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ENGINEERING CORRELATIONS FOR SOIL DEPOSITS IN TAIPEI

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Taipei, Taiwan 10410, R.O.C.***Key Words:** engineering correlation, soil deposits, Taipei soil deposit.

ABSTRACT

This study aims at establishing empirical correlations for the soil deposits in Taipei City. These correlations are valuable in the planning and preliminary design stages. For cohesive soils, correlations between (1) virgin compression index and natural water content, (2) virgin compression index and initial void ratio, (3) coefficient of consolidation and liquid limit, and (4) angle of shearing resistance and plasticity index are proposed. Profiles of overconsolidation ratio (OCR) as well as strength ratio are presented. Relationships between the coefficient of earth pressure at rest and the angle of shearing resistance and the plasticity index have also been carefully studied. For cohesionless soils, correlations between (1) permeability and effective particle size, and (2) angle of shearing resistance and standard penetration test (SPT) N -value are presented. Due to inherent variations in the Taipei soil deposits, there are practical limitations and the correlations presented in the paper should be used with great care.

臺北市土層工程性質之相關性分析

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摘 要

本研究之目的在利用相關性分析建立臺北市土層工程參數之經驗公式，而此相關性分析之結果，將極有助於日後工程建設之規劃及初步設計。對於凝聚性土層，本文提出下列各項參數之關係式：(1) 原始壓縮指數與自然含水量，(2) 原始壓縮指數與初始孔隙比，(3) 壓密係數與液性限度，及 (4) 有效抗剪角與塑性指數。本項研究亦分別提出過壓密比及強度比對深度的剖面圖，並對靜止土壓力係數與有效抗剪角及塑性指數之關係加以研究。對於非凝聚性土層，則提出 (1) 透水性與有效粒徑，及 (2) 有效抗剪角與 SPT N 值之關係式。由於臺北市土層原有的變異性，因此這些關係式有其實際上之限制，而在使用時仍需審慎研判。

INTRODUCTION

Due to rapid development of the Taipei city, many construction projects have been undertaken during the last decade. A significant amount of geotechnical engineering experience has been accumulated during this period, many empirical

relationships correlating the index properties of soils to their engineering properties can therefore be reliably established. Once these correlations are obtained, engineering properties of soils can be estimated through relatively convenient and simple tests. For future construction projects such as the mass rapid transit system and the storm and sanitary sewage system, these correlations are urgently needed for engineering planning and

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preliminary design.

The purpose of this study is to systematically establish empirical correlations for the soil deposits in Taipei. Some of these correlations are the result of a recent study on the geotechnical mapping of Taipei City [11, 18]. These correlations were rigorously evaluated and interpreted before they could be properly incorporated into the current framework. The geology of the Taipei Basin is briefly described below, serving as background material for this paper. Correlations for both cohesive soils and granular deposits are presented. Comparisons of correlations of the Taipei soil deposits to correlations of other soil deposits published elsewhere are also made for reference.

GEOLOGY OF THE TAIPEI BASIN

Taipei city is located in a triangular shaped basin (the Taipei Basin) in the northern part of Taiwan (see Fig. 1). The Taipei Basin is enclosed by the Tatun Volcanic Group, Linkou Tableland and a hilly terrain of Tertiary sedimentary rock [11, 18]. There are three major rivers flowing into the Basin. They are the Keelung River, the Hsintien River, and the Tahan River. These three rivers all merge into the Tamsui River, which flows into the sea at the town of Tamsui.

The Taipei Basin is a tectonic basin which was formed by the settlement of nappes between thrusts in the foothill range of northern Taiwan during the Pliocene and Pleistocene periods. The primary strata in the Taipei Basin are sedimentary deposits of the recent Quaternary period and the bedrock formation of the Tertiary period. Underlying a thin layer of topsoil, about 1 m to 6 m thick, the "unconsolidated" formation in the Taipei Basin can be divided into two major strata. Immediately below the topsoil is the Sungshan Formation, which extends to a depth of about 60 m below the existing ground surface. This stratum consists mainly of a soft, compressible, and slightly plastic clayey silt and silty clay layer, interstratified with fine sand layers containing high silt content. The silt content in the various layers varies from 10 to 95 per cent. Due to the high percentage of silt-size particles, the Sungshan Formation is often referred to as the Taipei Silt Stratum [19]. Underlying the Sungshan Formation is the Chingmei Formation. This is a coarse sand and gravel layer approximately 90 to 130 m in thickness.

This paper concentrates on the study of the Sungshan Formation, which is of primary interest to geotechnical engineers. From the geological point of view, the Sungshan Formation can be generally subdivided into six layers, as shown in

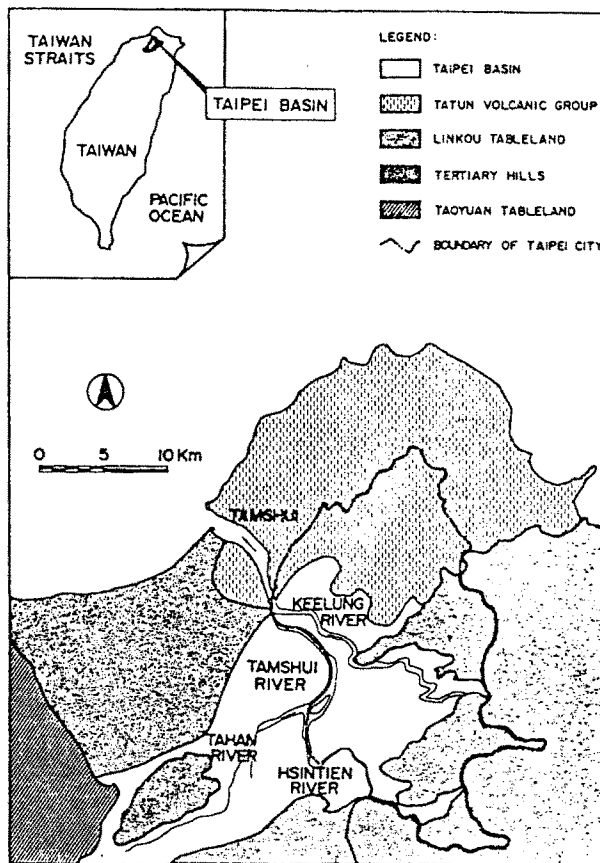


Fig. 1. Geological conditions of the Taipei Basin and its surrounding area [11].

Table 1. The thickness and sequence of these layers vary somewhat from area to area. Those shown in Table 1 are typical soil deposits adjacent to the Tamsui River, referred to as Zone T2 by Moh and Associates, Inc. (MAA) [18]. Three of the six layers are cohesive soils, i. e., layers VI, IV and II, and the other three are non-cohesive, i. e., layers V, III and I. As for the areas along the Keelung River, the silty clay layer becomes the primary layer with very little or no sand sublayers.

Two sets of correlations are proposed in this paper, one for cohesive deposits and the other for cohesionless deposits. At the present time, it is believed that these two sets of correlations are sufficient for the purpose of preliminary estimation. Future studies, however, will entail a more complete investigation of these correlations. They should be aimed at establishing correlations for all of the areas, including the various layers in these areas.

COHESIVE SOILS

1. Stress history

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Table 1. Typical profile of Taipei Sungshan Formation [18]

Sublayer	Soil description	U.S.C.*	Thickness (m)	w_n (%)	w_L (%)	I_p (%)	Gradation				SPT N -value
							Gravel (%)	Sand (%)	Silt (%)	Clay (%)	
VI	Grayish black clayey silt/silty clay	ML, CL	4.5	31.2	35.8	12.9	0	10	58	32	5
V	Gray silty fine sand	SM	10.1	26.3	—	—	1	76	19	4	10
IV	Gray silty clay/clayey silt	CL, ML	9.0	32.1	34.3	12.0	0	8	61	31	8
III	Gray silty fine sand	SM	10.6	23.9	—	—	0	60	34	6	21
II	Gray silty clay/clayey silt	CL, ML	8.4	27.2	30.3	9.2	0	8	67	25	19
I	Bluish gray silty sand with gravel	SM	4.5	20.3	—	—	1	63	29	7	31

* Unified soil classification.

pressure ($\bar{\sigma}_{vm}$) profile has been regarded as the single most important task in predicting long-term consolidation settlements as well as short-term stability problems [12]. In this study, more than 200 incremental one-dimensional oedometer test results were collected to provide an overview of the $\bar{\sigma}_{vm}$ profile of the cohesive soil deposits in Taipei. Casagrande's method [5] was used to estimate the $\bar{\sigma}_{vm}$ in this study. Figure 2 presents the variation of the overconsolidation ratio (OCR) in relation to depth, z , in which OCR is defined as the ratio between the $\bar{\sigma}_{vm}$ and the effective overburden pressure ($\bar{\sigma}_{vo}$). It should be noted that the test results were obtained from projects located in many different areas around the city. Figure 2, then, does not imply a cohesive soil layer 50 m thick which extends from the ground surface to a depth of 50 m. Rather, it shows the characteristics of cohesive soil layers which were encountered at different depths, depending on the borehole locations. That is the reason why Fig. 2 presents a continuous profile rather than three discrete segments corresponding to the 3 distinct layers as shown in Table 1. Figure 2 clearly indicates that the lower deposits are only slightly overconsolidated to normally consolidated, whereas the upper cohesive deposits are more heavily overconsolidated. In order to provide a convenient expression for preliminary estimation, engineering judgement was exercised to propose the following equation to describe the OCR profile as shown in Fig. 2:

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

Due to the data scatter, this equation should be used with great care at shallow depths (say, less than 5 m). It is noted that the estimated OCR is described to decrease continuously with increasing depth. At a depth of 40 m, OCR is only 1.08. From a practical point of view, cohesive deposits below this depth can generally be treated as normally consolidated. It should also be noted that the exact mechanism causing the overconsolidation of the Taipei deposits is not

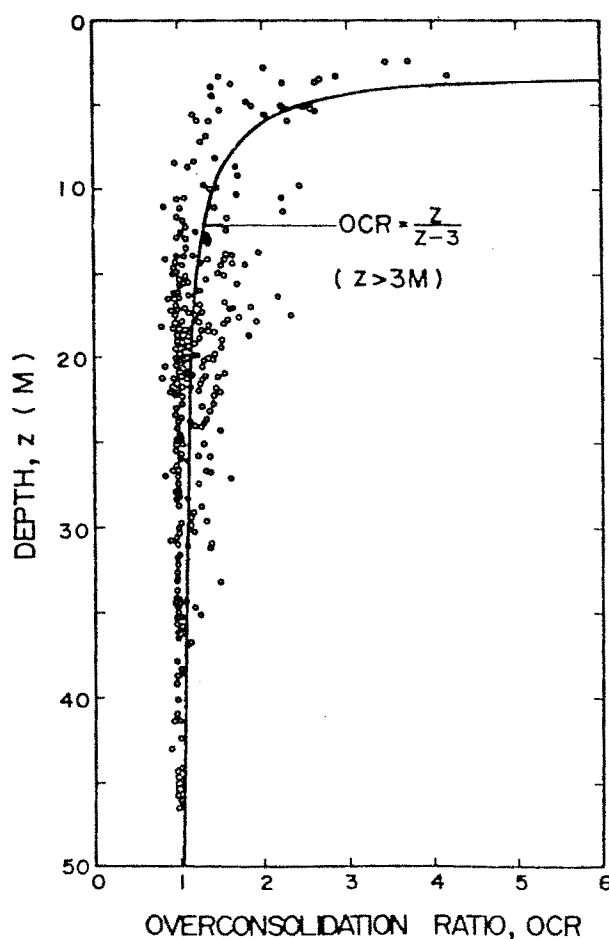


Fig. 2. Relationship between OCR and depth for the cohesive deposits in Taipei.

clearly understood yet. Further study of the geological history of the deposits is required.

2. Virgin compression index

The problem of settlement is one of the major engineering concerns for foundations founded on the cohesive deposits in Taipei. A correlation between the virgin compression index (C_c), which controls the magnitude of the compression in the normally consolidated range, and the natural water

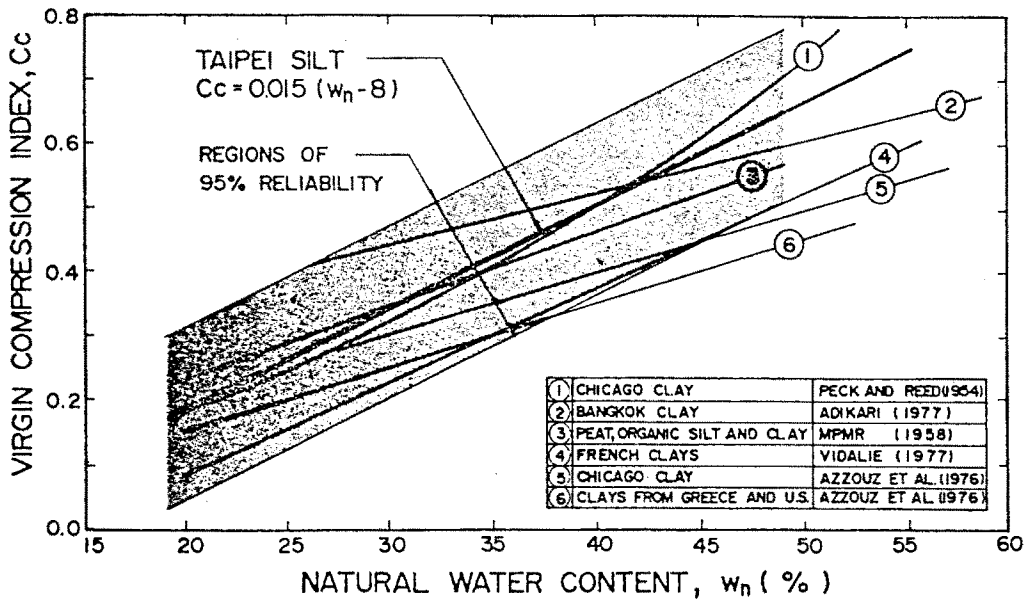


Fig. 3. Relation of virgin compression index to natural water content.

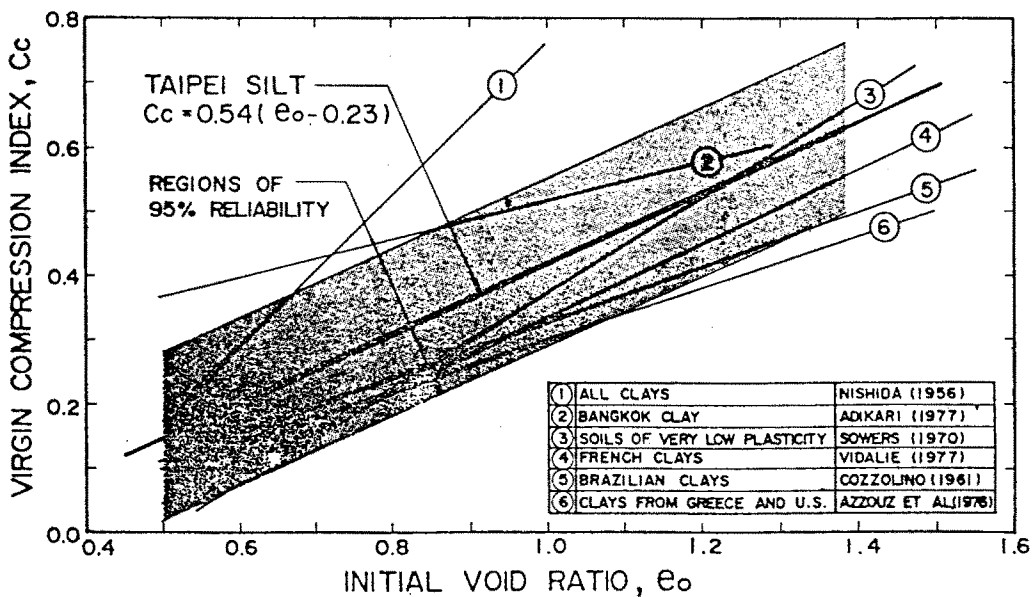


Fig. 4. Relation of virgin compression index to initial void ratio.

content (w_n) was established from 466 sets of conventional oedometer test results. Due to sample disturbance effect, the virgin compression index was corrected according to the method suggested by Terzaghi and Peck [28]. Based on linear regression analysis, the correlation between the virgin compression index and the natural water content can be expressed as follows:

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

where w_n is natural water content in per cent.

In order to illustrate the data scatter, regions of 95 per cent reliability of Eq. (2) are shown in

Fig. 3. Also plotted in Fig. 3 are the correlations for other soil deposits. Among them, the correlation for Chicago Clay proposed by Peck and Reed [23] appears to be very close to that of the Taipei soils.

Since the natural water content is closely related to the initial void ratio (e_o), the correlation between C_c and e_o can also be established at the same level of reliability:

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

Figure 4 presents the $C_c - e_o$ correlations for Taipei soils and other deposits.

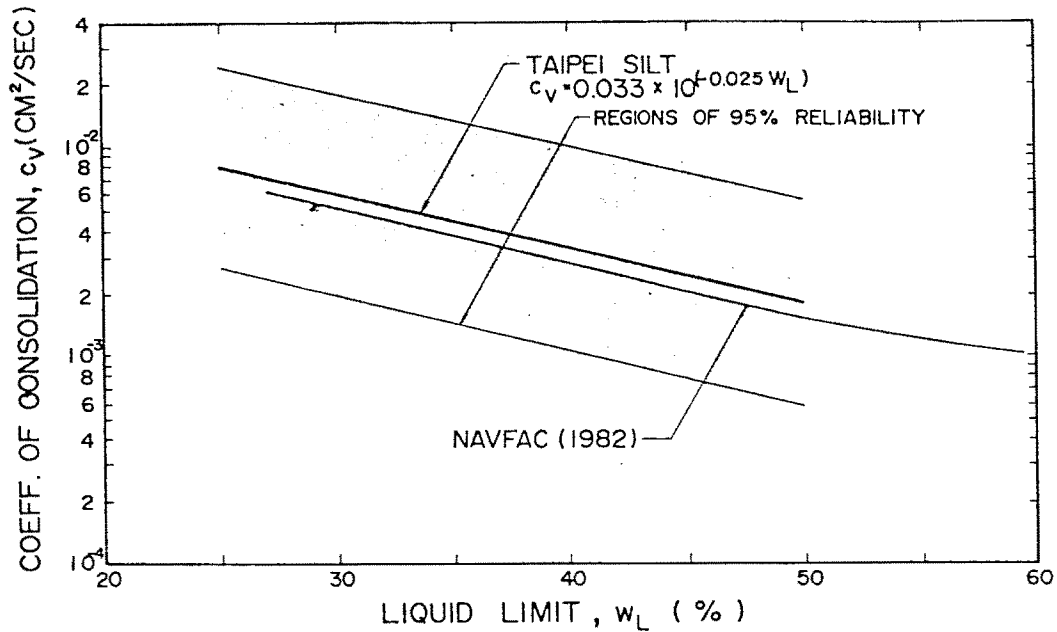


Fig. 5. Relation of coefficient of consolidation to liquid limit.

3. Coefficient of consolidation

Estimates of the rate of consolidation based on Terzaghi's theory require a determination of the coefficient of consolidation (c_v). The log time method [6] was used in this study to estimate c_v from the conventional oedometer test results. Similar to most of the clayey soils, the value of c_v of the cohesive soil deposits in Taipei is approximately constant in the normally consolidated (NC) range. The relationship between the c_v in the NC range and the liquid limit (w_L) can be described as follows:

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

where c_v is in cm^2/sec , and w_L in per cent.

Comparing Eq. (4) with the empirical correlations of U.S. NAVFAC [21], Fig. 5 shows that the correlation for undisturbed NC soils in Taipei is in fairly good agreement with that suggested by NAVFAC.

4. Angle of shearing resistance

The angle of shearing resistance ($\bar{\phi}$), which controls the effective stress-strength relationship, is dependent upon the stress history, anisotropy effect, drainage conditions, and various other factors. For local practice, the isotropically consolidated undrained triaxial compression test with pore pressure measurement (\overline{CIU} test) is widely used for determining the angle of shearing resistance of plastic soils. By running a \overline{CIU} test on normally consolidated samples from the Taipei Silt Stratum, the value of the $\bar{\phi}$ angle determined by either maximum obliquity or maximum deviatoric stress does not differ significantly.

Kenney [13] presented a relationship between $\bar{\phi}$ and I_p in which $\bar{\phi}$ was obtained from drained and undrained triaxial tests with pore water pressure measurements on normally consolidated natural and remolded marine, fresh water, and residual soils. Although his data have considerable scatter, he indicated a definite trend towards decreasing $\bar{\phi}$ value with increasing plasticity for I_p , varying from 8 to 100 per cent. Based upon the available \overline{CIU} test results for Taipei Silt, Eq. (5) is obtained by applying the least square method to correlate (NC range) with $\log(I_p)$:

$$\bar{\phi} = 41.0 - 7.6 \log(I_p) \quad (5)$$

where I_p is in per cent.

The comparison between Kenney's curve and the current study (see Fig. 6) indicates that for I_p , varying between 10 and 30 per cent, these two predictions are very close. However, the correlation coefficient for Eq. (5) is relatively low, i.e., only 0.5. This means that the value of $\bar{\phi}$ may not be accurately estimated from a measurement of I_p only.

5. Coefficient of earth pressure at rest

It is well recognized that a realistic simulation of the stress-strain-strength behavior of a soil element in the ground is difficult to achieve without an accurate estimation of the coefficient of the earth pressure at rest (K_0). Unfortunately, in the last few years, only a few projects in Taipei have carried out tests to determine the value of K_0 . For these projects, K_0 was determined by the procedures recommended by Poulos and Davis [26].

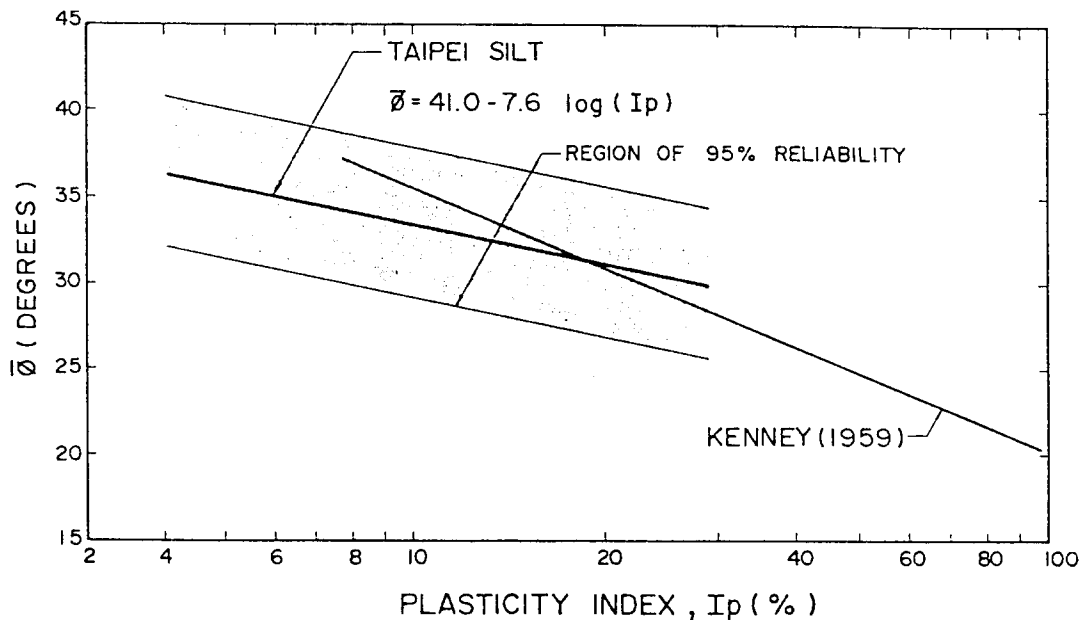


Fig. 6. Relation of angle of shearing resistance to plasticity index.

In 1965, Brooker and Ireland [4] developed a onedimensional compression test cell and auxiliary controls which enabled the measurement of radial stresses under the condition of zero lateral strain. They obtained the values of the angle of shearing resistance for Weald clay, London clay and Bearpaw shale from the available literature, and for Chicago clay and Goose Lake flour from drained direct shear tests. They then proposed an equation to correlate K_s with the effective angle of shearing resistance ($\bar{\phi}$) for these cohesive soils:

$$K_s = 0.95 - \sin \bar{\phi}. \quad (6)$$

This equation, in which $\bar{\phi}$ is obtained from \overline{CIU} tests, has been widely used by geotechnical engineers in Taipei in recent years. However, the validity of this equation has reached a stage of reevaluation.

Figure 7 presents K_s values for normally consolidated plastic Taipei Silt plotted against their angles of shearing resistance. These angles were determined from their corresponding K_s -consolidated undrained triaxial compression tests with pore pressure measurement ($\overline{CK_sU}$ test). In these tests, soil samples were first consolidated to the predetermined K_s state [26] and then sheared to failure in an undrained triaxial compression loading mode. As can be seen, Fig. 7 does not show an obvious relationship between K_s and $\bar{\phi}$ for the cohesive deposits in Taipei. As discussed earlier, \overline{CIU} tests are commonly used in local practice instead of the more complicated $\overline{CK_sU}$ tests. A preliminary study indicates that $\bar{\phi}$ determined from $\overline{CK_sU}$ tests is about 1° to 4° higher than $\bar{\phi}$ determined from \overline{CIU} tests, with an average of 3° . In other words, this implies that $\bar{\phi}$ determined from $\overline{CK_sU}$ tests

should be subtracted by 3° in order to substitute it into any correlation which is based on \overline{CIU} test results. Even if this simple correction is applied, K_s determined from Eq. (6) is still much lower than the measured K_s values. For $\bar{\phi}$ of \overline{CIU} tests varying from 27° to 35° ($\bar{\phi}$ of $\overline{CK_sU}$ varies from 30° to 38° , approximately), Eq. (6) will give K_s value from 0.50 to 0.38, which are relatively low compared with the available test results (see Fig. 7).

Since the available data are limited and scatter significantly, it can be concluded that a correlation between K_s and $\bar{\phi}$ cannot be reliably established at the present stage. More importantly, current study indicates that the direct application of Brooker and Ireland's correlation by using $\bar{\phi}$ from \overline{CIU} tests can be quite misleading. From a practical viewpoint, it is recommended that a range of 0.5 to 0.6 be used for preliminary estimation. Attempts were also made to set up a correlation between K_s and the plasticity index (I_p), but the results were unsatisfactory. A comprehensive study on the correlation between K_s and $\bar{\phi}$ is currently under investigation at MAA.

6. Undrained strength profile

Estimation of the undrained shear strength (s_u) is required in almost all projects in cohesive deposits involving bearing capacity analysis, retaining wall analysis, pile capacity analysis, and so on. For a long time, it has been recognized that the strength of clays is perhaps the most confusing subject area in soil engineering [13]. A complete treatment of undrained strength is well beyond the scope of this paper. Therefore, a simple undrained strength profile is proposed

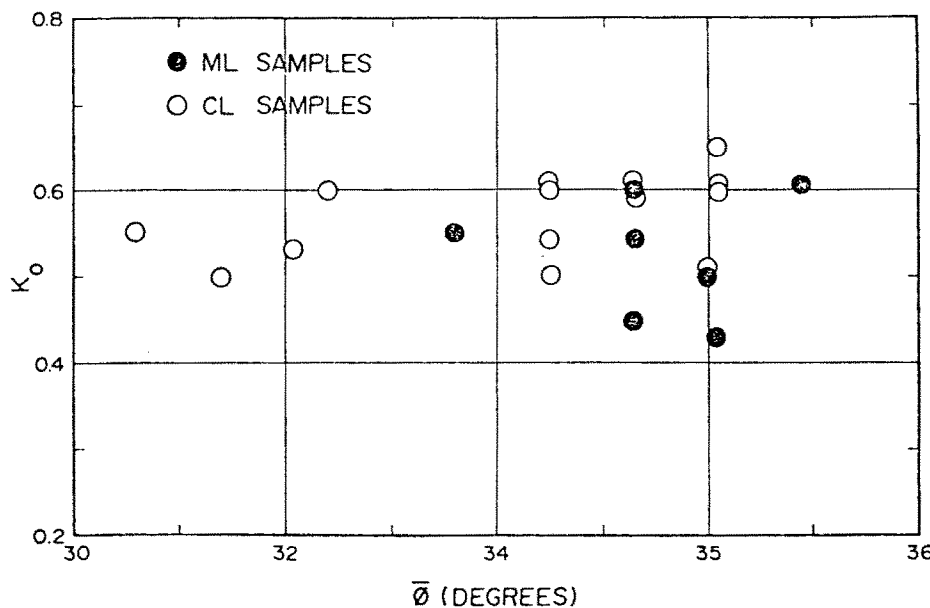


Fig. 7. Variation of K_0 with angle of shearing resistance.

instead. It should be emphasized that the test results were obtained from projects in many different locations around Taipei. Again, a strength profile as shown in Fig. 8 does not imply that the cohesive deposits in Taipei are continuous and not intermixed with sand layers. Rather, it indicates that the test results of cohesive soil samples were retrieved from different depths, depending on the borehole locations.

During the last decade, the saturated unconsolidated undrained 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. More than 120 sets of SUU test results were collected in this study. Figure 8 shows the variation of the normalized undrained shear strength (i.e., the strength ratio, $s_u/\bar{\sigma}_{vo}$), with respect to depth. It indicates that the strength ratio decreases with increasing depth for the upper deposits and remains almost constant for the lower deposits. It is noted that this trend is in accordance with the earlier discussion of the OCR profile (see Fig. 2). Based on a simplified picture of the strength behavior and stress history of the cohesive deposits in Taipei, Eq. (7) is thus proposed in order to provide a preliminary framework for practical use:

$$\frac{s_u}{\bar{\sigma}_{vo}} = 0.21(\text{OCR})^{0.9} \quad (7)$$

in which OCR can be substituted by Eq. (1), and Eq. (7) can be rewritten as follows:

$$\frac{s_u}{\bar{\sigma}_{vo}} = 0.21 \left(\frac{z}{z-3} \right)^{0.9}, \quad z > 3, z \text{ in } m. \quad (8)$$

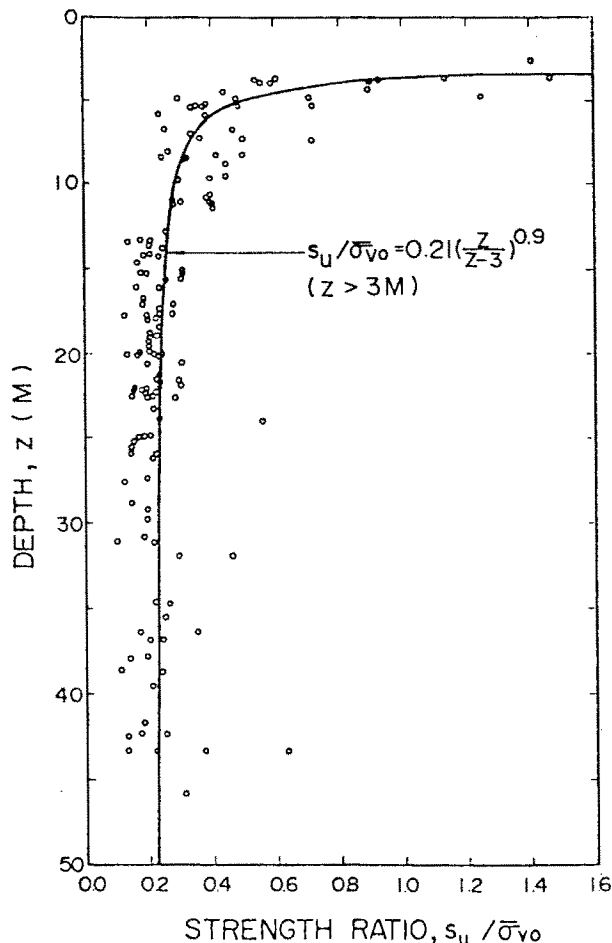


Fig. 8. Undrained shear strength profile of the cohesive deposits in Taipei.

It is interesting to compare the s_u determined from SUU test with that determined from conventional unconsolidated-undrained (UU) test [3],

which is called the "unsaturated" unconsolidated undrained (UUU) test by local engineers because the specimen was not artificially saturated before applying the confining pressure. For the cohesive soils in Taipei, a preliminary study indicates that the undrained shear strength determined from the SUU tests is approximately 75 per cent of that determined by the UUU tests.

It should be pointed out that the selection of type of test used to determine the undrained shear strength in engineering practice is beyond the scope of this paper. Equation (8) is proposed to describe the undrained shear strength profile of the Taipei Silt under the SUU condition of shearing.

COHESIONLESS SOILS

1. Permeability

Permeability (k) is one of the most critical parameters for construction projects, such as open excavation and tunnelling, in granular soils. The accurate determination of permeability is a very difficult task because the commonly adopted laboratory testing method for permeability will most likely yield results no more accurate than an order of magnitude, and field measurements may be significantly different from the laboratory results. Although field determination of permeability often provides a better indication of the average permeability, laboratory measurements can be used to obtain the relationship of permeability to the void ratio of the soil and are thus usually run whether or not field measurements are taken [15]. In Taipei, constant head permeability tests are commonly used. They are performed in triaxial cells in which saturated specimens are consolidated under various confining pressures.

In addition to the properties of the pore fluid, the permeability characteristics of a soil are dependent on particle size, void ratio, composition, fabric, and degree of saturation. These factors are closely related, and the effect of any single factor on permeability cannot be isolated. Therefore, it is very difficult to reliably correlate the permeability to any one of these factors. In order to provide a guideline for the estimation of permeability, this study compiled laboratory results of 55 silty sand samples (Unified Soil Classification: 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. The effective grain size of the soil sample, D_{10} , is defined as that size at which 10 per cent (by dry weight) of the sample is smaller than this grain size. Due to the high silt content of the Sungshan Formation, SP materials are rarely encountered in Taipei. Shown in Figs. 9(a) and

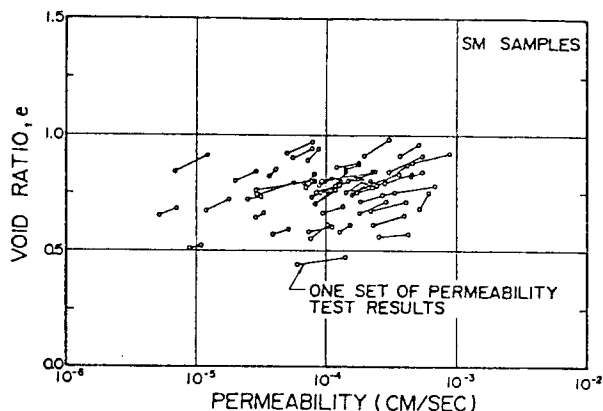


Fig. 9(a). Relation of void ratio to permeability (SM samples).

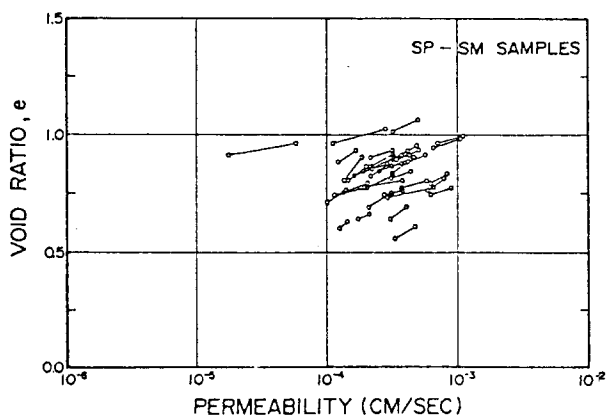


Fig. 9(b). Relation of void ratio to permeability (SP-SM samples).

9(b) are 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. For practical purposes, permeability is commonly correlated with D_{10} . Based on the available test results, Eq. (9) is suggested for the preliminary estimation of the permeability of granular soils in Taipei:

$$k = 19D_{10}^2 \quad (9)$$

where D_{10} is in cm, and k is in cm/sec. It should be noted that statistical analysis indicates that there is no significant difference between the empirical correlations of the SM and the SP-SM materials. Considering the inherent difficulty of permeability measurement, Eq. (9) is also found to be consistent with previous correlations proposed by Hazen [10] and Lane and Washburn [16].

Furthermore, if the variation of permeability with respect to the void ratio change is required, the parameter C_x , defined in the following equation, can be used to describe the relationship:

$$C_s = \frac{de}{d(\log k)} \quad (10)$$

For both SM and SP-SM deposits in Taipei, C_s is approximately equal to 0.28.

2. Angle of shearing resistance

The shearing resistance of granular soils is highly dependent on the size, shape, and gradation of the particles making up the soil. For a given soil, the strength is a function of the void ratio, confining pressure, ratio of loading, and so on. Since it is extremely difficult to obtain an "undisturbed" cohesionless sample without changing its original porosity, extensive application has been made in practice by correlating the angle of shearing resistance with the blow count obtained from standard penetration test, i.e., the N -value.

In general, the Mohr's envelope is curved for granular soils tested under a wide range of consolidation stresses. Nonetheless, for most engineering problems, the stresses are small enough that it is reasonable to use straight-line approximations without cohesion intercepts. In this study, the angles of shearing resistance, $\bar{\phi}$, of the cohesionless soils were determined from the results of direct shear tests conducted under relatively small normal stresses. It is well recognized that N -values should be corrected according to effective overburden pressure [9]. Therefore, excluding gravelly sand deposits, a correlation between the corrected N -value, i.e., N_c [25], and the angle of shearing resistance is obtained as follows:

$$\bar{\phi} = 28 + 1.3\sqrt{N_c} \quad (11)$$

where $N_c = 0.77 \log(195/\bar{\sigma}_{vo})N$; $\bar{\sigma}_{vo}$ in tons/m².

Figure 10 shows the correlation for the granular soils in Taipei and its regions of 95 per cent reliability. It indicates that most of the test results used in the correlation have N_c varying from 5 to 25. Figure 10 also shows that, for a given N_c value, the range of variation for the estimated $\bar{\phi}$ value is approximately 4°. Figure 10 also presents some other correlations [10, 17, 24] between $\bar{\phi}$ and SPT results which are commonly used by many engineers. It should be noted that the N -values used in those correlations were not corrected for effective overburden pressures. A direct comparison between these correlations and Eq. (11) cannot be made. Nonetheless, it is still recommended that these correlations be replaced by Eq. (11), which is established from a local data base with corrected SPT N -value.

CONCLUSIONS

This study provides a number of valuable empirical correlations for the soil deposits in

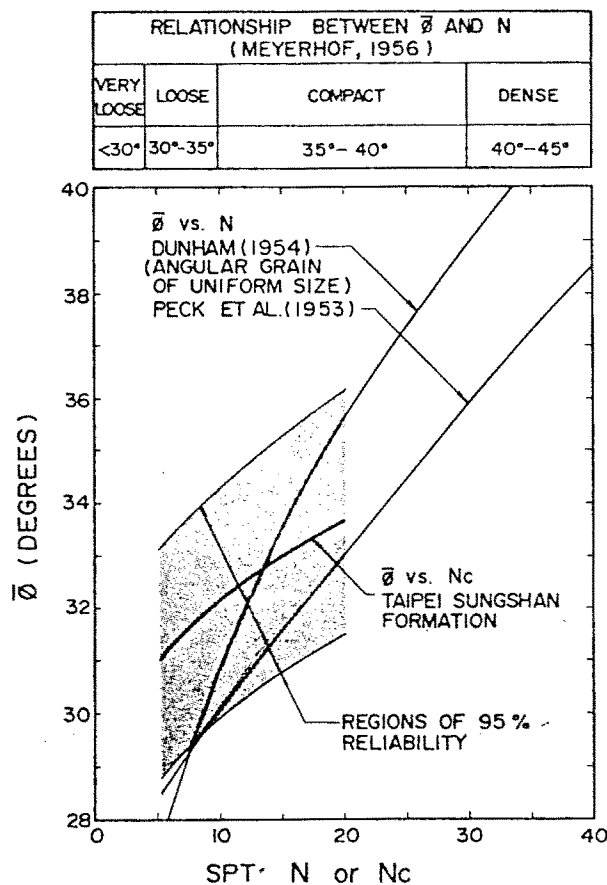


Fig. 10. Relation of angle of shearing resistance to SPT N -value.

Taipei. Except for a few parameters, these correlations have covered most of the commonly used engineering properties required in geotechnical design. This paper also presents the profiles of the OCR and the strength ratio which are believed to be essential in obtaining an overall picture of the engineering characteristics of the cohesive soil deposits in Taipei. Some of the correlations requiring further investigation have also been identified.

It should be noted that the most distinguishing feature of the Taipei Silt Stratum is the high percentage of silt-size particles. For plastic soils, silt content is always higher than 60 per cent. For granular deposits, silt-size particles may constitute 20 to 40 per cent of the total weight of the sample. This is the reason why the correlations for both the cohesive and cohesionless soils in Taipei differ from the correlations for many other deposits around the world with relatively pure clay or sand-size particles.

As mentioned above, the exact mechanism causing the overconsolidation of the upper deposits of Taipei Silt is still unknown. Also needed is a thorough understanding of the depositional characteristics of the three major rivers and their

influence on engineering properties. More detailed investigation of the geological conditions of the Taipei Basin need to be conducted from a geotechnical engineering viewpoint.

Obtaining "undisturbed" samples of cohesionless soil is always difficult. In Taipei, it is also difficult to obtain completely "undisturbed" samples of cohesive deposits because of the high silt content. Sampling practice has been greatly improved during the last decade, and piston sampling is widely accepted as routine. Some special techniques have been used on various occasions. However, during the period of data collection for this study, many of the test results were discarded due to sampling disturbance. It is believed that more investigations on sampling technique and its associated disturbance effects are urgently needed. In order to better understand the engineering characteristics of Taipei Silt, more advanced laboratory tests should be carried out on large block samples or samples made from reliable resedimentation techniques.

In-situ testing has been extensively used here in the last few years. The cone penetration test (CPT) is becoming increasingly popular for its unequalled ability to delineate soil stratigraphy and to measure soil properties rapidly and continuously. Therefore, it would be valuable to correlate the field measured parameters, especially from CPT results, to the engineering properties for the soils in Taipei. Furthermore, more in-situ test results should be incorporated into future study. For example, field permeability test results should be compared with laboratory permeability test results in order to improve the current correlation of permeability for practical use.

It should be emphasized that the correlations presented in this paper must be used with great care. These correlations are obtained from a fairly large yet still limited number of reliable test results. They should be regarded as "empirical" with a need for further improvements when more data become available. Geotechnical engineers should be aware that the Taipei Silt Formation is not uniform and its characteristics are complex. The correlations presented in the paper should be useful for planning and preliminary design purposes. They are not recommended for final and detail design.

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