29
	V	6.6	1.70	20.1	2.68	–	NP	2	73	18	5
	IV	20.0	1.37	35.1	2.72	35.2	12.9	0	4	61	35
	III	3.6	1.61	23.5	2.69	–	NP	0	54	36	11
	II	9.0	1.52	28.3	2.71	33.0	12.0	0	12	54	35
	I	4.7	1.68	20.9	2.68	–	NP	0	59	32	9

(To be Continued)

Table 2 Summary of Physical Properties of Subsoils (Continued)

Subzone	Sublayer	Thickness m	Dry Unit Weight, t/m³	Water Content, %	Specific Gravity	Liquid Limit, %	Plasticity Index, %	Particle Size Distribution, ※ %			
								Gravel	Sand	Silt	Clay
K2	VI	4.8	1.27	40.6	2.73	38.9	15.6	0	3	53	44
	V	4.9	1.41	33.1	2.67	–	NP	1	76	19	5
	IV	26.9	1.28	40.5	2.73	38.8	15.5	0	3	53	45
	III	4.2	1.56	26.4	2.69	–	NP	3	60	28	10
	II	7.4	1.43	31.2	2.70	33.7	12.0	0	9	58	33
	I	3.9	1.66	21.2	2.70	–	NP	4	68	18	9
H2	VI	5.6	1.39	33.0	2.71	35.0	13.6	0	8	57	35
	V	14.6	1.91	13.3	2.69	–	NP	22	52	20	5
	IV	6.7	1.46	31.0	2.71	31.2	10.1	0	10	65	25
	III	6.3	1.69	20.9	2.70	–	NP	2	68	25	7
	II	ND	1.44	30.8	2.71	33.2	9.5	0	16	66	18
	I	ND	1.72	21.3	2.66	–	NP	0	80	18	2

Nota : NP - Nonplastic
ND - Not sufficient data
※ - Percentages of particle size shown are average of different number of sample. sum of percentage of gravel, sand, silt and clay may not be 100%

not sufficient data for subzones T3, H1

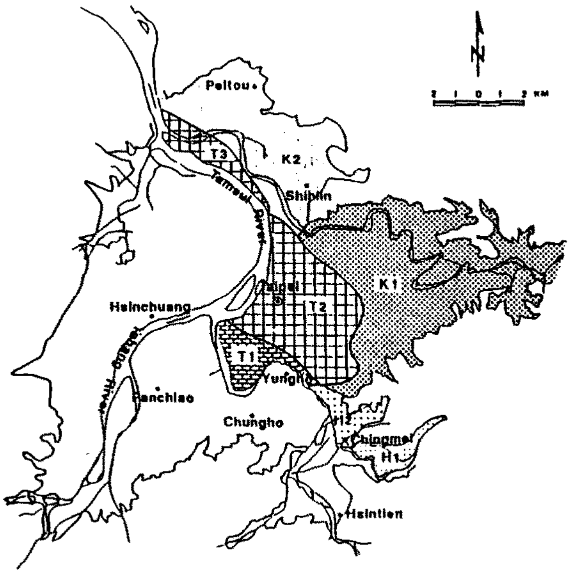

FIG. 3 LOCATION OF SUBZONES OF SUNGSHAN FORMATION

TABLE 3 SUMMARY OF ENGINEERING PROPERTIES OF SUBSOILS

Subzone	Sublayer	N blows/30cm	Compressibility				Shear Strength				Permeability
			e_0	C_c	c_v cm^2/sec	m_v cm^2/Kg	q_u Kg/cm^2	s_u Kg/cm^2	$\bar{c}$ Kg/cm^2	$\bar{\phi}$ deg.	k_v cm/sec
T2	VI	5	0.87	0.29	5.6E-03	2.5E-02	0.63	0.44	0	33.6	4.4E-07
	V	10	-	-	-	-	-	-	-	32.5	2.6E-04
	IV	8	0.94	0.39	6.4E-03	1.6E-02	0.96	0.53	0	32.3	1.4E-06
	III	21	-	-	-	-	-	-	-	33.3	1.7E-04
	II	19	0.84	0.33	6.3E-03	9.3E-03	1.74	0.53	0	35.5	-
	I	31	-	-	-	-	-	-	-	34.2	-
K1	VI	3	0.92	0.28	6.4E-03	2.9E-02	0.55	0.27	0	33.9	2.1E-07
	V	12	-	-	-	-	-	-	-	32.3	1.8E-04
	IV	4	1.06	0.46	3.8E-03	2.6E-02	0.73	0.40	0	31.3	-
	III	16	-	-	-	-	-	-	-	32.4	-
	II	14	0.88	0.36	3.4E-03	1.2E-02	1.46	0.55	0	31.6	-
	I	31	-	-	-	-	-	-	-	-	-
K2	VI	3	1.03	0.34	2.0E-04	2.8E-02	0.42	0.31	0	34.5	-
	V	5	-	-	-	-	-	-	-	32.4	-
	IV	3	1.18	0.50	3.2E-03	3.7E-02	0.64	0.34	0	30.9	3.1E-07
	III	20	-	-	-	-	-	-	-	35.3	-
	II	14	1.04	0.50	1.7E-03	1.6E-02	1.39	-	-	30.6	-
	I	19	-	-	-	-	-	-	-	-	-

TABLE 4 AN EXAMPLE ILLUSTRATING THE EFFECTS OF GROUNDWATER RECOVERY ON SOIL LIQUEFACTION

SUBLAYER	DEPTH m	FINES %	EXISTING N_1	AFTER GWL RECOVERY N_2	EXISTING F.S.1	AFTER GWL RECOVERY F.S.2
V	6.0	26	6	4	1.04	0.63
	9.0	29	17	15	1.66	1.21
	10.5	18	12	10	1.06	0.81
	13.5	39	15	13	1.34	1.10
III	21.0	19	17	12	1.27	0.93
	25.0	38	19	14	1.47	1.14

Assumptions: Earthquake M_L = 7.5 a_{max} = 0.18g

N_1 for GWL V : -5.4 m, III : -10.3 m

N_2 for GWL V & III : -1.5m

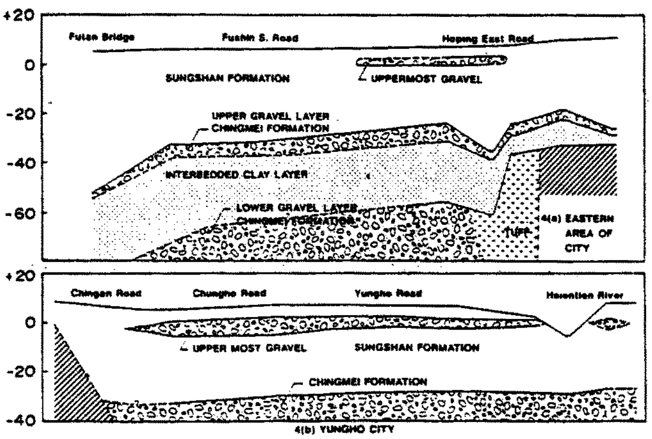

FIG. 4 DISTRIBUTION OF GRAVELS

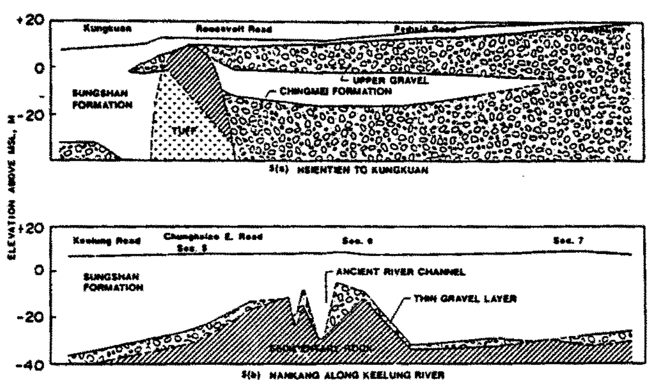

FIG. 5 DISTRIBUTION OF GRAVELS

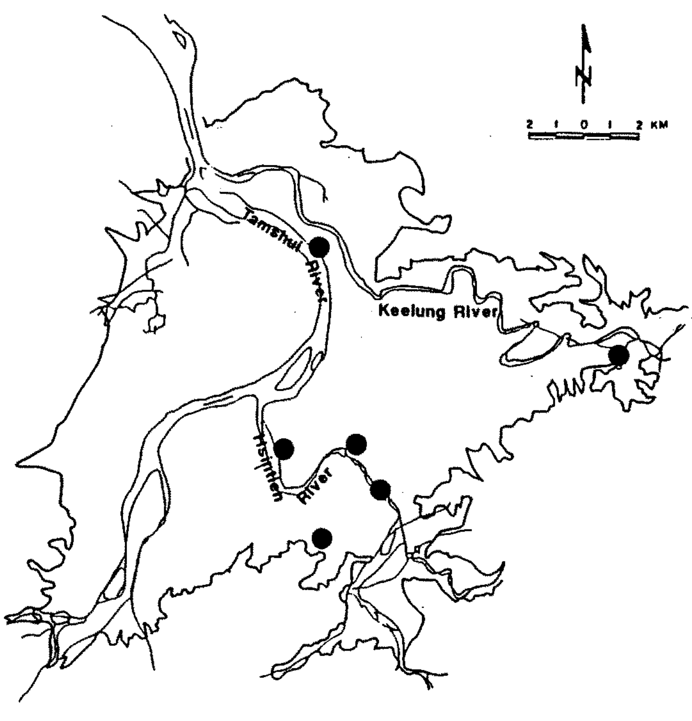

FIG. 6 LOCATION OF SITES WITH GAS EMISSION

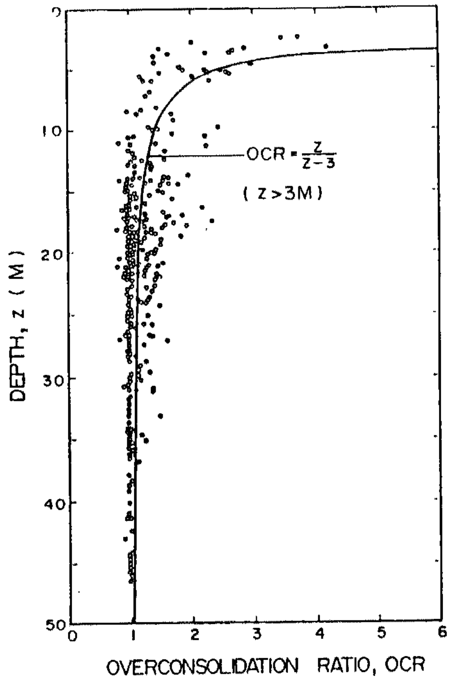

FIG. 7 RELATION OF OCR AND SOIL DEPTH FOR COHESIVE SOILS IN TAIPEI

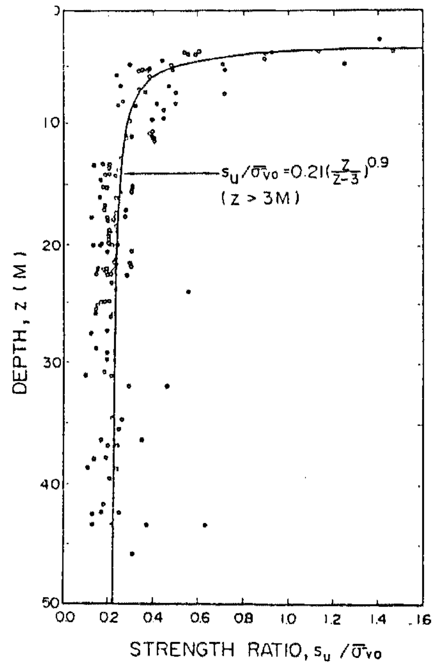

FIG. 10 UNDRAINED SHEAR STRENGTH PROFILE OF THE COHESIVE SOILS IN TAIPEI

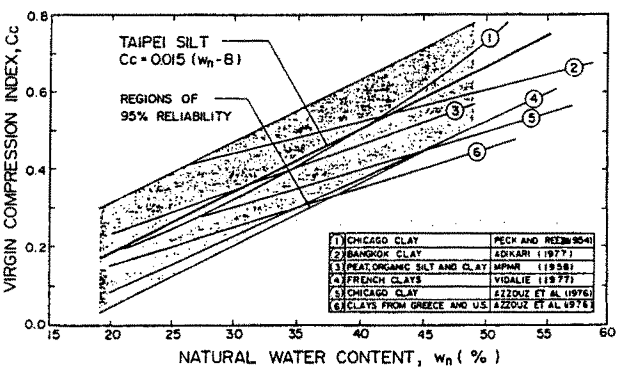

FIG. 8 RELATION OF VIRGIN COMPRESSION INDEX TO NATURAL WATER CONTENT

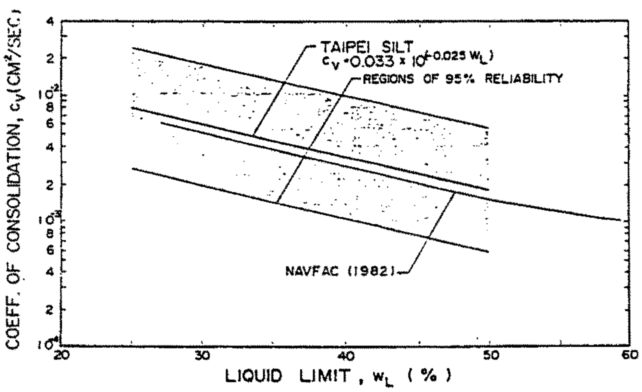

FIG. 9 RELATION OF COEFFICIENT OF CONSOLIDATION TO LIQUID LIMIT

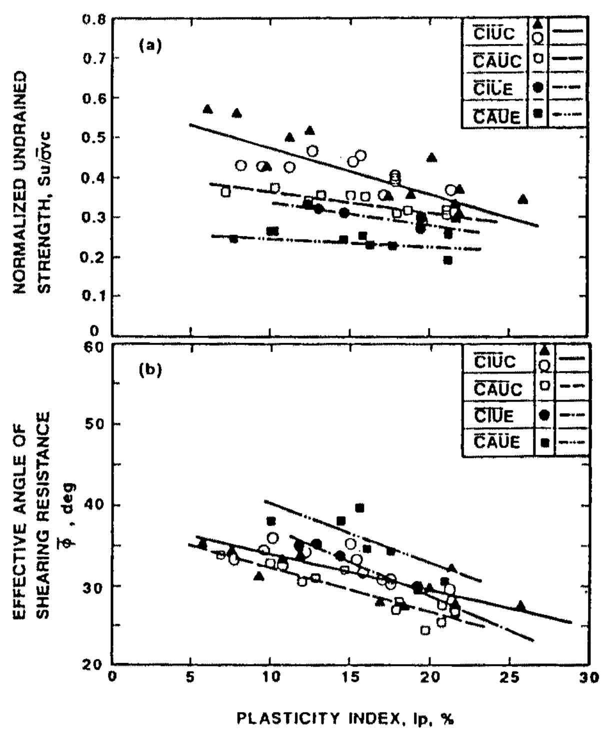

FIG. 11 CORRELATIONS BETWEEN UNDRAINED STRENGTH RATIO, EFFECTIVE ANGLE OF SHEARING RESISTANCE WITH PLASTICITY INDEX OF TAIPEI COHESIVE SOILS

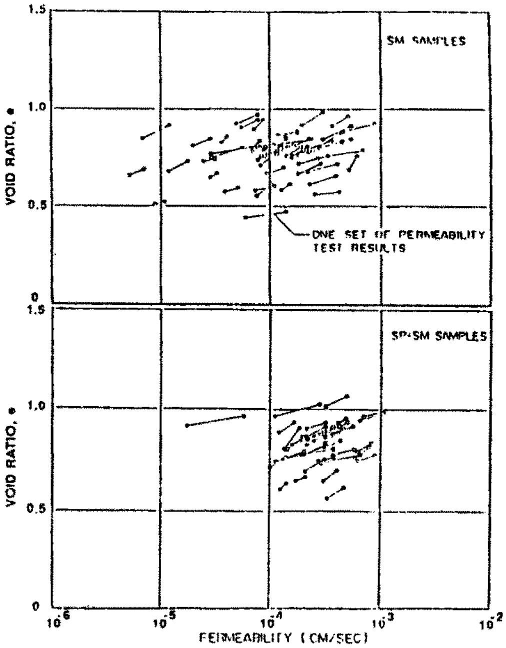

FIG. 12 RELATION OF VOID RATIO TO PERMEABILITY

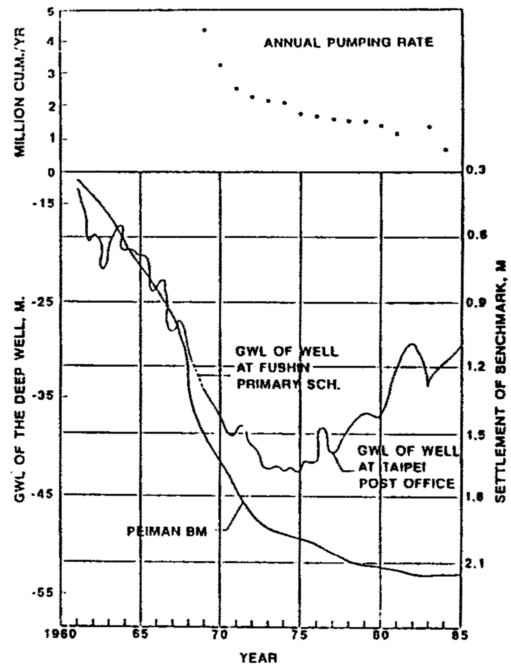

FIG. 14 ANNUAL PUMPING RATE, GROUNDWATER LEVEL AND SETTLEMENT AT CENTER OF TAIPEI CITY

RELATIONSHIP BETWEEN $\bar{\phi}$ AND N (MEYERHOF, 1956)			
VERY LOOSE	LOOSE	COMPACT	DENSE
<30°	30°-35°	35°- 40°	40°-45°

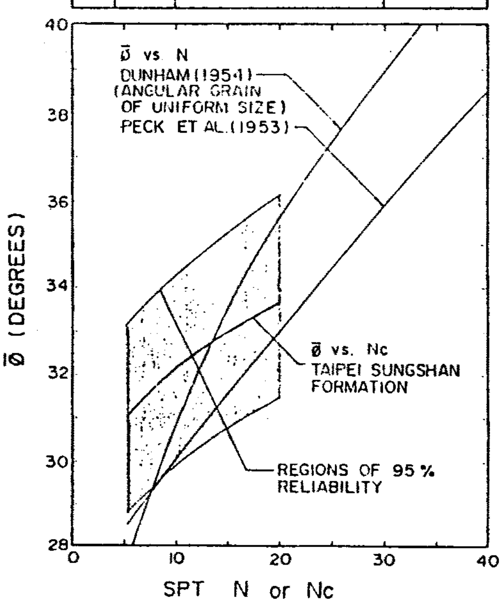

FIG. 13 RELATION OF ANGLE OF SHEARING RESISTANCE TO SPT N-VALUE

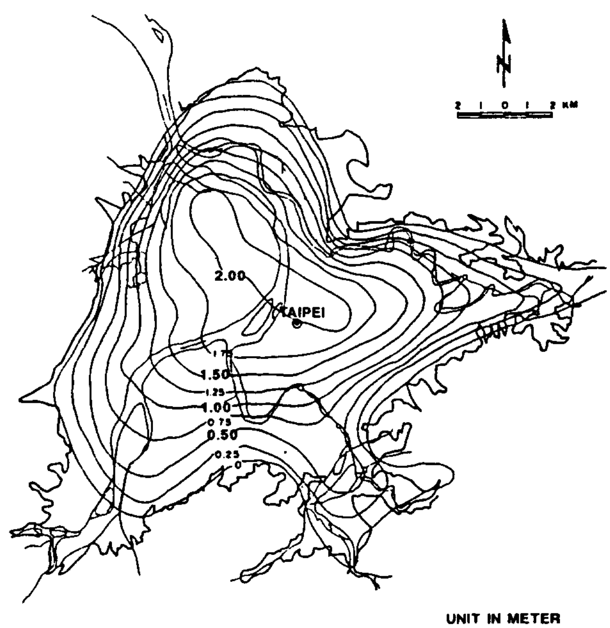

FIG. 15 CONTOURS OF GROUND SUBSIDENCE OF THE TAIPEI BASIN BETWEEN 1955 AND 1987 (AFTER WRPC, 1988)

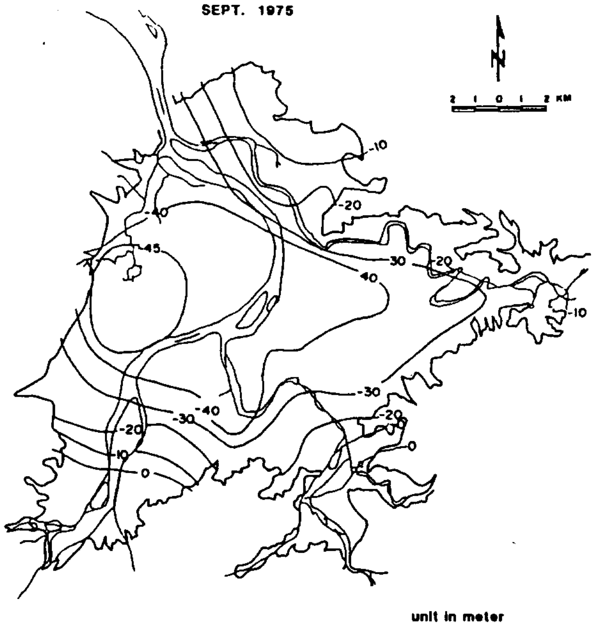

FIG. 16(a) GROUNDWATER LEVELS IN THE CHINGMEI GRAVEL 1975 (after WRPC 1976)

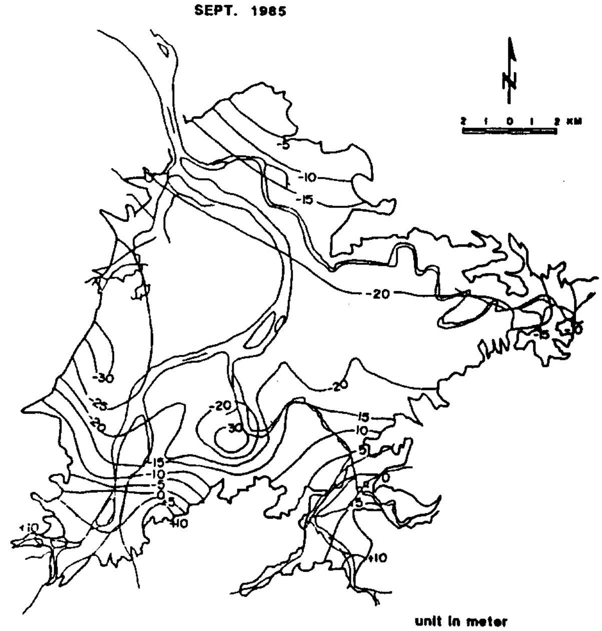

FIG. 16(b) GROUNDWATER LEVELS IN THE CHINGMEI GRAVEL 1985 (after WRPC 1986)

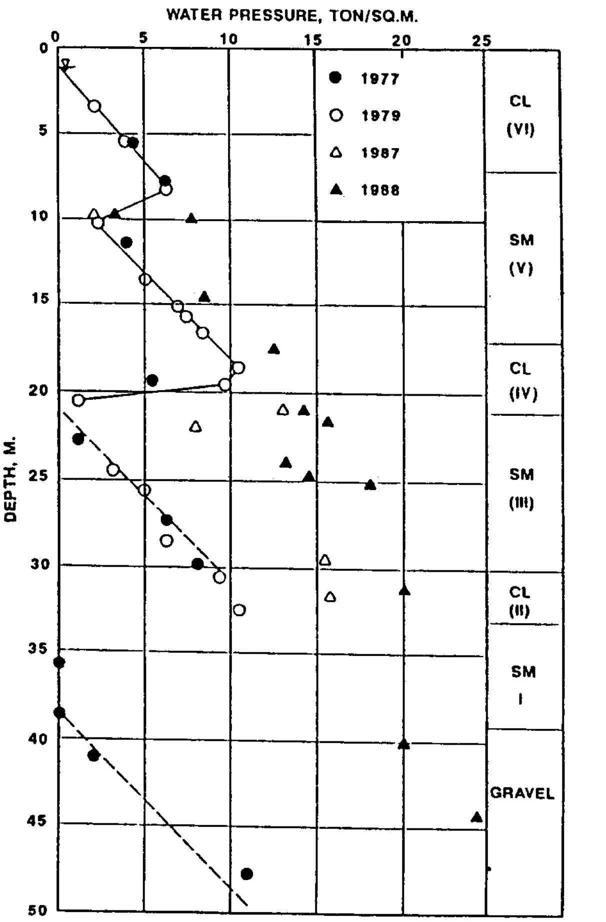

FIG. 17 MONITORED GROUNDWATER PRESSURE AT TAI-POWER HQ SITE

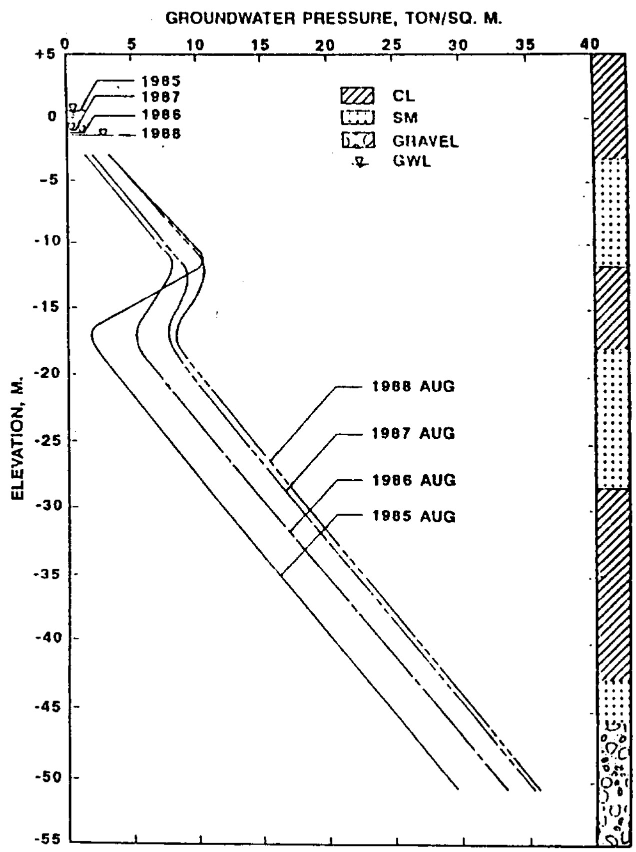

FIG. 18 RECOVERY OF GROUNDWATER PRESSURE AT TAIPEI RAILWAY MAIN STATION

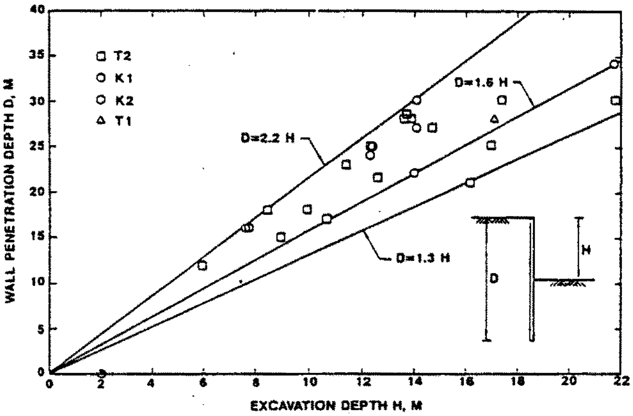

FIG. 19 RELATIONSHIP BETWEEN WALL PENETRATION DEPTH AND EXCAVATION DEPTH

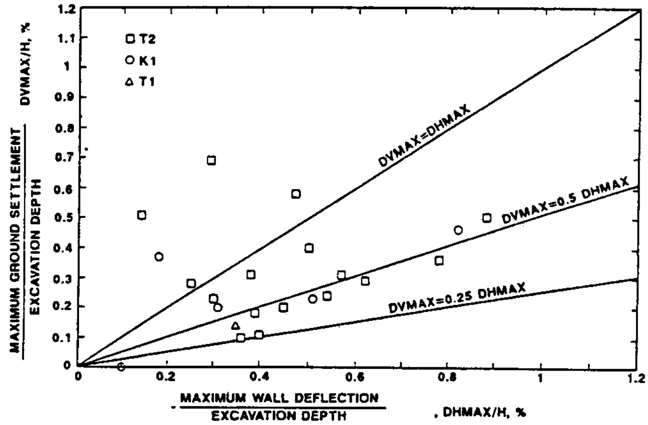

FIG. 22 RELATIONSHIP BETWEEN MAXIMUM GROUND SETTLEMENT AND MAXIMUM WALL DEFLECTION

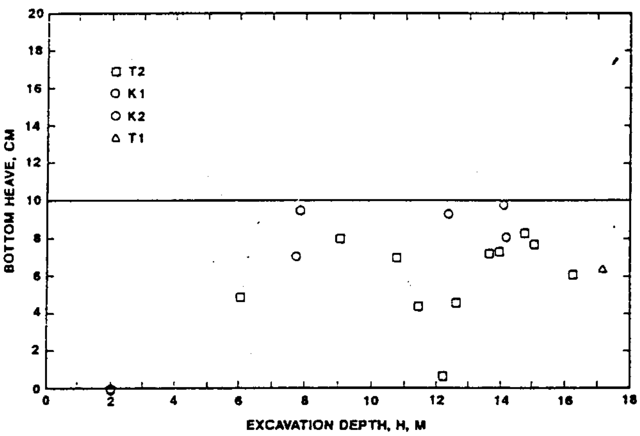

FIG. 20 RELATIONSHIP BETWEEN BOTTOM HEAVE AND EXCAVATION DEPTH

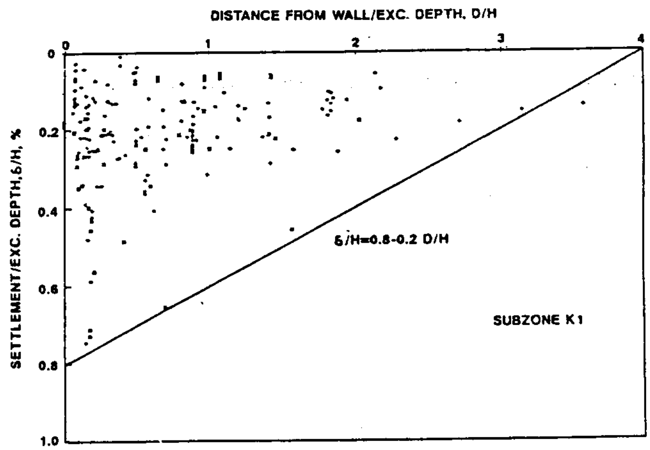

FIG. 23 SETTLEMENT PROFILE FOR EXCAVATION IN K1 SUBZONE

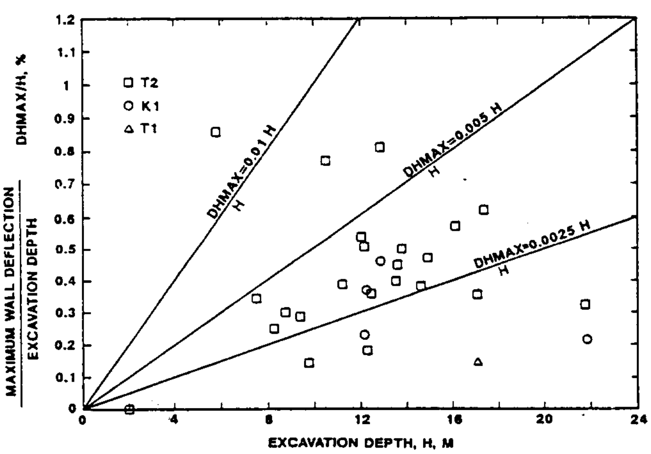

FIG. 21 RELATIONSHIP BETWEEN MAXIMUM WALL DEFLECTION AND EXCAVATION DEPTH

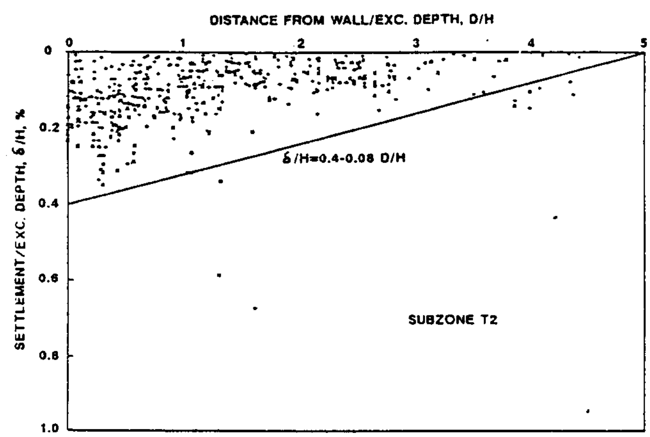

FIG. 24 SETTLEMENT PROFILE FOR EXCAVATION IN T2 SUBZONE

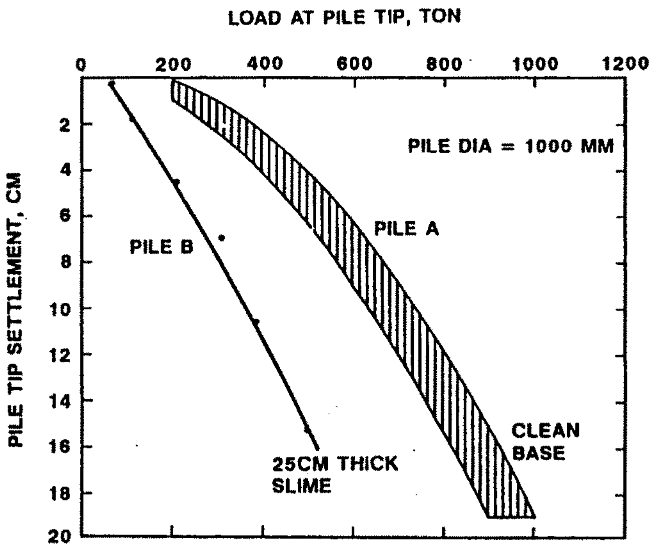

FIG. 25 END BEARING CAPACITY OF PILES RESTING ON GRAVEL LAYER - SUBZONE T2

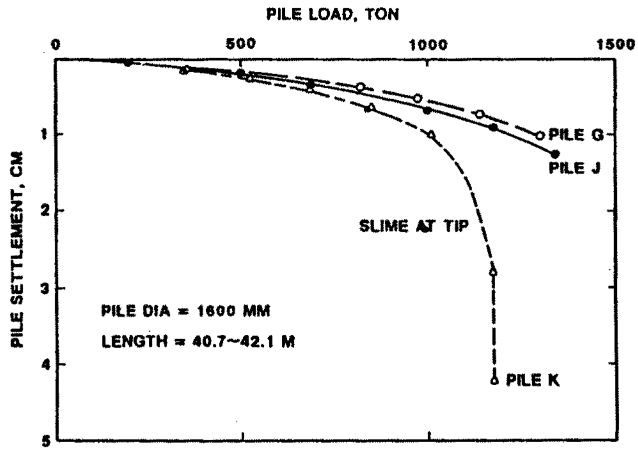

FIG. 27 RELATIONSHIP OF LOAD - SETTLEMENT OF PILES SOCKETED IN SANDSTONE

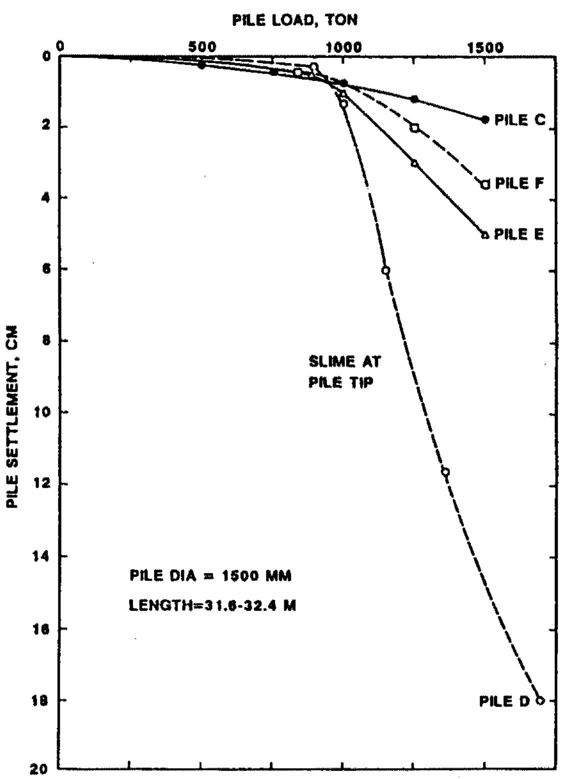

FIG. 26 RELATIONSHIP OF LOAD-SETTLEMENT FOR PILES RESTING ON THE UPPER GRAVEL LAYER OF SUBZONE K1