Doctoral Dissertation

Characteristics of Clayey Silts with Low Plasticity Focusing on the Undrained and Partially Drained Shear Strengths and New Approach to Soil Classification

非排水及び部分排水せん断強度に着目した低塑性粘土質シルトの特性と土の分類に関する新たな提案

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SYNOPSIS

In this thesis, characteristics of clayey silts with low plasticity located at western site of Korean peninsula were analyzed focusing on the evaluation of fully undrained and partially drained shear strengths. Several points of soil classification using Casagrande's plasticity chart were discussed and revised Polidori's plasticity chart were suggested through verification of applicability using Korean four different marine clayey soils.

Firstly, it was carried out to investigate the applicability and problems of soil classifications based on the plasticity charts for natural marine soils taken from four different Korean coastal areas, and also through comparison with the plasticity charts in Casagrande and Polidori. The liquid limit and plastic limit of fine-grained soils could be altered according to both the percentages of the silt and clay fraction, therefore, the silt fraction is proposed to be almost as important as the clay fraction in relation between the Atterberg limits and these compositional fractions. In conclusion, Polidori's revised plasticity chart based on the CF=30% borderline proposed by this study appears to be more appropriate for the classification of silt or clay. This is because no distinction exists above the A-line where the natural soils taken from the four different coastal areas lie on the empirical plasticity chart proposed by Casagrande, and these soils were classified as silt by Polidori's plasticity chart.

Secondly, a series of laboratory and in-situ tests were carried out to evaluate undrained strength of clayey silts with low plasticity at Incheon and Gunsan sites. The applicability of UC in determining the undrained strength of these soils was examined. It can be concluded that UC test is not suitable for evaluating the undrained shear strength of low plastic soils. Therefore, in evaluating the undrained shear strength of soil with a low plasticity index, effective confining pressure that corresponds to typical marine clay should be applied to a soil specimen before shearing in order to compensate for the lost residual effective stress. In this case, the CIU

(recompression test) proposed by Tsuchida and Mizukami can be quite useful in duplicating the in-situ shear strength of a soil. The undrained shear strength normalized by the yield consolidation pressure, s_u/p'_c, are presented for four coastal sites, that is, Busan/Gwangyang and Incheon/Gunsan having the characteristics of high and low plasticity, respectively. The field vane shear strengths, $s_{u(FVT)}$, were compared with the unconfined compressive strength, $q_u/2$ which has been used as a representative testing method in Korea.

Many researchers have suggested that the undrained shear strength normalized by the yield consolidation pressure, s_u/p'_c depends on I_p. However, the undrained shear strength normalized by the yield consolidation pressure, s_u/p'_c is in the range of 0.25 to 0.35, independently of the plasticity index, I_p except for s_u/p'_c, using $q_u/2$ values in case of soils having a low plasticity such as Incheon and Gunsan clayey silts.

Bjerrum's correction factor has been commonly applied to evaluate the mobilized undrained shear strength using the field vane test in Korea. However, the corrected undrained shear strengths using Bjerrum's correction factor, including Morris and Williams' method were considerably underestimated for Korean marine clay when compared with the $q_u/2$ values that have been used as the mobilized undrained shear strength for practical design in Korea.

Thirdly, partial drainage characteristics of clayey silts with low plasticity were analyzed using laboratory and in-situ tests. When estimating whether low plastic soil is under partially drained conditions or not, the approach recently proposed by Schnaid et al., can be quite useful. This approach is based on plotting the normalized cone resistance, Q_t, versus the pore pressure parameter, B_q in combination with the strength incremental ratio s_u/σ'_{vo}, from the CPTU data. It is evident that two-thirds of the results fall in the range where $B_q < 0.3$, corresponding to the domain in which partial drainage prevails when testing normally consolidated soils at a standard rate of penetration (2cm/s).

Fourthly, the comparison of stability analysis regarding fully undrained and partially drained concepts for a low plastic soil, respectively, were carried out to check economic feasibility of each design concept. The replacement depths were estimated by applying a series of internal friction angles to consider its sensitivity under partially drained conditions. The estimated replacement depths (Compulsory replacement method) and replacement ratio, a_s (SCP method) were drastically changed with a slight change of internal friction angles. Accordingly, selecting appropriate strengths in stability analysis is the most important thing. According to back analysis from field measurement data at completed construction sites applied compulsory replacement method and the analysis of CPTU based strengths, the internal friction angles under partially drained conditions ranged from $\varphi'=5^\circ$ to $\varphi'=15^\circ$. At the present stage, it is very difficult to select appropriate the value of φ' to design improvement of clayey silts under partially drained conditions beneath gravity type structures such as breakwaters, quaywalls and revetments. Therefore, when applying partially drained concept in designing soft ground improvement, it is recommended that the lowest values of φ', $\varphi'=3\text{-}5^\circ$ be applied to stability analysis from the view point of safety side.

From now on, further in-depth study should be carried out regarding the applicability of each design concept focusing on the evaluation of partially drained strength (φ') of the intermediate soil through detailed analysis of dissipation trend in CPTU data.

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CHAPTER 1
INTRODUCTION

1.1 BACKGROUND AND OBJECTIVES

In most cases of soil structure design, soil is classified for simplicity as either sand or clay based on whether the soil structure is dominantly constituted of particles or fines content, and the appropriate design methods are used to properly represent the soil's behavior. For example, when an external load is applied to cohesionless soils such as sands and silts, contact forces arise between the soil particles from the friction between these particles as they make contact with one another.

For sandy soil, the state of its density (or packing) is important since the state of the forces is transmitted between the soil particles. The shear strength (τ) of sandy grounds may be calculated by using the following equation:

$$\tau = \sigma'_n \tan\varphi' \tag{1.1}$$

where σ'_n and φ' are the effective normal stress and the internal friction angle, respectively.

Geotechnical engineers prefer to perform total stress analysis for clayey soils, particularly for short-term analysis, because there is uncertainty in predicting the excess pore water pressure and the soils have a practically undrained nature due to very low permeability. In such cases, τ can be expressed in terms of the undrained shear strength s_u ($\tau = s_u$). However, most soils in the natural state are generally composed of a combination of sand, silt, clay, etc. and it is difficult to simply classify such soils into either sand or clay because they possess both properties of sand and clay. These kinds of soil are called intermediate soils, and the design code for the construction of port and harbor facilities in Japan classifies the ground into clayey or sandy soil according to a soil size fraction of 0.074 mm present in the ground. When more than 80% of a soil is composed of sand-sized particles, the soil is classified as sandy ground, and an effective stress analysis is applied to the ground stability analysis. In such an analysis, the soil parameter φ' is generally estimated by using the Standard Penetration Test (SPT). On the other hand, when less than 50% of the soil is sand, the soil is considered to be clay, and therefore the total stress analysis is applied to the design of various facilities on such ground. Soils with between 50% and 80% of sand-sized content are called intermediate soils because these show an intermediate behavior between sand and clay. For intermediate soils, if the permeability of soil is less than 10^{-4} cm/s, the stability analysis and the evaluation of s_u follow those of clayey soils, if the permeability of soil is more than 10^{-4} cm/s, the stability analysis and the evaluation of shear strength follow those of sandy soil (Japanese Port Association, 2007). In addition, Geotech Note No. 2 (Japanese Geotechnical Society, 1992) states that ground with a plasticity index between NP and 25%, permeability between 10^{-7}cm/s and 10^{-4}cm/s, and a coefficient of consolidation between 144cm^2/day and 14,400cm^2/day can be classified as intermediate soils. Unfortunately, until now, a classification code for intermediate soils did not exist in Korea.

In Japan, the undrained strength, s_u, has been generally evaluated by using the Unconfined Compression (UC) test, and it is taken to be half of the unconfined compression strength obtained from such a test. However, at present, most countries, with the exception of Japan and Korea, seldom use the UC test to evaluate the s_u of cohesive soils. For example, according to Eurocode 7.2 (2007), the UC test is prescribed as an index test, and reports on the soil investigations at Bothkennar (Hight et al., 1992) provide a good example of this fact, since no UC test was reported in these investigations. Although the UC test has been severely criticized and its validity has been questioned by various researchers, its use has been well established for Japanese soils in which Unconfined Compression tests yield reliable results, provided that samples of good quality are used (Tsuchida and Mizukami, 1991). Therefore, the UC test and the Standard Penetration Test are some of the most common tests used for clayey and sandy soils, respectively.

Extensive geotechnical data obtained from laboratory and in-situ tests have been accumulated in a database containing UC test results. The reason for this is to provide an automatic balance, or so-called lucky harmony, among the many factors that contribute to over-estimation and under-estimation of the actual strength value. These factors that mainly affect the UC test results are the sample disturbance, strain rate effect, and anisotropy of the strength. The undrained strength, which is to be used for the design of the structure, is presented by the following equation, in which these factors are used:

$$s_u = (q_{u(mean)}/2)\ c_1 c_2 c_3 \tag{1.2}$$

where

$q_{u(mean)}$: mean of the unconfined compression strength

c_1: correction coefficient for the reduction of the strength due to the sample disturbance

c_2: correction coefficient for strength anisotropy

c_3: correction coefficient for strain rate.

According to Tsuchida (2000), the unconfined compression strength in an in-situ effective stress condition is about 30% less than that of an ideal sample. Accordingly, the value of c_1 will be 1.0/0.7 = 1.43. The extension strength of clay is about 70% of the compression strength, and since compression is the mode of shearing for q_u, the correction coefficient for the anisotropy, c_2, will be (1.0 + 0.7)/2 = 0.85. Many studies have been carried out on the effect of the strain rate on the undrained strength of clay, and an important problem is how much the strain rate is to be considered for the practical design of earth structures. Ladd and Foott (1974) recommended a rate ranging from 0.008 to 0.015%/min, and Bjerrum suggested a rate of 2 to 3×10^{-5} %/min. According to the case studies reported by Nakase (1967), it took only a few to several hours from the beginning of movement to the completion of a failure for most of the observed slip failures of port facilities to occur. A strain rate of 0.01%/min seems to be adequate for the stability analysis, since it approximately corresponds to a few to several hours. It is assumed that the strain rate to be used for the design is 0.01%/min; therefore, the conventional strain rate of the Unconfined Compression test, 1.0%/min, is 100 times faster, and the correction coefficient, c_3, for the difference of the strain becomes 0.85 based on the change in the undrained strength, with a strain rate obtained from triaxial compression and extension tests for marine clays of various plasticities (Hanzawa, 1977; Tsuchida et al., 1989). Therefore, the following equation can be obtained:

$$c_1 c_2 c_3 = 1.43 \times 0.85 \times 0.85 = 1.00 \quad (1.3)$$

Based on the above relationships, $q_u/2$ has been considered to be an accurate value of the mobilized undrained strength. Basically, in Korea, while the practical design for soft clayey ground has been carried out by using a $q_u/2$ value, the results of the Unconsolidated Undrained (UU) test, Field Vane Test (FVT), and CPTU test are used as accessorial methods. However, the strength of the intermediate soil, as evaluated through a UC test, is likely to be considerably underestimated.

When a soil specimen is retrieved from the ground and is exposed to the atmosphere, part of the in-situ effective stress in the specimen remains in the form of negative pore pressure. This negative pore pressure is called residual effective stress (p'_r). Even though the specimen is tested under unconfined conditions, the residual effective stress acts as a confining pressure. However, if the value of p'_r in the specimen is reduced due to a disturbance of the specimen during sampling, handling, extruding, or trimming, then the $q_u/2$ value will be reduced due to swelling. According to Tanaka et al. (1996), the order of p'_r for high-quality samples is around 1/5 to 1/6 of the in-situ vertical effective stress (p'_{vo}) for normally consolidated and slightly over-consolidated clays. Tanaka et al. (2001) investigated two soils with a low plasticity: intermediate soil from Ishinomaki, Japan and lean clay from Drammen, Norway. They suggested that the p'_r values measured for the Ishinomaki intermediate soil and the Drammen clay were much smaller than those measured for ordinary clay, for which p'_r is approximately 1/6 p'_{vo}. Accordingly, the loss of the residual effective stress (p'_r) is too significant to be compensated for by factors overestimating the strength. Therefore, the validity of the test for clayey soils is not applicable to intermediate soils. Until now, the UC test has been invariably carried out to evaluate the undrained shear strength of such soils, without giving any consideration to alternative methods.

Nakase et al. (1972) proposed a method to correct the unconfined compression strength of intermediate soils according to the content of sand-sized particles or the I_p (Low Plasticity) value. For many Japanese soils, the correlation between I_p and the content of sand-sized particles has been conducted. However, when this method has been applied to correct the $q_u/2$ of intermediate soils, the value has been reported to be considerably underestimated. Therefore, it is possible to make a false judgment about the design of ground treatment work that supports structures on such soils and thereby to cause a waste of large amounts of money.

In this study, a series of laboratory and in-situ tests, such as CPTU and FVT, were carried out to evaluate undrained strength of soil samples obtained from Incheon and Gunsan, Korea. The suitability of UC testing was examined with respect to determining the undrained strengths of these soils. An appropriate method to evaluate the design parameters for low I_p soils was then discussed. In the design of soft ground improvements, the strength incremental ratio (s_u/p'_c) is one of the most important factors used to evaluate the appropriate time of staged embankment loading. In Korea, Skempton's equation has been widely used to evaluate strength incremental ratio. Therefore, the field applicability of this equation to clayey silts with low plasticity was reviewed by comparing its results with those obtained from laboratory and in-situ tests.

And, the drainage characteristics of clayey silts with low plasticity in Korea were estimated by using the piezocone penetration test (CPTU) and back analysis results (based on where bearing capacity of soft ground equals the embankment load when applying compulsory replacement method) including UU-tests (Kim et al., 2010, Baek et al., 2014). Because the undrained or partially drained shear strength should not be larger than the fully drained shear strength, in every possible case, based on the long term stability, the estimation of the shear strength under partially drained conditions has been discussed.

The comparison of stability analysis regarding fully undrained (UC and recompression tests) and partially drained concepts for clayey silt with low plasticity was carried out to evaluate applicability of the above approaches by using compulsory replacement method and SCP method, respectively.

Meanwhile, the classification of soil types is important on the basis of the utilization of data obtained from soil analyses as well as in establishing a communicative language connecting the engineers in the field. The classification of soils, of course, does not eliminate the requirement of a detailed analysis of soil laboratory experiments that measure the engineering properties of the soil. However, it is a fact that, properties of engineering are parallel to properties of indices and classification of soils. Therefore, an engineer can know how to solve problems by knowing how to classify the soil at a construction (Holtz and Kovacs, 2002).

The classification of coarse-grained soils as gravel and sand is usually carried out by a sieve analysis. The grouping of fine-grained soils as silt and clay in almost all cases is performed by the use of Atterberg limits. Currently, the distinction between coarse and fine-grained soils is made by a No. 200 sieve or a grain size of 75 μm, based on the Unified Soil Classification System (USCS). Early interest in the various modes by which a fine grained soil interacts with moisture content led to the development of the 'soil consistency concept'. Largely, through the work of Atterberg and Casagrande, the Atterberg limits and related indices have become characteristics of assemblages of soil particles. The Atterberg limits have been correlated with properties such as swelling, shrinkage, compressibility, permeability and shear strength. These correlations, though empirical, could be developed into a rational approach if the mechanisms controlling the Atterberg limits and the mechanical properties of the soil are carefully considered (Sridharan et al., 1988). The data that define the plasticity of soils (Atterberg, 1911) can be included on Casagrande's plasticity chart (1932, 1948, 1958), in which the plasticity index, PI, is plotted against the liquid limit, LL. For the finest soil component (below the No. 200 sieve), the terms "silt" and "clay" are used, respectively, to distinguish materials exhibiting lower plasticities from those with higher plasticities. Based on Casagrande's plasticity chart, the material passing through the No. 200 sieve is considered as "silt" if the liquid limit and plasticity index plot below the A-line on the plasticity chart, and as "clay" if the liquid limit and plasticity index plot above the A-line on the chart (all Atterberg limits tests are based on material passing through a No. 40 sieve). The foregoing definition holds for inorganic silts and clays and for organic silts, but is not valid for organic clays, since these latter soils plot below the A-line.

On the other hand, Polidori (2003) questioned the position of clays above the A-line on Casagrande's plasticity chart, and reversely was doubtful about the placing of silt-size material below the A-line as well as the significance of the distance of points from the A-line. The extensive study conducted by Polidori (2003) based on the experimental data from the literature and his experimental results, demonstrated a plasticity chart, which was completely different from that of Casagrande. In fact, the respective positions of the silt and clay zones were reversed, with the argument that, because Casagrande's chart was defined empirically without considering the clay content of soils, the clay and silt zones on Casagrande's chart are not accurate.

For this reason, Polidori (2003, 2007) suggested a new plasticity chart based on the Atterberg limits of artificial soil mixtures. That is, the clay fraction, CF (below 2 μm) of each soil was lowered in successive steps by adding sand to obtain changes of 10% in weight of the CF down to a minimum of 10%, or the Atterberg limits of the mixtures with sand were calculated on the basis of the data for pure minerals. It then becomes necessary to verify the applicability of his new plasticity chart to natural common soils.

In this study, through a comparison of the plasticity charts by Casagrande and Polidori, the applicability and problems of soil classification based on both plasticity charts for natural marine clayey and silty soils in Korean were investigated and a new method was developed.

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CHAPTER 2
LITERATURE REVIEW

2.1 CLASSIFICATION OF FINE-GRAINED SOILS WITH PLASTICITY CHARTS

2.1.1 COMPARISON OF PLASTICITY CHARTS BETWEEN CASAGRANDE AND POLIDORI

The liquid limit and plastic limit are determined by relatively simple laboratory tests and engineers have used the tests extensively for finding the correlation of several physical soil parameters as well as for soil identification. As shown in Fig. 2.1, the date that define the plasticity of soils (Atterberg, 1911) can be included on Casagrande's (1932, 1948) plasticity chart, in which the plasticity index, I_p is plotted against the liquid limit, LL. This chart is divided into two zones separated by the A-line [$I_p = 0.73$ (LL-20)], allowing us to distinguish the points that lie above the A-line (inorganic clays, C) from the points that lie below the line (silty soils, M). All the plasticity charts reported by the various standards for classifying fine soils have this characteristic in common. The A-line was defined by Casagrande (1948) empirically on the basis of experimental evidence. According to the ASTM standard (D2482), both inorganic and organic (O) silts and clays are classified as low (L) or high (H) plasticity according to whether their liquid limit value is lower or higher than 50, respectively. Finally, the low plasticity silty clay group (CL-ML) is defined by plasticity index values between 4 and 7. Polidori (2003) proposed new plasticity chart using the consistency limits reported in the literature. This research was born out of the need to answer the following questions:

(a) Why do inorganic soils represented on Casagrande's plasticity chart, with clay fractions (CF < 2 μm) that are smaller than their silt and/or sand fractions (2-425 μm), lie above the A-line in the clay zone?

(b) Conversely, why does pure kaolinite lie below the A-line in the silt zone?

(c) What is the significance of the distance of the points plotted on Casagrande's plasticity chart from the A-line?

He investigated the index properties that the liquid limit is proportional to the percentage of clay (< 2μm) for clay percentages that are not too low (Seed et al. 1964b; Nagaraj et al. 1987; Tan et al. 1994; Kumar and Muir Wood, 1999). The regression line on a graph of liquid limit against clay percentage passes through the origin of the axes. The same occurs for both the plastic limit PL and, as a consequence, for the plasticity index (Seed et al. 1964b). Seed et al. concluded that the linear correlation of the liquid and plastic limits with the clay content holds until the volume of the water-clay system becomes greater than the voids of the non-clay fraction in the mixtures. Polidori (2003) calculated the Atterberg limits of the mixtures with sand on the basis of the data for pure minerals, assuming that, as reported by Seed et al. (1964b), the liquid and plastic limits are proportional to the clay contents, as follows; $LL = (C/100) \times LL_c$ and $PL = (C/100) \times PL_c$ where C is the percentage < 2μm, LL_c is the liquid limit of the clay fraction only, and PL_c is the plastic limit of the clay fraction only. Polidori (2003) has considered mixtures with a sand content ranging from 10% to a maximum value of 70%. In Fig 2.2, the data from Mesri and Cepeda-Diaz, 1986 and calculated as reported by Seed et al. (1964b) are plotted on Casagrande's plasticity chart. All the plasticity index values of the montmorillonite mixtures lie in the high plasticity clay zone near the U-line, whereas pure kaolinite lies in the low plasticity silt zone. It can be maintained that the mixtures composed mainly of sand (> 50%) also lie in the clay zone when plotted on Casagrande's plasticity chart. Polidori (2003) concluded that, for montmorillonite, the value of the plasticity index corresponding to 100% CF constitutes an outer edge of the zone of the mixtures with high clay contents (≥ 50%), and the point corresponding to the mixture with 50% clay marks the boundary between the zone of mixtures with a high clay content (CF ≥ 50%) and the area of mainly sandy mixtures (CF < 50%). The same trend can be made regarding the kaolinite data. Connecting the kaolinite data points with the data points of montmorillonite based on the same percentage of clay contents, the zones where mixtures with the same CF lie are defined in Fig. 2.3. The lines corresponding to 100% and 50% CF, designated by Polidori (2003) as the C-line and the 0.5C-line, respectively, define the zone of mixtures with CF ≥ 50%. The zone of mainly sandy mixtures (CF < 50%) is found above the 0.5C-line. It differs from Casagrande's plasticity chart, whose positions are reversed when compared with Polidori's plasticity chart. Polidori (2003) emphasized that a mixture with higher silt and/or sand contents, with the same liquid limit, shows an even higher plasticity index because it must contain clay minerals that are increasingly expandable as the clay content decreases. In other words, for a given liquid limit value, a drop in the clay content is accompanied by an increase in the plasticity index, and as a consequence the ratio (A = PI divided by percentage < 2 μm), which defines the activity of clay minerals (Lambe, 1951; Skempton, 1953), increases.

If the soils lying above the A-line in the clay zone have higher plasticity index values than the silts below the A-line for a given liquid limit value, Casagrande's plasticity chart can be valid. As PI = LL – PL, the values of the plastic limit of the clays should be less than the values for silt with the same liquid limit, contrary to what has been shown in previous studies (Polidori, 2003; Seed et al. 1964b). In order for the silt zone to lie under the clay zone (Casagrande's plasticity chart), the values of the plasticity index of the mixtures (Fig. 2.3) with CF < 100% should lie under the C-line. In this case, the values of the plasticity limit of the mixtures plotted in Fig. 2.3 should increase as the clay content decreases, contrary to what occurs. As shown in Fig. 2.4, Polidori (2003) proposed the new plasticity chart based on Fig. 2.3. On the new plasticity chart, for inorganic soils, the distance of a point from the C-line should be inversely proportional to its clay content. It means that the pure clay minerals lie on the C-line. The inorganic soils that lie on the 0.5C-line have a CF = 50%, whereas those that lie at the greatest distance from the C-line, near the U-line, have the lowest clay contents.

Polidori (2007) suggested the revised plasticity chart calibrated using additional experimental data (Fig. 2.5). The main things are the intercept value of the 0.5c-line and the C-line are extended to the W_L axis, to fully designate the clay zone where the inorganic soils with platey clay minerals lie.

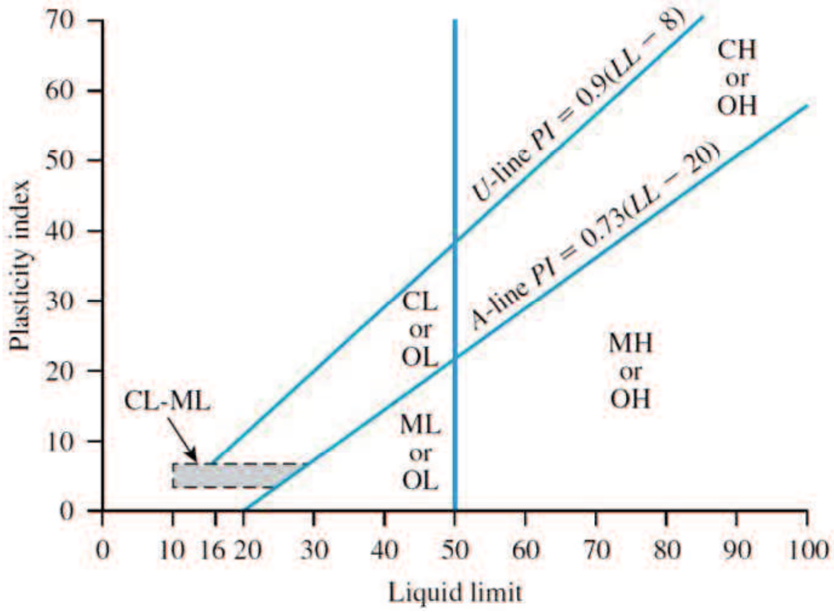


Fig. 2.1 Casagrande’s plasticity chart

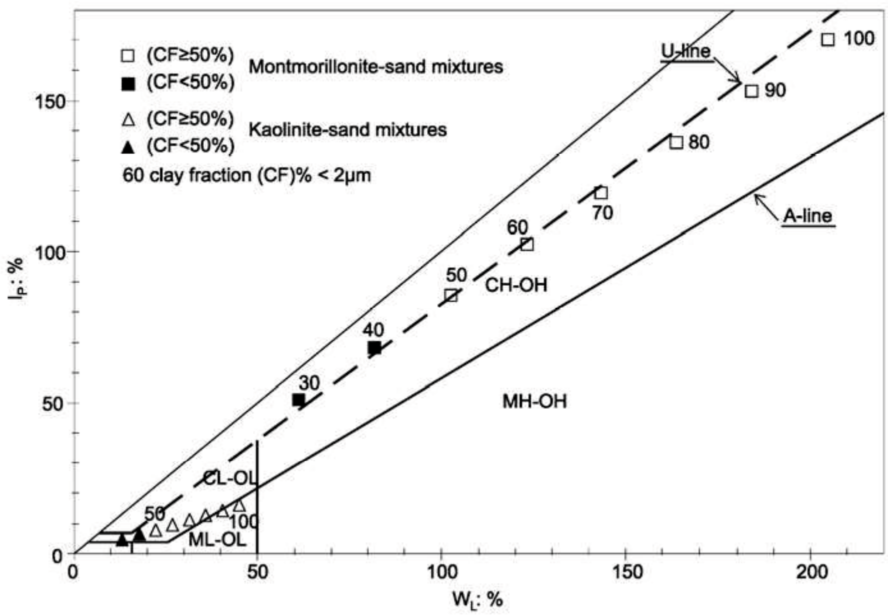


Fig. 2.2 Location on Casagrande’s plasticity chart of pure kaolinite and montmorillonite clay samples and their respective mixtures with fine silica sand (Polidori, 2003)

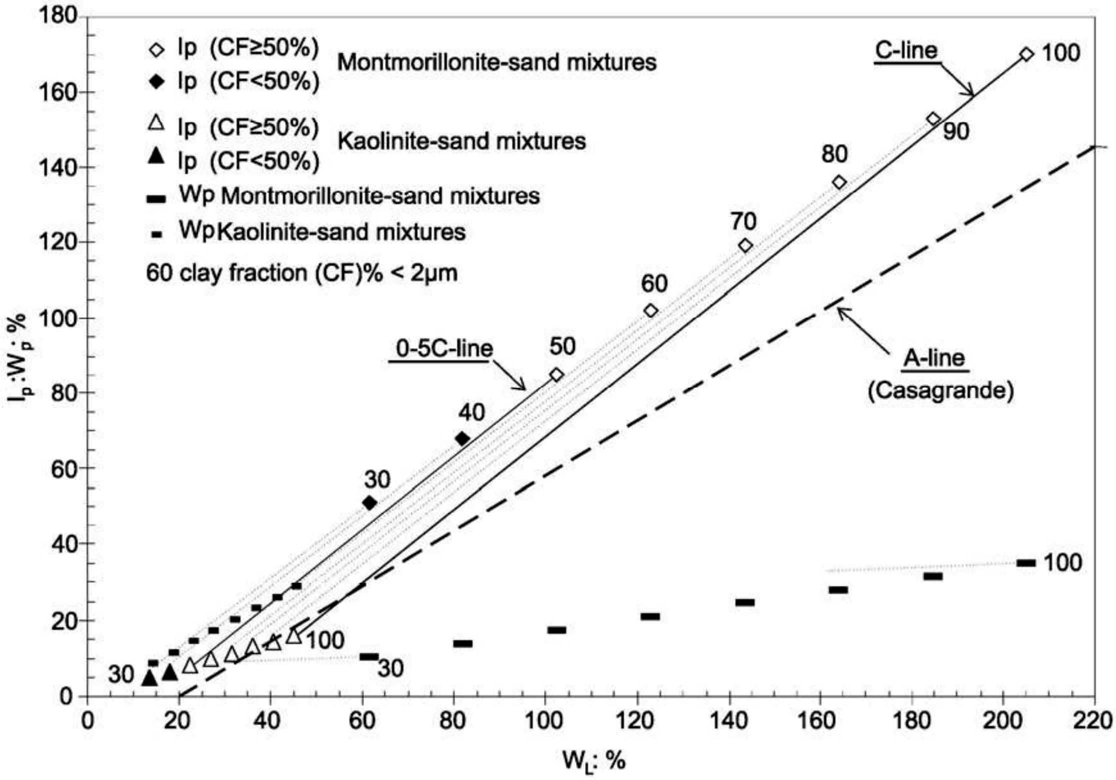


Fig. 2.3 Plasticity index, I_p, and plastic limit, W_P, as functions of liquid limit, W_L. The dashed lines define the zone where mixtures having the same clay fraction (CF) lie (Polidori, 2003)

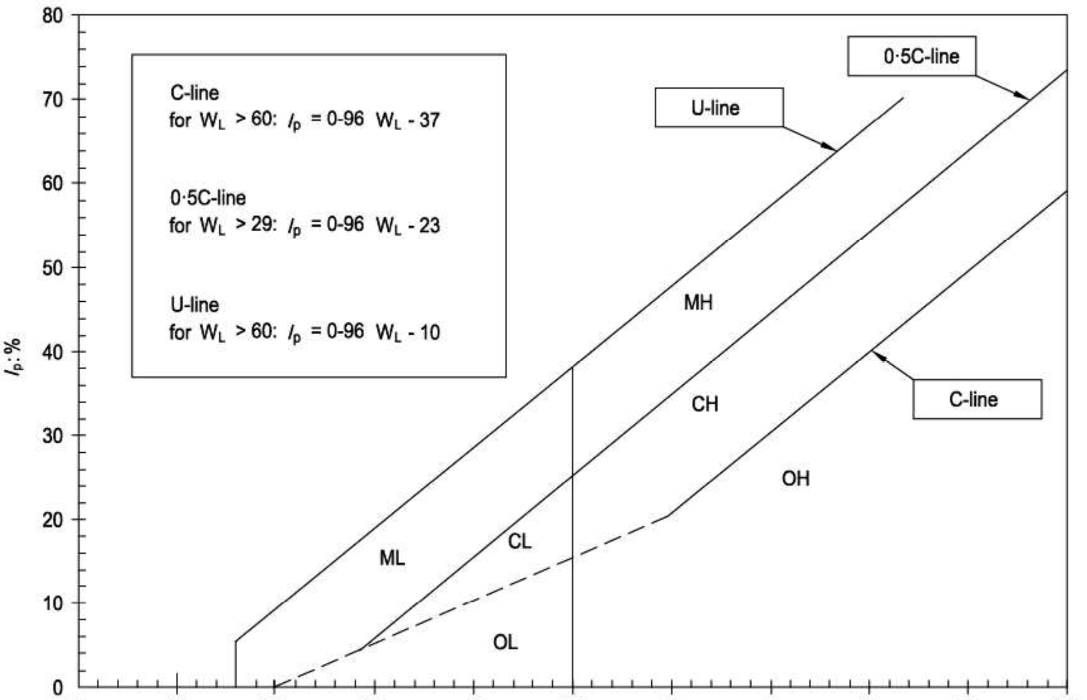


Fig. 2.4 Polidori's plasticity chart (Polidori, 2003)

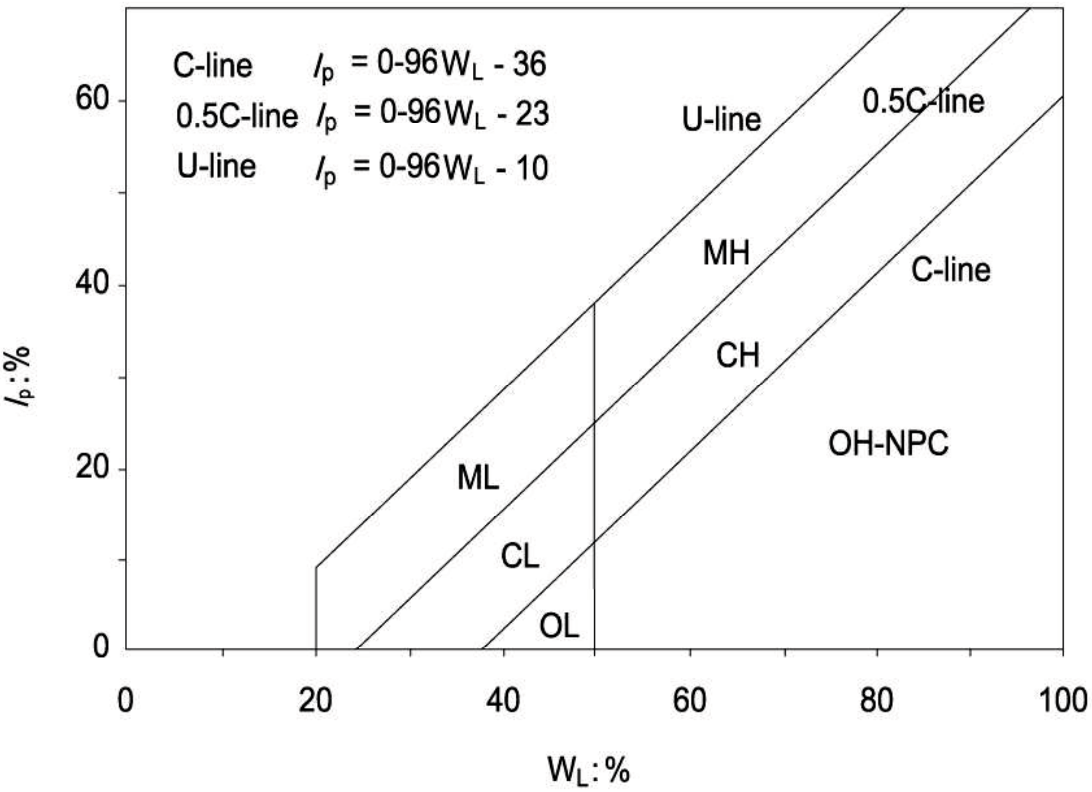


Fig. 2.5 Polidori's revised plasticity chart (Polidori, 2007)

Let us take a closer look at the different points of view, that is, whether the plasticity chart comes from the clay and silt size fraction or from their plasticity behavior.

Sridharan et al. (1988) in his study of the Atterberg limits of natural soils suggested that the natural soils which have been subjected to physical and chemical weathering and other depositional processes, have a wide distribution of particle sizes and physic-chemical and mechanical properties. Consequently, natural soils do not exhibit trends anticipated from the results with artificial soils, as their various constituents directly fail to influence soil properties on the basis of the quantity present. To be precise, the Atterberg limits of natural fine-grained soils have no correlation with their percentage clay size fractions.

On Casagrande's plasticity chart, for a given liquid limit value, the soils lying above the A-line in the clay zone show higher plasticity index values than the silts below the A-line. Regarding the liquid limit, the plastic limit, and their numerical difference, that is, PI=LL-PL, the values of the plastic limit of the clays (CF $\geq$ 50%) should be less than the values for silt

(CF < 50%) with the same liquid limit, contrary to what has been demonstrated in Polidori's (2003) and Seed et al., (1964b)'s studies. Fig. 2.6 shows the location of 125 soil samples taken from the literature on the plasticity chart (Seed et al., 1964a; Lupini et al., 1981; Skempton, 1985; Wasti and Bezirci, 1986; Burland, 1990; Di Maio and Fenelli, 1994). All the soils (except for 3 soils) lie above Casagrande's A-line, regardless of their clay size content, and between the C-line and the U-line. The data by Nagaraj and Jayadeva (1983) offers another example of the same case. All the soils plotted on Casagrande's plasticity chart lie above the A-line in the clay zone, except for the data of 3 soils which lie below the A-line in Casagrande's silt zone. Based on these cases, Polidori (2009) suggested that the boundary A-line is equivalent to Polidori's C-line because the residual soils lie below both the boundaries, as shown in Fig. 2.7. Because no distinction exists above the A-line where the common soils lie on the empirical plasticity chart proposed by Casagrande, Polidori's new plasticity chart appears to be more appropriate for classifying silt or clay based on the Atterberg limits. This specific content is treated in more detail later using the natural common fine-grained soils in Korea.

Meanwhile, Prakash and Sridharan (2004) discussed the fundamental issues of Polidori's new plasticity chart (Polidori, 2003). The classification of soil into silt or clay through a plasticity chart is purely based on plasticity, that is, the liquid limit and the plasticity index. In other words, the term "silt size fraction" is different from the term "silt", and the term "clay size fraction" is different from the term "clay". The type and quantity of clay minerals with their associated exchangeable cations present in fine-grained soil control the extent of the soil plasticity and volume change behavior rather than the CF. For the same bentonite clay, depending upon the exchangeable cation, the liquid limit can vary several fold, whereas marginal variations are seen in the plastic limit (Sridharan et al., 1986). If the montmorillonitic clay mineral content in the CF is more, the soil subsequently exhibits clayey behavior and high plasticity, that is, relatively high LL and low PL. If the kaolinitic clay mineral content is more, then the soil exhibits silty behavior and low plasticity, that is, relatively low LL and high PL. A given soil may have a CF of less than 50%, and if the quantity of kaolinitic clay minerals within that is prominent and the CF contains relatively inert particles, then the soil lies below the A-line. Therefore, Casagrande's plasticity chart classifies the given soil into silt or clay based on a behavioral viewpoint rather than on the soil content based on particle size (Prakash and Sridharan, 2004). According to Prakash and Sridharan (2004), even though kaolinite is clay mineral based on its particles being coarser in size, it is normally of silt size. Kaolinite clay mineral contributes to the silty type of soil behavior. It has a higher plastic limit and hence less plasticity. Therefore, pure kaolinite lies below the A-line in the silt zone.

Meanwhile, Polidori (2004) emphasized that considerations regarding inorganic soils with platey clay minerals are not valid for residual inorganic soils containing non-platey clay minerals (allophone, halloysite, attapulgite) because their characteristics (high plastic limit, low plasticity index, high residual strength; Kenney, 1967; Lupini et al., 1981; Wesley, 1992; Mitchell, 1993) are very different from those of platey clay minerals for which the plasticity chart was developed. Therefore, these soils might lie below or above C-line according to their particular characteristics. He explained that the reason why some soils lie in a zone that does not correspond to their clay fractions might be mainly attributable to the poor precision of the standard method for determining the plastic limit, including for residual inorganic soils or organic soils. Based on the colloidal activity of clays, defined by Skempton (1953), $A = I_p/CF$ ($\% < 2\mu m$), the plasticity of a given soil depends on both the type (A) and the clay fraction (CF). If the relation between LL, PL and CF is not linear, then Skempton's linear relation (I_p, CF) would not be correct. Therefore, it should be adopted a linear variation of LL and PL with the CF content. Polidori (2004) emphasized that the classification of inorganic soils as silts or clays from a behavioral standpoint must be pursued exclusively in the zone between the U-line and the C-line in which the soils lie.

Meanwhile, the grouping of fine-grained soils as silt and clay is almost always carried out according to Atterberg limits. As mentioned above, the distinction between the coarse and fine-grained soils is made by the No. 200 sieve, whereas the soil consistency tests to designate fine-grained soils as silt or clay have been solely performed on materials passing through the No. 40 sieve (with a sieve mesh size of 425 μm), not on materials passing through No 200 sieve. Regarding the classification of soils based on their particle size, the British Standard (BS 1377) distinguishes soil types according to the following criteria: clay = soil fraction with particles < 2 μm, and silt = soil fraction with particles 2-60 μm. Sand is designated as having a grain size from 60-2000 μm, and is further subdivided into fine, medium and coarse sand, corresponding to particle size intervals of 60-200 μm, 200-600 μm and 600-2000 μm, respectively, indicating that material coarser than 75 μm but finer than 425 μm is termed as medium-grade sand. Therefore, when conducting the Atterberg tests using a small percentage of material finer than 75 μm, the material prepared may include a significant amount of coarse material. Why is some amount of coarse-grained soil included in a test intended to make a distinction between fine-grained soils? An extensive literature review was performed to seek an answer to this question by Kayabali (2011). However, no studies were encountered addressing this issue.

Meanwhile, according to engineering field manual (United States Department of Agriculture, USDA, 1975), although fines are soils particles that pass No. 200 sieve, Atterberg limits are determined on materials that pass the No. 40 sieve. This apparent inconsistency occurs because Atterberg limits were established and measured on materials passing No. 40 sieve many years before the No. 200 sieve was established as the largest size for fines.

Table 2.1 summarizes several of the grade scales used in engineering today and as with the scale in Fig. 2.8, there remains considerable disagreement as to the boundary between gravel, sand, silt and clay because the sizes are arbitrary. It is interesting to note that the USCS does not subdivide soils finer than sands according to grain size. Rather, the USCS uses plasticity for distinguishing silt from clay.

As shown in Fig. 2.8, a Swedish soil scientist Atterberg engaged in ceramics and agriculture work distinguished coarse grained soils from fine grained soils based on materials finer than 0.2mm and called “Mo” in Swedish. This “Mo” means silt soils or fine sand. Therefore, it is presumed that initially, Atterberg (1914) developed an agricultural classification system (Table 2.2) based on plasticity as determined from his liquid and plastic limit tests using materials finer than 0.2mm. The Atterberg limits were later adopted by the US Bureau of Public Roads in the late 20’s and the test procedures were simplified. Casagrande (1932, 1948, 1958) researched and standardized the Bureau of Public Roads version of the Atterberg limits tests and developed the plasticity chart after considerable testing for distinguishing between silts and clays of high and low plasticity using materials finer than 0.425mm (No. 40 sieve).

Around the same time, the revised Public roads and American Association of State Highway and Transportation Officials (AASHTO) classification system (1945) was developed to assess the load bearing characteristics of subbase and subgrade soils for a highway. It is necessary to carry out to sieve analysis using No. 40 sieve for group classification (A-1 and A-3). Therefore, it can be inferred that Atterberg limits tests using materials finer than No. 40 sieve are fixed to satisfy both sides (USCS and AASHTO system) in the period.

Meanwhile, Kayabali (2011) compared the Atterberg limits obtained by using soil passing the No. 40 sieve versus soil passing the No. 200 sieve with sixty soil samples. Fig. 2.9 shows that liquid limits for soil passing the No. 200 sieve are higher than their corresponding values for passing the No. 40 sieve. It reveals that liquid limit for -#200 soils are 10-20% higher than liquid limit for -#40 materials at least more than half of 60 pairs of soils. Fig. 2.10 shows the comparison between the plastic limits from -#200 materials and those from -#40 material. Similar to the comparison of liquid limits for -#200 and -#40 materials, the plastic limits of the -#200 materials are also higher than plastic limits of -#40 materials for the same soils. The overall degree of deviation of the plastic limits of -#200 materials from the plastic limits of -#40 materials is somewhere between 10-20 percent for approximately half of sixty pairs of soils. According to Kayabali (2011), when classifying the soil based on the -#200 material, soil classification can be changed compared to the -#40 material of the same soil. Fig. 2.11a illustrates the most changes for the two pairs of soils, which shifted from CL to MH, where both the plasticity level (as L versus H) and the type of soil (C versus M) changed. Fig. 2.11b illustrates the change of soil type, as the plasticity level remains the same. Fig. 2.11c indicates the shift from low plasticity to high plasticity for silty soils. Fig. 2.11d shows that a number of CH soils shift to MH. If the plastic soils are classified based on the Casagrande's plasticity chart, as fine contents increase, the soils can be classified as silty soils (ML or MH), such as the Kayabali's research.

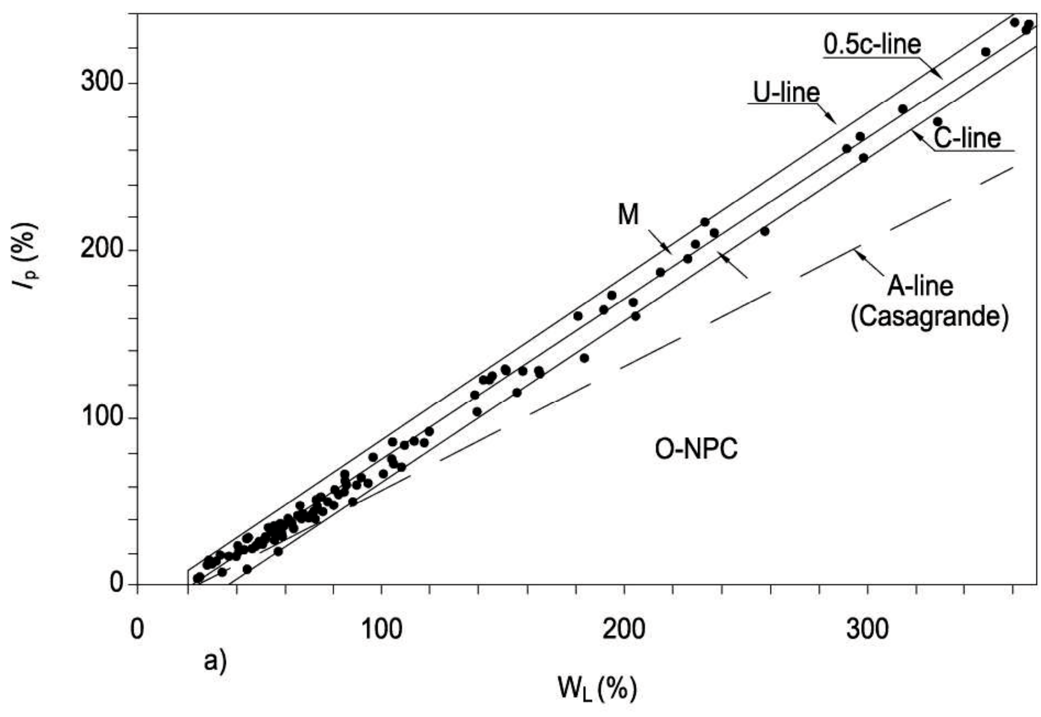


Fig. 2.6 Location of 125 soil samples from literature on the plasticity chart (Polidori, 2007)

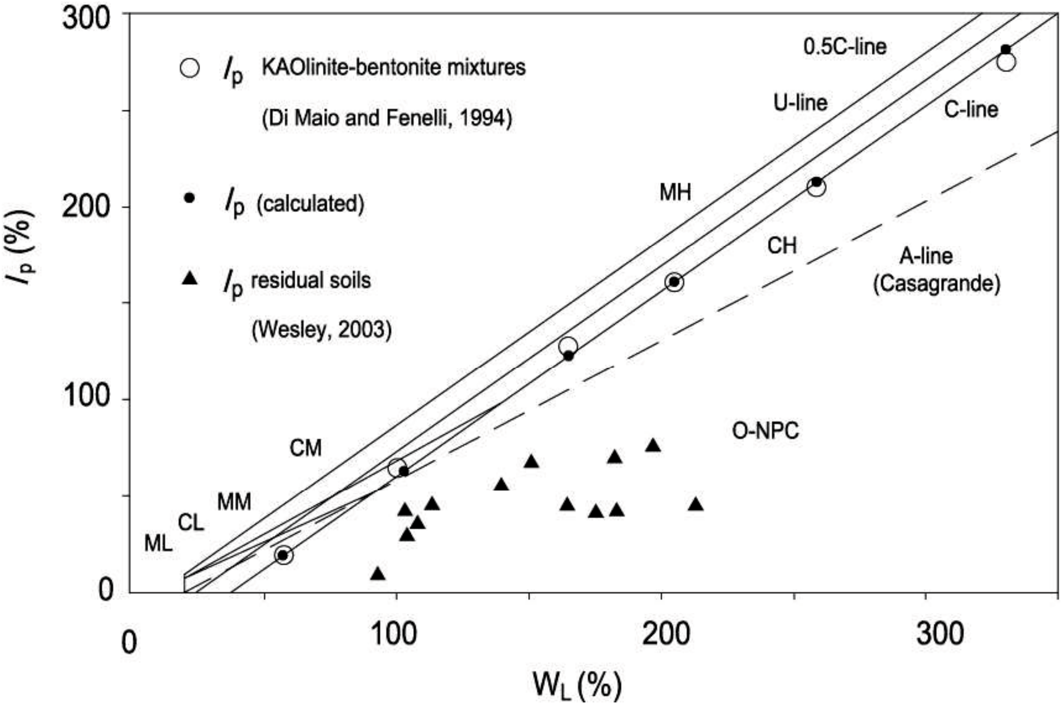


Fig. 2.7 Location of data from literature on the plasticity chart of Casagrande and Polidori (Polidori, 2009)

Table 2.1 Grain size ranges according to several engineering soil classification systems

Soil Type	USCS Symbol	Grain Size Range (mm)			
		USCS	AASHTO	USDA	British Standard and M. I. T.
Gravel	G	76.2 to 4.75	76.2 to 2	> 2	> 2
Sand	S	4.75 to 0.075	2 to 0.075	2 to 0.05	2 to 0.06
Silt	M	Fines < 0.075	0.075 to 0.002	0.05 to 0.002	0.06 to 0.002
Clay	C		< 0.002	< 0.002	< 0.002

Table 2.2 Atterberg Soil Classification (1914)

Major divisions	Secondary divisions / Description
A	Clays (Plastic soils) I.-Sticky Clays (Highly Plastic) This group contains only the heaviest clays II.-Loamy Clays (Not sticky) Subdivided into medium heavy and fairly heavy clays
B	Loams (Nonplastic, More or Less Cohesive Soils I.-Fairly Heavy Loams Clayey Loams II.-Light Loams Sandy Loams and loose soils

C	Sand, Mo, and Silt Soils (Noncohesive Soils) I.-Capillarity Greater than 34 cm Fine-grained sandy soils; dust loess; subdivided by mechanical analysis II.-Less than 34 cm Coarse, dry sandy soils, useful only for forestry

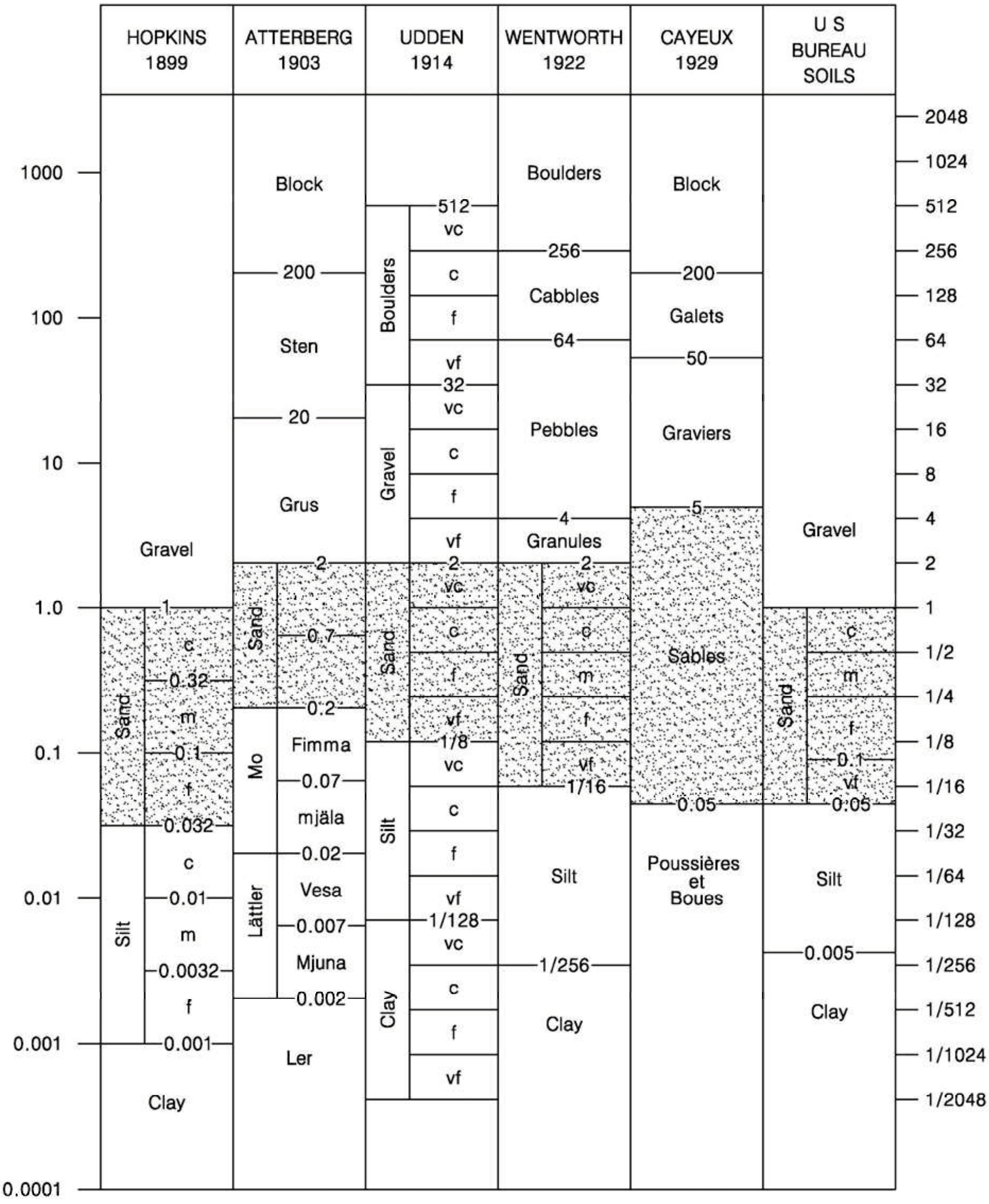


Fig. 2.8 Representative grade scales of Early Soil Scientists (Richard and Kenneth, 1987)

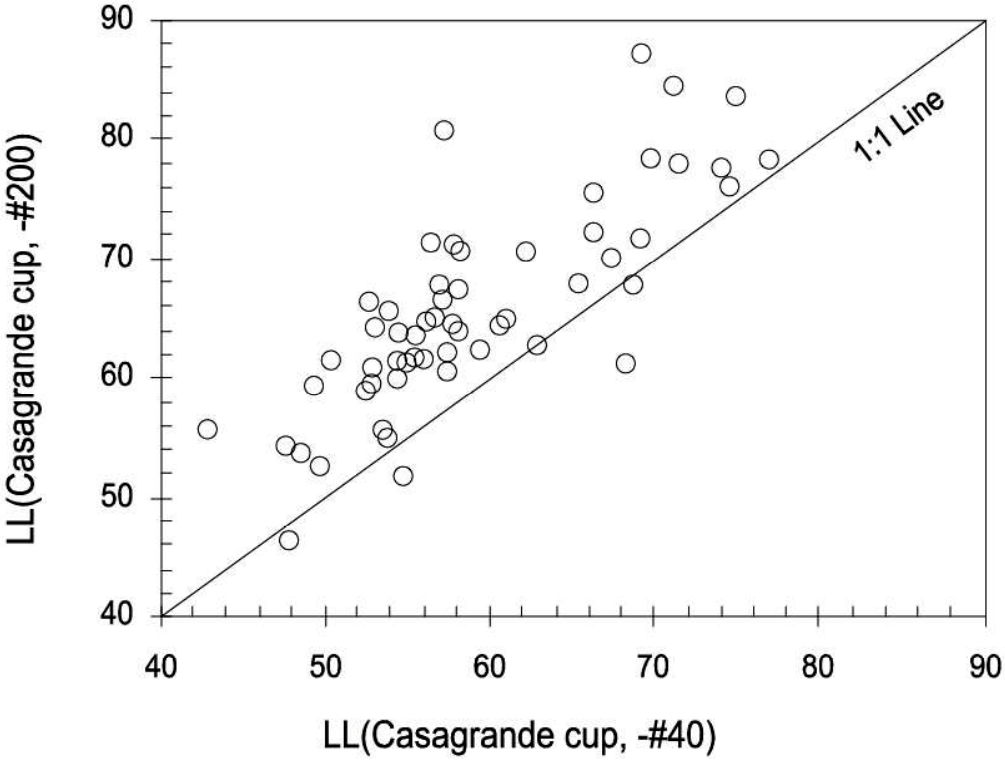


Fig. 2.9 Comparison between the liquid limits of -#200 materials and liquid limits of -#40 materials (Kayabali, 2011)

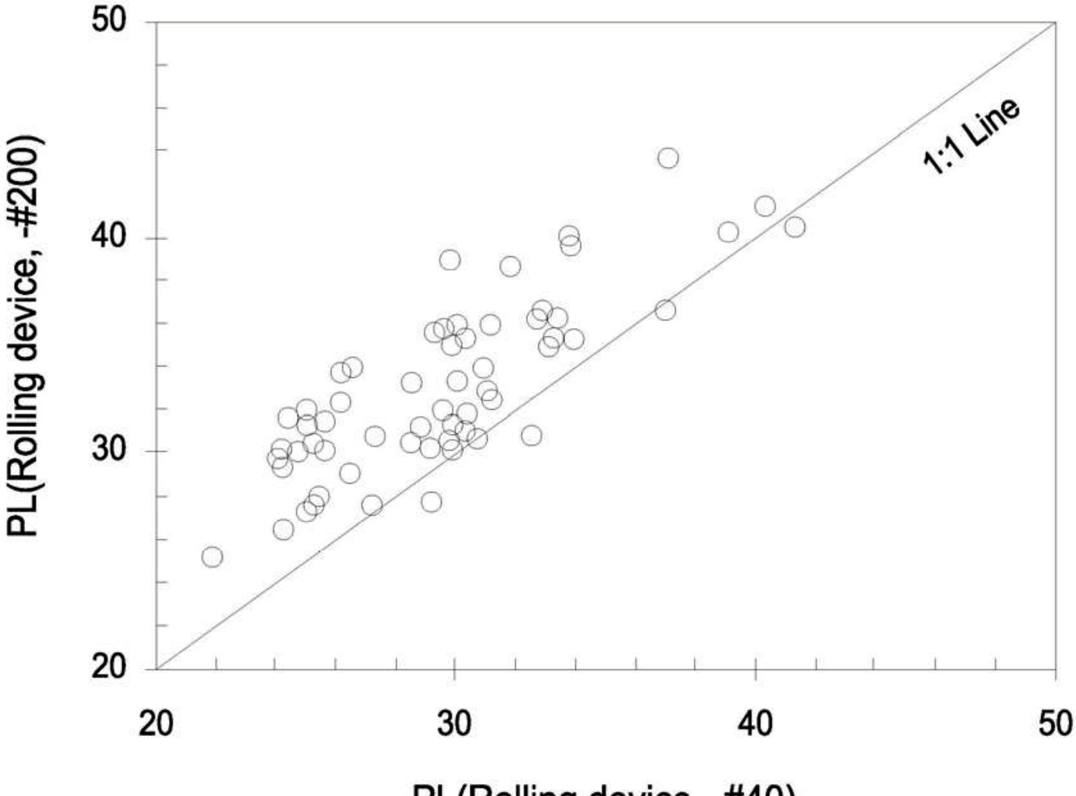
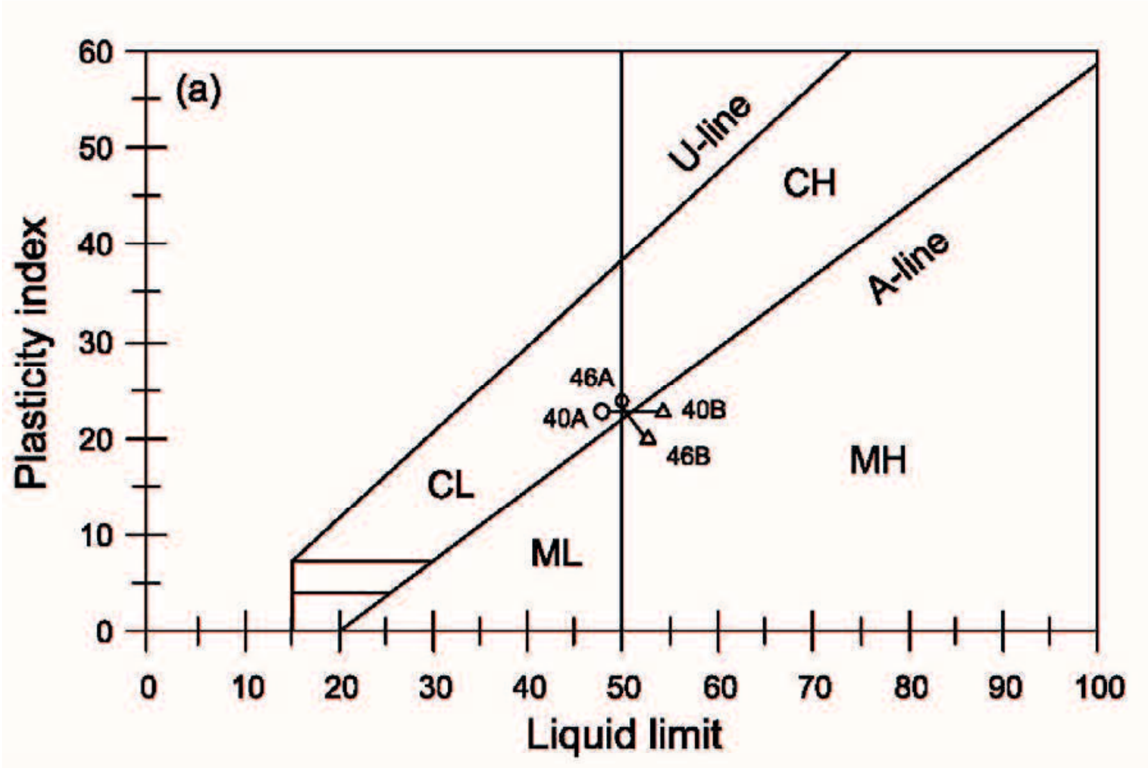
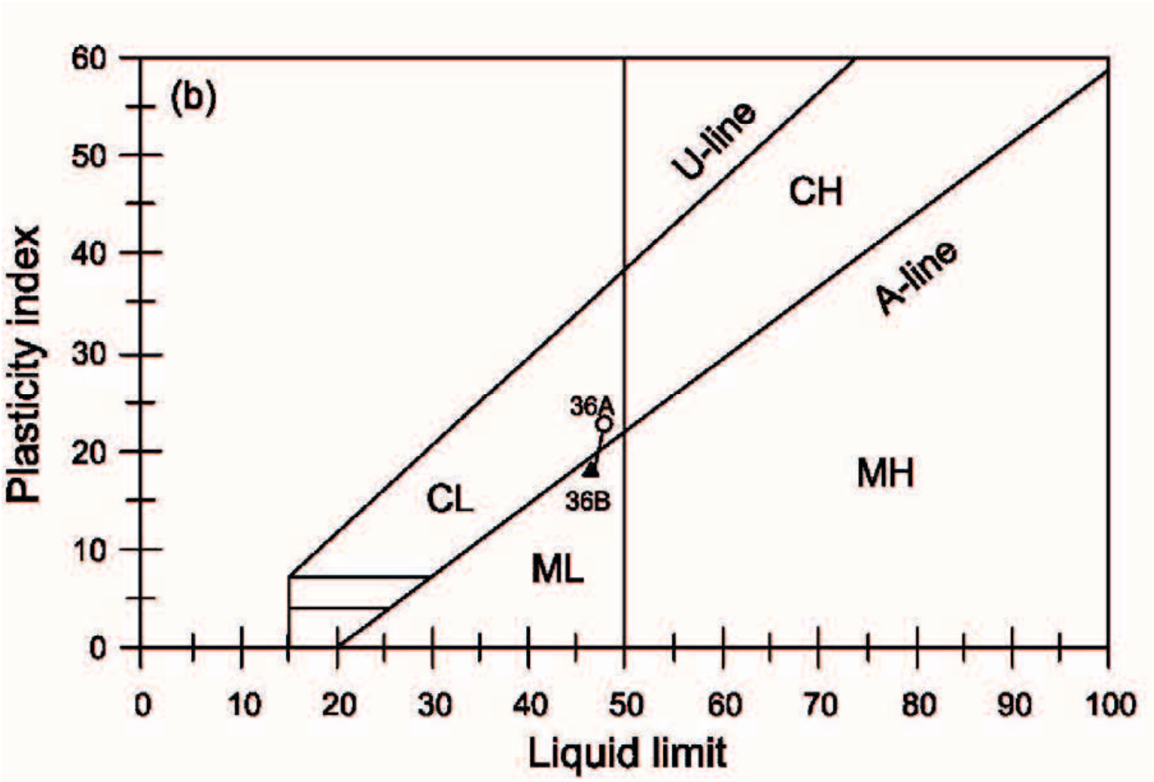


Fig. 2.10 Comparison between the plastic limits of -#200 materials and the plastic limits of -#40 materials (Kayabali, 2011)


(a)
U-line
CH
A-line
46A
40A
40B
46B
CL
MH
ML
Plasticity index
Liquid limit
0
10
20
30
40
50
60
70
80
90
100


(b)
U-line
CH
A-line
36A
36B
CL
MH
ML
Plasticity index
Liquid limit
0
10
20
30
40
50
60
70
80
90
100

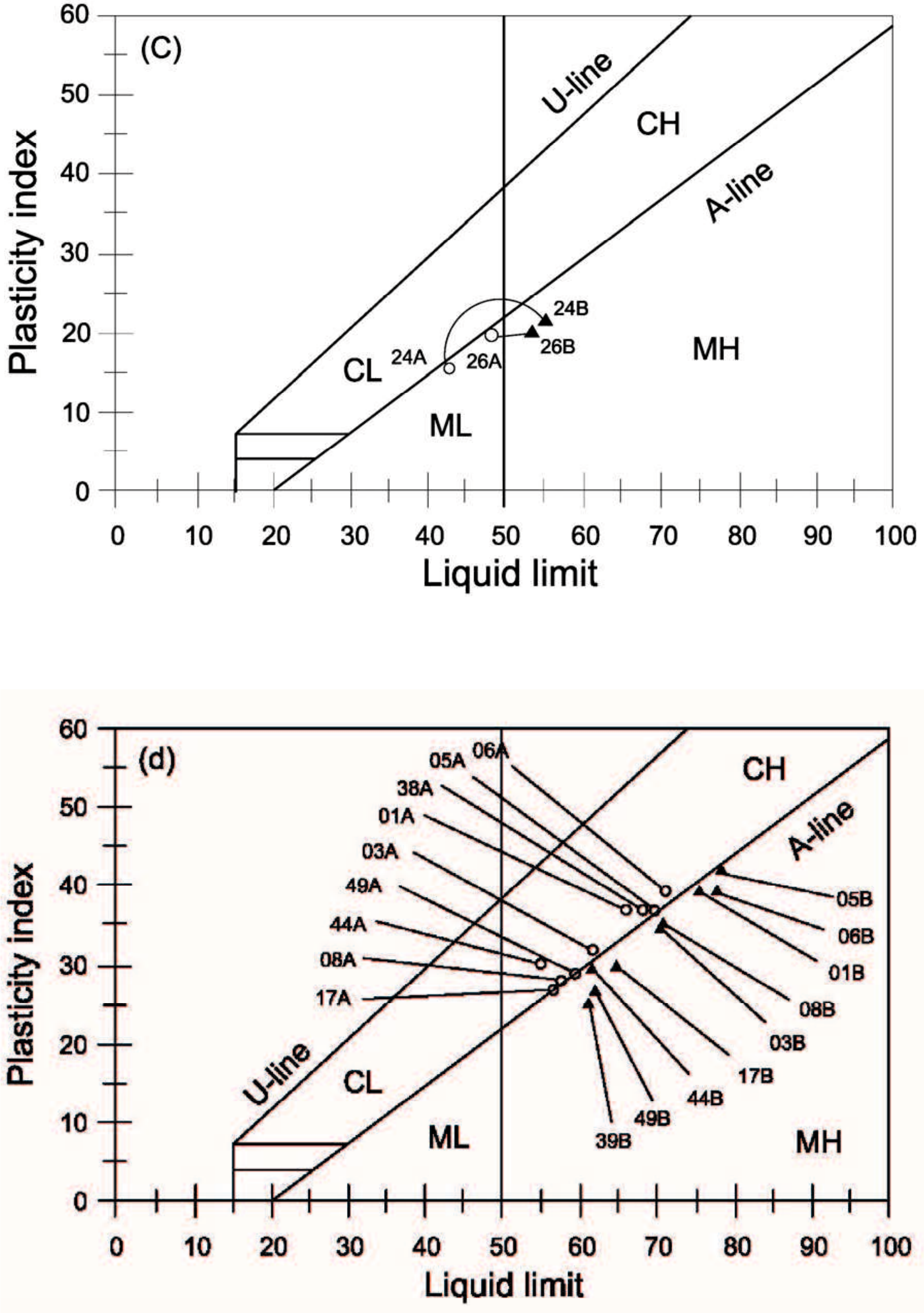


Fig. 2.11 Comparison between soil classification of -#200 materials and -#40 materials based on Casagrande's plasticity chart (Kayabali, 2011)

2.1.2 FIELD INDENTIFICATION TESTS FOR SOIL CLASSIFICATION

Coarse-grained soils such as gravel, sand, gravelly sand, etc, can be readily identified by inspection. Traces of silt and/or clay are somewhat more difficult to identify when mixed with these materials but may not be of much importance unless the quantity is over 5 to 10 %. The sedimentation test described for fine-grained soils may be used to determine if significant quantities of silt, very fine sand, or clay are present. Fine-grained soils can be identified using some, or all, of the following tests performed on approximately the materials finer than 0.425mm (remove the larger particles by hand rather than actually sieving)

1) DILATANCY (or pore water mobility reaction to shaking)

Prepare a pat of moist soil with a volume of 1 to 3 cm^3, using enough water to make the soil soft but not sticky. Place the pat in the open pale of one hand and jar the hand vigorously with the other. If the soil is fine sand, silt, or silty fine sand, the inertia force due to jarring will force the water to the surface of the soil pat, and it will appear wet or glossy. When the sample is manipulated, this surface water will disappear. In soils with substantial clay this test procedures no reaction.

2) DRY STRENGTH (resistance of dry lumps to crushing)

Mold a pat of soil to about the consistency of putty by adding water as necessary. Allow the pat to completely dry and then test the crushing strength by breaking or crumbling between the fingers. The dry strength increases with increasing plasticity. High dry strength is characteristics for clays of the CH group, lesser dry strength for CL and MH soils, and very low to nonexistent strength for OL and ML soils. Fine sand, silt, and sand-silt mixtures possess almost no dry strength. Note that one may do this test approximately on naturally dried in-situ soil.

3) TOUGHNESS (consistency near the plastic limit)

Take a specimen of about 1 cm^3 and mold it to the consistency of putty. Proceed to roll the soil in the palm (or on a smooth surface) into a thread about 3 mm in diameter. When the pat of soil crumbles and loses its plasticity, the plastic limit has been reached. The higher the resistance of the 3 mm thread to pulling apart, the higher the position of the soil on the plasticity chart with respect to the A line. A weak thread which is easily crumbled indicates silts or inorganic clays of low plasticity. Highly organic clays are also very weak but may feel spongy at the plastic limit.

4) SEDIMENTATION

Place about 50 g (more for gravelly soils) in a glass jar, such as a beaker, test tube, glass graduate, or other jar on the order of 150 mm deep, with water to fill the jar. Vigorously shake for several minutes and allow to stand. Gravel and coarse sand will settle almost instantly. Medium to very fine sand will take not more than 1 to 3 minutes; silt will take not more than 15 minutes. Clay will take only slightly longer unless a deflocculating agent is added. The relative thickness of the sediments in an indication of percentages of various grain sizes.

5) COLOR

In general, dark colors such as black, gray, and dark brown indicate organic soils.

6) ODOR

Organic soils usually have a distinctive smell of decaying materials. This test should be applied to fresh samples which are still wet. Roots, pieces of weeds, wood, plants, etc., may be visually present as further aid.

7) FEEL

Sands and silts dray rapidly and can be dusted from the hands easily. Clay tends to leave considerable discoloration after drying and the hands may have to be washed to remove all traces. Clay tends to be smooth to the touch or to leave a smooth streak when a spatula blade is moved across a wet mass. Silts and sands are rough and gritty and leave grain marks when a spatula blade is moved across a wet lump (J. E. Bowles, 1984).

2.1.3 JAPANESE PLASTICITY CHART BASED ON CASAGRANDE'S

Japanese Geotechnical Society (formerly Japanese Society of Soil Mechanics and Foundation Engineering, usually abbreviated to JGS) established standization committee for making soil identification and classification method in 1970. This committee proposed Japanese United Soil Classification System, USCS in 1973 for the first time after carrying out symposium and discussion many times.

Yamada and Imai (1971) proposed field identification method with the naked eye for soil classification based on USCS method (Casagrande's plasticity chart). Fig. 2.12, 2.13 show methods for estimating plasticity index (PI) and liquid limit (LL) through comparison the number of count for toughness test with each value obtained from laboratory tests, respectively. As shown in Fig. 2.14, the higher the resistance of the 3 mm thread to pulling apart, the higher the position of the soil on the plasticity chart with respect to the A-line. A weak thread which is easily crumbled indicates silts or inorganic clays of low plasticity (below the A-line). Organic clays are also very weak and lie below the A-line (approximately lower than 20% of plasticity index, PI). As shown in Fig. 2.15, high dry strength is characteristic for clays of the CH group, lesser dry strength for CL and MH soils, and very low to nonexistent strength for OL and ML soils. Fine sand, silt, and sand-silt mixtures possess almost no dry strength.

While Casagrande (1932) proposed two zones separated by the A-line on his plasticity chart to distinguish silt from clay, JGS-affiliated committee (1973) suggested unsuitability of the A-line (Casagrande's plasticity chart) for Japanese clayey soils because of the similarity in physical and mechanical properties on the upper and lower position adjacent to the A-line based on a comparison with field identification and laboratory test results in soil classification (see Fig. 2.14, 2.15). Based on the case outlined above, as shown in Fig. 2.16, this committee (1973) proposed Japanese plasticity chart by adding C and D-line. The zone surrounded by A, C and D-lines is designated as C'H, which is equivalent to CH in the zone that lie above the A-line on its chart, due to the same physical and mechanical properties. However, JGS (1996) suggested a modified version of plasticity chart at subcategories under method of classification of geomaterials for engineering purposes. C, D-line, in other words, C'H was eliminated from plasticity chart as a simplicity process. Ogawa and Matsumoto (1978) showed physical properties obtained from harbor districts in Japan to support these characteristics. As shown in Fig. 2.17, almost all soil test data lie on the upper and lower position adjacent to the A-line, especially, liquid limits (LL) having a range of 100% ± 30% lie mainly below the A-line, that is, in zone surrounded by A, C, D-lines. The higher the liquid limit and plasticity index, the higher the position of the soil on the plasticity chart with respect to the A-line. Meanwhile, Fig. 2.18 shows position of soil date from Japanese coastal areas including Japanese C, D-lines and Casagrande's A-line on Polidori's plasticity chart. Almost all soil test date lie on the upper and lower position adjacent to the Polidori's C-line with a slope of approximately 45 degrees rather than Casagrande's A-line. It can be confirmed that Japanese marine soils does not lie above the 0.5C-line (silt zone) except for some data lower than liquid limits of approximately 80%. Polidori's plasticity chart is valid for inorganic soils that contain platey clay minerals (residual soil containing non-platey clay minerals and organic soil lie below Casagrande's A-line and Polidori's C-line, respectively). It seems that Polidori's plasticity chart does not suffice to classify Japanese marine soils. However, the thread rolling method is simple, but it has long been criticized since it is considered highly operator-dependent. For example, Whyte (1982) reported that the plastic limit of a soil determined in different laboratories ranged from 19% to 39% with an average plastic limit of 23%. JGS-affiliated research committee on soil consistence (1995) carried out round robin tests to analyze highly operator-dependent characteristic. The committee showed that the range of fluctuation in liquid limit and plastic limit obtained from each test is from 10% to 20% due to the problems on preparation for soil specimens as well as a personal and laboratories' error. This tendency towards highly operator-dependent characteristic was noticeable in plastic limit. Therefore, further experimental data and studies are necessary to judge applicability to Japanese fine-grained soils of Polidori's plasticity chart.

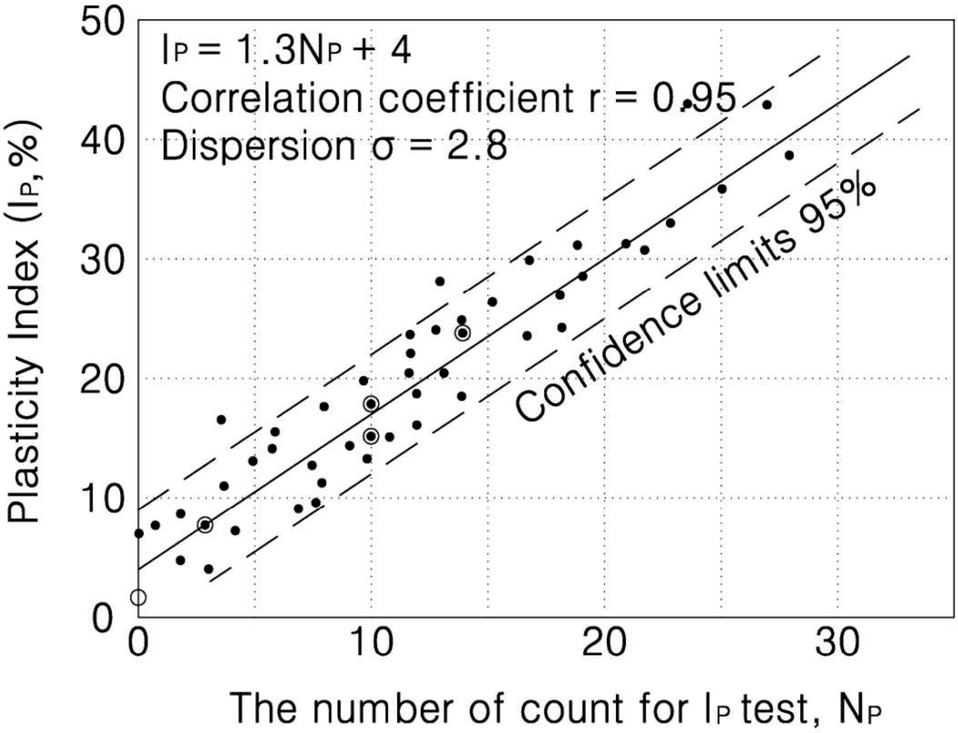


Fig. 2.12 Relations between plasticity index (I_P) and the number of count for toughness test (Yamada and Imai, 1971)

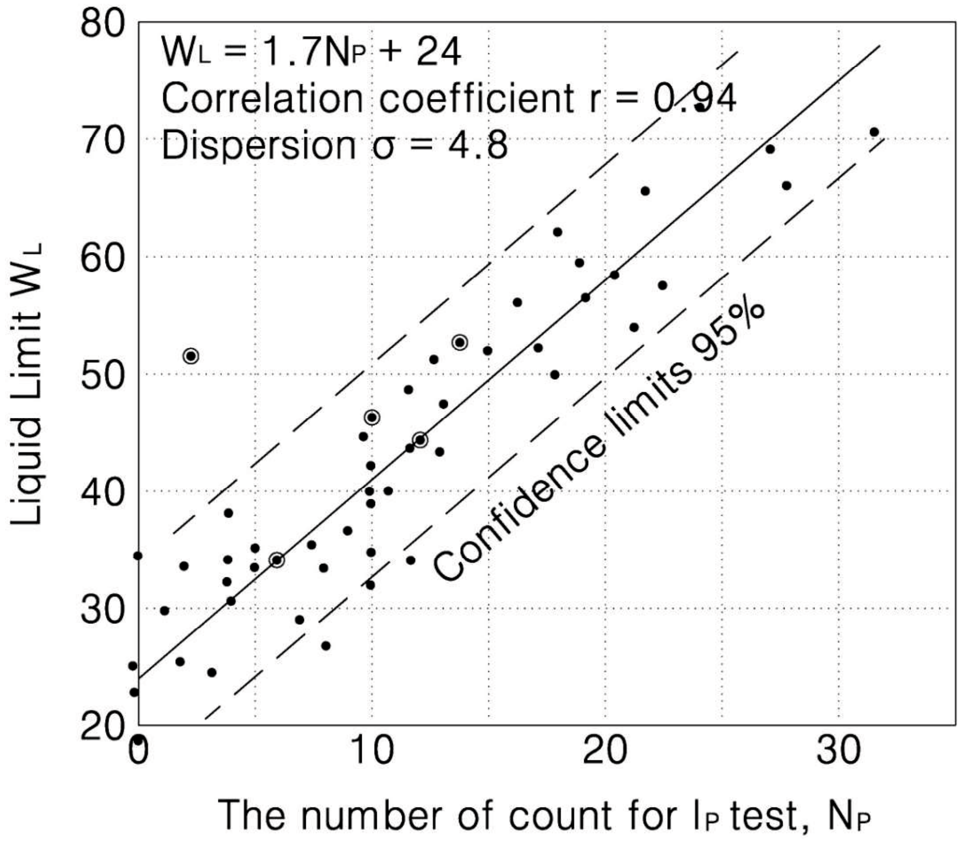


Fig. 2.13 Relations between liquid limit (W_L) and the number of count for toughness test (Yamada and Imai, 1971)

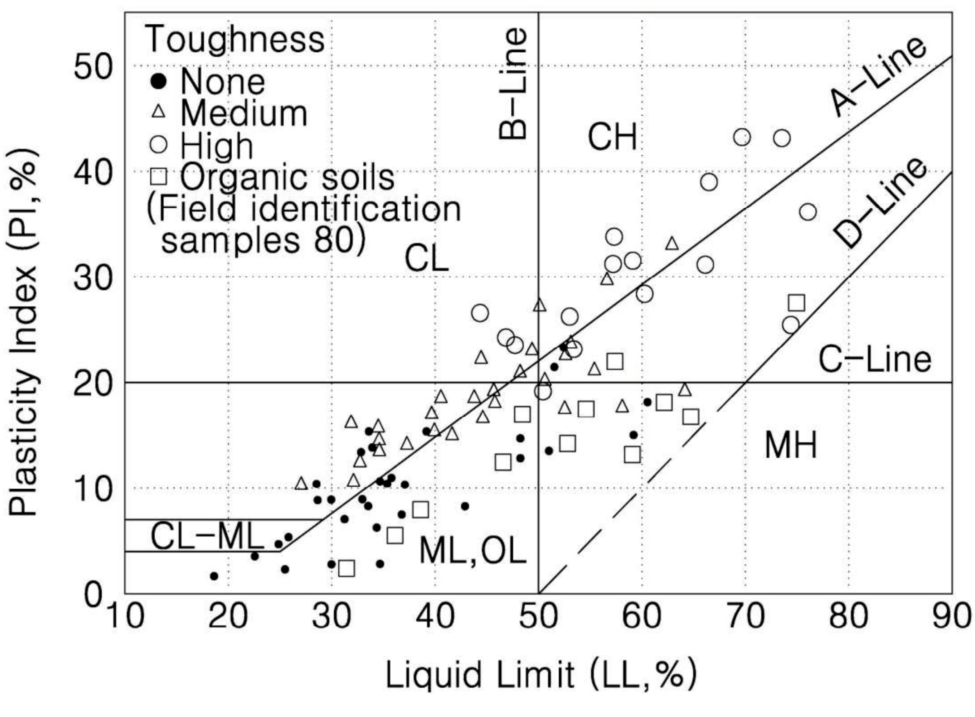


Fig. 2.14 Field identification using toughness method (Consistency near the plastic limit) based on Casagrande's plasticity chart (Yamada and Imai, 1971)

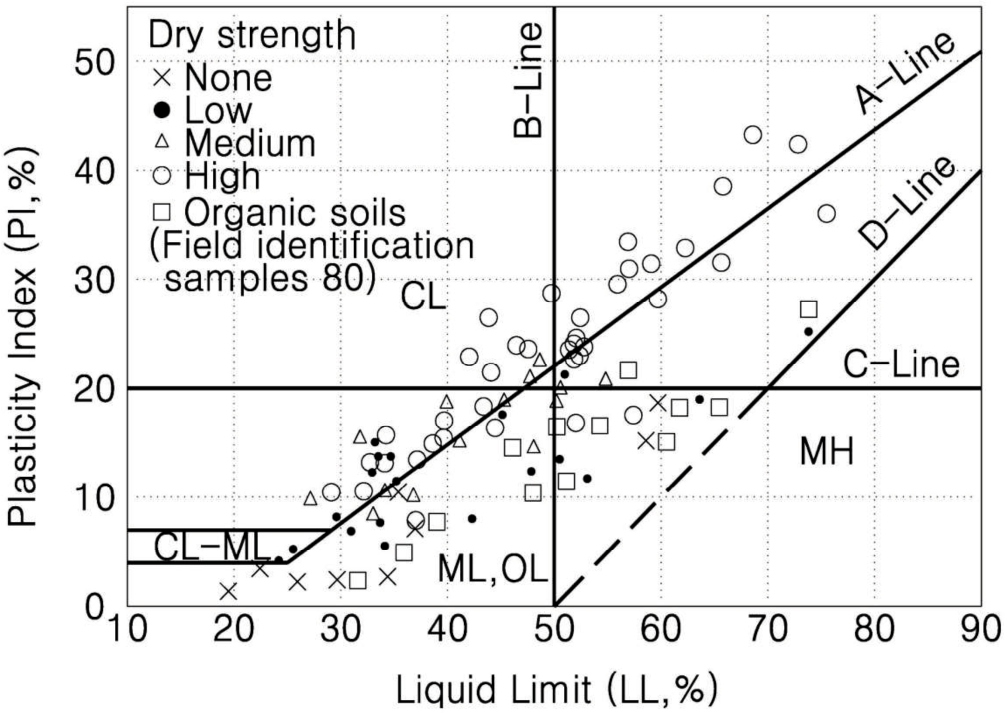


Fig. 2.15 Field identification using dry strength method (Resistance of dry lumps to crushing) based on Casagrande's plasticity chart (Yamada and Imai, 1971)

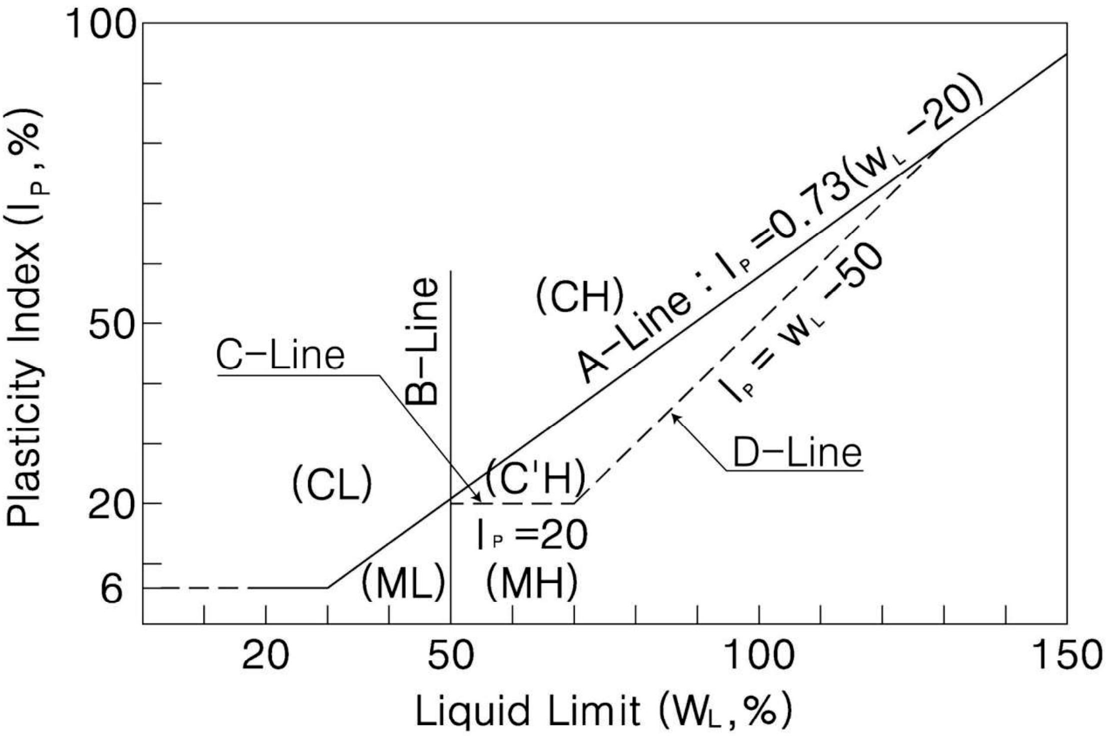


Fig. 2.16 Japanese plasticity chart (JGS, 1973)

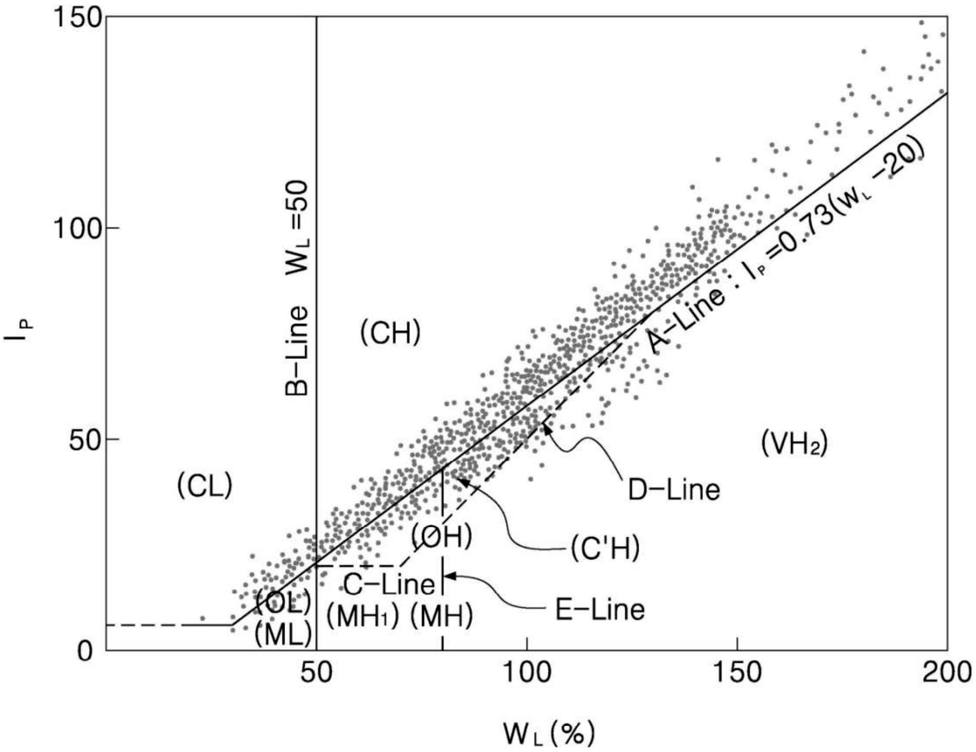


Fig. 2.17 Position of soil date from Japanese coastal areas including Japanese C, D-lines on Casagrande's plasticity chart (Ogawa and Matsumoto, 1978)

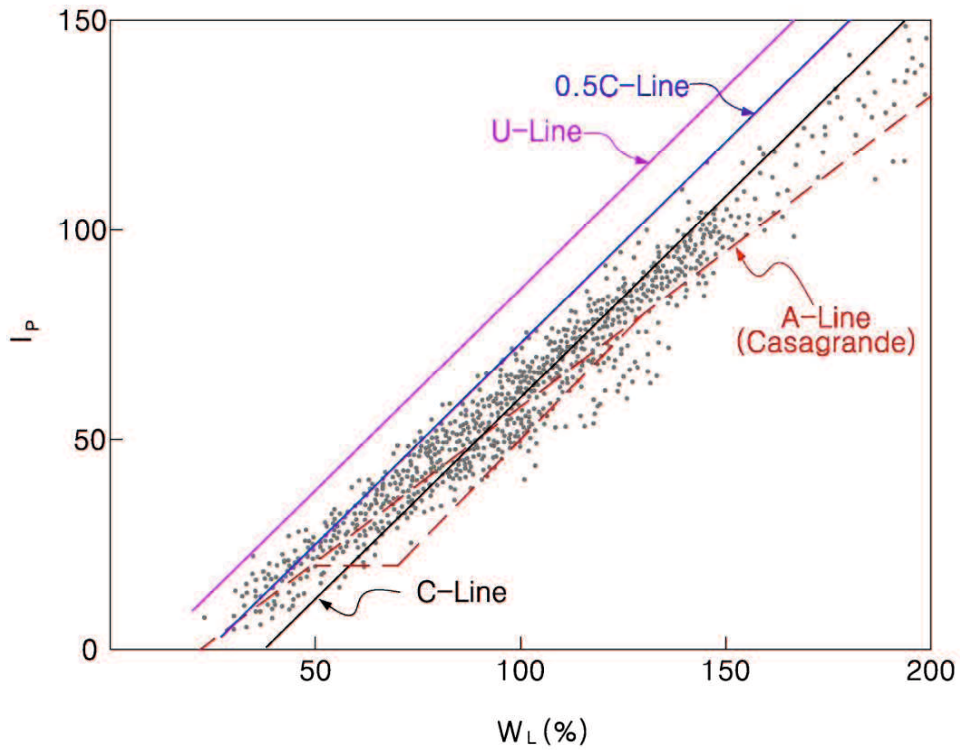


Fig. 2.18 Position of soil date from Japanese coastal areas including Japanese C, D-lines and Casagrande's A-line on Polidori's plasticity chart (Ogawa and Matsumoto, 1978)

2.2 UNDRAINED SHEAR STRENGTH CHARACTERISTICS OF INTERMEDIATE SOILS WITH LOW PLASTICITY

2.2.1 IMPORTANT FACTORS OF UNDRAINED SHEAR STRENGTH OF CLAYS

In most cases of soil structure design, soil is classified for simplicity as either sand or clay based on whether the soil structure is dominantly constituted of particles or fine content, and the appropriate design methods are used to properly represent the soil's behavior. For example, when an external load is applied to cohesionless soils such as sands and silts, contact forces arise between the soil particles from the friction between these particles as they make contact with one another.

For sandy soil, the state of its density (or packing) is important since the state of the forces is transmitted between the soil particles. The shear strength (τ) of sandy grounds may be calculated by using the following equation:

$$\tau = \sigma'_n \tan\varphi' \tag{2.1}$$

where σ'_n and φ' are the effective normal stress and the internal friction angle, respectively.

Geotechnical engineers prefer to perform total stress analysis for clayey soils, particularly for short-term analysis, because there is uncertainty in predicting the excess pore water pressure and the soils have a practically undrained nature due to very low permeability. In such cases, τ can be expressed in terms of the undrained shear strength s_u ($\tau = s_u$). However, most soils in the natural state are generally composed of a combination of sand, silt, clay, etc., and it is difficult to simply classify such soils into either sand or clay because they possess both properties of sand and clay. These kinds of soil are called intermediate soils, and the design code for the construction of port and harbor facilities in Japan classifies the ground into clayey or sandy soil according to a soil size fraction of 0.074 mm present in the ground. When more than 80% of a soil is composed of sand-sized particles, the soil is classified as sandy ground, and an effective stress analysis is applied to the ground stability analysis. In such an analysis, the soil parameter φ' is generally estimated by using the Standard Penetration Test (SPT). On the other hand, when less than 50% of the soil is sand, the soil is considered to be clay, and therefore the total stress analysis is applied to the design of various facilities on such ground. Soils with between 50% and 80% of sand-sized content are called intermediate soils because these show an intermediate behavior between sand and clay. For intermediate soils, if the permeability of soil is less than 10^{-4} cm/s, the stability analysis and the evaluation of s_u follow those of clayey soils, if the permeability of soil is more than 10^{-4} cm/s, the stability analysis and the evaluation of shear strength follow those of sandy soil (Japanese Port Association, 2007). In addition, Geotech Note No. 2 (Japanese Geotechnical Society, 1992) states that ground with a plasticity index between NP and 25%, permeability between 10^{-7} cm/s and 10^{-4} cm/s, and a coefficient of consolidation between 144 cm^2/day and 14,400 cm^2/day can be classified as intermediate soils. Unfortunately, until now, a classification code for intermediate soils did not exist in Korea.

In Japan, the undrained strength, s_u, has been generally evaluated by using the Unconfined Compression (UC) test, and it is taken to be half of the unconfined compression strength obtained from such a test. However, at present, most countries, with the exception of Japan and Korea, seldom use the UC test to evaluate the s_u of cohesive soils. For example, according to Eurocode 7.2 (2007), the UC test is prescribed as an index test, and reports on the soil investigations at Bothkennar (Hight et al., 1992) provide a good example of this fact, since no UC test was reported in these investigations. Although the UC test has been severely criticized and its validity has been questioned by various researchers, its use has been well established for Japanese soils in which Unconfined Compression tests yield reliable results, provided that samples of good quality are used (Tsuchida and Mizukami, 1991). Therefore, the UC test and the Standard Penetration Test are some of the most common tests used for clayey and sandy soils, respectively.

Extensive geotechnical data obtained from laboratory and in-situ tests have been accumulated in a database containing UC test results. The reason for this is to provide an automatic balance, or so-called lucky harmony, among the many factors that contribute to over-estimation and under-estimation of the actual strength value. These factors that mainly affect the UC test results are the sample disturbance, strain rate effect, and anisotropy of the strength. The undrained strength, which is to be used for the design of the structure, is presented by the following equation, in which these factors are used:

$$s_u = (q_{u(mean)}/2)\ c_1 c_2 c_3 \tag{2.2}$$

where

$q_{u(mean)}$: mean of the unconfined compression strength

c_1: correction coefficient for the reduction of the strength due to the sample disturbance

c_2: correction coefficient for strength anisotropy

c_3: correction coefficient for strain rate.

1) SAMPLE DISTURBANCE

It is well known that the strength of clay sample is greatly dependent on the disturbances during boring and sampling and the greater the disturbance the smaller the strength. Ladd and Lambe (1963) explained the effect of sample disturbance by the effective stress changes during the process of sampling, such as drilling, tube sampling, extrusion from tube and trimming. Fig. 2.19 illustrates a hypothetical effective stress path for a normally consolidated clay element during tube sampling. The soil of the in-situ effective stress condition is called “ideal sample” and the point P with an effective stress of σ'_{ps} corresponds to the condition of perfect sampling, in which no disturbance has been given to the sample except that involved with the release of the in-situ deviator stress. During the sampling and testing process, disturbances are given to the specimen and the excess pore water pressure within the specimen builds up. Finally point

F with an effective stress of σ'r represents the residual effective stress of the actual specimen at the start of an unconfined compression test.

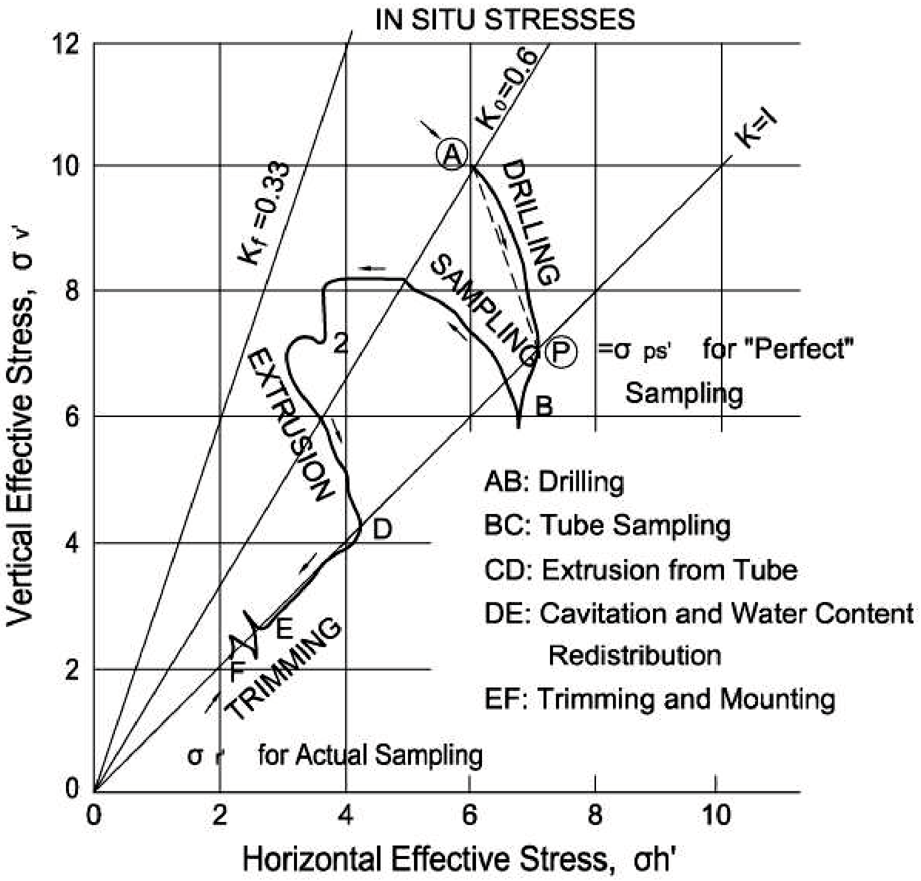


Fig. 2.19 Hypothetical effective stress path during boring and sampling (Ladd and Lambe, 1963)

Okumura (1974) carried out a comprehensive study on the disturbance effects and proposed the disturbance ratio defined by the following equation

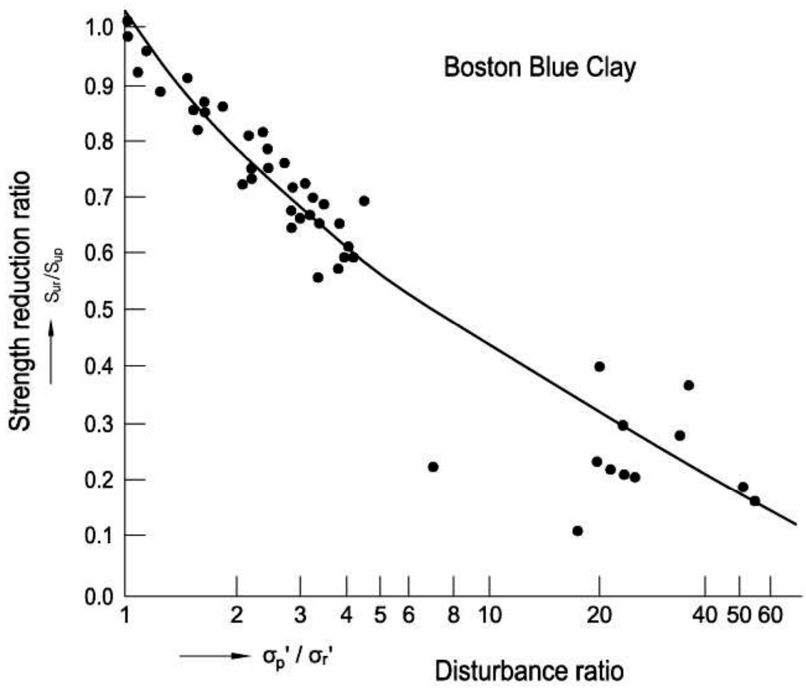
$$R= \sigma'_{ps}/\sigma'_{r} \tag{2.3}$$

where

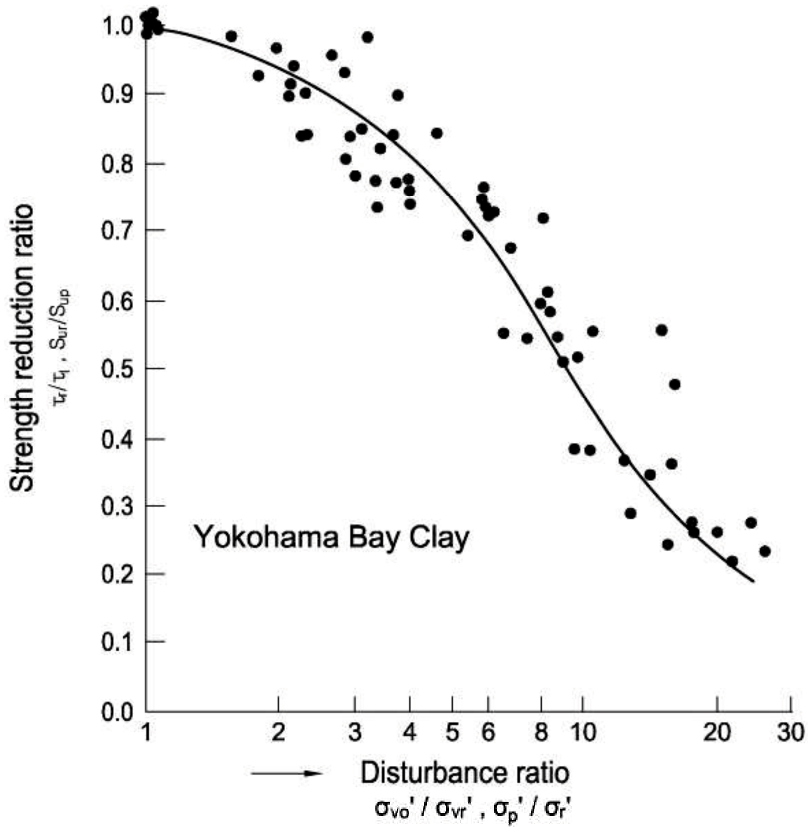
σ'_r = the residual effective stress of the specimen

σ'_{ps} = the effective stress of a perfect sample

Okumura carried out a series of triaxial tests and simple shear tests of reconstituted clay, in which the effect of disturbance was simulated by giving the shear stresses to one dimensionally, consolidated specimens in laboratory tests. Okumura showed that the strength of a perfect sample is in the range of 88 to 93% of an ideal sample and each clay showed a unique relationship between strength reduction ratio, defined as s_{ur}/s_{up} and disturbance ratio. The s_{ur} is the strength of the sample, and s_{up} is the strength of perfect sample. Fig. 2.20(a) and 2.20(b) illustrate the relationship of strength reduction ratio against disturbance ratio for Boston Blue Clay and Yokohama Clay, respectively. As shown in Fig. 2.20, there are some differences in the relations among the clays. Okumura measured directly the residual effective stress σ'_r of undisturbed samples of Kinkai Clay, and reported that the disturbance ratios calculated in Eq. 2.3 ranged from 3 to 6. Recently several researchers have also tried to interpret the sample quality by measuring σ'_r (Hatakeyama and Matsumoto, 1992, Kudoh et al., 1994, Mitachi and Kudoh, 1996, Tanaka et al., 1996, Shogaki and Maruyama, 1998). Tanaka et al. (1996) concluded that the measured residual effective stress of undisturbed samples obtained with a Japanese standard sampler is in the range of 1/4 to 1/7 of effective overburden pressure, which means the disturbance ratio ranges from 2 to 4, as the σ'_{ps} is about 50% of the effective overburden pressure. As for the Boston Blue Clay in Fig. 2.20(a), it was reported that the mean values of σ'_r/σ'_{ps} of medium to soft clay samples collected by tube sampler were 0.2 (Jamiolkowski et al., 1985); accordingly, the mean disturbance ratio was 5. Fig. 2.20 shows that, even if the disturbance ratio is the same, the strength reduction ratio is different for different clays. For example, when R=4, the reduction of strength is 15-25% for Yokohama Clay and about 40% for Boston Blue Clay. Assuming that the disturbance ratio of samples collected by JIS sampler ranges from 2 to 5 and that Fig. 2.20(b) is applicable to Japanese marine clays, the strength reduction will be 5-30% to the perfect sample and 15-40% to the ideal sample. This means that on average, the unconfined compression strength is about 30% smaller than that of an ideal sample in the in-situ effective stress condition.



(a) Boston Blue Clay



(b) Yokohama Bay Clay

Fig. 2.20 Strength reduction ratio versus disturbance ratio (Okumura, 1974)

2) STRENGH ANISOTROPY

The anisotropy of shear strength of soil can be divided into two types, inherent anisotropy and stress system induced anisotropy. The former results from the difference in particle arrangement in the sedimentation process and the latter from the rotation of principal stresses during shear. Practically, a concept of "combined anisotropy" (Ladd et al., 1977) is used, since the separation of the two anisotropies is very difficult. Fig. 2.21 ideally shows the types of shear strength mobilized along a slip surface. The strength mobilized in different zones is different from each other, and this difference is called the anisotropy. Comparing the undrained strengths of compression, extension and simple shear, the compression strength is the highest and the extension strength is the lowest for most soft clays. Accordingly, the ratio between extension and compression strengths represents the degree of strength anisotropy. The strength ratio of extension strength s_{ue}, over compression strength, s_{uc}, is plotted versus plasticity index, I_p, in Fig. 2.22, where data on Japanese marine clays are included. Bjerrum (1972) emphasized the fact that strength anisotropy becomes greater as the plasticity decreases. As shown in Fig. 2.22, the value of s_{ue}/s_{uc} ranges from 0.5 for low plasticity clay to 1.0 for high plasticity clay. Considering the plasticity index of most Japanese marine clays ranges between 30 and 70, it can be said that the strength ratio s_{ue}/s_{uc} is 0.70 ± 0.1 (Tsuchida, 2000).

Berre (1979) proposed the following strength, s_u, corrected on anisotropy for the case where a flat level clay deposit is subjected to loading such as an embankment;

$$s_u = (s_{uc}+2s_{us}+s_{ue})/4$$

Because the simple shear strength, s_{us} is almost the same as the average strength between compression and extension strengths (Hanzawa and Tanaka, 1992), the corrected strength is nearly to $(s_{uc}+s_{ue})/2$.

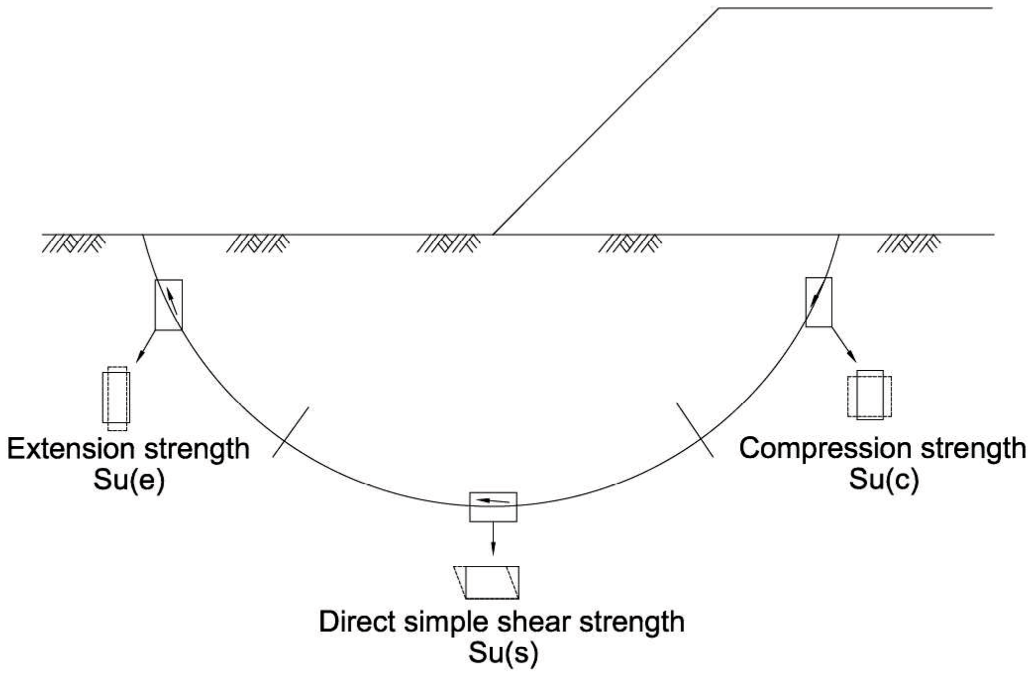


Fig. 2.21 Strength anisotropy (Tsuchida, 2000)

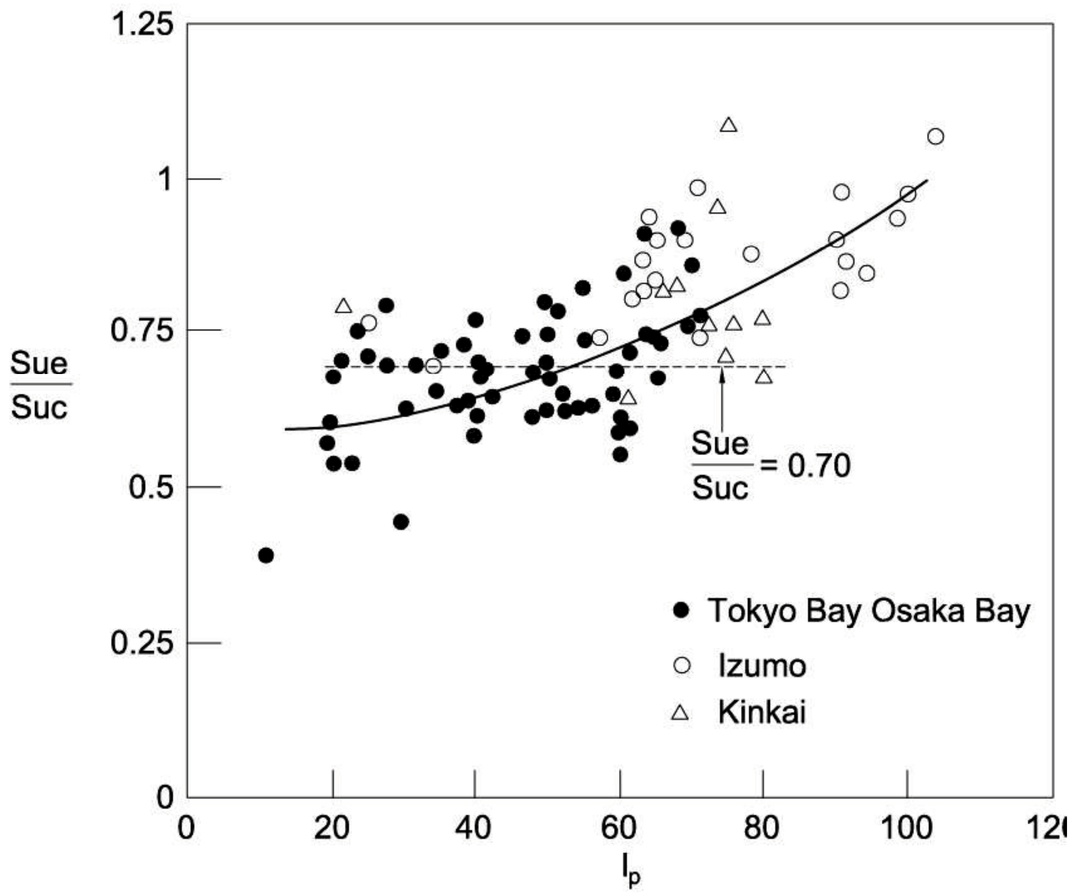


Fig. 2.22 Strength ratio, Sue/Suc with plasticity index (Tsuchida, 2000)

3) STRAIN RATE EFFECT

A lot of researches have been carried out on the effect of strain rate on the undrained strength of clay. Fig. 2.23 indicates the change in undrained strength with strain rate obtained from triaxial compression and extension tests for marine clays of various plasticities (Hanzawa, 1977; Tsuchida et al., 1989). It seems that the effects of the strain rate on undrained strength are not so different among clays whose plasticity indices range from 29 to 62. An important problem is how much strain rate is to be considered for the practical design of earth structures. Ladd and Foote (1974) recommended a rate ranging from 0.008 to 0.015%/min, and Bjerrum recommended a rate of 2 to 3×10^{-5} %/min. According to the case histories reported by Nakase (Nakase, 1967), it took a few to several hours for most observed slip failures of port facilities from the beginning of movement to the completion of failure. The strain rate of 0.01%/min, which corresponds to a few to several hours approximately, seems to be proper for stability analysis.

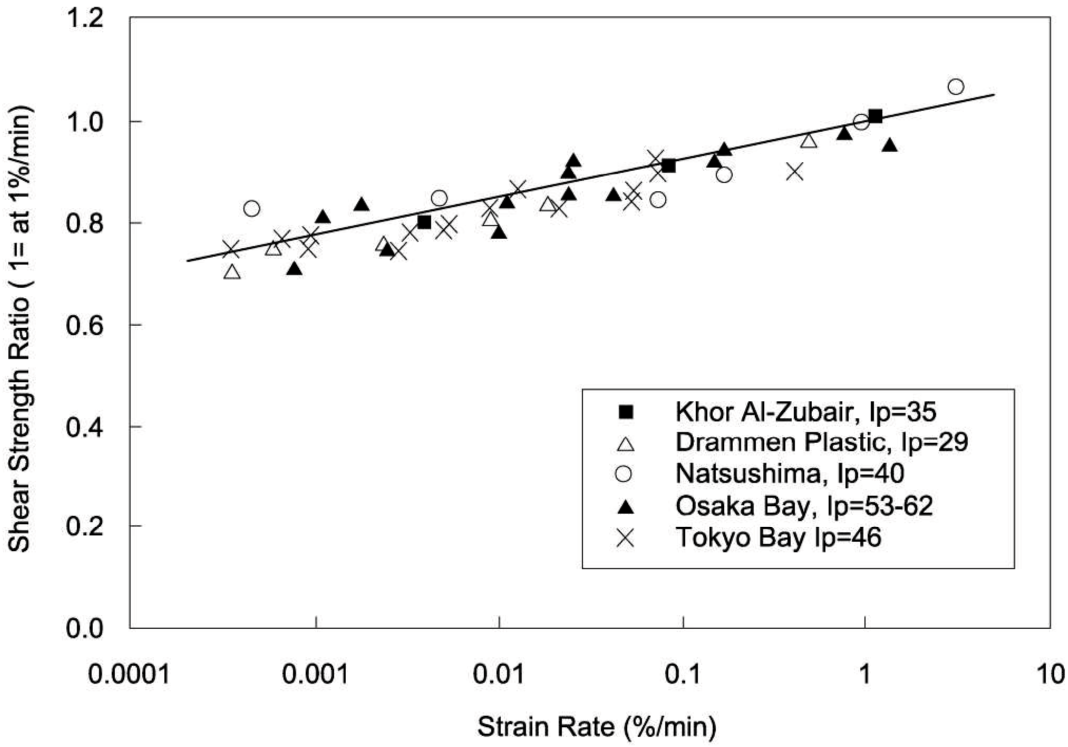


Fig. 2.23 Change in undrained strength with strain rate (Hanzawa, 1977; Tsuchida et al.,

1989)

According to Tsuchida (2000), the unconfined compression strength in an in-situ effective stress condition is about 30% less than that of an ideal sample. Accordingly, the value of c_1 will be 1.0/0.7 = 1.43. The extension strength of clay is about 70% of the compression strength, and since compression is the mode of shearing for q_u, the correction coefficient for the anisotropy, c_2, will be (1.0 + 0.7)/2 = 0.85. Fig. 2.23 indicates the effect of strain rate on the undrained strength. Assuming that the strain rate to be used for the design is 0.01%/min, the conventional strain rate of the Unconfined Compression test, 1.0%/min, is 100 times faster, and the correction coefficient, c_3, for the difference of the strain becomes 0.85 based on the change in the undrained strength, with a strain rate obtained from triaxial compression and extension tests for marine clays of various plasticities (Hanzawa, 1977; Tsuchida et al., 1989). Therefore, the following equation can be obtained:

$$c_1c_2c_3 = 1.43 \times 0.85 \times 0.85 = 1.00 \tag{2.4}$$

Based on the above relationships, $q_u/2$ has been considered to be an accurate value of the mobilized undrained strength. Basically, in Korea, while the practical design for soft clayey ground has been carried out by using a $q_u/2$ value, the results of the Unconsolidated Undrained (UU) test, Field Vane Test (FVT), and CPTU test are used as accessorial methods.

2.2.2 APPLICABILITY OF UNCONFINED COMPRESSION TEST FOR INTERMEDIATE SOILS WITH LOW PLASTICITY

1) UNCONFINED COMPRESSION STRENGTH FOR THE LOW PLASTICITY SOILS

The undrained shear strength ($q_u/2$) profile of the Ishinomaki intermediate soil obtained from the UC test is plotted in Fig. 2.24. The figure also shows the $q_u/2$ profile for the marine clay layer that overlies the intermediate soil. The $q_u/2$ value for the upper clayey layer increases with a strength depth ratio of 2kPa/m. This corresponds to a strength incremental ratio of 0.33, which is a common figure for typical Japanese Marine clays (Tanaka, 1994). The actual $q_u/2$ values obtained for the intermediate soil are much smaller than the linearly extrapolated strength from the clay layer. Nakase et al. (1972) proposed a method to correct the unconfined compression strength of intermediate soils according to the content of sand-sized particles or the I_p (Low Plasticity) value. For many Japanese soils, the correlation between I_p and the content of sand-sized particles has been conducted. However, although this method has been applied to correct the $q_u/2$ of intermediate soils, the value has been reported to be considerably underestimated. Fig. 2.25 shows the profile of strain at failure (ε_f) for the Ishinomaki intermediate soil. The order of ε_f has been considered to be a good indicator of sample quality: the better the quality sample, the smaller the ε_f is. As shown in Fig. 2.25, for the intermediate soil, however, a specimen with a small $q_u/2$ value does not always show a large ε_f. It is usual that the ε_f in the UC test is between 2 and 3% for typical Japanese marine clays, provided that the soil sample is retrieved by a proper sampling method. As shown in Fig. 2.25, for most of the depths at the Ishinomaki site, the failure strain is of the same order as that of typical Japanese marine clays. Therefore, the order of magnitude of ε_f cannot be an indicator for judging whether the sample quality is good enough to provide a suitable $q_u/2$ value for design.

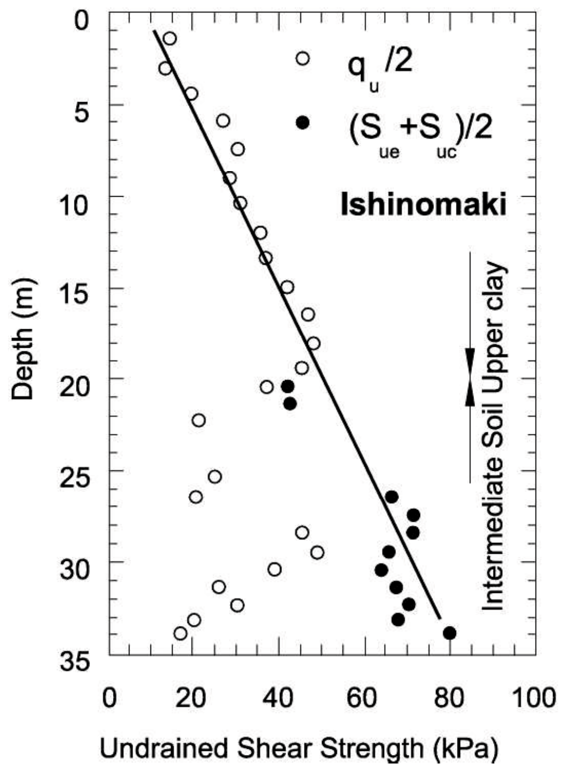


Fig. 2.24 $q_u/2$ from UC test and average strength from recompression triaxial tests for Ishinomaki intermediate soil (s_{uc} and s_{ue} measured by compression and extension triaxial tests, respectively) (Tanaka et al., 2001)

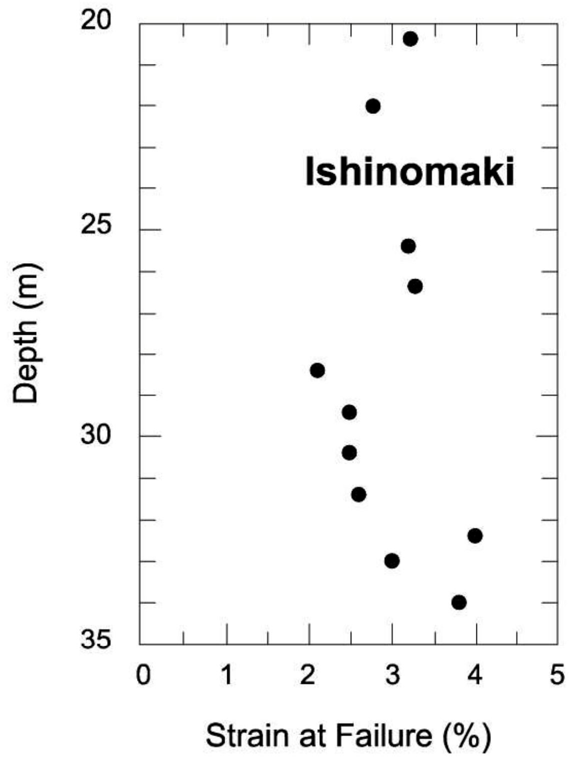


Fig. 2.25 Strain at failure of UC test for Ishinomaki soil (Tanaka et al., 2001)

Fig. 2.26 shows the $q_u/2$ profile for the Drammen clay, together with the undrained shear strength estimated by the CPTU, assuming a cone factor (N_{kt}) of 15. At depths shallower than 12m, the $q_u/2$ value coincides, more and less, with the strength from CPTU. At depths larger than 15m, however, the $q_u/2$ values are much smaller than the strengths estimated by CPTU. The shear strength estimated by CPTU is directly influenced by the N_{kt} factor, which may be dependent on soil properties. However, it is hard to believe that the N_{kt} factor suddenly changes at a depth of 15m. Therefore, it may be inferred that the UC test could not properly evaluate the undrained shear strength at depths larger than 12m. As shown in Fig. 2.27, ε_f for the Drammen clay is relatively larger than that for the Japanese soft clays. The order of ε_f becomes larger especially when the depth exceeds 12m, where significant discrepancy between $q_u/2$ and s_u from CPTU was observed.

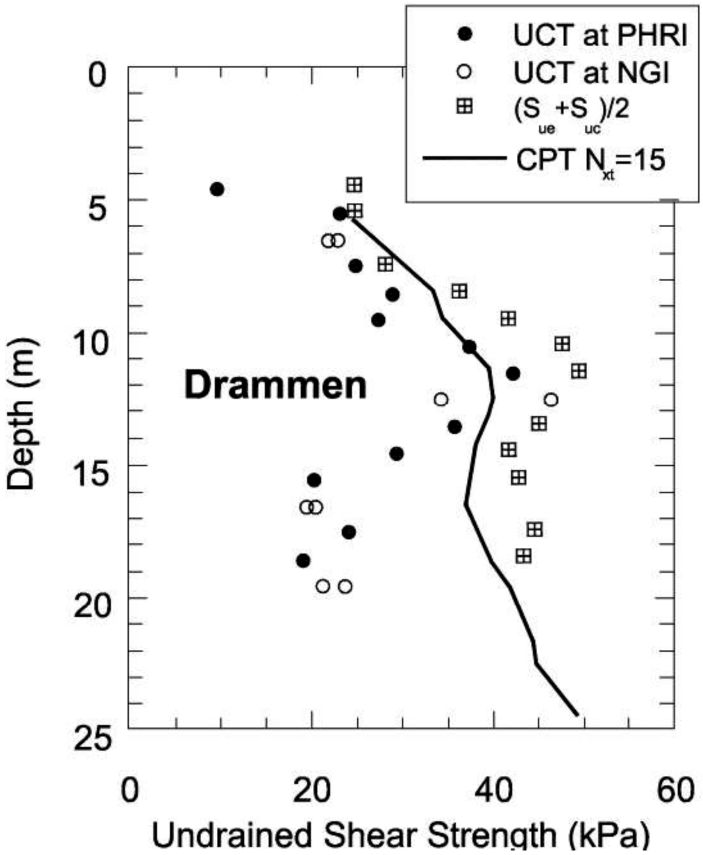


Fig. 2.26 $q_u/2$ from UC test, mean strength from recompression triaxial tests and the strength estimated by CPTU for Drammen clay (Tanaka et al., 2001)

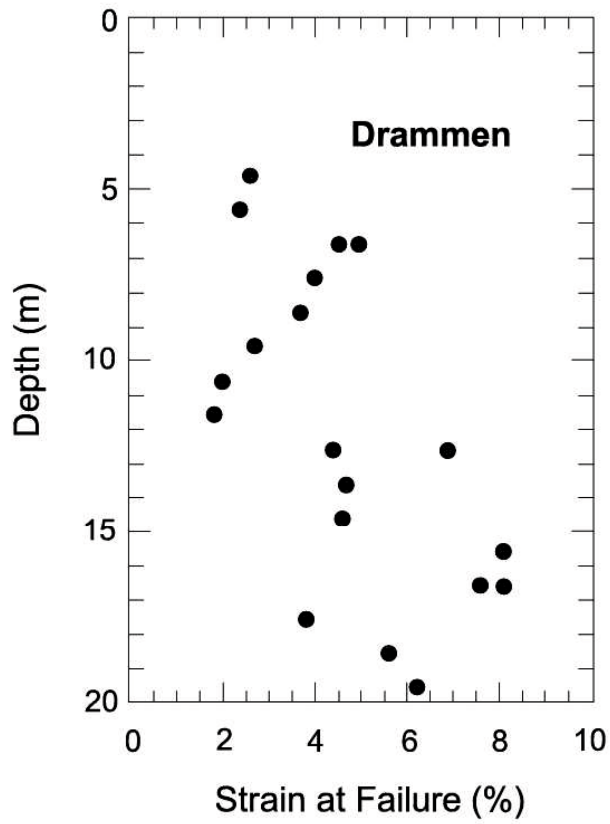


Fig. 2.27 Strain at failure of UC test for Drammen clay (Tanaka et al., 2001)

2) RESIDUAL EFFECTIVE STRESS

When a soil specimen is retrieved from the ground and is exposed to the atmosphere, part of the in-situ effective stress in the specimen remains in the form of negative pore pressure. This negative pore pressure is called residual effective stress (p'_r). Even though the specimen is tested under unconfined conditions, the residual effective stress acts as a confining pressure. However, if the value of p'_r in the specimen is reduced due to a disturbance of the specimen during sampling, handling, extruding, or trimming, then the $q_u/2$ value will be reduced due to swelling. According to Tanaka et al. (1996), the order of p'_r for high-quality samples is around 1/5 to 1/6 of the in-situ vertical effective stress (p'_{vo}) for normally consolidated and slightly over-consolidated clays. Fig. 2.28 shows the p'_r value measured for clays in various parts of the world. Values of p'_r for all the soil types are of the order of 1/5 or 1/6 of p'_{vo} except for the Drammen clay.

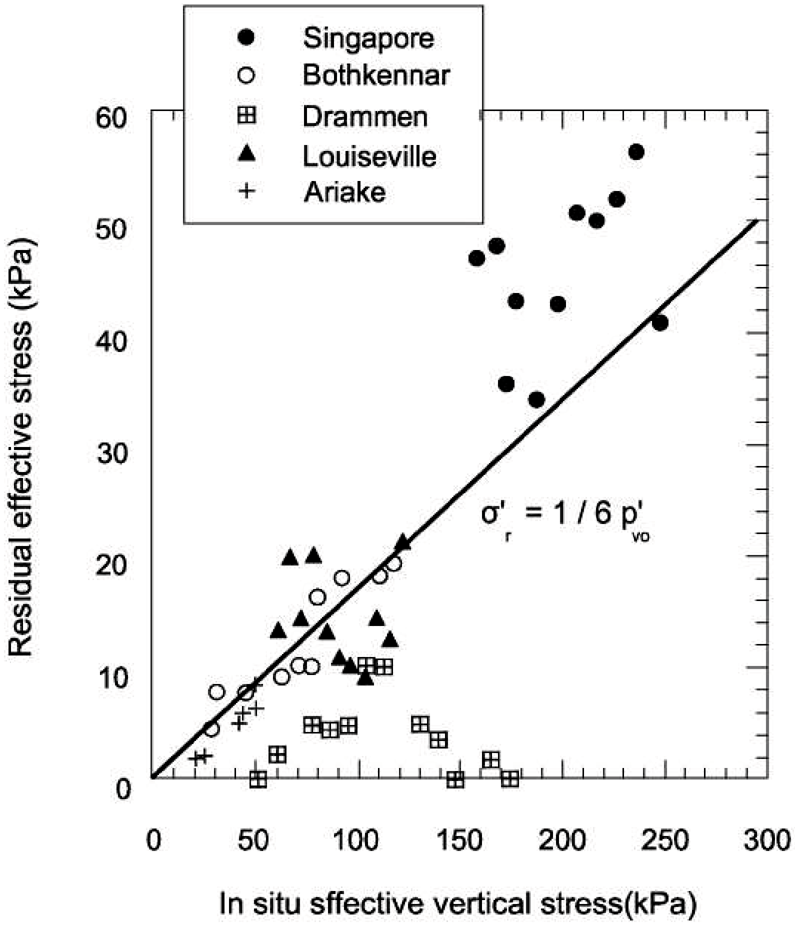


Fig. 2.28 Relation between residual effective stress and in-situ effective burden pressure for various regions (all samples collected by Japanese standard sampling method) (Tanaka et al., 2001)

Tanaka et al. (2001) investigated two soils with a low plasticity: intermediate soil from Ishinomaki, Japan and lean clay from Drammen, Norway. The p'_r values measured for the Ishinomaki intermediate soil and the Drammen clay are shown in Fig. 2.29. It is found that the p'_r values measured for the Ishinomaki intermediate soil and the Drammen clay were much smaller than those measured for ordinary clay, for which p'_r is approximately $1/6p'_{vo}$. Accordingly, the loss of the residual effective stress (p'_r) is too significant to be compensated for by factors overestimating the strength. Therefore, the validity of the test for clayey soils is not applicable to intermediate soils. In evaluating the undrained shear strength of soil with a low plasticity index, effective confining pressure should be applied to a soil specimen in order to compensate for the lost residual effective stress.

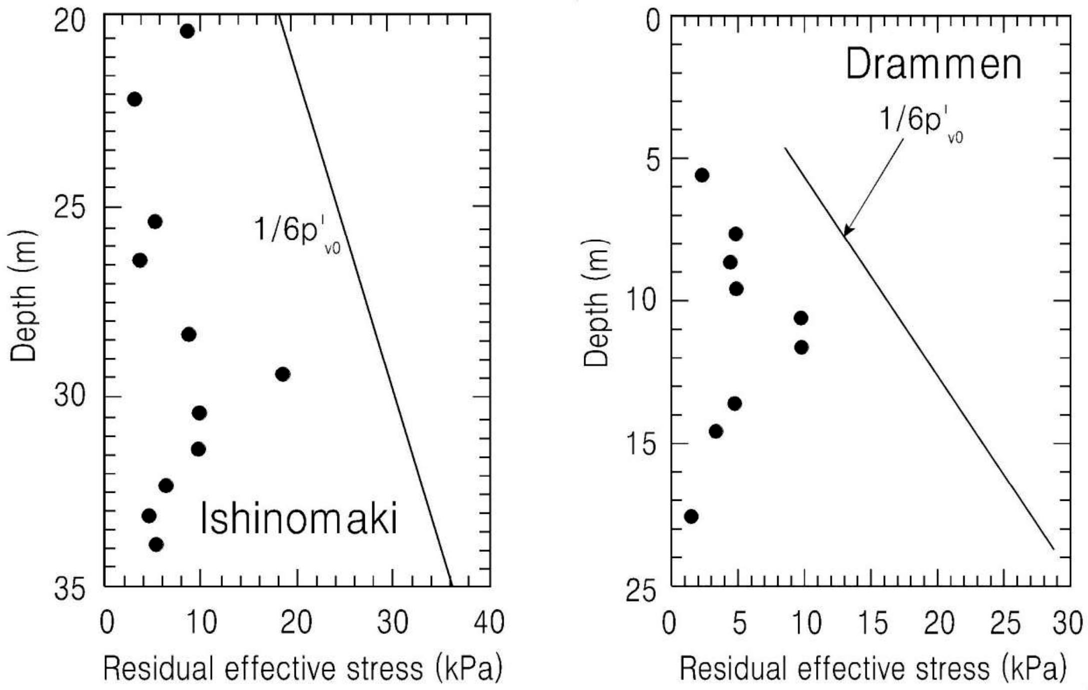


Fig. 2.29 Distribution of residual effective pressure for Ishinomaki intermediate soil and Drammen clay (Tanaka et al., 2001)

2.2.3 ESTIMATION OF UNDRAINED SHEAR STRENGTH BY USING CPTU DATA

The Piezocone Penetration Test (CPTU) has been widely used for several decades because it is the most effective in-situ test method that can be used to practically obtain the soil properties. The CPTU has been mainly used for three applications: (1) to estimate the soil properties through an appropriate correlation; (2) to directly perform geotechnical design from the CPTU data; and (3) to determine the subsurface stratigraphy. Numerous attempts have been made over the years to develop reliable analytical models to simulate the cone penetration process and to derive the proper correlations with soil properties when empirical CPTU results are used. In particular, many researchers have attempted to deduce the undrained shear strength (s_u) of the cohesive soils from the measured cone resistance by utilizing a correlation between s_u and cone resistance for a given site. Estimating s_u from CPTU through the use of the net cone resistance (q_t-σ_{vo}) is done, as shown in the following equation:

$$s_u=(q_t-\sigma_{vo})/N_{kt} \tag{2.5}$$

where s_u: undrained shear strength

q_t: corrected cone resistance for pore water pressure effects

σ_{vo}: in-situ total overburden pressure

N_{kt}: termed cone factor.

Aas et al. (1986) presented a correlation between cone factor and the plasticity index obtained from Norwegian and North Sea clays. They used the following average values of s_u in the laboratory.

$$s_u=(s_{uc}+s_{ud}+s_{ue})/3 \tag{2.6}$$

Where s_{uc}, s_{ud} and s_{ue} are the undrained shear strengths from triaxial compression, direct simple shear and triaxial extension tests, respectively.

The results suggested that the value of cone factor increased as PI increased, and the value of cone factor varied from 12 to 21. La Rochella et al. (1988) did not find any correlation between the cone factor and the PI, when they compared CPTU data with results of FVT on Eastern Canada clays. The data obtained by La Rochella et al. (1988) showed a slightly smaller cone factor varying from 11 to 18. Tanaka et al. (1996) investigated marine clays in Japan and found that the cone factor varied from 8 to 16 when the CPTU data were compared with $q_u/2$ data and from 9 to 14 when the CPTU data were compared with FVT data. Tanaka et al. (1996) reported that there exists no special correlation between the cone factor and PI, supporting the observation by La Rochella et al. (1988). Jang et al. (2001) investigated marine clays in Korea and found that the cone factor varied from 9.7 to 14.3 with OCR, based on comparing 46 CPT data of 10 test sites with undrained shear strength obtained from FVT and laboratory triaxial tests. Jang et al. (2001) also suggested that there exists no special correlation between the cone factor and PI. Kjekstad et al. (1978) and De Ruiter (1982) investigated how N_{kt} increased as OCR increased. On the other hand, Tanaka and Sakagami (1989) and Fukasawa et al. (2004) indicated that the relationship between the cone resistance and the undrained shear strength is not affected by the state of consolidation of the ground.

2.2.4 APPLICABILITY OF NORMALIZED UNDRAINED SHEAR STRENGTH (s_u/p'_c) BY USING PLASTICITY INDEX

Skempton (1957) established the following empirical equation for normally consolidated clays based on field vane test results.

$$\frac{s_u}{p'_{vo}} = 0.11 + 0.0037\,(PI) \tag{2.7}$$

where s_u and p'_{vo} are the undrained shear strength and the effective overburden pressure, respectively. This statistical relationship has been found to be broadly valid over a wide range of types of sedimented clays. In particular, when the equation is used for designing soft ground improvement in Korea, it has been used as a design standard in almost all cases, such as in port and harbor design codes. This relationship is usually used to estimate the undrained shear strength of samples from Atterberg limit tests, if the deposit is known to be normally consolidated. In addition, if the values of s_u and I_p have been determined, the correlation is also utilized to evaluate whether a deposit is preconsolidated or not. Since it was presented that s_u, normalized by the effective overburden pressure, s_u/p'_{vo}, increases with increasing I_p, many researchers have discussed this correlation. It has been known that, granted that the ground is in a normally consolidated state, sometimes, the yield consolidation pressure (p'_c) is bigger than effective overburden pressure, and that because of delayed consolidation effects such as cementation and secondary consolidation, the undrained shear strength for these delayed consolidated marine clays is also proportional to p'_c. It is well known that $s_{u(FVT)}$ is strongly influenced by OCR. Jamiolkowski et al. (1985) suggested a more general form to express the field vane shear strength, as shown in equation (2.8):

$$(s_{u(v)}/p'_o)_{oc}=(s_{u(v)}/p'_o)_{nc}OCR^m \tag{2.8}$$

where $(s_{u(FVT)}/p'_o)_{oc}$ is the strength ratio of the overconsolidated clay and $(s_{u(FVT)}/p'_o)_{nc}$ is that of the normally consolidated one. Since many test results have suggested that the m values are nearly equal to 1.0, the field vane shear strength for most natural clays is proportional to p'_c (Leroueil and Jamiolkowski 1991).

As shown in Fig. 2.30, this s_u/p'_c ratio seems to increase with increasing I_p (Larsson 1980). In addition, it can be seen that the s_u/p'_c ratio obtained by Bjerrum (1954) in the low I_p range and by Hansbo (1957) in the high I_p range, measured in Scandinavia, depends on I_p. According to Hansbo's original reference quoted by Larsson (1980), s_u was not normalized by p'_c but by p'_o. In addition, the I_p values obtained by Hansbo (1957) decreased with depth. Therefore, it could be assumed that high I_p values were obtained near the ground surface. Clays in the near ground surface are likely to be in the overconsolidated state, because of the variation in the ground-water level or the desiccation effect. Since, in this case, s_u was increased by the overconsolidation effect, it could be considered that due to this effect, s_u/p'_o was bigger than s_u/p'_c for the same I_p.

On the other hand, there have been strong criticisms to uniquely relating s_u to the I_p value. Based on extensive unconfined compression, triaxial and direct shear test results on alluvial and diluvial clays of the Osaka District by Mikasa (1967, 1968) have suggested that there is no unique relationship between s_u and I_p, and insisted that physical properties like I_p could not work as an index for directly evaluating the shear strength characteristics of clay. Schmertman and Morgenstern (1977) and Hanzawa (1982, 1983) also insisted that s_u is much more dependent on regional variations, such as the formation process, than the I_p value. Leroueil and Jamiolkowski (1991) suggested that the s_u/p'_c ratios for non-organic Eastern Canadian clays do not depend on I_p and have a range between 0.25 and 0.35 for irrespective of I_p. As shown in Fig. 2.31, another result regarding the independency of s_u/p'_c from I_p was suggested by Tanaka (1994), based on Japanese clays together with the ratios for Bothkennar clay in Great Britain, which are plotted using data from Nash et al. (1992). In both cases, it can be seen that meaningful relationships do not exist between s_u/p'_c and I_p. The ratios for Japanese marine clays range from 0.20 to 0.35. It seems that the s_u/p'_c ratios for Japanese marine clays for same I_p are a little bigger than that for Bothkennar clay.

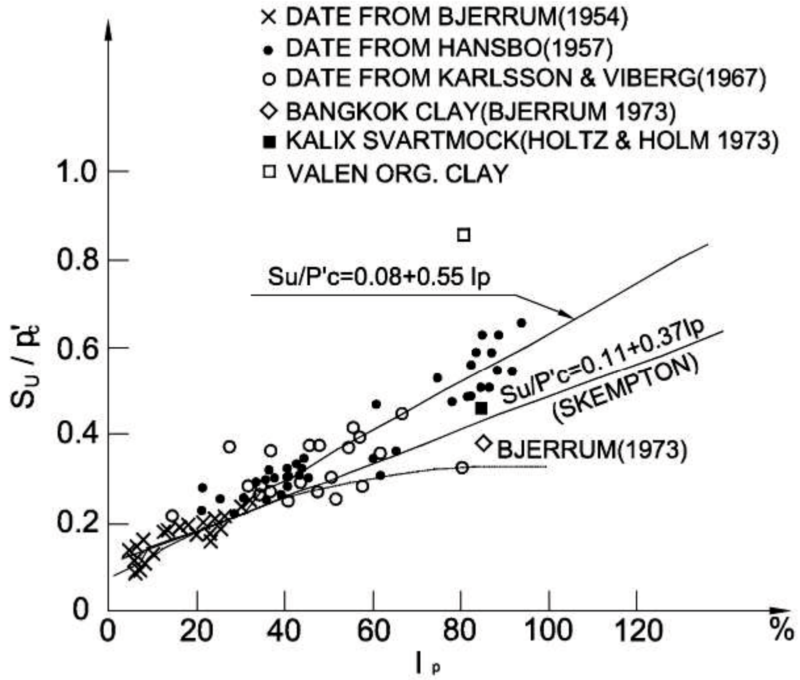


Fig. 2.30 The field vane strength normalized by yield consolidation pressure versus plasticity index (Larsson 1980)

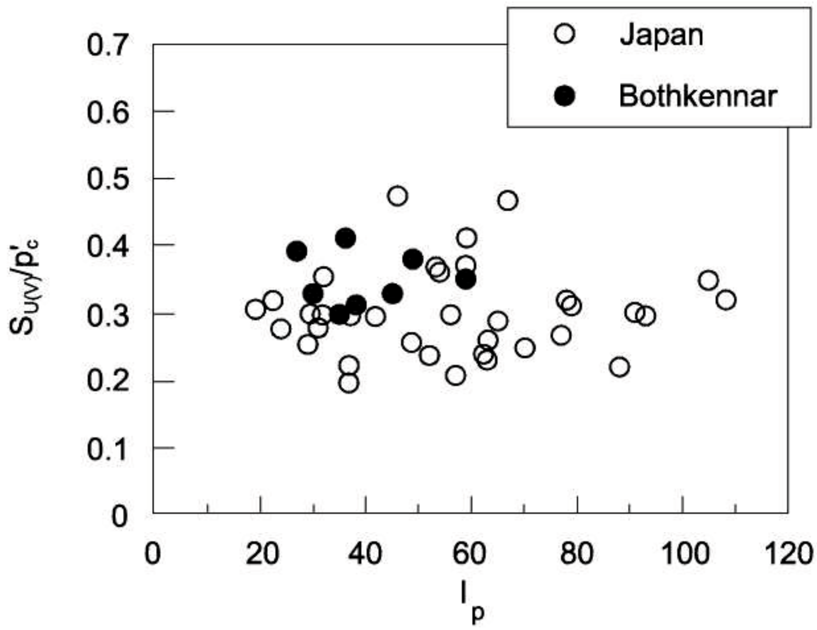


Fig. 2.31 Comparison of vane shear strength normalized by yield consolidation pressure for Japanese and Bothkennar clays (Nash et al. 1992, Tanaka 1994)

Meanwhile, normalized undrained shear strength depends on the mode of shear and differs in different directions in the ground. The vane undrained shear strength is the average mobilized value for the modes of shear that occur along a circular surface of sliding in field instabilities. The values of undrained shear strength mobilized in different directions can be measured by laboratory tests that simulate district modes of shear. The tests most widely used are triaxial compression, triaxial extension, and direct simple shear.

As shown in Fig. 2.32, $s_u/p'_{c(CK_oTCT)}$ is bigger than $s_u/p'_{c(CK_oTET)}$ and $s_u/p'_{c(DST)}$ is equal to the average of $s_u/p'_{c\ (CK_oTCT)}$ and $s_u/p'_{c(CK_oTET)}$ (Hanzawa and Tanaka, 1992). The ratios by Ladd (1973) indicate a symmetry above and below 1.0, and locate more apart from 1.0 as PI decreases, which demonstrates that $s_{u(DST)}$ is equal to the average strength of $s_{u(CK_oTCT)}$ and $s_{u(CK_oTET)}$ and the strength anisotropy is greater the lower the PI. But, such correlations could not be observed in the ratios by Berre and Bjerrum (1973). It is difficult to find out any correlation between these ratios and PI from Hanzawa and Tanaka (1992).

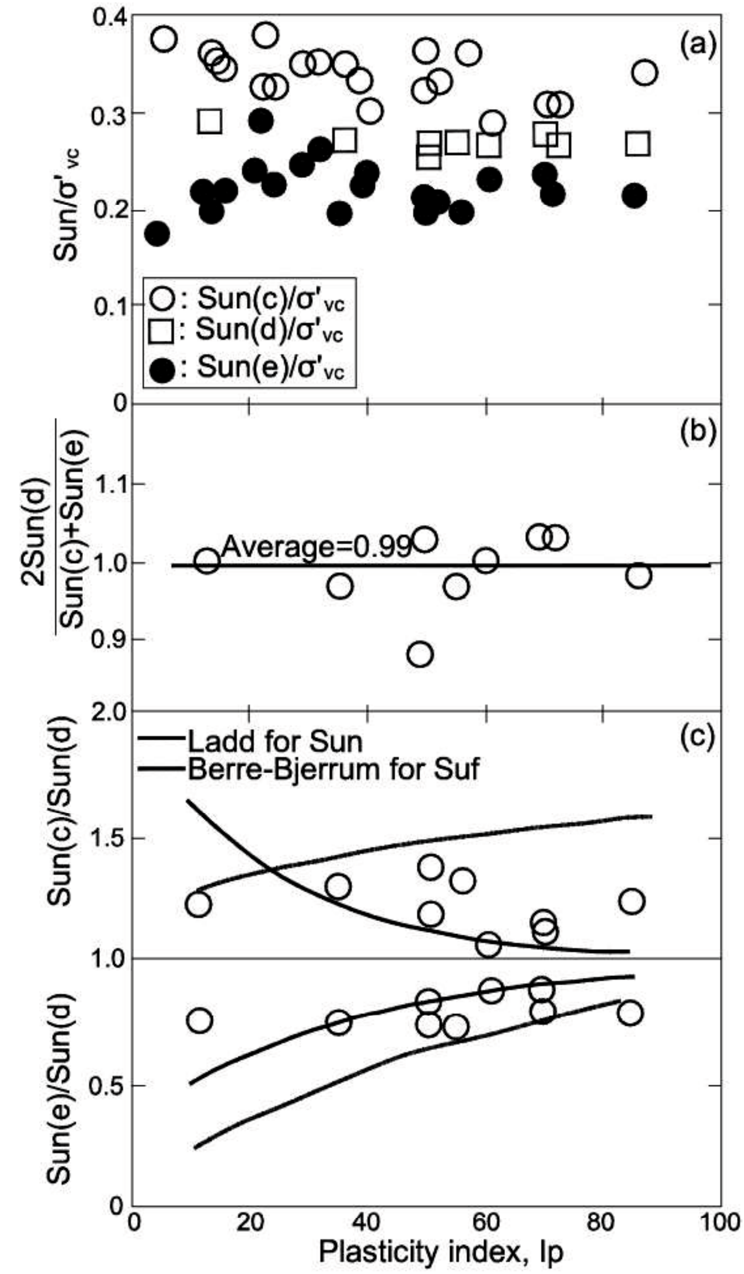


Fig. 2.32 Normalized undrained shear strength with different methods plots versus plasticity index

2.2.5 MOBILIZED UNDRAINED VANE SHEAR STRENGTH USING CORRECTION FACTORS

In general, it is considered that the vane strength is not equal to the mobilized strength, and that a correction factor has to be introduced before the vane strength could be used in a stability analysis. For example, a computed factor of safety greater than one for a failed embankment on a soft clay deposit indicated that $S_{u(FVT)}$ was greater than the average strength value mobilized along the surface of sliding at failure. Reduction factors were then recommended by the Swedish Geotechnical Institute for the field vane undrained shear strength (Osterman 1960). Bjerrum (1973) analyzed well-documented embankment, footing, and excavation failures in terms of the field vane undrained shear strength. The computed factors of safety based on circular arc analyses were plotted against the plasticity indices of the soils and a straight line was fitted through the data points. Using this line, Bjerrum recommended the in-situ vane correction factor μ=1/FS shown in Fig. 2.33. The mobilized undrained shear strength is computed from

$$S_{u(mob)} = \mu\, S_{u(FVT)}\ ,\ \mu = 1.7 - 0.54\log(I_p) \tag{2.9}$$

Additional data for embankment failures, compiled by Ladd et al. (1977) Menzies and Simons (1978) and Tavenas and Leroueil (1980) show more scatter of the individual cases from the Bjerrum correction curve. The scatter in part is related to differences in equipment and procedures used for determining the vane strength and plasticity index. The scatter is also related to the influence on the computed factor of safety of ignoring the three-dimensional end effects of the surface of sliding (Azzouz et al., 1981, 1983), and of the difference in strengths mobilized within the embankment and in the crust on the underlying foundation soil (Lefebvre et al, 1988). As pointed out Ladd and Foott (1974), it is assumed that the discrepancy in measured values is primarily due to the strain rate effect, and then it was concluded that the strain rate effect on s_u is greater the higher the I_p value, as shown in Fig. 2.33. However, it should be pointed out that the $s_{u(FVT)}$ value is affected by the clay properties such as whether or not sand seams, shells and organic materials are contained in the clay. Such clays give a higher $s_{u(FVT)}$ value, resulting in a lower correction factor, μ. The factor of safety is also dependent on the drainage conditions during construction. These points are not clear in the method proposed by Bjerrum.

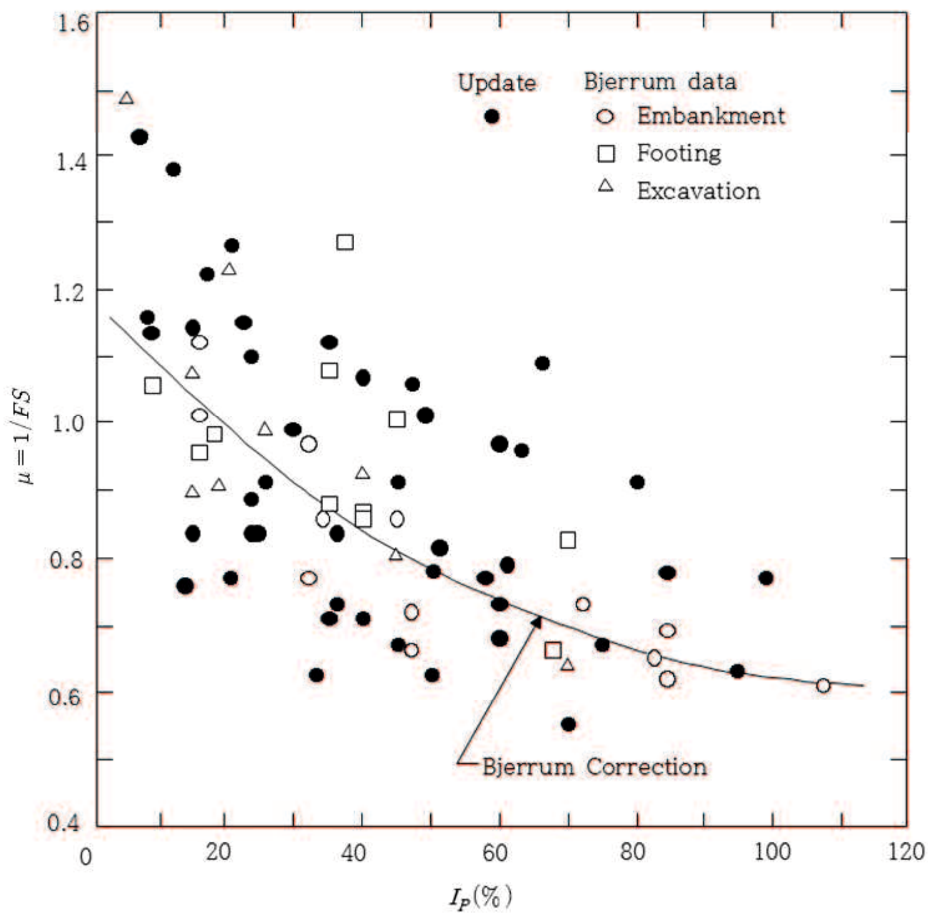


Fig. 2.33 Bjerrum's field vane correction factor

Meanwhile, Morris and Williams (1994) proposed correction factors for the field vane shear strength using theoretical analysis in terms of effective stress parameters. The strength estimates are based on known in-situ stresses and soil parameters derived from laboratory tests.

A correction factor, μ_v, was presented to compensate for pore-pressure and shearing-rate effects. Based on a review of the vane shear strength and Atterberg limit data from 28 different clays from various sites, it has been shown that strong correlations exist between μ_v and I_p and w_L, as shown in Fig. 2.34. These correlations have similar trends, and there is a statistical significance to the analogous correction between Bjerrum's (1973) total stress field vane correction factor μ and I_p. The correction factors are as follows.

$$\mu_v = 1.18\exp(-0.18I_p)+0.57 \qquad \text{for } I_p>5$$

$$\mu_v = 7.01\exp(-0.08w_L)+0.57 \qquad \text{for } w_L>20 \tag{2.10}$$

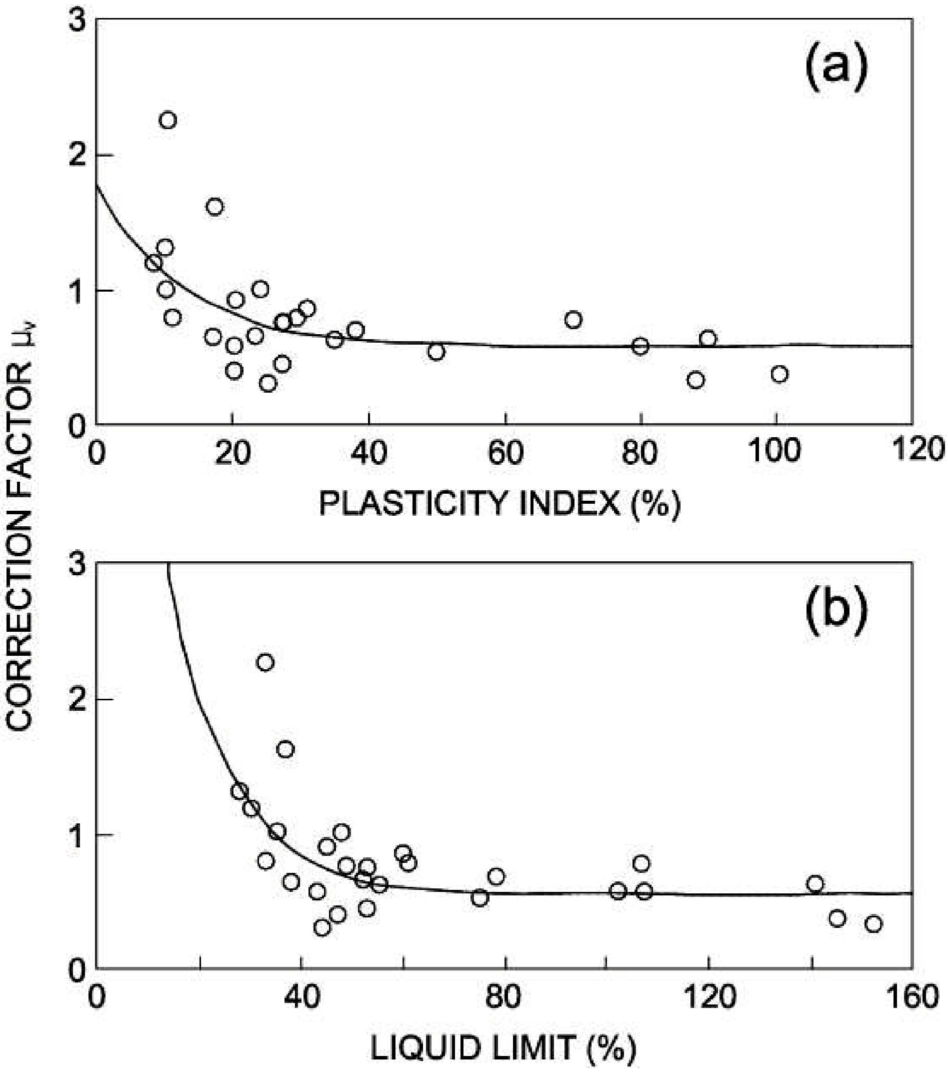


Fig. 2.34 Correction factor versus (a) the plasticity index I_p and (b) the liquid limit w_L (Morris and Williams 1994)

Meanwhile, Mesri (1975) showed that by associating Bjerrum's correction factor in Fig. 2.33 with the relationship between $s_{u(FVT)}/p'_c$ and the plasticity index in Fig. 2.35, the mobilized undrained shear strength for stability analysis can be expressed, independently of I_p as shown in Eq. (2.9).

$$s_{u(mob)}=0.22p'_c \tag{2.11}$$

This equation was derived from Bjerrum's data, which consists of s_u/p'_o and p'_c/p'_o for young as well as aged clays, as well as results for μ. Although the mobilized undrained shear strength, obtained by normalizing the yield consolidation pressure shows a constant value, independently of the plasticity index, this equation was derived from based on combining Bjerrum's curve in which $s_{u(FVT)}$ normalized by the yield consolidation pressure, $s_{u(FVT)}/p'_c$ increases with increasing I_p with Bjerrum's field vane correction factor. So, intrinsically, this equation is influenced by plasticity index, I_p. Eq. (2.11) suggests that the incremental strength ratio for the preloading method must be 0.22.

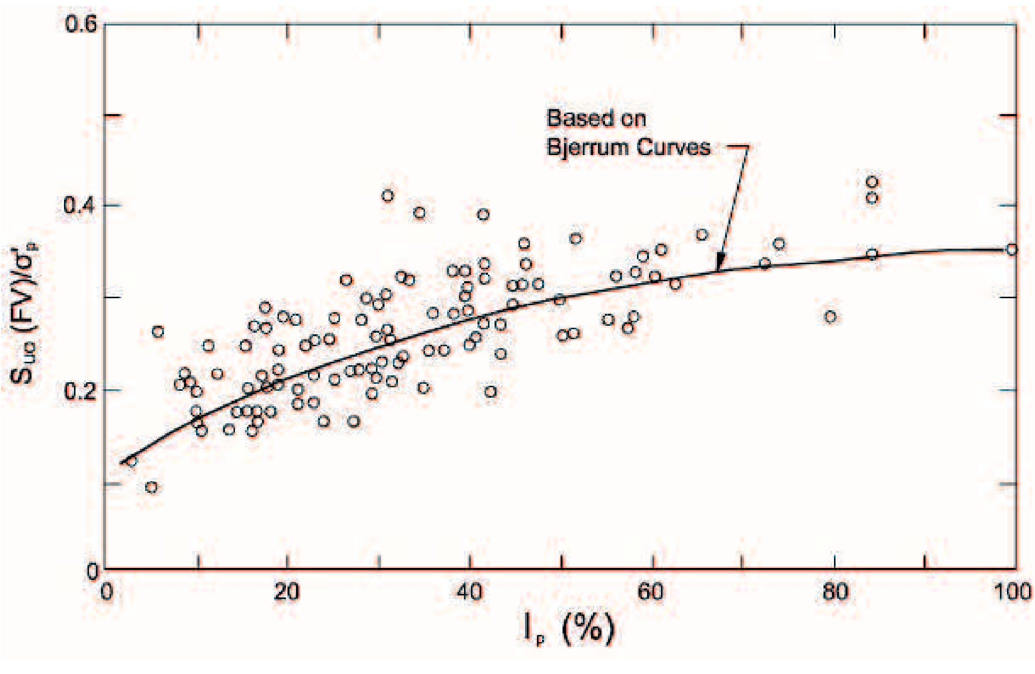


Fig. 2.35 Undrained shear strengths from field vane tests on inorganic soft clays and silts (Mesri 1975)

2.3 THE ASSESSMENT OF PARTIALLY DRAINED CONDITIONS

2.3.1 THE ASSESSMENT OF PARTIALLY DRAINED CONDITIONS FROM PHYSICAL PROPERTIES OF SOILS

According to the design code for the construction of port and harbor facilities in Japan, soils with between 50% and 80% of sand-sized content are called intermediate soils because these show an intermediate behavior between sand and clay. For intermediate soils, if the permeability of soil is less than 10^{-4} cm/s, the stability analysis and the evaluation of s_u follow those of clayey soils, if the permeability of soil is more than 10^{-4} cm/s, the stability analysis and the evaluation of shear strength follow those of sandy soil (Japanese Port Association, 2007). Fig. 2.36 shows the relations between sand contents, plasticity index and consolidation coefficients. Consolidation coefficients, c_v dramatically increased at around 50% of sand content and 20% of plasticity index, respectively (Kamei, 1992).

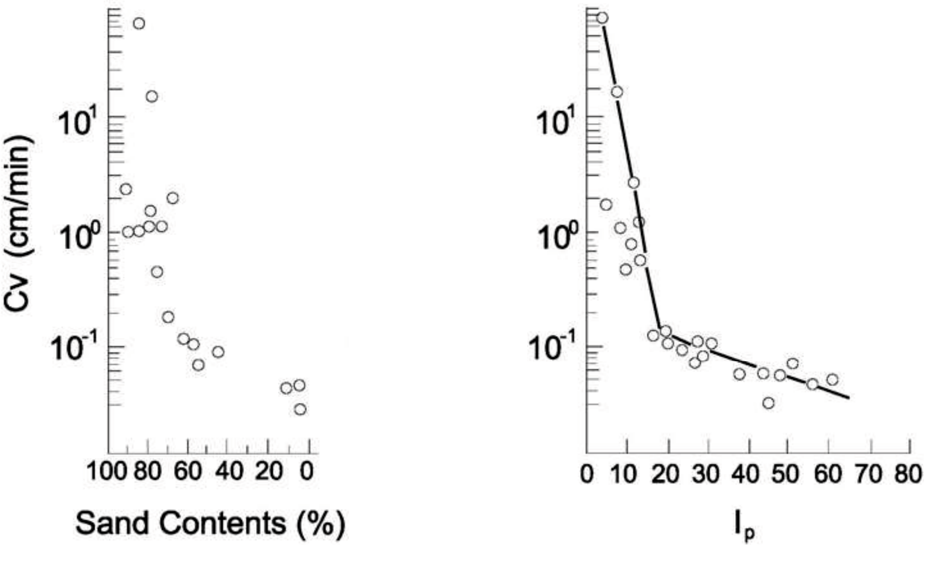
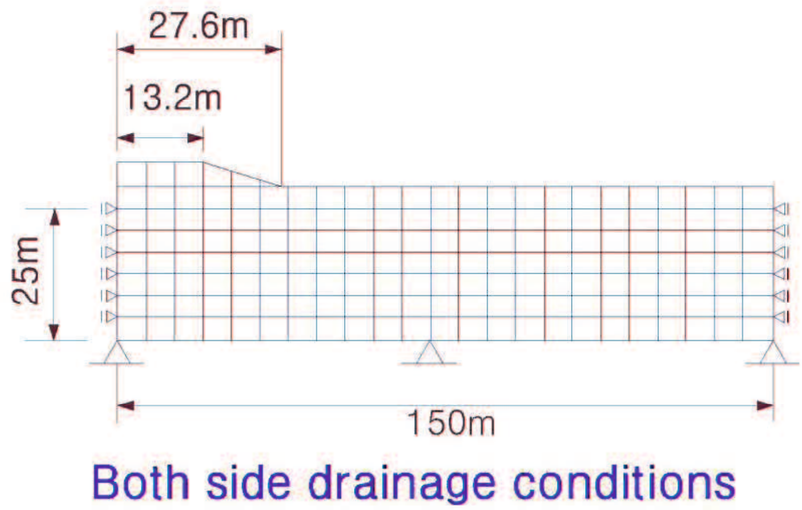
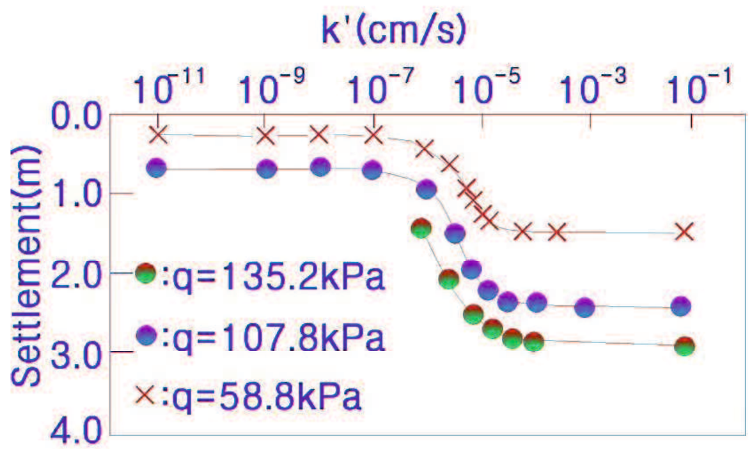


Fig. 2.36 The relations between sand contents, plasticity index and consolidation coefficients (Kamei, 1992)

In addition, Asaoka et al. (1989) analyzed whether or not the partially drained conditions were present, based on conducting a Finite Element Method (FEM) analysis that focused on changes in the permeability of the soft ground; that is, the settlement of the soft ground dramatically increased as the embankment loading increased on the soft ground at a range of permeability between 10^{-7} cm/s and 10^{-4} cm/s. As shown in Fig. 2.37, Asaoka et al. (1989) suggested that the range of the partial drainage was between 10^{-7} cm/s and 10^{-4} cm/s, based on the FEM analysis. Geotech Note No. 2 from previous researches, (Japanese Geotechnical Society, 1992) states that ground with a plasticity index between NP and 25%, permeability between 10^{-7} cm/s and 10^{-4} cm/s, and a coefficient of consolidation between 144 cm^2/day and 14,400 cm^2/day can be classified as intermediate soils.



(a) FEM modeling considering the loading sequence



(b) Settlement of the completed construction ground

Fig. 2.37 Effect on the changed permeability of clayey ground concerning partially drained behavior (Asaoka, 1989)

2.3.2 THE ASSESSMENT OF PARTIALLY DRAINED CONDITIONS FROM CPTU DATA

The conventional cone penetration test (CPTU) occurs under conditions that range from fully undrained in clay to fully drained in clean sand at the standard penetration rate of 2 cm/s according to the International Reference Test Procedure (IRTP, 1999) and the ASTM standard (ASTM D 5778). This standard penetration rate is specified regardless of soil type. For the intermediate soils-silty sands, clayey sands, sandy silt, etc.-partially drained conditions pertain. Complications can arise in estimating the soil behavior type and properties, as well as interpretation of pore pressure dissipation data, for CPTUs conducted under partially drained conditions. Drainage conditions during cone penetration tests (CPTU) are largely dependent on the rate of dissipation of soils and therefore on the distribution characteristic of the grain size of soils. Measurements of the pore pressure (u) are therefore extremely useful for assessing the characteristic grain size of soils in addition to the other piezocone records to furnish the geotechnical design parameters which depend on whether the penetration is fully drained or undrained. It is generally accepted that undrained penetration occurs in soft to firm clay soils when a standard piezocone is inserted at 2 cm/s speed. On the other hand, standard CPTUs in intermediate soils such as silts and clayey sands vary from undrained to partially or fully drained penetration. The degree of drainage is associated with the content of fine-grained particles with soil interlocking and therefore with soil permeability and compressibility properties. However, the fact that the penetration rate affects the value of the cone penetration resistance q_c for these soils was not taken into account at the time that standards were prepared for the CPTU. Therefore, the direct quantitative evaluation of material properties using procedures which were developed for the drained behavior of sand or the undrained behavior of clay cannot be appropriate. It may be considered that in interpreting CPTU data, the permeability/drainage effects are of primary importance and strength, density, and overburden effects are of secondary importance. The assessment of the drainage conditions during penetration is therefore the first step to analyze the piezocone parameters in such soils. Physically, drainage conditions during penetration are important because, if the penetration rate is sufficiently low for a given soil, the soil ahead of the cone consolidates during penetration, thereby developing larger shear strength and stiffness than it would have under undrained conditions. The closer the conditions are to fully drained during penetration, the higher the value of q_c. Another physical process that is at play for soils with large clay content for penetration under fully undrained conditions is the effect of the rate of loading on shear strength. The higher the penetration rate is, the larger the undrained shear strength, s_u is, and the larger the q_c.

Kim et al. (2006) conducted field CPTUs in the state of Indiana, US under various penetration rates to clarify the rate effect on the cone tip resistance due to the soil type in the field and investigate the effects of drainage conditions during penetration on q_c by varying the penetration rate and the type of soil. The average of silt and clay contents is 73%, 17%, respectively. The average of liquid limit (LL) and plasticity index (PI) is 31%, 12%, respectively. Therefore, this ground can be classified as intermediate soil, which is likely to be under partially drained conditions. As shown in Fig. 2.38 and Fig. 2.39, values of cone resistance for velocities of 20 mm/s and 1 mm/s were almost the same (around 0.8 MPa), but the excess pore pressure for 1mm/s decreased to an average 210 kPa, compared to an average 270 kPa for 20 mm/s. Considering the cone resistance for 20 mm/s and 1 mm/s penetration rates, there is no apparent rate effect. In contract, the decrease in the excess pore pressure at 1 mm/s indicates that partially drained conditions are in effect around the cone tip. In this range, the increase of cone resistance due to the change in drainage conditions is small and compensated for by the decrease in pore pressure acting on the surface of the cone tip. The increase in cone resistance due to the change of drainage conditions appears clearly for velocities less than 1 mm/s. With decreasing cone velocity below 1 mm/s, q_t increases significantly and excess pore pressure decreases. Cone resistance at the lowest velocity of 0.05 mm/s was about 2 MPa, 2.5 times larger than the cone resistance for 20 mm/s. Even with the lowest velocity (0.05 mm/s), partially drained conditions are still in effect because the measured pore pressure at 0.05 mm/s was around 65 kPa, and this value is still greater than the hydrostatic pressure of 45 kPa at that depth.

The cone resistance and excess pore pressure under partially drained conditions measured for 0.2 mm/s, 0.1 mm/s, and 0.05 mm/s fluctuate. This is because the degree of consolidation of the surrounding soils undergoes rapid change under partially drained conditions.

The average value of cone tip resistance, pore pressure, and friction sleeve versus cone penetration rates are shown in Fig. 2.40. Average cone tip resistance at 0.2 mm/s penetration rate increased about 70% as it dropped from 1 mm/s, and cone tip resistance at 0.05 mm/s increased up to 2.3 times the value at 20 mm/s penetration rate.

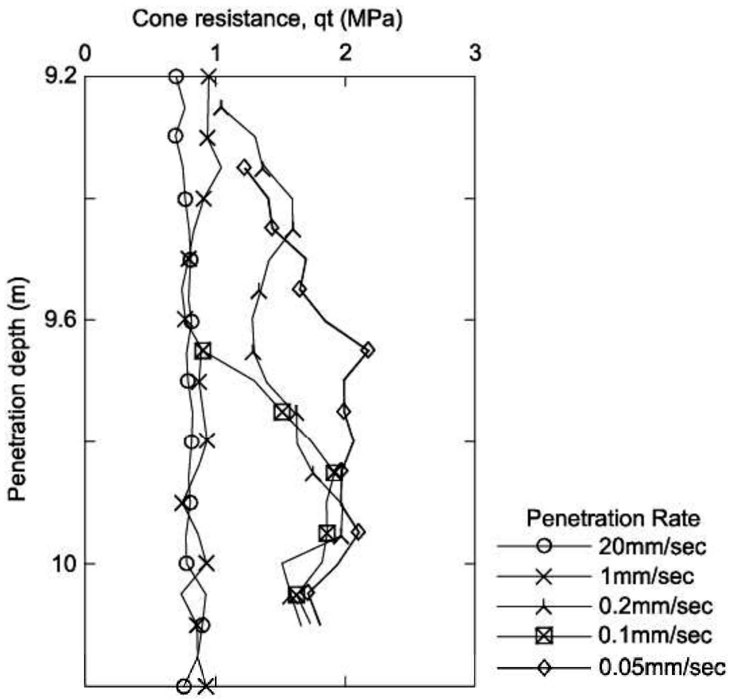


Fig. 2.38 Cone tip resistances measured at various penetration velocities for clayey silts (Kim et al. 2006)

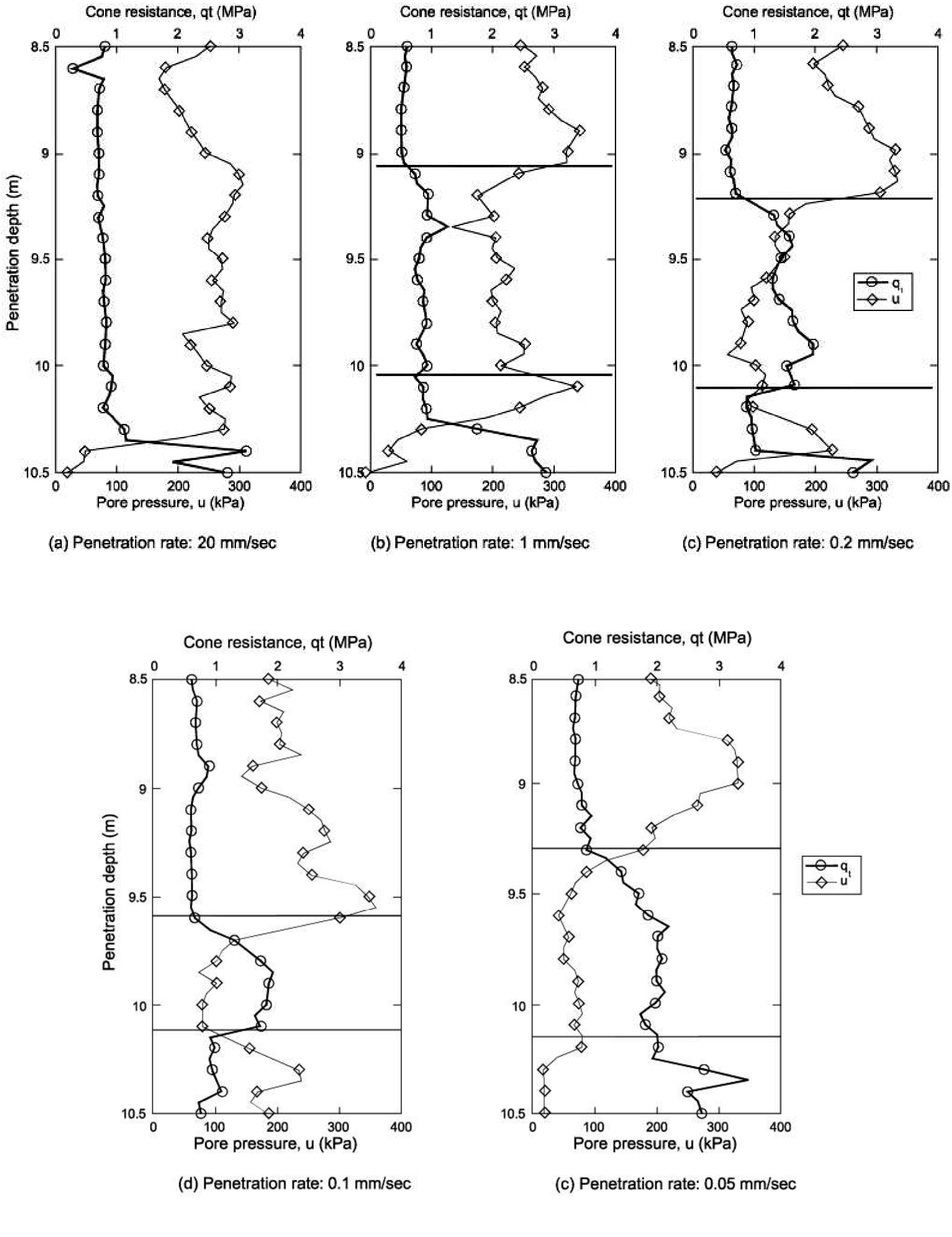


Fig. 2.39 Cone tip resistance and excess pore pressure results with varying penetration velocities (Kim et al. 2006)

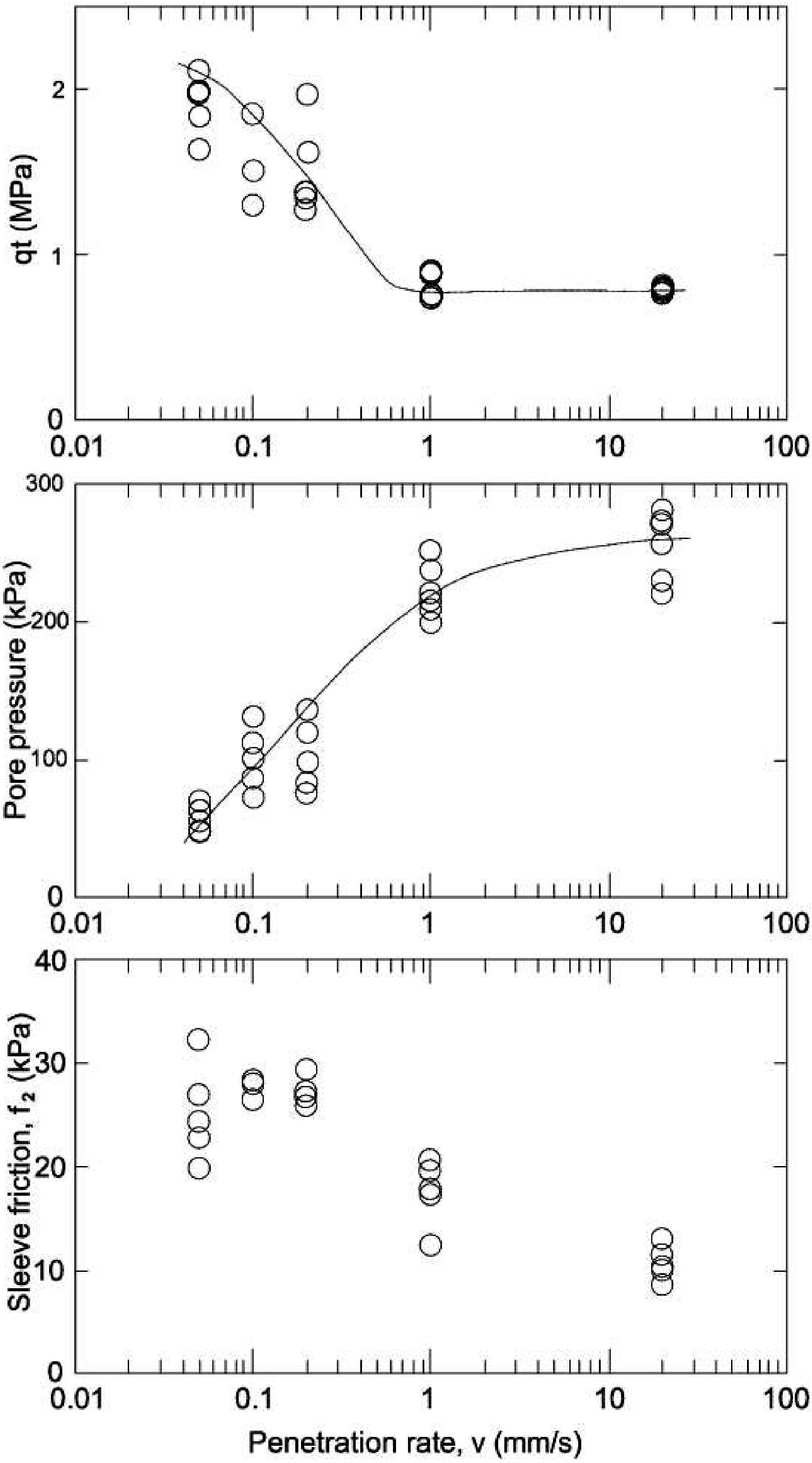


Fig. 2.40 Effect of penetration rate on cone tip resistance, pore pressure, and friction sleeve (Kim et al. 2006)

The inverse of varying soil properties (specifically the coefficient of consolidation, c_h) for a given penetration rate is to vary the penetration rate within a given soil (constant c_h). Results from variable penetration rate tests have been effective in demonstrating how penetrometer measurement changes (e. g., Randolph and Hope, 2004; Chung et al. 2006; Schneider et al. 2008; Kim et al. 2008; Jaeger et al. 2010) as a function of the normalized penetration velocity, which can be defined as $V = \upsilon d/c_h$ where υ = cone penetration rate (conventionally = 20 mm/s); d = penetrometer diameter (generally 35.7 or 43.7 mm); and c_h = coefficient of consolidation for conditions where the primary direction of pore water flow is horizontal. Fully undrained penetration typically occurs when V is larger than about 30-100 and fully drained penetration occurs when V is less than about 0.03-0.01 (Randolph 2004).

The majority of research to date, both numerical and analytical, has been performed on soils that are contractive [e. g., (near) normally consolidated kaolin and sand-clay mixtures], resulting in a decrease in the cone shoulder pore pressure (u_2) and an increase in the tip resistance (q_t) as the degree of partial consolidation increases with decreasing V. Notable exceptions to these trends are experimental results by Silva (2005) and Schneider et al. (2007), whose data with silica flour produced opposing trends because of significant dilatational effects. The former case is observed in the field for contractive soils (e. g., normally consolidated clays, loose silty sands), whereas the latter case is consistent with strongly dilative soils (e. g., highly overconsolidated (OC) clay and dense silty sand; e. g., Lunne et al. 1997). A single framework for both contractive and dilatantive soils is desirable, but at this stage sufficient research has only been performed on contractive soils for generalized analysis.

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CHAPTER 3
APPLICABILITY OF POLIDORI'S NEW PLASTICITY CHART TO KOREAN FINE-GRAINED SOILS

3.1 GENERAL ASPECTS

As shown in Fig. 3.1, the Incheon and Gunsan sites are located in the western area of the Korean peninsula, while the Busan and Gwangyang sites are located in the southern area of the Korean peninsula. As shown in Fig. 3.2, at the Incheon and Gunsan sites, soils show characteristics of relatively low plasticity when compared with that of the Busan and Gwangyang sites. The Incheon and Gunsan sites show characteristics of clayey silts with low plasticity. The water content (W_N), liquid limit (W_L) and plastic limit (I_P) of the Incheon and Gunsan soils vary considerably with depth, as shown in Fig. 3.3. Due to the seasonal variation of river flow, deposited sand components might be altered with each season. As shown in Table 3.1, the natural water content of Incheon soils varies from 20% to 57%. The liquid limit and plastic limit of this site are 24%-67% and 17%-32%, respectively. Based on the grain size distribution, the average percentages of distribution of sand, silt, clay in Incheon area are 20%, 64%, 16%, respectively. The natural water content of Gunsan soils varies from 21% to 69%. The liquid limit and plastic limit of this site are 24%-73%, 15%-33%, respectively. In addition, the average percentages of the distribution of sand, silt and clay in the Gunsan area are 26%, 50% and 24%, respectively. The clays in these regions were mostly composed of illite, kaolinite and the chlorite type of clay minerals, X-ray diffraction analysis revealed that the predominant rock forming mineral was quartz. On the other hand, the Busan and Gwangyang sites have characteristics of high compressibility, low shear strength, and thick deposited marine clayey soils. The natural water content of Busan soils, which was deposited to have thicknesses of 40-45 m, varies from 21% to 100%. The liquid limit and plastic limit of this site are 30%-89% and 16%-43%, respectively. Based on the grain size distribution, the average percentage distribution of sand, silt and clay in the Busan area are 5%, 56% and 39%, respectively. The clay layer thickness of the Gwangyang site varies from 25 m to 30 m. The range in natural water content is 28%-140%. The liquid limit and plastic limit of this site are 28%-125%, 17%-39%, respectively. Based on X-ray diffraction analysis, the clay minerals in these regions were composed of illite, kaolinite and chlorite, and the predominant clay mineral was illite.

Table 3.1 Physical properties of soils taken from different coastal areas, Korea

Site	W_N (%)	W_L (%)	W_P (%)	I_P (%)	Sand (%)	Silt (%)	Clay (%)
Incheon	20-57 (34)	24-67 (35)	17-32 (22)	4-39 (14)	10-86 (20)	7-92 (64)	2-50 (16)
Gunsan	21-69 (39)	24-73 (41)	15-33 (22)	3-44 (18)	8-90 (26)	5-88 (50)	4-49 (24)
Busan	21-100 (62)	30-89 (70)	16-43 (32)	14-54 (38)	3-42 (5)	40-72 (56)	11-55 (39)
Gwangyang	28-140 (86)	28-125 (84)	17-39 (30)	14-87 (54)	4-66 (6)	21-75 (52)	12-65 (42)

Round brackets show average values

As shown in Fig. 3.2, almost all soils taken from different coastal areas in Korea lie above the A-line in the clay zone according to the Casagrande's plasticity chart, which are classified as CL or CH. However, in case of Incheon and Gunsan soils, in particular by in-situ observations or hands-on soils tests, almost all soils in these areas might be classified as silts or intermediate soils or clays containing lots of sand, which are different from the soil classification according to Casagrande's plasticity chart. Therefore, an extensive literature review was performed to seek a solution to this problem of soil classification.

Fig. 3.1 Location of Incheon, Gunsan, Gwangyang, and Busan in the Korean peninsula

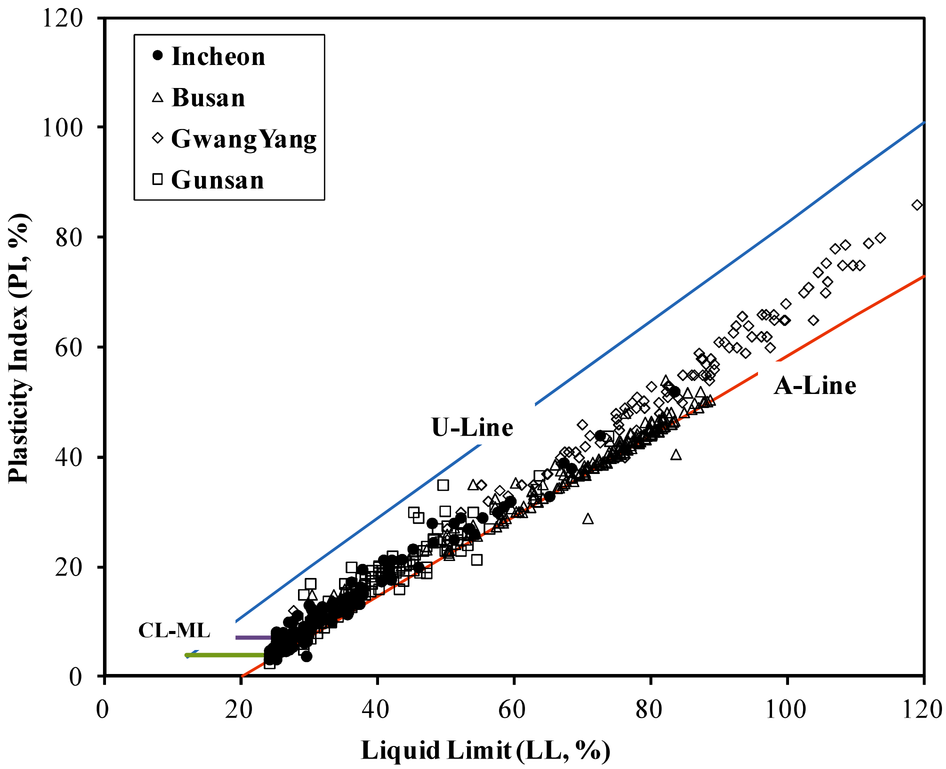
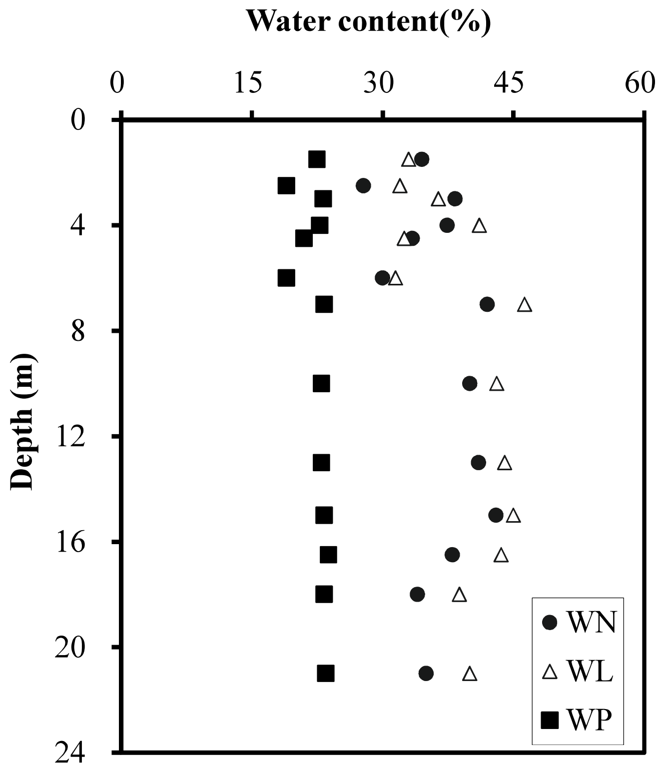
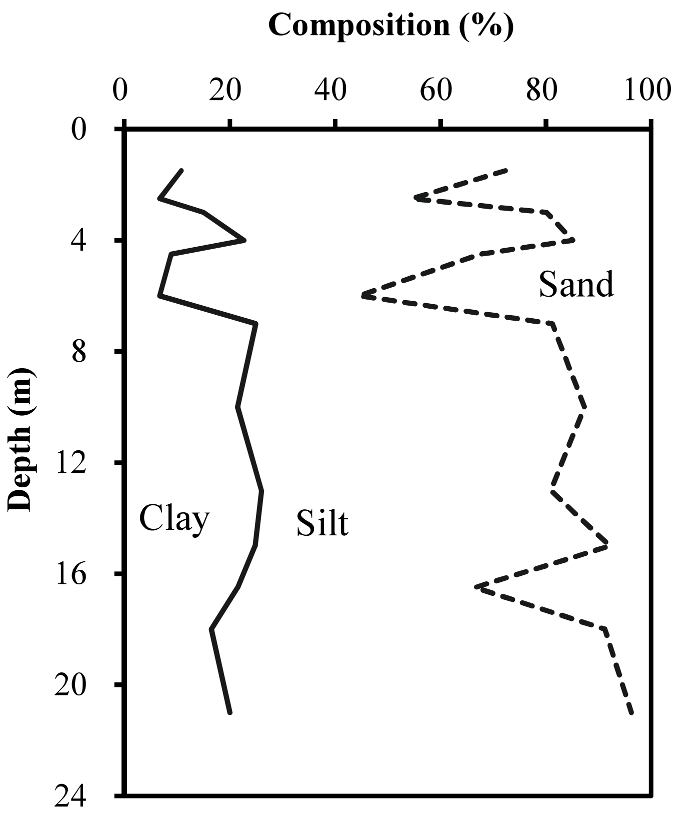
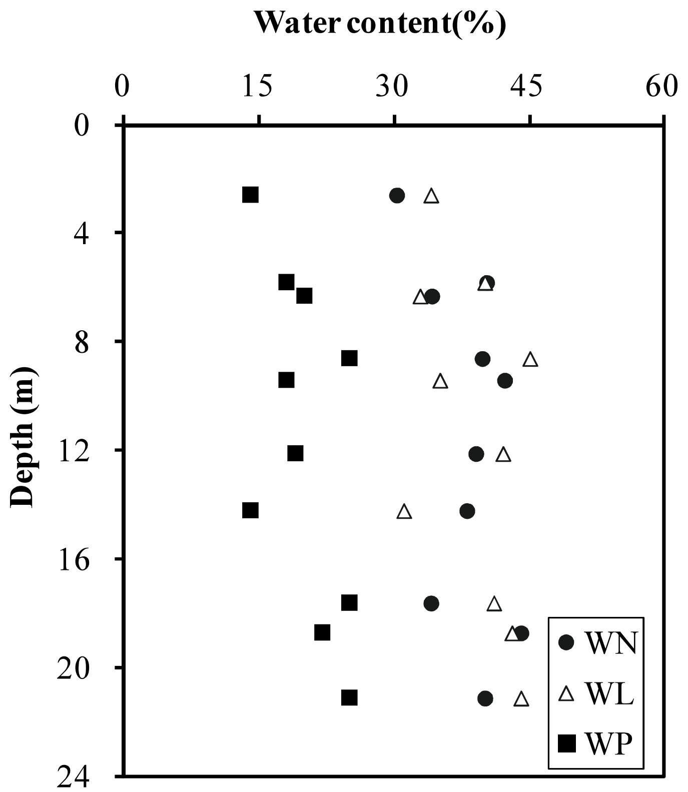


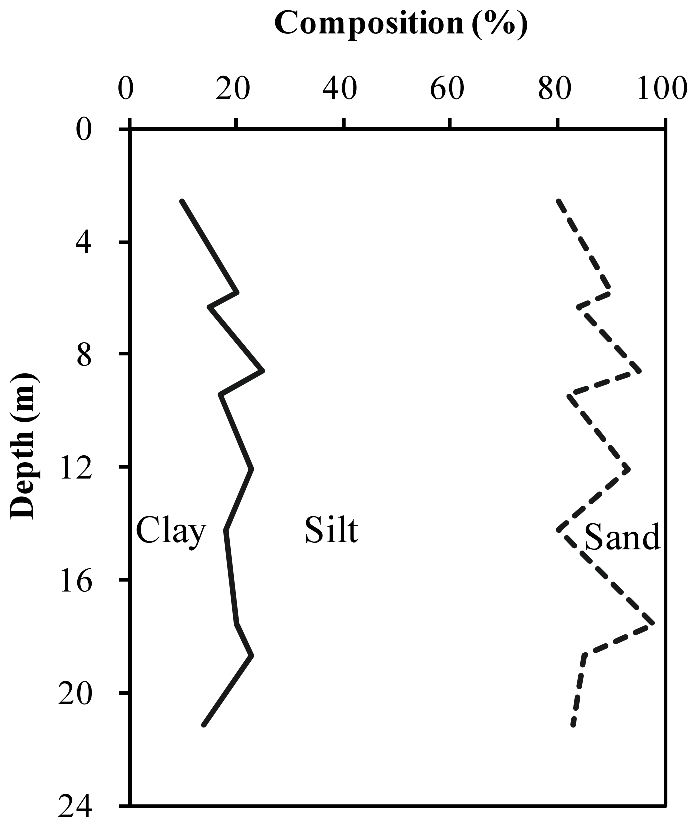
Fig. 3.2 Position of soil samples from four different Korean coastal areas on the plasticity chart





(a) Incheon site





(b) Gunsan site

Fig. 3.3 Soil composition and index properties from the Incheon and Gunsan sites

3.2 THE PLASTICITY CHARTS AND INDEX PROPERTIES

3.2.1 LIQUID LIMIT

Fig. 3.4 shows the liquid limit as a function of the clay fraction of the soils taken from four different Korean coastal areas. The liquid limit of each soil, except for those with clay fraction approximately equal to or smaller than 10%, more or less increases linearly with the clay fraction. The average linear line is seen to pass through the origin, which is in agreement with the result by Seed et al. (1964b), Nagaraj et al. (1995), Kumar and Muir Wood (1999) and Polidori (2007). As mentioned in Polidori's paper (2007), the liquid limit equation valid for inorganic soils containing platey clay minerals is $LL=k_1CF$, where the LL-slope k_1 depends on the factors that influence the plasticity of the soils, that is, the type of clay minerals and absorbed cations, pH, degree of crystallinity, etc.

Therefore, even soils with the same LL (or PI) value may have very different characteristics because of the amount and type of their clay minerals. The LL-slopes k_1 of the soils of the Busan and Gwangyang areas are steeper than those of the Incheon and Gunsan areas.

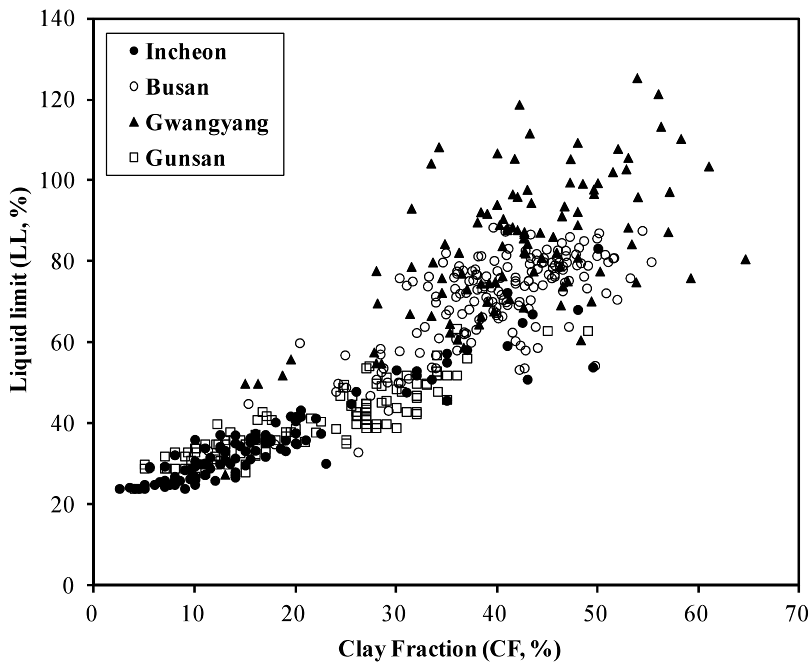


Fig. 3.4 The liquid limit, W_L, as a function of the clay fraction, CF (< 2 μm) of soils taken from the four different Korean coastal areas

3.2.2 PLASTIC LIMIT

As shown in Fig. 3.5, the plastic limit, W_P as a function of the clay fraction, CF shows less linearity and relatively little variation when compared with the data of the liquid limit. Except for those with clay fraction approximately equal to or smaller than 10%, that is, a low clay fraction, the best link between the plastic limit and the clay fraction is a linear relationship. The regression lines of the plotted mixtures intersect the PL axis randomly between 10% and 15%. The trend of the W_P to be a function of the clay fraction, CF, of the soils is similar to that of Polidori's approach (2007). However, the intercept value of the PL in this study is slightly bigger than that of Polidori's (2007).

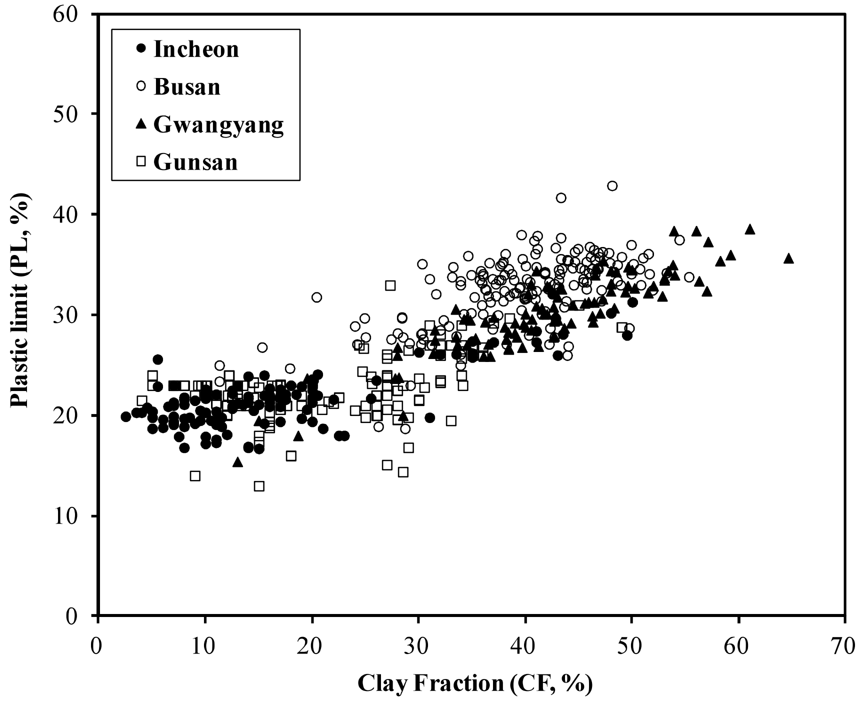


Fig. 3.5 The plastic limit, W_P as function of the clay fraction, CF (< 2 μm) of soils taken from four different Korean coastal areas

3.2.3 ACTIVITY

Since the adsorbed water that surrounds the clay particles causes the plasticity of the soil, it can be expected that the type of clay minerals and their proportional amounts in a soil will have an effect on the liquid limit and the plastic limit. To separate them, the ratio of the plasticity index to the clay fraction (percentage by weight of particles finer than 2 μm) was termed as “the activity” which is a very useful concept (Skempton, 1953). For many types of clay, a plot of the plasticity index versus the clay content yields a straight line passing through the origin. The slope of the line for individual clay gives the activity. This difference in the slope is due to the diverse plasticity characteristics of the various types of clay minerals. The activity is used as an index for identifying the swelling potential of clay soils.

Meanwhile, Seed et al. (1964a) studied the plastic properties of several artificially prepared mixtures of sand and clay and concluded that, although the relationship between the plasticity index and the clay fraction is linear, as suggested by Skempton, it may not always pass through the origin. Skempton’s (1953) definition of “active”, “normal” and “inactive” clays has been well demonstrated. Clays which have an activity of less than 0.75 are “inactive” clays, and those with an activity greater than 1.25 are “active” clays, while those between the two are classified as “normal” clays. The higher the activity of a soil, the more important the influence of the clay fraction on its properties and the more susceptible their values are to changes in factors such as the type of exchangeable cations and pore fluid composition. As shown in Fig. 3.6, Incheon soils have been categorized as inactive or normally active clays (A=0.85), Gunsan soils are also classified as inactive or normally active clays (A=0.86), Busan soils as normally active clays (A=0.98) and Gwangyang soils as normally active or active clays (A=1.31).

According to Mitchell (1993), the activity of montmorillonite varies from 1 to 7, illite from 0.5 to 1, and that of kaolinite is less than 0.5. Based on these classifications of clay minerals using the activity chart, the clays in the Incheon and Gunsan regions were mostly composed of illite and kaolinite types of clay minerals. Moreover, the predominant clay mineral in the Busan and Gwangyang regions is illite. These results exhibit conformity with results obtained by the x-ray diffraction analysis conducted in this study.

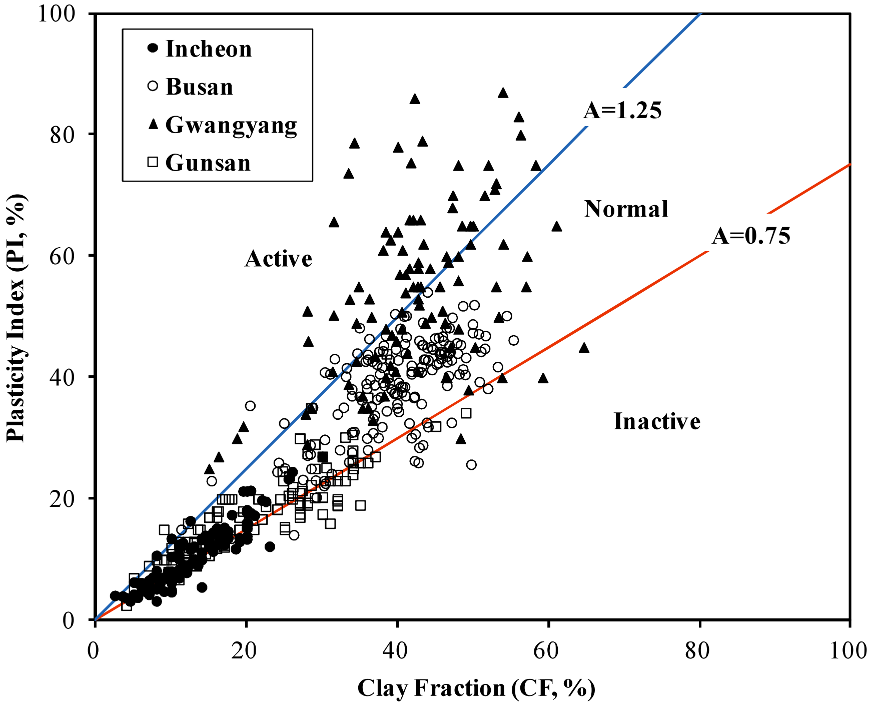


Fig. 3.6 The activity of soils taken from four different Korean coastal areas

3.2.4 COMPARISION OF SOIL CLASSIFICATIONS USING THE TWO PLASTICITY CHARTS

The plasticity index as a function of the liquid limits for the soils taken from the four different Korean coastal areas are shown on the Polidori's plasticity chart, including in which A-line (Casagrande) is shown in Figs. 3.7-11. As can be seen in Figs. 3.7-10, the soils taken from the four different areas are clearly separated into two groups: the ones that are composed mainly of silt and/or sand (CF $\leq$ 30%) and the others composed of clay and/or silt (CF > 30%). Almost all soils lie above Casagrande's A-line regardless of their clay contents.

As shown in Figs. 3.7-3.8, the soils in the Incheon and Gunsan areas are classified as silt based on CF=30% by the revised Polidori's plasticity chart, and this soil classification is in agreement with in-situ observation and hands-on soil tests. These soils, however, are classified as clay by Casagrande's plasticity chart, that is, almost all the soils of the two areas are located on the A-line. Meanwhile, as shown in Figs. 3.9-3.10, the soils in the Busan and Gwangyang areas are mostly classified as clay by both the Casagrande and the revised Polidori's plasticity chart. Therefore, as mentioned earlier, if a soil is classified as silt having the characteristic of low plasticity such as those of the Incheon and Gunsan areas, the problem of soil classifications is exclusively in relation to silt, because almost all cases are classified as clay by Casagrande's plasticity chart.

This general frame for soil classifications based on the clay fraction in this study is similar to that of Polidori, except for the borderline of the clay fraction percentage, that is, the threshold for the classification of silt or clay in this study and also in Polidori's study is based on CF=30% and CF=50%, respectively. Why does this difference in the borderline in soil classifications based on the clay fraction percentage occur? Firstly, let us start with soil classifications based on Unified Soil Classification System (USCS).

In the USCS, the soil is classified into two broad categories, coarse or fine grained soils, which are classified based on the percentage by dry weight (< 50% and ≥ 50%, respectively) of the specimen passing through the No. 200 (75 μm) sieve. Therefore, the borderline for soil classification is whether more than 50% of soil passes through the No. 200 sieve or not. Using the same analogy, according to Polidori's plasticity chart, the classification of soils into clays and silts is based on CF=50%. Additional comments will carry over about this, but Polidori might be tied down by the 50% reference point in his soil classifications as in the USCS.

Meanwhile, the grouping of fine-grained soils as silt and clay is almost always carried out according to Atterberg limits. As mentioned above, the distinction between the coarse and fine-grained soils is made by the No. 200 sieve, whereas the soil consistency tests to designate fine-grained soils as silt or clay have been solely performed on materials passing through the No. 40 sieve (with a sieve mesh size of 425 μm), not on materials passing through No 200 sieve. Regarding the classification of soils based on their particle size, the British Standard (BS 1377) distinguishes soil types according to the following criteria: clay = soil fraction with particles < 2 μm, and silt = soil fraction with particles 2-60 μm. Sand is designated as having a grain size from 60-2000 μm, and is further subdivided into fine, medium and coarse sand, corresponding to particle size intervals of 60-200 μm, 200-600 μm and 600-2000 μm, respectively, indicating that material coarser than 75 μm but finer than 425 μm is termed as medium-grade sand. Therefore, when conducting the Atterberg tests using a small percentage of material finer than 75 μm, the material prepared may include a significant amount of coarse material. Why is some amount of coarse-grained soil included in a test intended to make a distinction between fine-grained soils? An extensive literature review was performed to seek an answer to this question by Kayabali (2011). However, no studies were encountered addressing this issue.

Kayabali (2011) compared the Atterberg limits obtained by using soil passing the No. 40 sieve versus soil passing the No. 200 sieve with sixty soil samples.

As mentioned earlier, a Swedish soil scientist Atterberg engaged in ceramics and agriculture work distinguished coarse grained soils from fine grained soils based on materials finer than 0.2mm and called “Mo” in Swedish. This “Mo” means silt soils or fine sand. Therefore, it is presumed that initially, Atterberg (1914) developed an agricultural classification system based on plasticity as determined from his liquid and plastic limit tests using materials finer than 0.2mm. The Atterberg limits were later adopted by the US Bureau of Public Roads in the late 20’s and the test procedures were simplified. Casagrande (1932, 1948, 1958) researched and standardized the Bureau of Public Roads version of the Atterberg limits tests and developed the plasticity chart after considerable testing for distinguishing between silts and clays of high and low plasticity using materials finer than 0.425mm (No. 40 sieve).

Around the same time, the revised Public roads and American Association of State Highway and Transportation Officials (AASHTO) classification system (1945) was developed to assess the load bearing characteristics of subbase and subgrade soils for a highway. It is necessary to carry out to sieve analysis using No. 40 sieve for group classification (A-1 and A-3). Therefore, it can be inferred that Atterberg limits tests using materials finer than No. 40 sieve are fixed to satisfy both sides (USCS and AASHTO system) in the period.

According to the previous studies (Seed et al., 1964b; Polidori, 2003), the liquid and plastic limits are both linked by a linear relationship to their clay size content for clay percentages which are not too low, that is, Atterberg limits may be only related to the percentage of the clay fraction and the type of clay minerals. Conceptually, the liquid limit and plastic limit are directly related to the grain size and the specific surface. The soil passing the No. 200 sieve has a greater specific surface than the soil passing the No. 40 sieve. Therefore, the soil passing the No. 200 sieve should have a greater liquid limit and plastic limit than the soil passing the No. 40 sieve for the same soil sample. On the other hand, as shown in Figs. 3.4 and 3.5, because the data of the plastic limit show relatively little variation (especially according to the type of clay minerals) in comparison with the data of the liquid limit for the same clay fraction, the plasticity index for soil passing the No. 200 sieve is expected to be slightly greater than the soil passing through No. 40 sieve; that is, the increase rate of the liquid limit is higher than that of the plastic limit for the same clay fraction percentage. It then follows that the plot of the soil passing the No. 200 sieve on the Casagrande plasticity chart should be located slightly upward on the right side of the plot of the soil passing the No. 40 sieve. This shift becomes rather significant when the class of a fine-grained soil is near the borderline, that is, when soil that classified as clay can be determined to be silt by Casagrande's plasticity chart. Therefore, the increase in the liquid and plastic limits for the soil passing the No. 200 sieve with respect to those of the soil passing the No. 40 sieve causes remarkable changes in both soil classifications and plasticity levels. As mentioned above, even extreme cases such as shifts from CL to MH are also possible. In other words, if the plasticity index values are equal, soils that have the highest liquid limit values (and therefore lie below the A-line) exhibit silt behavior. In the last analysis, the clay and silt zones on Casgrande's plasticity chart are not accurate. From this viewpoint, if the Atterberg limits and activity, A=PI/CF (% < 2 μm) defined by Skempton (1953) are only related to the clay fraction (Seed et al., 1964b; Polidori, 2003, 2007, 2009), what is the role of silt on the Atterberg limits and activity? Polidori's plasticity chart might be suggested based on the relationship between the Atterberg limits and the clay fraction percentage, mostly using artificial soil mixtures, ignoring the content of the silt fraction, that is, the role of silt on the plasticity chart. The Atterberg limits for artificial soil mixtures with a CF < 25%-30% are not linearly proportional to the clay size content of soil (Seed et al, 1964b, Polidori, 2007), while natural soils with a CF < 10%-15% showed a similar trend in this study. Therefore, the difference of the CF in the lower limit of the linear relation between the Atterberg limits and the clay fraction in this study and Polidori's study varies from approximately 15% to 20%. According to Seed et al. 1964b, and Polidori, 2003, 2007, the linear relation (LL, CF) holds good until the volume of the clay-water system is greater than the volume of the void of the non-clay fraction in the mixtures, or in other words, until the non-clay particles are still not in contact with one another. From this viewpoint, this phenomenon may be only maintained by the clay fraction, that is, the clay phase, without considering the silt fraction. However, as mentioned above, because the liquid limit and plastic limit of fine-grained soils could change according to both the percentages of the silt and the clay fractions, the silt fraction could be almost as important as the clay fraction in the relation between the Atterberg limits and the relevant fraction. Otherwise, the Atterberg limits and the plasticity chart must be defined by the clay fraction, as is the activity. From the experimental data collected, it is considered that a fine-grained soil that has about a 30% clay fraction could behave like clays. This is because enough fine grained soils (silt and clay fraction) exist to fill the voids between the coarse-grained soils and hold them apart. In other words, all the round particles containing clay fraction of more than 30% are scattered in the soil-water system, that is, the behavior of the soil is dominated by the clay phase. In addition, almost all soils taken from four different coastal areas have been classified as low plasticity or as medium inorganic clay (CL) by Casagrande's plasticity chart, as shown in Table 3.1 While using the data obtained from the grain size analysis, the excessive silt concentration in the soil (50%-64%) supports its specification as an ML type soil assigned according to Polidori's soil classification. However, soils having clay fraction of more than 50% might not exist in Korea, except for a few of the soils taken from the Busan and Gwangyang areas. Therefore, according to Polidori's soil classification, unrealistically, almost all soils in Korea might be classified as silt, including high plastic clayey soils taken from the Busan and Gwangyang areas. Because no distinction exists above the A-line where the natural soils taken from four different coastal areas lie on the empirical plasticity chart proposed by Casagrande, and these soils were classified as silt by Polidori's plasticity chart, from Polidori's revised plasticity chart based on the borderline of CF = 30% proposed by this study, it appears that it is more appropriate to classify these soils as silt or clay. To understand the reasons for the difference of the borderline of the CF in this study and Polidori's soil classifications, further experimental data and studies are necessary.

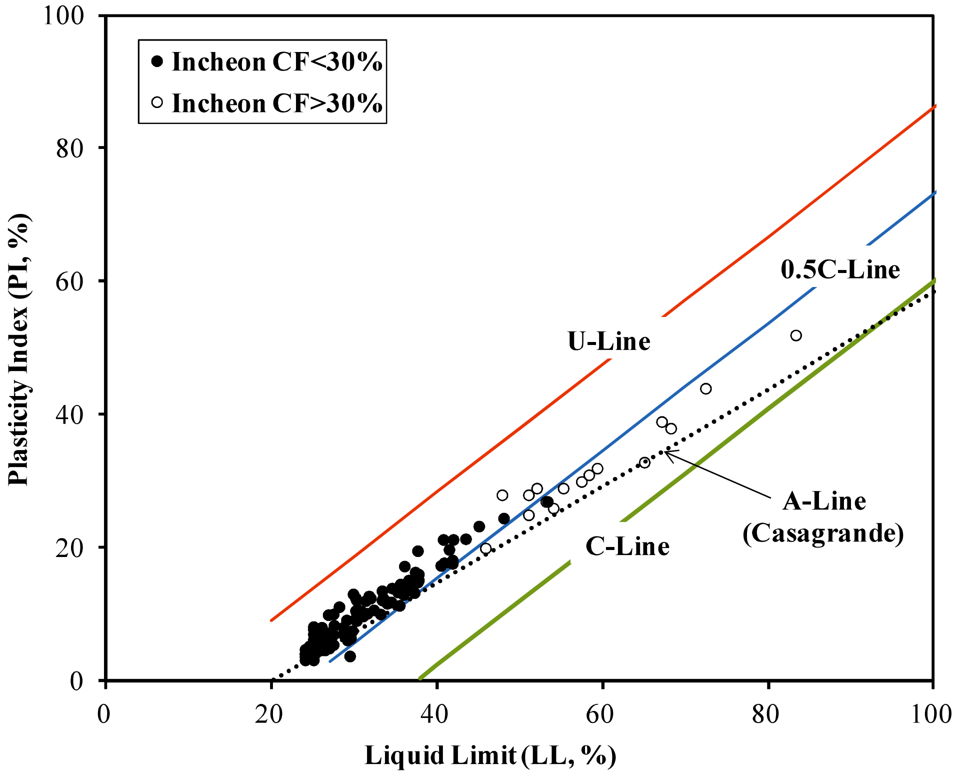


Fig. 3.7 Position of Incheon soils including Casagrande's A-line on Polidori's plasticity chart

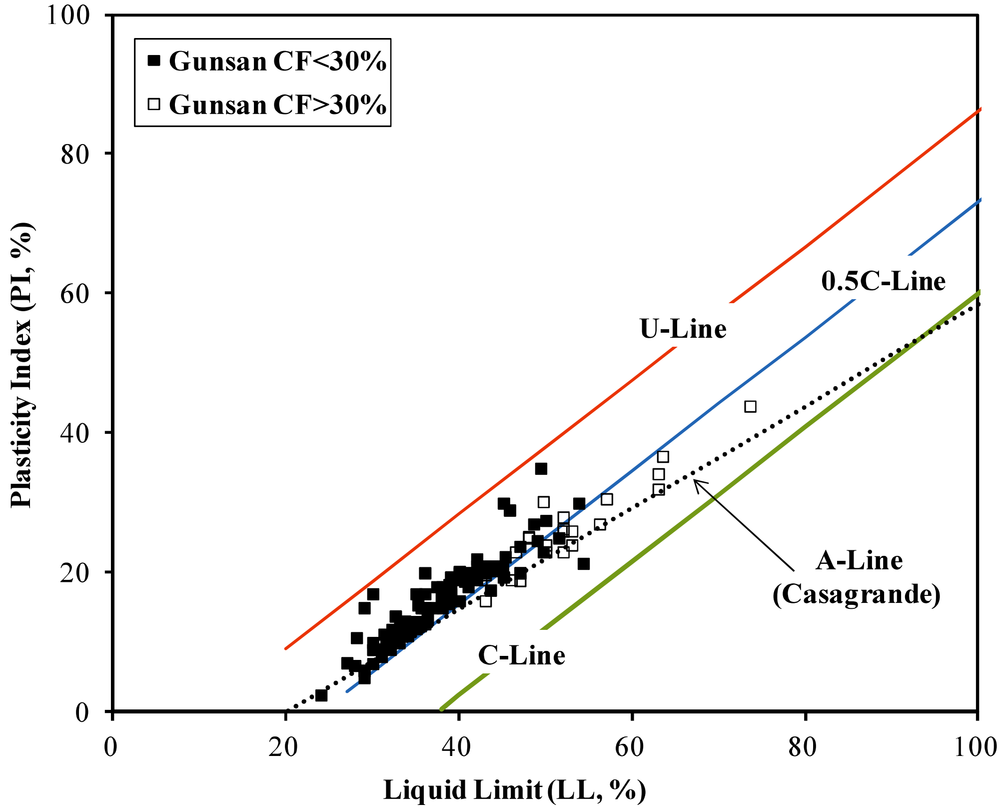


Fig. 3.8 Position of Gunsan soils including Casagrande's A-line on Polidori's plasticity chart

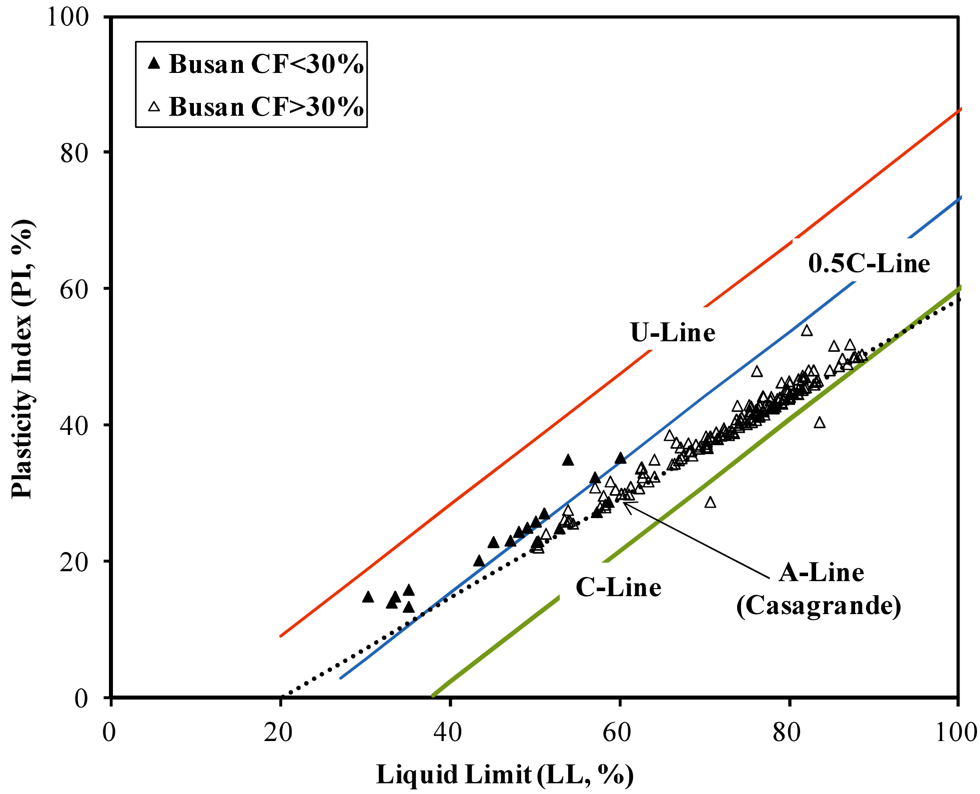


Fig. 3.9 Position of Busan soils including Casagrande's A-line on Polidori's plasticity chart

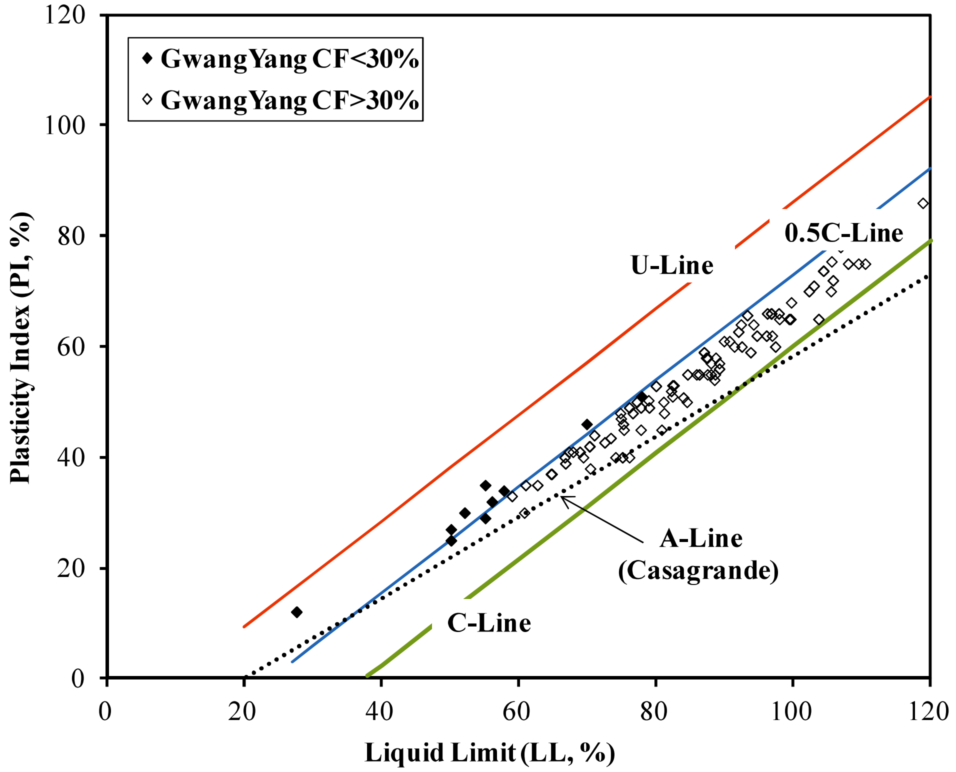


Fig. 3.10 Position of Gwangyang soils including Casagrande's A-line on Polidori's plasticity chart

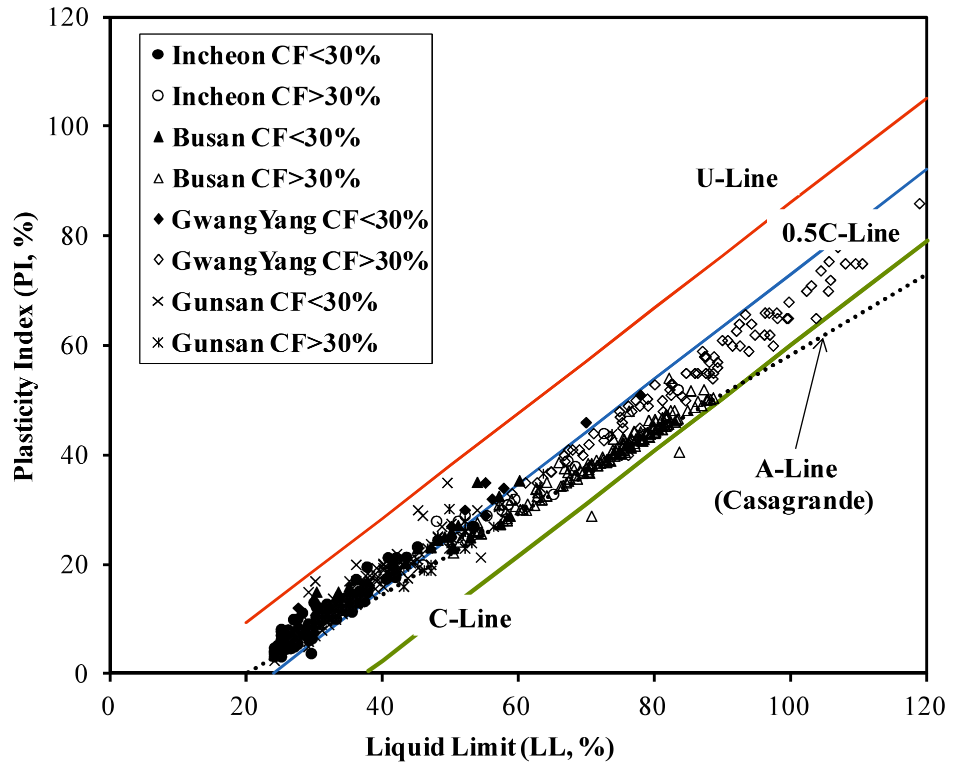


Fig. 3.11 Position of soils taken from four different Korean coastal areas including Casagrande's A-line on Polidori's plasticity chart

3.3 CONCLUSIONS

In this study, through a comparison of the plasticity charts of Casagrande and Polidori, the applicability and problems of soil classifications based on both plasticity charts for natural marine soils taken from four different Korean coastal areas were investigated.

Almost all the soils taken from the four different Korean coastal areas have been classified as low plasticity or medium inorganic clay (CL) by Casagrande's plasticity chart, while using the data obtained from grain size analysis, the excessive silt concentration in the soil (50% - 64%) supports its specification as an ML type soil assigned according to Polidori's soil classification.

Almost all the soils were classified as the silt according to Polidori's plasticity chart, which is based on the relationship between the Atterberg limits and the clay fraction percentage, mostly from using artificial soil mixtures, while ignoring the content of the silt fraction, that is, the role of silt in the plasticity chart. Because the liquid limit and plastic limit of fine-grained soils could change according to both the percentages of the silt and clay fractions, the silt fraction could be almost as important as the clay fraction in the relation between the Atterberg limits and the relevant fraction.

Based on the experimental data in this study, and because no distinction exists above the A-line where the natural soils taken from the four different coastal areas lie on the empirical plasticity chart proposed by Casagrande and because these soils were classified as silt by Polidori's plasticity chart, Polidori's revised plasticity chart based on the borderline of CF = 30% proposed by this study appears to be more appropriate for classifying between silt or clay. To understand the reasons for the difference in the borderline of the CF in soil classifications in the present study and Polidori's soil classifications, further experimental data and studies are necessary.

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CHAPTER 4
UNDRAINED SHEAR STRENGTH CHARACTERISTICS OF CLAYEY SILTS WITH LOW PLASTICITY

4.1 TESTING METHOD

4.1.1 LABORATORY TEST

1) INDEX TESTS

Liquid limit and plastic limit tests and particle size distribution tests were performed according to ASTM 4318, ASTM D422 and ASTM D2487. Casagrande method was used to determine the liquid limit of soils.

2) CONVENTIONAL OEDOMETER TEST

The standard oedometer tests were carried out on a cylindrical specimen of saturated soil with usual dimensions of 60 mm in diameter and 20 mm in height, respectively. The two porous stones at the top and bottom of the sample allowed a two-way drainage of the sample. The load was doubled at each increment and a loading time of 24 hours was applied at each increment. Yield consolidation pressure was obtained using the Casagrande method.

3) UNCONFINED COMPRESSION (UC) TEST

The testing method followed ASTM D2166. A specimen with a 100mm height and a 50mm diameter was sheared at an axial strain rate of 1%/min.

4) ISOTROPIC UNDRAINED RECOMPRESSION TRIAXIAL (CIU) TEST

The recompression test method proposed by Tsuchida and Mizukami (1991) was used in this test, and a soil specimen is isotropically consolidated in a triaxial cell for two hours at its mean in-situ effective stress $(p'_{vo}+2p'_{ho})/3$ where p'_{vo} and p'_{ho} are the in-situ vertical and horizontal stress, respectively). In this study, an earth pressure at rest (k_o) of 0.5 was assumed. Therefore, an isotropic consolidation pressure of $2/3p'_{vo}$ was applied to each specimen for two hours before shearing. Each specimen was sheared under undrained conditions at an axial strain rate of 0.1%/min. The nominal dimension of the specimen used in this test was the same as that of the UC test.

4.1.2 IN-SITU TEST

1) PIEZOCONE PENETRATION TEST WITH PORE PRESSURE MEASUREMENT (CPTU)

The cone used in the investigation of Incheon sites followed ASTM 5778. The cone has a projected cross section area of $10cm^2$, and conical angle of 60°. The filter for pore pressure measurement is located behind the shoulder.

2) FIELD VANE TEST (FVT)

The FVT apparatus used for this investigation was of the penetration type and sheared without utilizing a borehole. The dimensions of the vane were 40mm in diameter (D), 80mm in height (H) and 1mm thick. The FVT method followed the ASTM D2573. The vane was rotated at a constant rate of 6 degrees/min.

4.2 PHYSICAL PROPERTIES OF CLAYEY SILTS WITH LOW PLASTICITY

The Incheon and Gunsan sites are located in the western region of the Korean peninsula, and the Busan and Gwangyang sites are located in the southern region. The Incheon and Gunsan sites have relatively low plasticity compared to the plasticity of the Busan and Gwangyang sites, as shown in Fig. 4.1.

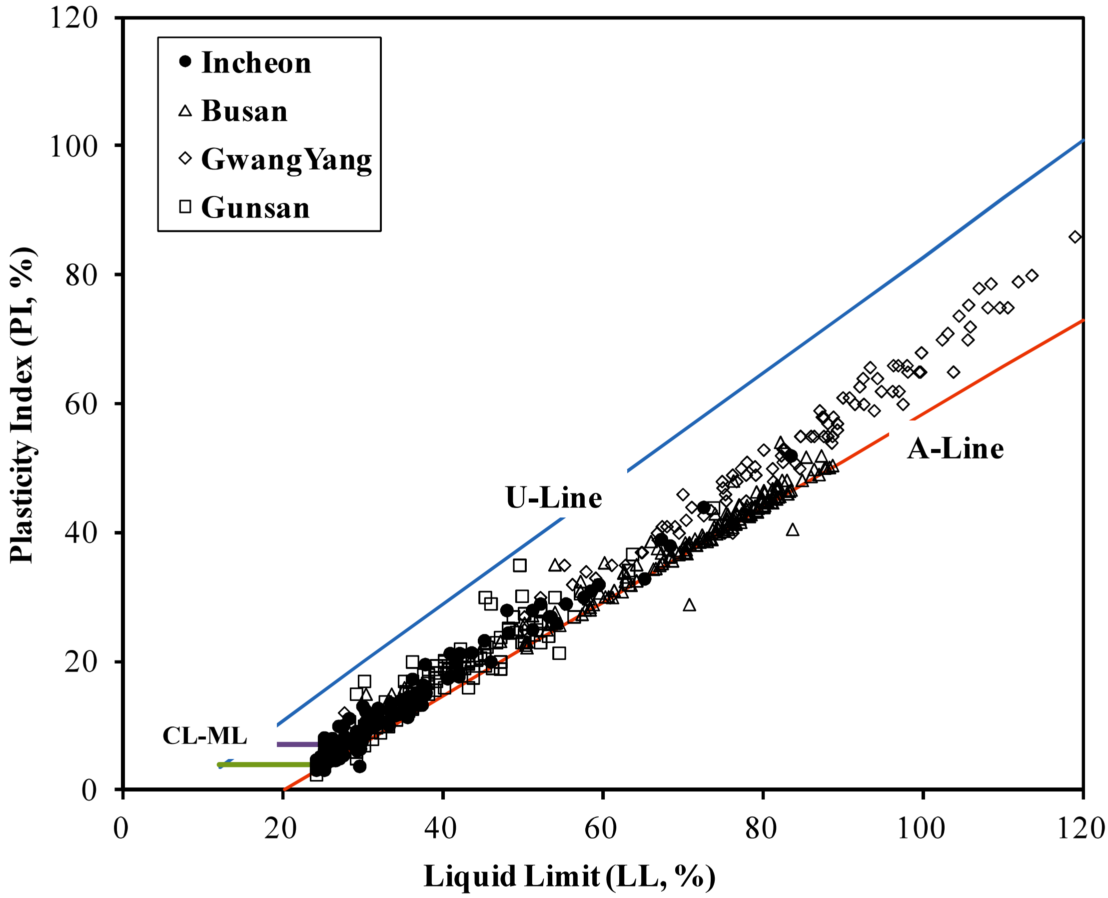


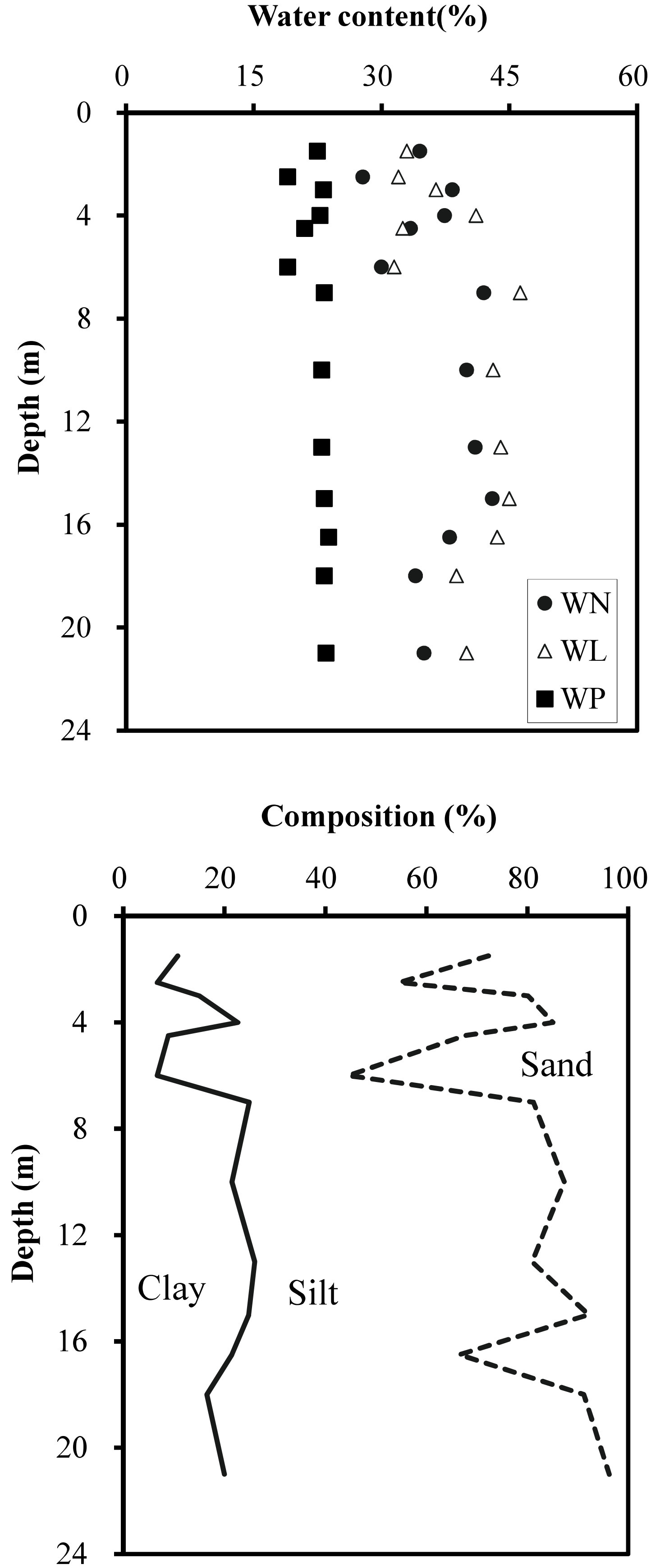
Fig. 4.1 Casagrande's plasticity chart

The Incheon site is located at a sea water depth of 6m and is about 20m thick. As shown in Fig. 4.2, the water content (W_N), liquid limit (W_L), and plastic limit (W_P) of clayey silt with low plasticity at the Incheon site varied considerably with depth. Due to seasonal variations of the river flow, the deposited sand components might change from season to season. The grain size distribution is also shown in Fig. 4.2 and sand, silt, and clay compositions were divided using two demarcation lines. Clay was here defined as soil with particles smaller than 0.002mm. Based on the grain size distribution only, the design code for port construction in Japan states that almost the entire profile could be classified as clay because the percentage of sand-sized particles in this layer was less than 50%. However, as shown in Fig. 4.2(a), soils located at depths of around 2 m and 6 m might be classified as intermediate soils. Fig. 4.2(b) indicates that since the percentage of sand-sized particles at depths shallower than 5m was approximately greater than 50%, soils at these depths could be classified as intermediate soils. Soils at depths deeper than 12 m could also be considered as intermediate soils. The percentage of sand at site B was somewhat greater than that at site A. Finally, the grain size distribution of site C is somewhat similar to that of Site A, except for some depths which contained a lot of sands. The layers with high sand contents had lower water contents. On the other hand, layers that contained abundant fine-grained soils had higher water contents. Generally, clayey silt at Incheon contained abundant sand- or silt-sized particles, and the I_p value was nearly proportional to the clay content.

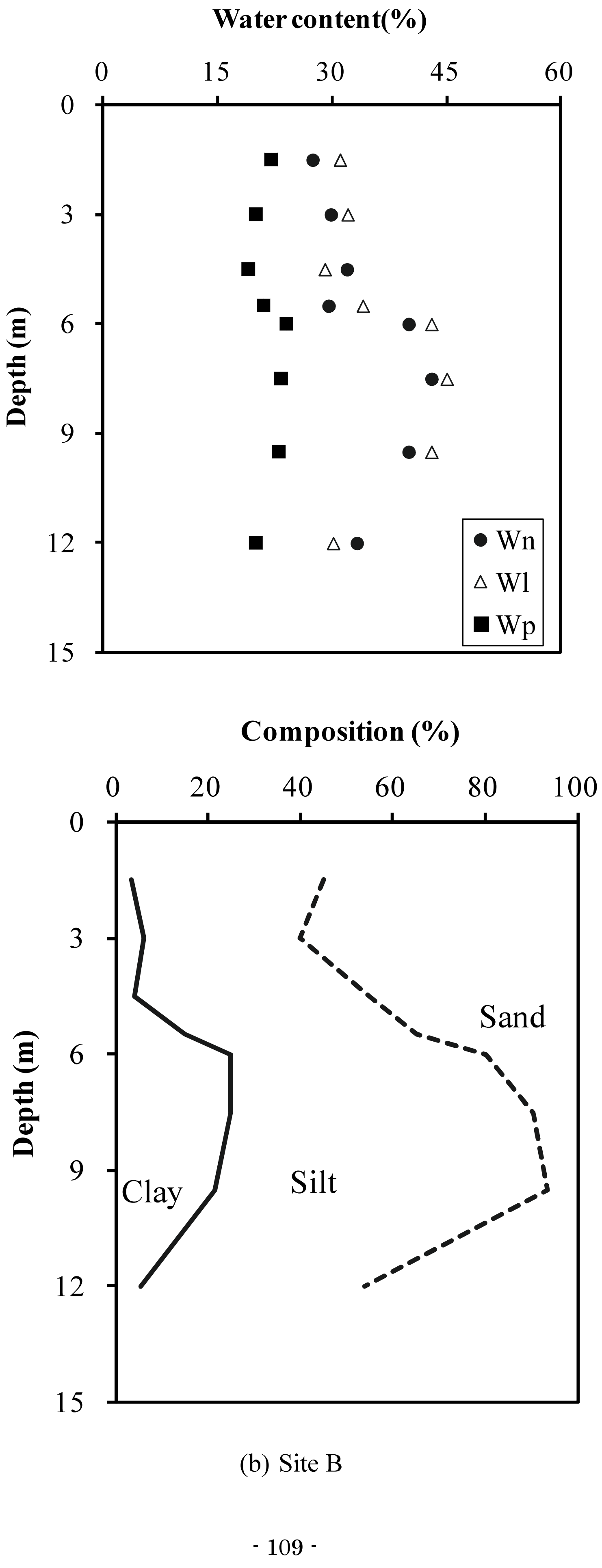
Gunsan site is located at sea water depth of 5m and is about 20m thick. As shown in Fig. 4.3(a), water content (W_N), liquid limit (W_L), plastic limit (W_P) of clayey soil with low plasticity at Gunsan site varied considerably with depth. Due to the seasonal variation of river flow, deposited sand components may be changed in each season such as Incheon soil. The grain size distribution is shown in Fig. 4.3(b). Based on the grain size distribution only, the design code for port construction in Japan states that almost the entire profile could be classified as clay because the percentage of sand-sized particles in this layer was less than 50%.

Gunsan clayey silt contains a lot of sand or silt sized particles and its PI value is nearly proportional to its clay content such as Incheon soil.

In order to judge whether or not the ground should be classified as an intermediate soil, the ground should be first classified as a coarse-grained soil based on the above approach. However, since the averages of silt and clay contents at the Incheon and Gunsan sites are 56%, 15% and 87%, 19%, respectively, for the entire profile, these grounds cannot be classified as intermediate soils, except at some depths, when only the grain size distribution criteria mentioned above are applied. However, according to Geotech Note No. 2 (Japanese Geotechnical Society, 1992), the soils may be classified as intermediate soils, since the average plasticity index are 18%, 21%, respectively, and the coefficient of consolidation at Incheon and Gunsan sites are 770cm^2/day, 730cm^2/day, respectively. This intermediate soil classification method, which considers soil plasticity and the consolidation parameter in addition to the grain size distribution, can be a more suitable classification mechanism than that of the port design code.


Water content(%)
0
15
30
45
60
Depth (m)
0
4
8
12
16
20
24
WN
WL
WP
Composition (%)
0
20
40
60
80
100
Depth (m)
0
4
8
12
16
20
24
Clay
Silt
Sand

(a) Site A



Water content(%)
0
15
30
45
60
Depth (m)
0
3
6
9
12
15
Wn
Wl
Wp
Composition (%)
0
20
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60
80
100
Depth (m)
0
3
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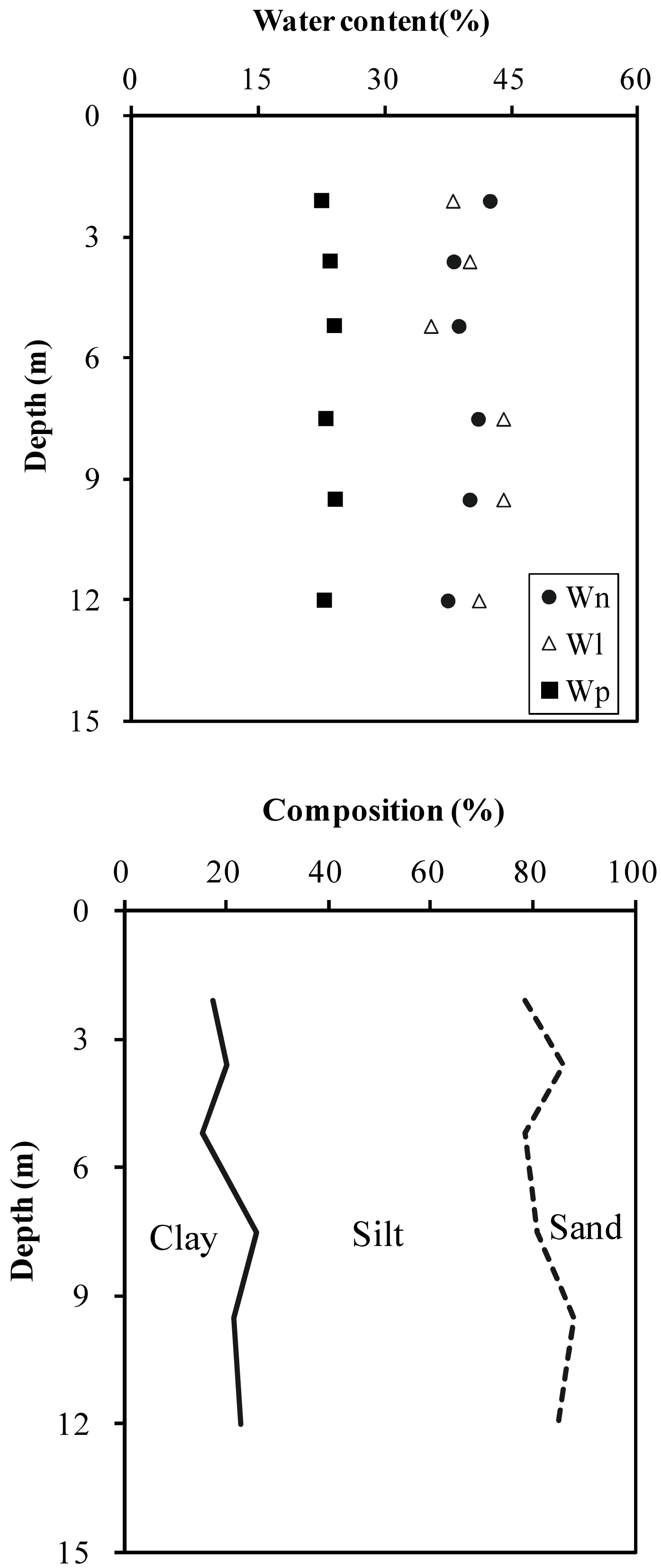
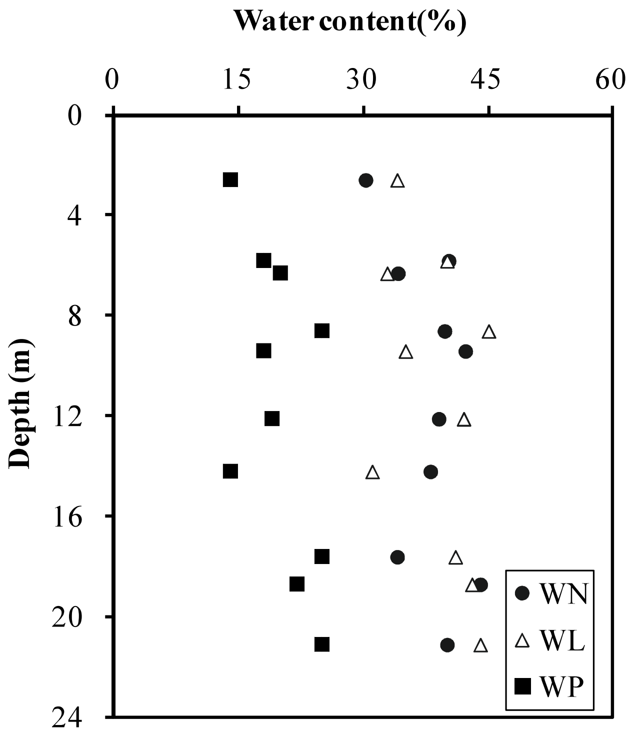
12
15
Sand
Clay
Silt

(b) Site B

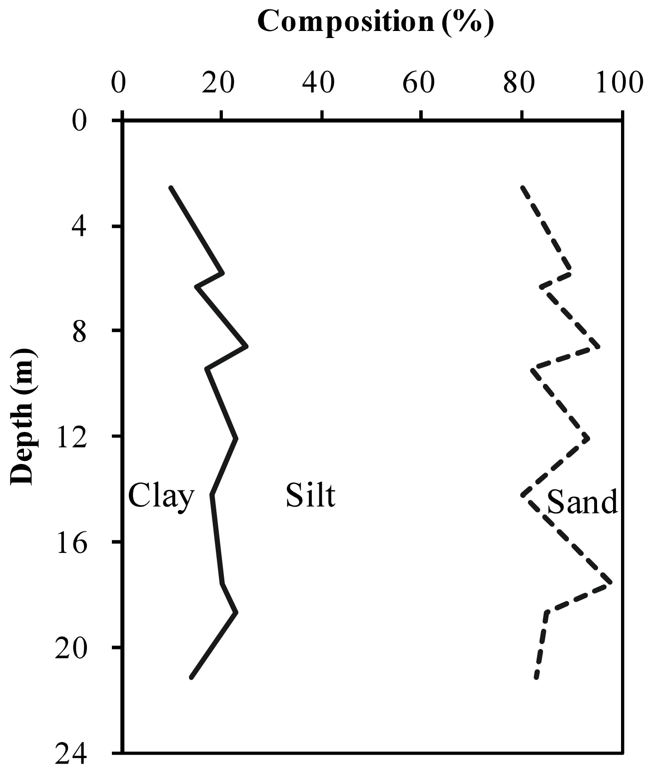


(c) Site C

Fig. 4.2 Soil composition and index properties of Incheon clayey silt



(a) Water contents, Liquid limits, Plastic limits with depth at Gunsan site



(b) Classification of soils based on particle size with depth at Gunsan site

Fig. 4.3 Soil composition and index properties of Gunsan clayey silt

4.3 COMPARISON OF RESULTS BETWEEN UC AND CIU (RECOMPRESSION TEST)

4.3.1 APPLICABILITY OF UC TEST FOR CLAYEY SILT WITH LOW PLASTICITY

The undrained shear strengths ($q_u/2$) of the Incheon and Gunsan soils obtained from the UC test were presented in Fig. 4.4-4.5, respectively. The $q_u/2$ values for Incheon and Gunsan soils increased with a strength depth ratio of 1kPa/m, 1.4kPa/m, respectively. This value was much smaller than the values from common marine clay layer. And, these values corresponded to a strength incremental ratio (s_u/p'_c) of 0.18, 0.19, respectively. As can be seen in Fig. 4.4, the $q_u/2$ value at depth of 12m was only 15kPa and its strength incremental ratio (s_u/p'_c) was as small as 0.1 for Incheon site. And, as shown in Fig. 4.5, the $q_u/2$ value at depth of 12m was only 19kPa and its strength incremental ratio (s_u/p'_c) was also as small as 0.14 for Gunsan site. According to the previous research (Nakase et al, 1972), layers have been known to contain a lot of sand, resulting in a small $q_u/2$. However, this study found no direct correlation between the percentage of the sand content and the $q_u/2$ value. For example, as shown in Fig. 4.2(a), at depths of 7m and 13m, the sand content was almost the same, at about 20%. However, as shown in Fig. 4.4, the $q_u/2$ values at these depths varied from 15kPa to 35kPa. The same trend could be found in Gunsan clayey silt. Therefore, it is not enough to explain about low value of UC test for Incheon and Gunsan clayey silts. As shown in Fig. 4.6-4.7, it has been believed that the order of strain at failure is a good indicator of sample quality. The better the quality of the sample, the smaller the strain range at which a failure occurs. The strain at failure of the UC specimen for two sites showed approximately less than 4% at most depths. The strain at failure in the UC test for typical Korean marine clays in Korea is known to be less than 4%, provided that the sample is retrieved using a proper sampling method. Therefore, it is difficult to judge the quality of the sample of Incheon and Gunsan clayey silts using only the UC test result. As mentioned earlier, the UC test has been generally conducted due to the existence of an automatic balance between the factors that overestimate and underestimate the undrained strength.

When a soil specimen is retrieved from the ground and is exposed to the atmosphere, part of the in-situ effective stress in the specimen remains in the form of negative pore pressure. This negative pore pressure called residual effective stress (p'_r). Even though the specimen is tested under unconfined conditions, the residual effective stress acts as a confining pressure. However, if the value of p'_r in the specimen is reduced due to disturbance of specimen during sampling, handling, extruding or trimming, and then the $q_u/2$ value will be reduced due to swelling. According to Tanaka et al (1996), the order of p_r' for high quality samples is around 1/5 to 1/6 of the in-situ vertical effective stress (p'_{vo}) for normally consolidated and slightly over-consolidated clays.

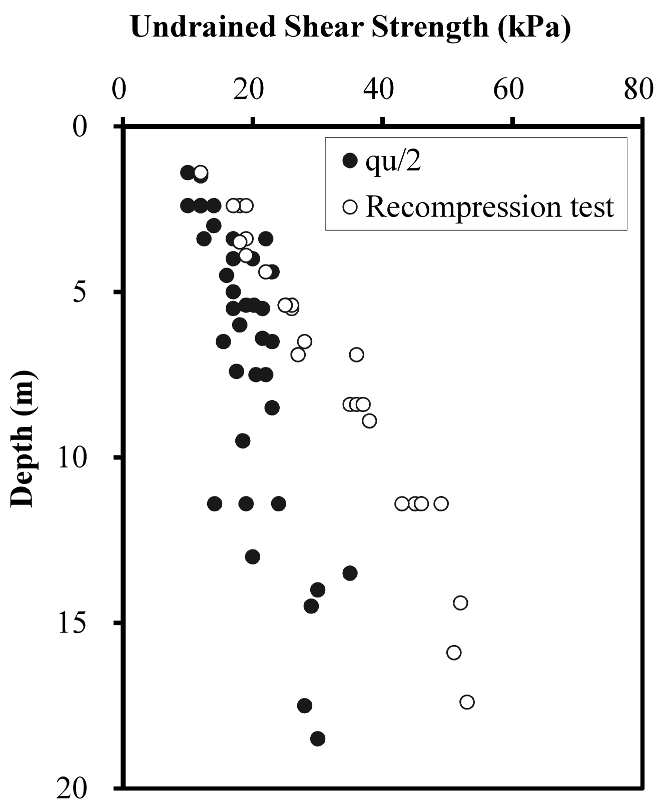


Fig. 4.4 Undrained shear strength from q_u test, recompression test for Incheon clayey silt

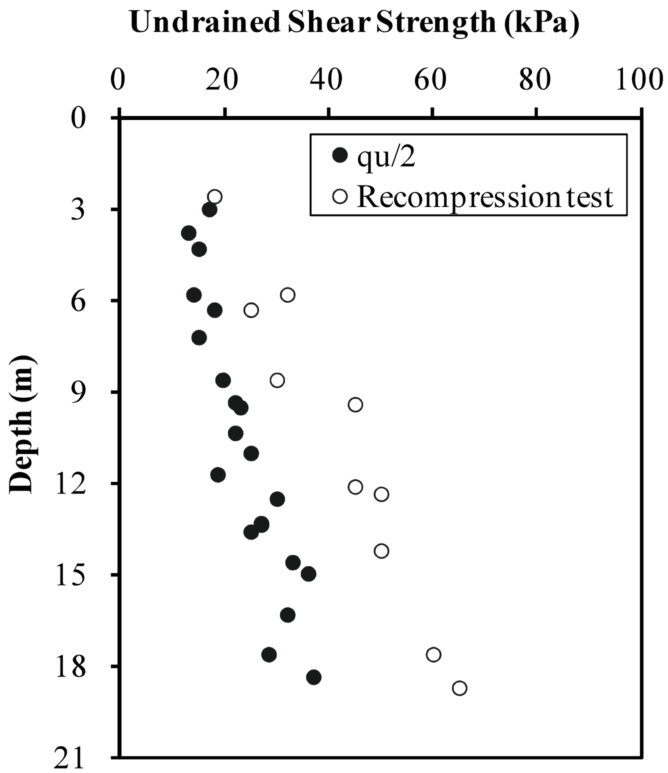


Fig. 4.5 Undrained shear strength from q_u test, recompression test for Gunsan clayey silt

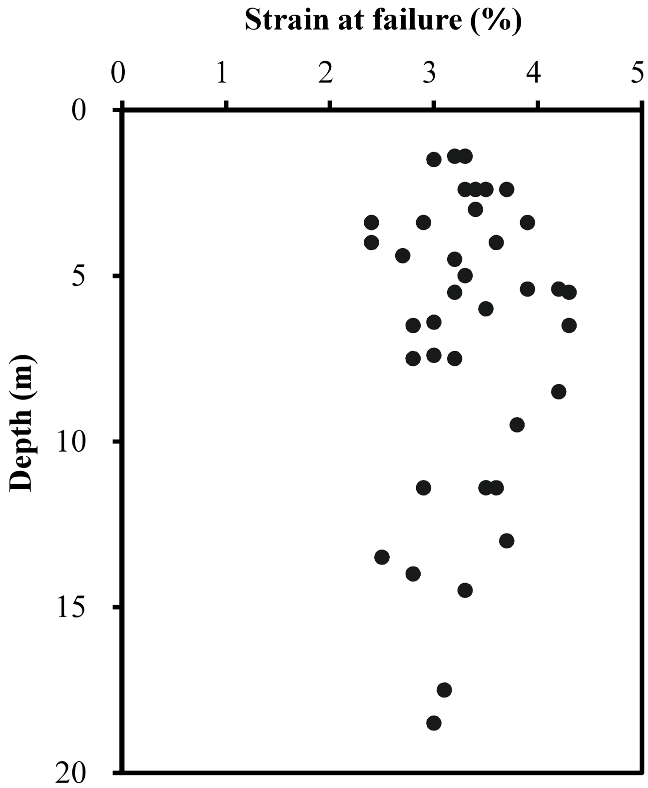


Fig. 4.6 Strain at failure of unconfined compression test for Incheon clayey silt

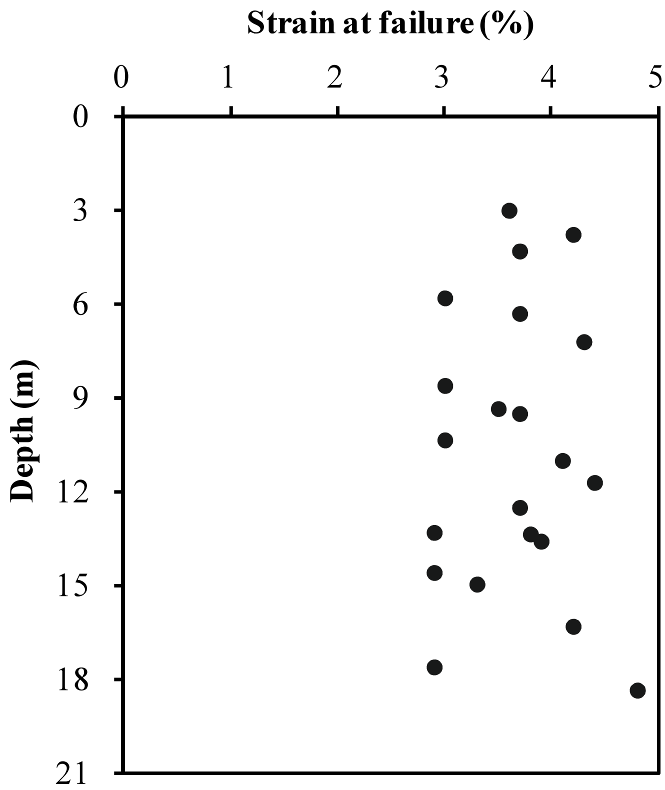


Fig. 4.7 Strain at failure of unconfined compression test for Gunsan clayey silt

Tanaka et al. (2001) investigated two soils with low plasticity; an intermediate soil from Ishinomaki, Japan and lean clay from Drammen, Norway. As shown in Fig. 4.8, although these two soils had the same order in the plasticity index (PI), there was a significant difference in the grain size distribution between them. The Ishinomaki intermediate soil contained a lot of sand or silt sized particles. Its PI value was nearly proportional to its clay content. On the other hand, Drammen clay consisted of a large proportion of rock flour due to the rock grinding, which contained few clay minerals. Tanaka et al. (2001) suggested that the p'_r values measured for Ishinomaki intermediate soil and the Drammen clay were much smaller than those measured for ordinary clay, for which p'_r is approximately $1/6p'_{vo}$. Tanaka et al. (2001) observed that these clays have a strong correlation between p'_r and $q_u/2$. These results indicated that the UC test is not suitable to evaluate the undrained shear strength of low plasticity soils. Therefore, when the undrained shear strength of soil with a low plasticity index is evaluated, the effective confining pressure that corresponds to typical marine clay should be applied to the soil specimen before shearing in order to compensate for the lost residual effective stress.

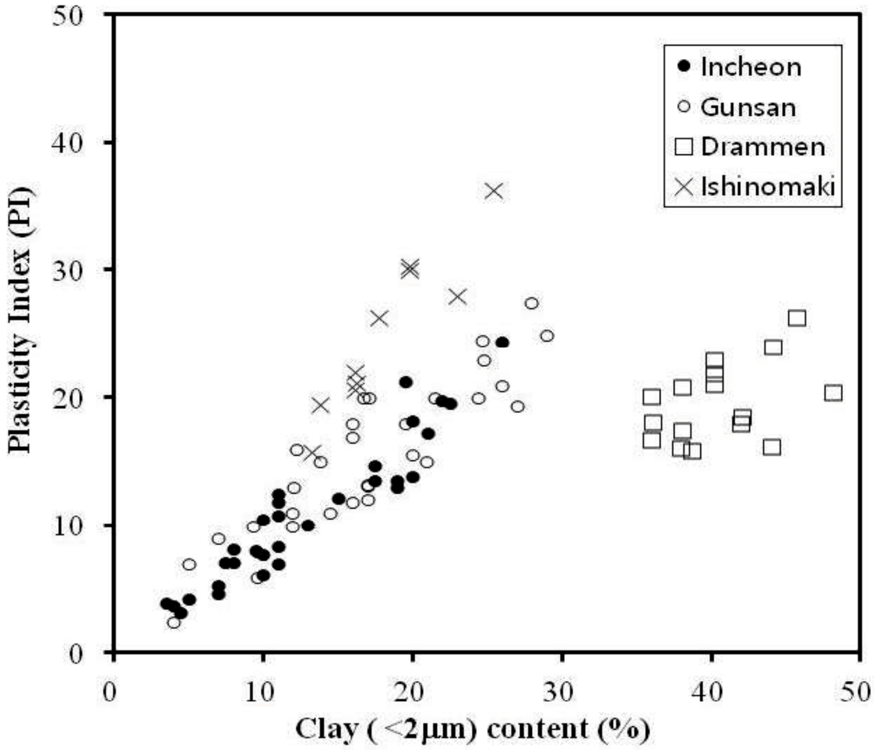


Fig. 4.8 Comparison of activity from four different low plasticity soils

4.3.2 RECOMPRESSION TESTING METHOD USING CIU TRIAXIAL TEST AND ITS RELATION TO $q_u/2$ STRENGTH

Hanzawa (1982) proposed a method that evaluates the mobilized undrained shear strength (s_{umob}) to be used as a design parameter in the stability analysis. By analyzing several failed case histories in construction field, he claimed that his proposed method is able to predict failures in field. In his method, a soil specimen is consolidated under k_o conditions to the in-situ effective stresses and then sheared under undrained conditions at an axial strain rate of 0.01%/min. To take into account the strength anisotropy, the average undrained shear strength (s_{uave}) from the compression and the extension tests ($s_{uave}=(s_{uc}+s_{ue})/2$) is used, where s_{uc} and s_{ue} are strengths measured by the compression and the extension tests, respectively.

Meanwhile, elaborate triaxial testing methods such as Hanzawa's method are time consuming process and expensive to be carried out in practice. Therefore, Tsuchida and Mizumaki (1991) proposed a simplified testing method to evaluate s_{umob}, where a specimen is isotropically consolidated under the mean effective stress, before shearing in undrained condition, as mentioned earlier. The s_{umob} values obtained from the CIU test are multiplied by the correction factor of 0.75 regardless of PI. This factor of 0.75 in the CIU test is derived assuming strength anisotropy ratio (s_{ue}/s_{uc}) of 0.70 (Tsuchida and Tanaka, 1995). For considering strain rate effect in the CIU test, based on the experience on the strain rate effect for Japanese marine clays, 7% reduction in strength was applied regarding one logarithm scale of the axial strain rate, comparing with CK_oU test results (Tsuchida and Tanaka, 1995). As shown in Fig. 4.4-4.5, s_{umob} values obtained from recompression test for Incheon and Gunsan clayey silt with low plasticity, unlike $q_u/2$ values, increased considerably with depth. The s_{umob} values by the recompression test for Incheon and Gunsan clayey silt with low plasticity increased with a strength depth ratio of 2.8kPa/m, 2.5kPa/m, respectively. These values correspond to a strength incremental ratio (s_u/p'_c) of 0.32, 0.30, respectively, which are common values for typical Korean marine clays.

Without back analyzing case histories, it is very difficult to confirm whether or not the recompression testing method provides a reasonable strength. This issue is addressed in more detail in the following section.

4.4 NORMALIZED UNDRAINED SHEAR STRENGTH (s_u/p'_c)

Fig. 4.9-4.10 show e-logP curves for the Incheon and Gunsan clayey silts. And, the p_c values obtained at various depths are plotted in Fig. 4.11-4.12. The incremental load ratio was unity and the duration of each step loading was 24 hours. Because the Incheon and Gunsan clayey silts with low plasticity were deposited under seabed and have never been exposure to the atmosphere since then, it is apparent that the layers are geologically normally consolidated. As shown in Fig. 4.11-4.12, however, the p_c values are somewhat bigger than the in-situ effective burden pressure (p'_{vo}) and the ranges of OCR are 1.1-2.8, 1.2-3.0, respectively, for Incheon and Gunsan sites. This over-consolidation's characteristics might be due to aging effect as well as cementation.

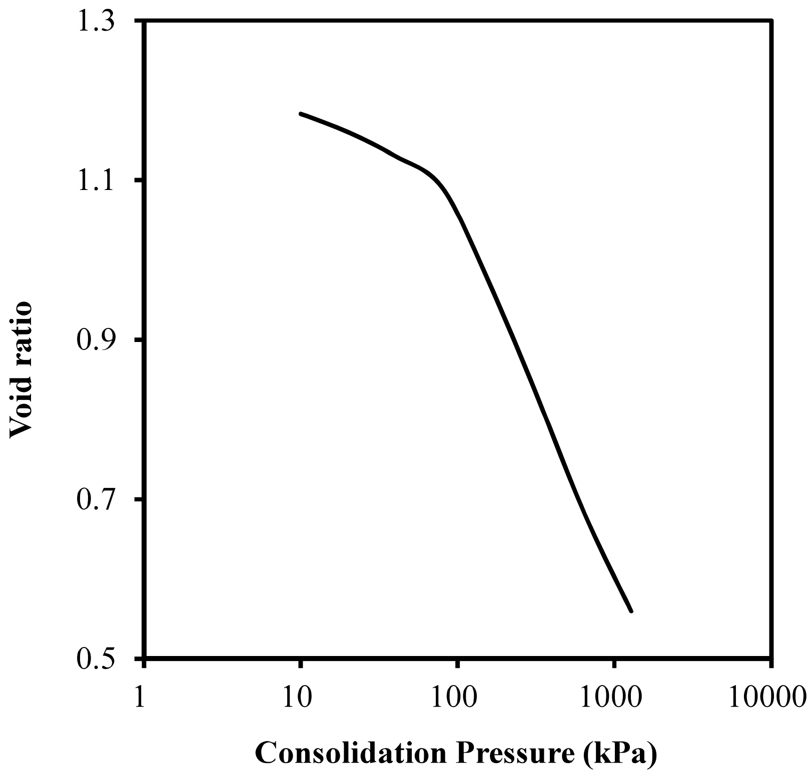


Fig. 4.9 A typical e-logP curve for Incheon clayey silt at depth=8m

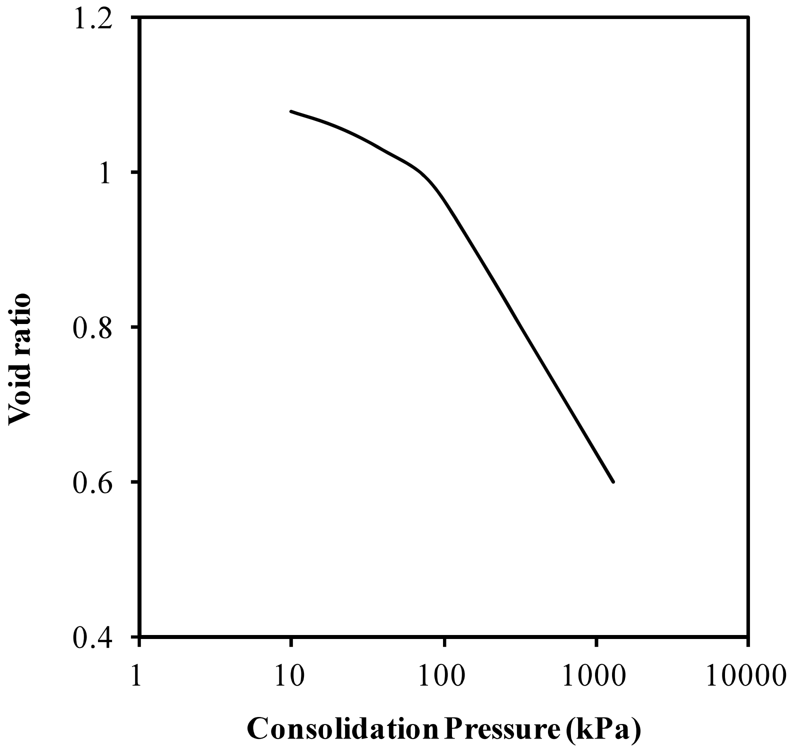


Fig. 4.10 A typical e-logP curve for Gunsan clayey silt at depth=9m

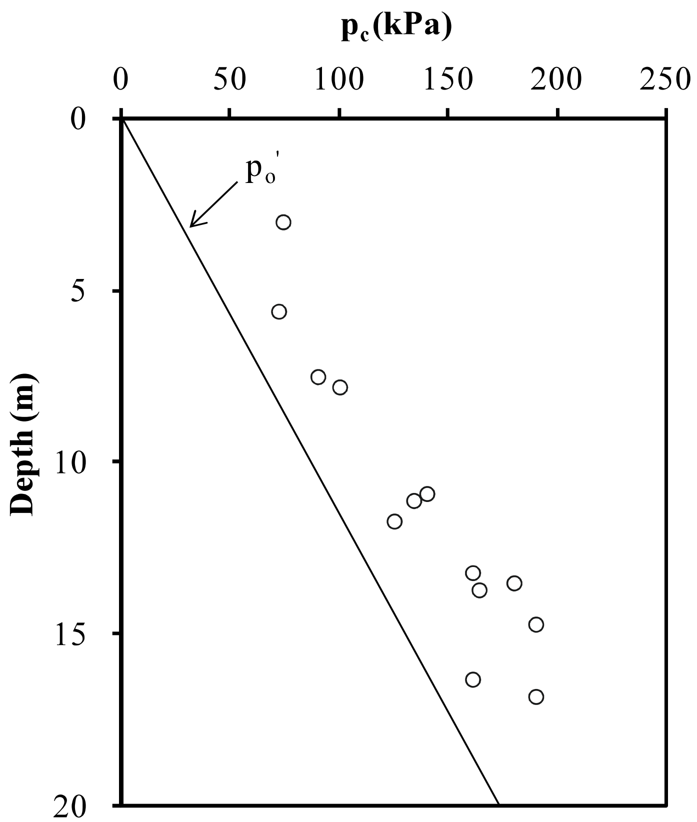


Fig. 4.11 Yield consolidation pressure with depth for Incheon site

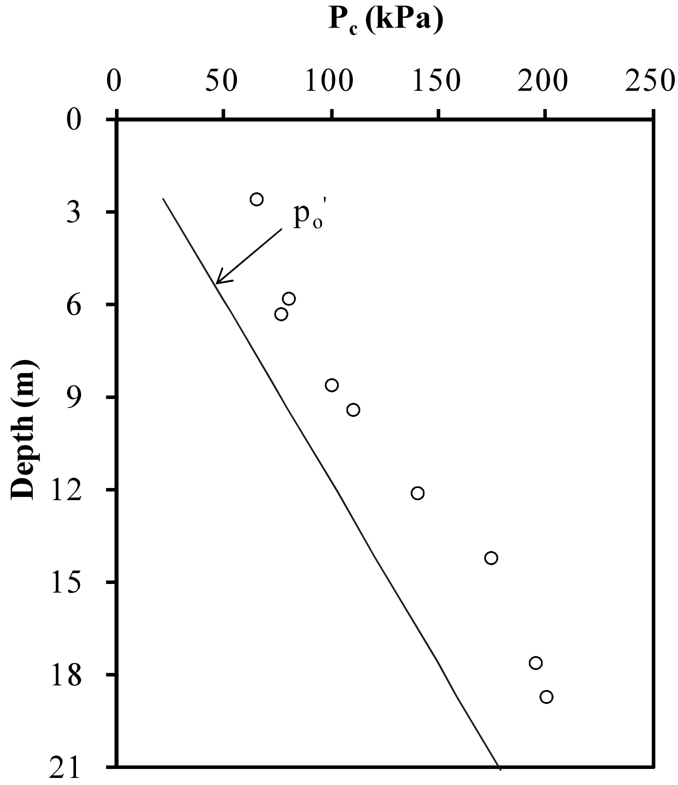


Fig. 4.12 Yield consolidation pressure with depth for Gunsan site

Fig. 4.13 shows the s_u/p'_c ratios obtained by the UC, CIU (recompression), and FVT tests of soils taken from the four coastal areas. In case of the Incheon and Gunsan areas, the average s_u/p'_c ratios obtained by the UC test were 0.18 and 0.19, respectively, which are nearly the same values as those estimated using Skempton's equation. Therefore, the s_u/p'_c values for Korean marine clays have been considered to be proportional to I_p, and Skempton's equation yields reliable values. As mentioned earlier, the reason why the UC test has been continuously conducted is the existence of an automatic balance between the factors which overestimates and other factors which underestimates the undrained strength. When a soil specimen is retrieved from the ground and is exposed to the atmosphere, part of the in-situ effective stress in the specimen remains in the form of negative pore pressure. This negative pore pressure called the residual effective stress (p'_r). Even though the specimen is tested under unconfined conditions, the residual effective stress acts as a confining pressure. However, if the value of p'_r in the specimen is reduced due to the disturbance of the specimen during sampling, handling, extruding or trimming, and then the $q_u/2$ value will be reduced due to swelling.

Meanwhile, the average s_u/p'_c ratios obtained by the CIU test from the Incheon and Gunsan clayey silts were 0.32 and 0.30, respectively. The average s_u/p'_c ratios obtained by the FVT test from the Incheon and Gunsan clayey silts were 0.36 and 0.32, respectively. Since the undrained shear strength obtained by FVT for the clayey silt containing sandy soil tended to be much higher than that obtained for ordinary clay, it seems that the $s_u/p'_{c(FVT)}$ ratio gave a larger result than the $s_u/p'_{c\ (CIU)}$ ratio. In case of Busan and Gwangyang areas, the average s_u/p'_c ratios obtained by the UC test were 0.30 and 0.32, respectively, similar to that of the Incheon and Gunsan areas, which are nearly the same values as those estimated using Skempton's equation. The average s_u/p'_c ratios obtained by FVT from the Busan and Gwangyang soils were 0.31 and 0.33, respectively.

Prior research and this research on the s_u/p'_c ratio showed that it varies from place to place, in particular at a small I_p. In the northern areas of the world, such as in Scandinavia (Drammen clay is a representative case in this area) and in eastern Canada, clay with a low plasticity is widely distributed, even though it contains many fine particles. Although the fine particles can be classified as clay according to the United Soil Classification System (USCS), they contain no clay minerals because they are produced by glacial grinding of rock and are sometimes called "rock flour" (La Rochelle, 1992). When the vane is inserted into such clay, a high pore water pressure is assumed to be generated due to the low compressibility and the highly developed structure. For this reason, the undrained shear strength can be considerably reduced. In areas where such clay is encountered, disturbances caused mainly by the insertion of the vane are one of the most serious problems for the measurement of the shear strength (La Rochelle et al., 1988).

Kimura and Saitoh (1983) suggested that the strength reduction can be recovered by dissipation of the excess pore water pressure, although it takes a long time for such dissipation to occur. However, Korean marine clays with a low plasticity generally contain many sand particles, similar to Japanese marine clay, which has sand seams in the clay layers in most cases. As a result, the permeability is considerably higher than that of clay in Scandinavia. For example, the coefficient of consolidation, c_v, obtained by the conventional oedometer test for the Incheon and Gunsan clayey silts, are around 770 cm^2/day and 730 cm^2/day for average I_p of 18% and 21%, respectively. In contrast, the data from the offshore Norway site indicates that c_v is around 27 cm^2/day when I_p is in the range of 15-17% (Grozic et al., 2003). Therefore, although a large pore water pressure develops when the vane is inserted into the Incheon and Gunsan clayey silts, the excess pore water pressure can be dissipated readily due to the high permeability, and the degree of the strength reduction can be minimized. When the UC test is conducted for the Incheon and Gunsan clayey silts, a negative pore water pressure, which acts as a confining pressure, tends to be lost because the permeability of the Incheon and Gunsan clayey silt with a low I_p, is large. As a result, the Incheon and Gunsan clayey silts mobilize only a small strength due to the loss of the negative pore water pressure under the UC test conditions.

Tanaka (2000) suggested that the q_u value is heavily underestimated even when the p'_r is high if the soil structure is destroyed due to a low-quality sampling,. Whether or not the soil structure has been destroyed can be confirmed by checking the shape of the e-logP curve. In case of an undestroyed soil structure, a clear p'_c can be observed and the gradient c_r before p'_c is very small. On the other hand, as shown in Fig. 4.9-4.10 for a low-quality sample, the gradient c_r before p'_c is large even though p'_c is clearly observed. Unfortunately, even though the residual effective stress measurement test has not been conducted for the Incheon and Gunsan clayey silts, the reduction of q_u for the Incheon clayey silt with low plasticity can be inferred from the destruction of the soil structure and the loss of the residual effective stress in the specimen. Therefore, a recompression test, such as CIU, should be conducted to compensate for the loss of the residual effective stress. Based on prior research and the present research, the s_u/p'_c ratio is in the range of 0.25 to 0.35, regardless of the plasticity index (I_p) of Korean marine clays. However, the UC test results from the Incheon and Gunsan areas are similar to the results for the Japanese marine clays. Therefore, it can be inferred that the recompression test provides a much more adequate shear strength value for clayey silt with low plasticity, unlike the UC test.

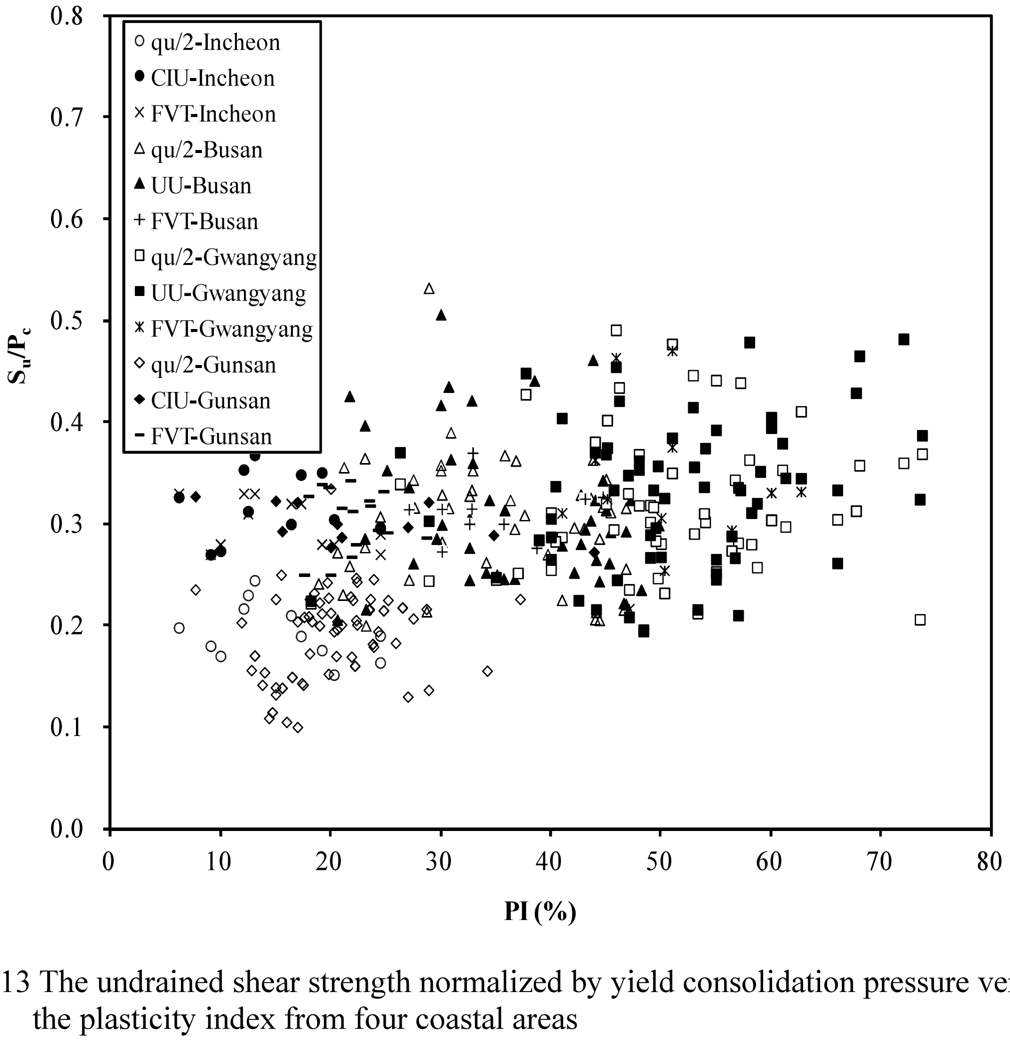


Fig. 4.13 The undrained shear strength normalized by yield consolidation pressure versus the plasticity index from four coastal areas

4.5 COMPARISON OF RESULTS BETWEEN UCT and FVT

The ratios of $s_{u(FVT)}$ to $q_u/2$ are plotted against I_p in Fig. 4.14. The values of $s_{u(FVT)}$ are apparently greater than those of $q_u/2$ value in particular when I_p is less than 25. Since the values of $s_{u(FVT)}/\ p'_c$ and $s_{u(CIU)}/\ p'_c$ for the clays with smaller I_p do not decrease as shown in Fig. 4.13, the increase in $s_{u(FVT)}/(q_u/2)$ may be attributed to the reduction of the values of q_u. As already pointed out, the permeability of Korean clayey silts with small I_p values tends to easily lose the negative pore pressure under UC test conditions. As a result, clay with small I_p values mobilizes only a small amount of strength in accordance with the reduction of the amount of negative pore pressure in the specimen. Except for clay with small I_p, the ratio of $s_{u(FVT)}/(q_u/2)$ ranges from 0.9 to 1.2.

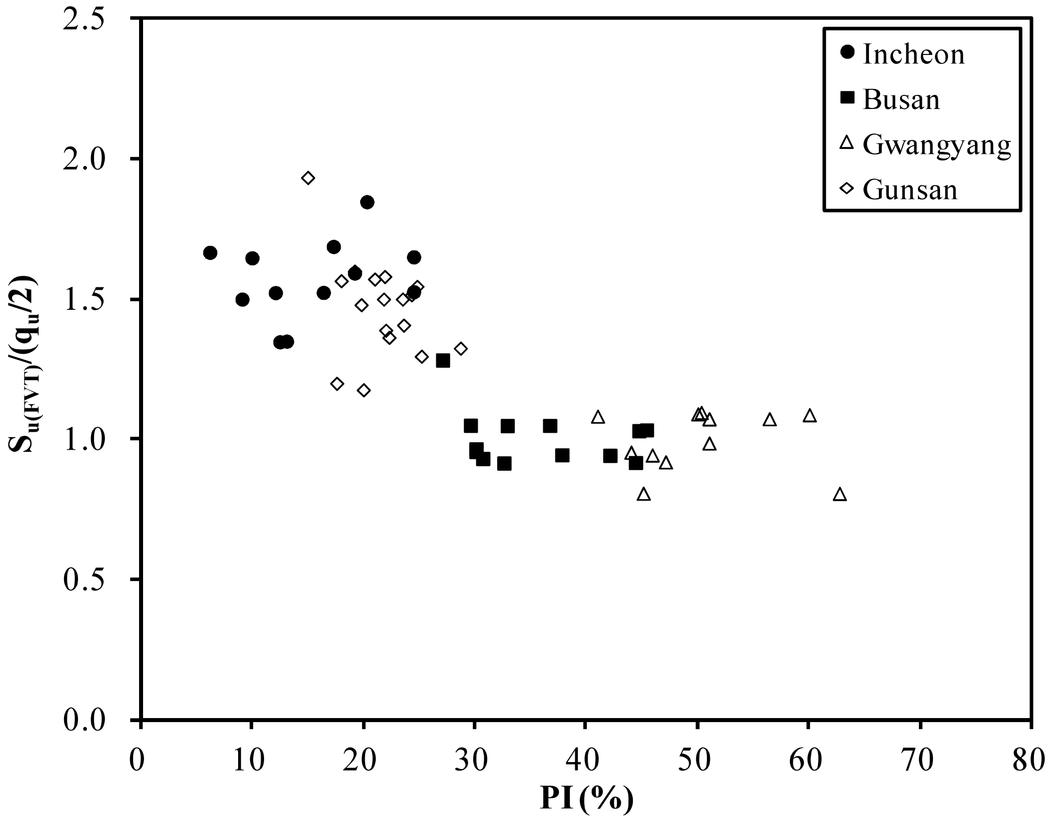


Fig. 4.14 Strength ratio of $s_{u(FVT)}$ to $q_u/2$ versus I_p

4.6 BJERRUM'S, MORRIS AND WILLIAMS' CORRECTION FACTORS

The ratio of $s_{u(FVT)}$ corrected by μ to $q_u/2$ is plotted against I_p in Fig. 4.15. However, if both the $q_u/2$ and $s_{u(mob)}$ values corrected by μ are identical, the ration must be unity. As shown in Fig. 4.15, the ratio seems to be influenced by I_p. When I_p is greater than 30, the ratio becomes less than unity, the higher the value of I_p, the greater is this trend. Meanwhile, the ratio of $s_{u(FVT)}$ corrected by μ_v to $q_u/2$ is plotted against I_p in Fig. 4.16 using Morris and Williams' correction factor in the same way. When I_p is greater than 30 such as in Bjerrum's correction factor, the ratio becomes less than unity, the higher the value of I_p, the greater is this trend. In addition, $s_{u(mob)}$ corrected by Morris and Williams' correction factor is much smaller than $s_{u(mob)}$ corrected by Bjerrum's correction factor. It is inferred that, according to Bjerrum's, Morris and Williams' correction methods, Korean structures supported on subsoils where I_p is over 30 should have failed because $s_{u(mob)}$ is overestimated by the q_u method. Fortunately, we have never had such failures at least for port facilities designed by using the $q_u/2$ value. However, there might be a possibility that the q_u method overestimates the mobilized shear strength, and a safety factor fortunately does compensate for the shear strength estimated unsafely by the q_u method. A suitable safety factor F_{sp} is actually used for the practical design procedure for the q_u method. The purpose of F_{sp} may be to cope with uncertainties of design parameters, such as $s_{u(mob)}$ or the unit weight of the foundation soil, as well as the external forces imposed on the structures. The value of F_{sp} is specified according to the importance and the size of the structures by the technical standards for port and harbor facilities and has a value of 1.3 for ordinary stability (Japanese Port Association, 2007). To make F_{sp} equivalent to the correction factor in the q_u method, the factor μ_q to $q_u/2$ becomes 0.77 when F_{sp}=1.3. It can be seen in Fig. 4.15-4.16 that the lowest value of ($\mu s_{u(FVT)}/(qu/2)$) and ($\mu_v s_{u(FVT)}/(q_u/2)$) are rather smaller than 0.77. Even when applying Bjerrum's, Morris and Williams' method to evaluate $s_{u(mob)}$, the proper safety factor is required to consider the uncertainties of the design parameters and external forces on the structure, and the errors arising from the stability analysis are the same as those in the q_u method. Therefore, it is concluded that the corrected vane shear strength significantly underestimates the mobilized shear strength of Korean marine clays, as it does Japanese marine clays.

Meanwhile, as mentioned earlier, Mesri (1975)'s equation was derived from Bjerrum's data, which consists of s_u/p'_o and p'_c/p'_o for young as well as aged clays, as well as results for μ. Although the mobilized undrained shear strength, obtained by normalizing the yield consolidation pressure shows a constant value, independently of the plasticity index, this equation was derived from based on combining Bjerrum's curve in which $s_{u(FVT)}$ normalized by the yield consolidation pressure, $s_{u(FVT)}/p'_c$ increases with increasing I_p with Bjerrum's field vane correction factor. So, intrinsically, this equation is influenced by plasticity index, I_p. It suggests that the incremental strength ratio for the preloading method must be 0.22. Based on plasticity index dependent characteristics of undrained shear strength, this equation might be applied to Korean marine clayey soils. However, in Korea, in contrary, an incremental strength ratio of approximately 0.3 is usually adopted in practice except for low plastic soils such as those of the Incheon and Gunsan areas. Although confirming investigation testing to confirm the results, such as FVT, CPT and the q_u test at completed construction sites, are not included, based on testing results from the four coastal areas, it can be concluded that Mesri's proposed relation of $s_{u(mob)}/p'_c$ is quite underestimated when it is applied to Koreans marine clays.

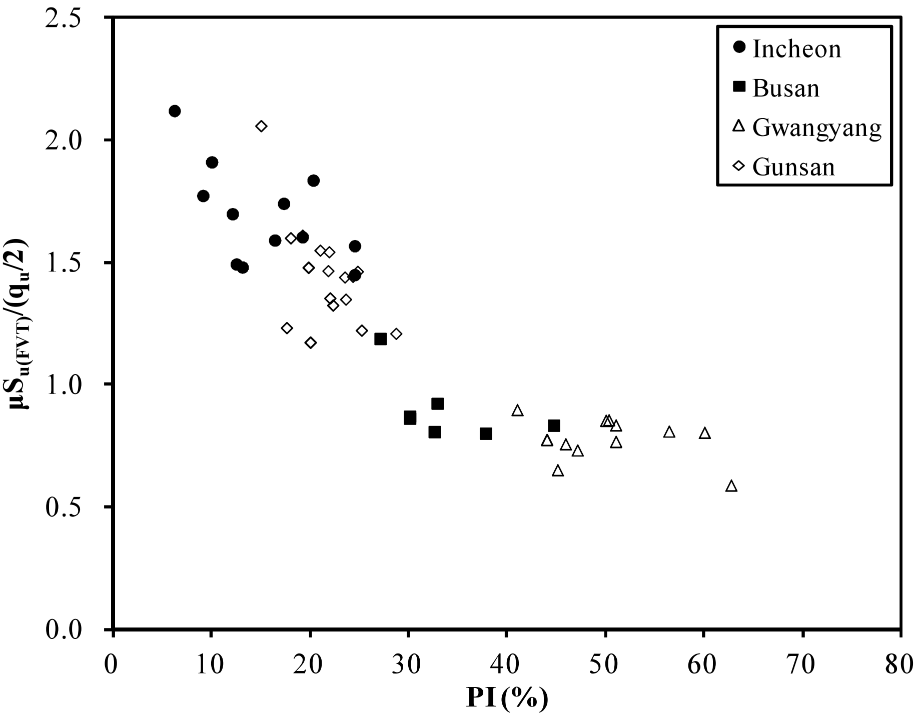


Fig. 4.15 Comparison of vane strength corrected by Bjerrum's factor and $q_u/2$

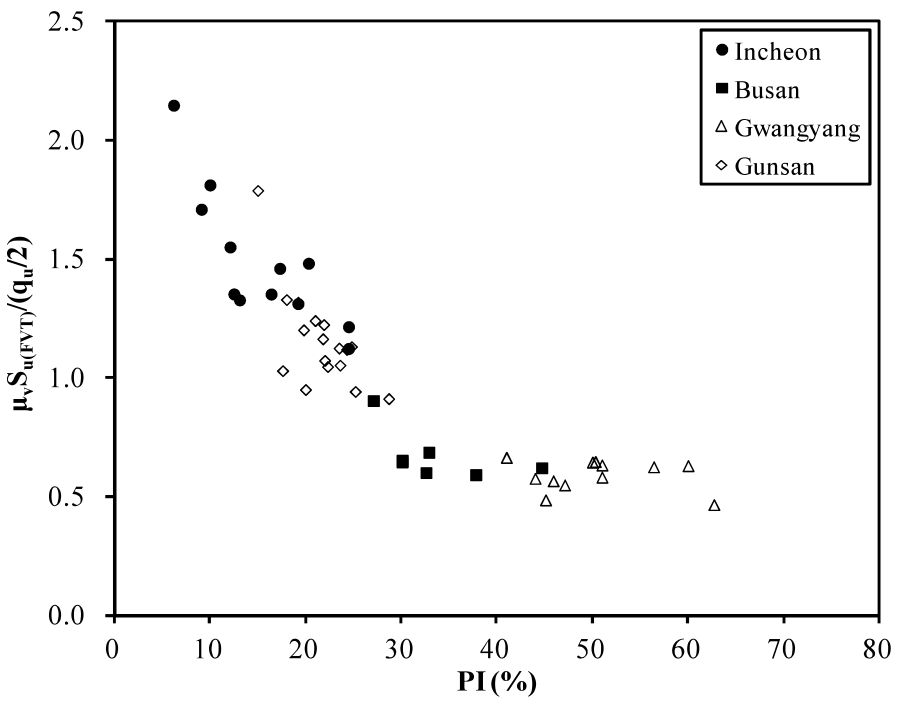
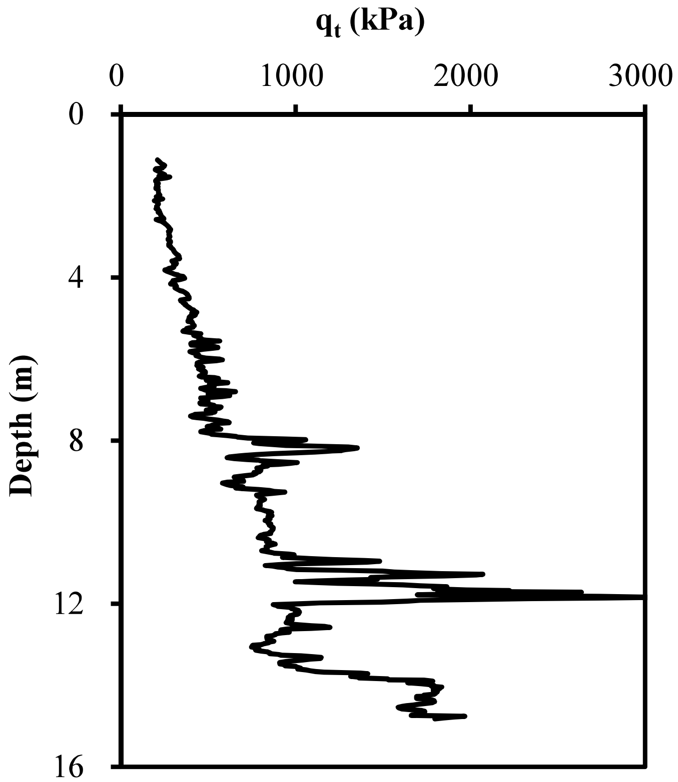


Fig. 4.16 Comparison of vane strength corrected by Morris and William's factor and $q_u/2$

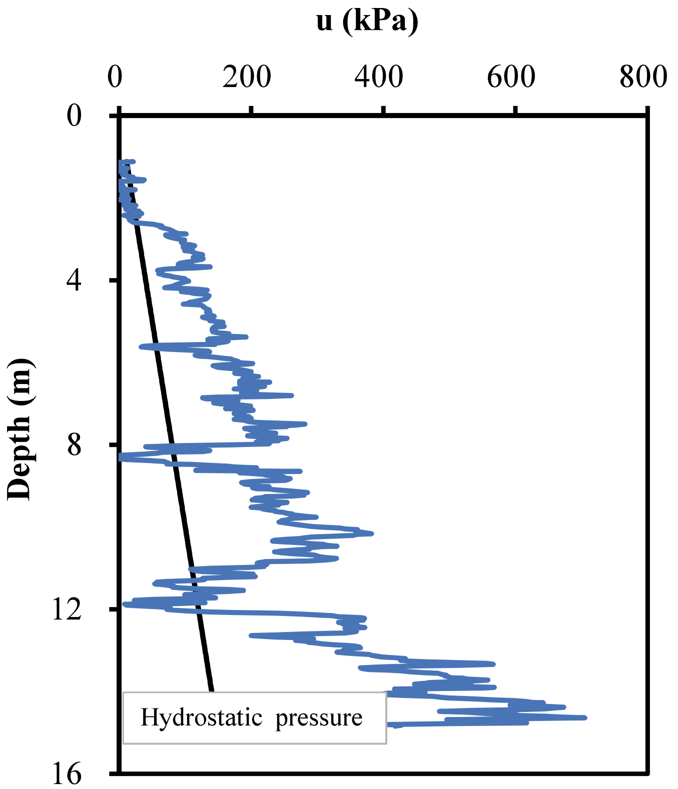
4.7 ANALYSIS OF FIELD TEST BY CPTU AND FVT

The layered nature of the Incheon clayey silt with low plasticity is well depicted in Fig. 4.17 by the CPTU test. The CPTU tests were conducted at a distance of around 1km from each sampling site. As shown in Fig. 4.17, the corrected cone resistance (q_t), which is calculated by considering the effective area of the cone, the influence of the pore water pressure acting on the filter, and the sleeve friction (f_s) that fluctuates with increasing depth, all indicate the existence of many small layers. It may be seen that the pore water pressure (u) is invariably small at depths where a large q_t is observed, which indicates a permeable sand seam at those locations. In particular, many such sandy layers can be seen at depths of around 8m and at a depth of 12m at site A. At depths shallower than 5m and at a depth of around 11m at site B, the fluctuations of cone resistance with increasing depth are considerably larger than those at other depths. At these depths, the excess pore pressures are smaller than the hydrostatic pressure, which indicates the existence of many permeable silty or sandy soil layers at these locations. And, at depths shallower than around 3m and at deeper than around 9m at site C, the cone resistances were larger than the other depths. However, the excess pore pressures for almost the whole depth of the grounds were larger than hydrostatic pressure, except for depths shallower than around 3m.

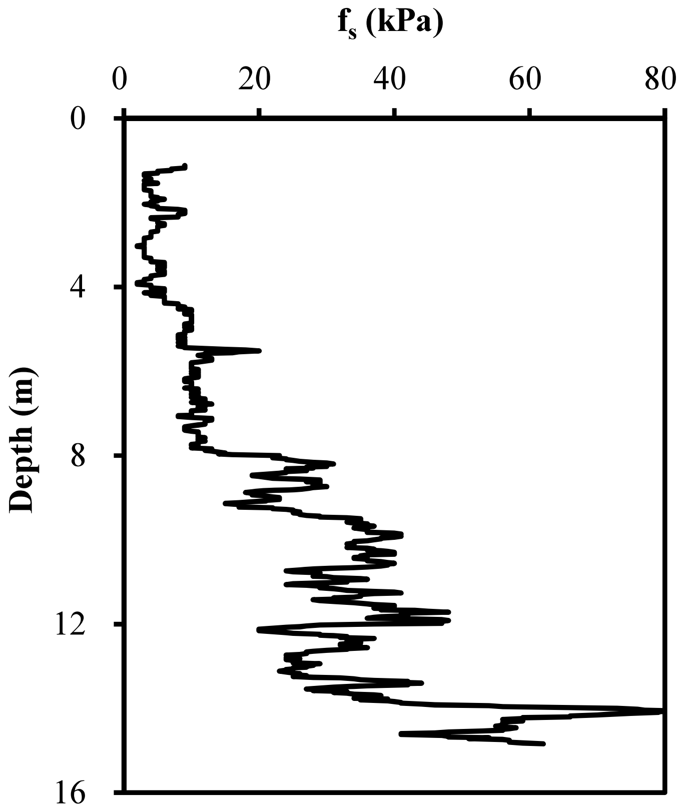
On the other hand, CPTU results do not necessarily correspond to the particle size distribution profile, since the grain size distribution at each level is calculated from a single point. On the other hand, the CPTU data are obtained only from around the tip of the cone. Therefore, it is likely that the particle size distribution profile from the undisturbed tube sample does not provide precise distribution characteristics for the entire depth of 1 meter. In this case, CPTU can be a powerful tool to avoid making an incorrect judgment regarding the soil distribution characteristics of clayey silts with low plasticity.



qt (kPa)
0
1000
2000
3000
0
4
8
12
16
Depth (m)

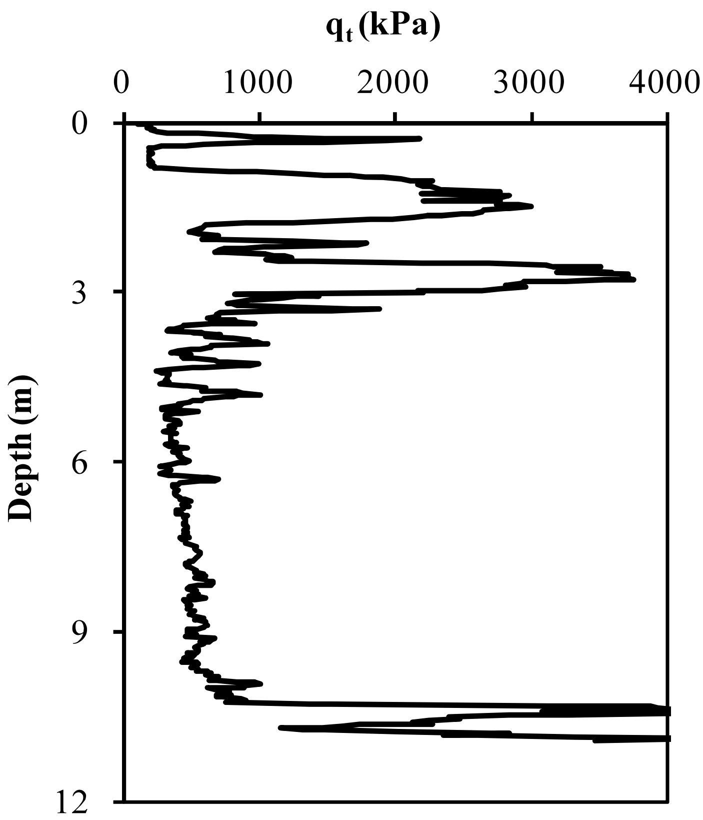


u (kPa)
0
200
400
600
800
0
4
8
12
16
Depth (m)
Hydrostatic pressure

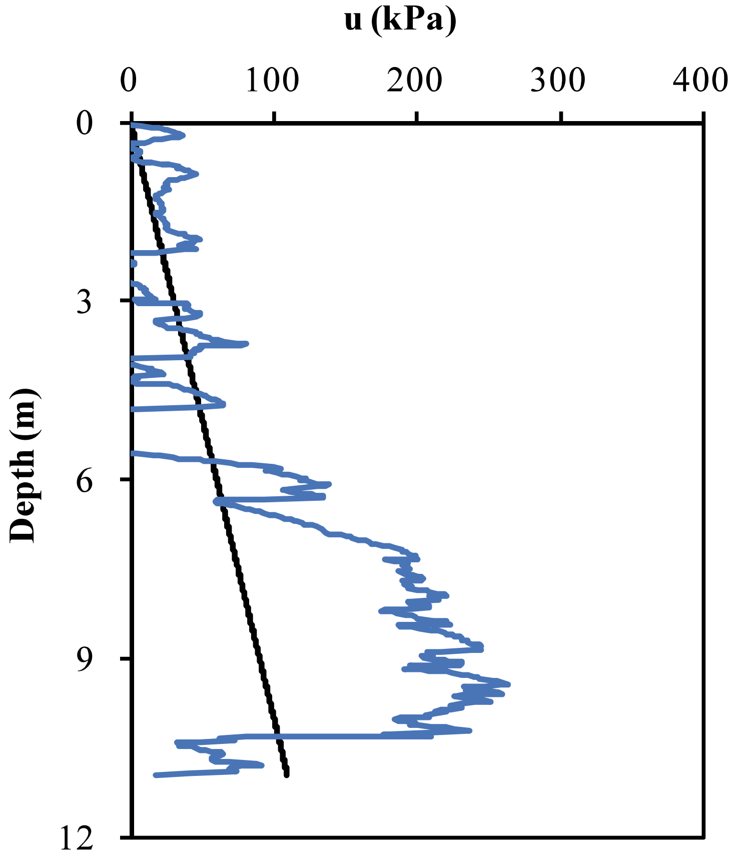


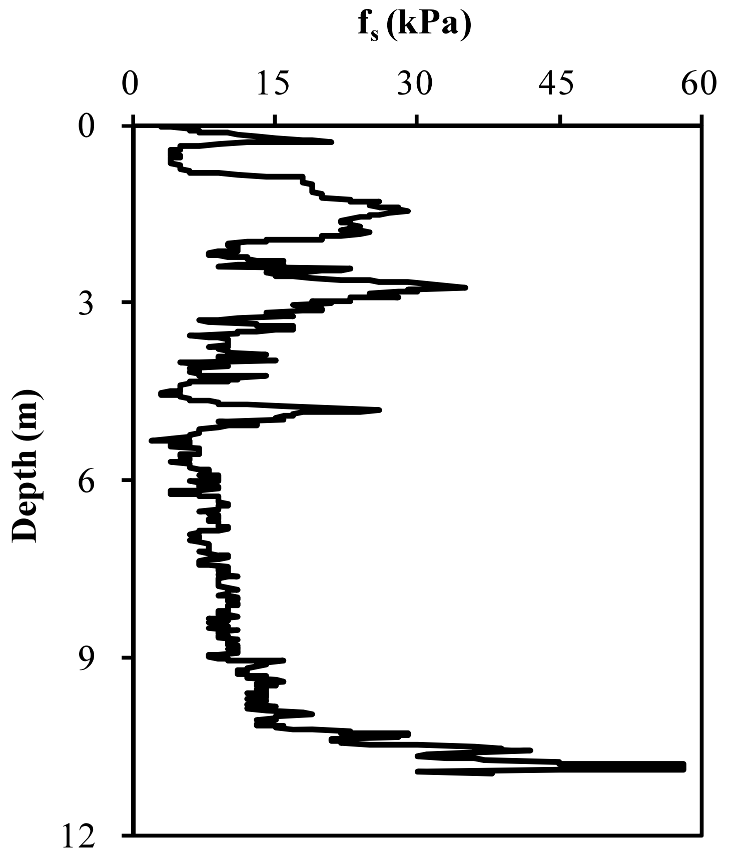
fs (kPa)
0
20
40
60
80
0
4
8
12
16
Depth (m)

(a) Site A

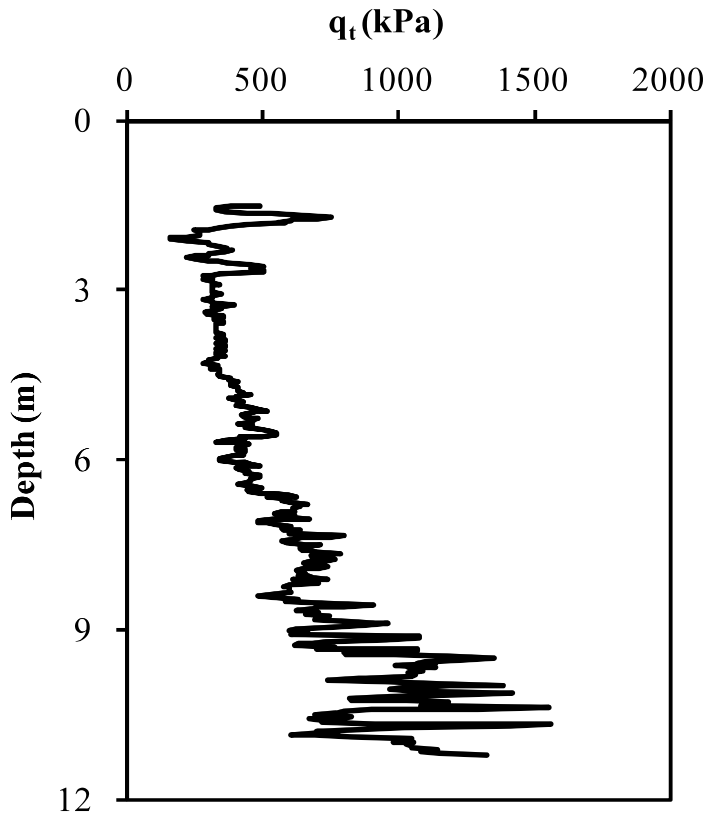


qt (kPa)
0
1000
2000
3000
4000
0
3
6
9
12
Depth (m)

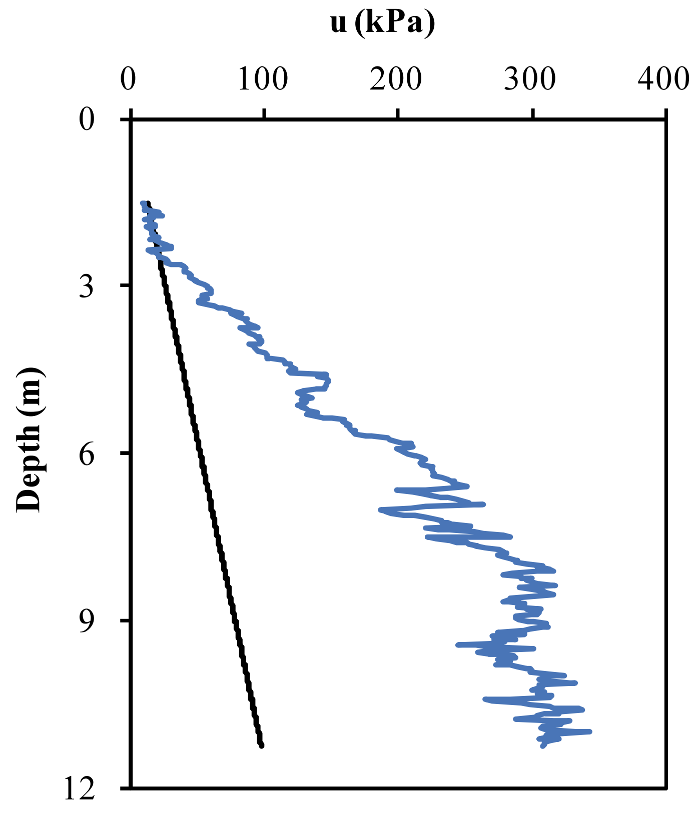

u (kPa)
0
100
200
300
400
0
3
6
9
12
Depth (m)


f_s (kPa)
0
15
30
45
60
0
3
6
9
12
Depth (m)

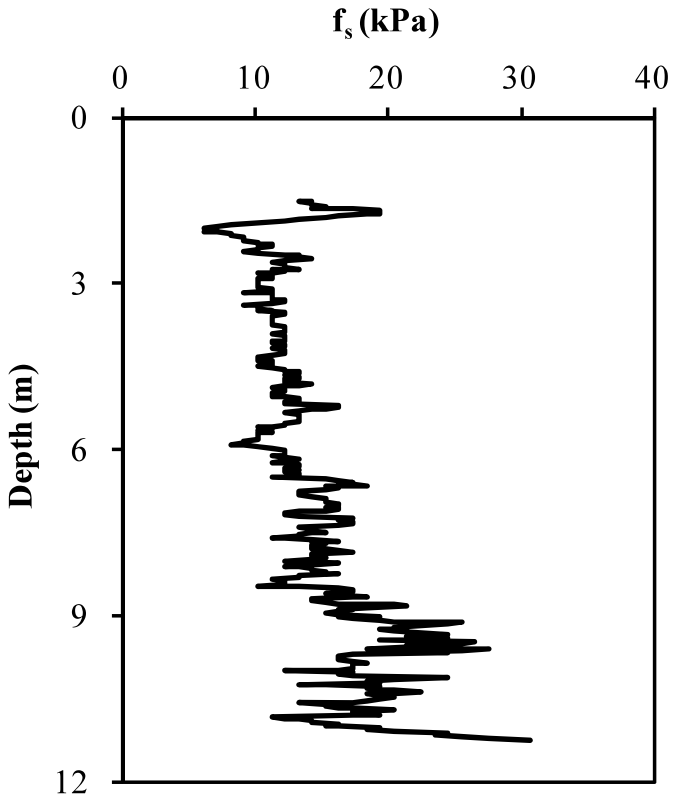
(b) Site B



q_t (kPa)
0
500
1000
1500
2000
0
3
6
9
12
Depth (m)



u (kPa)
0
100
200
300
400
0
3
6
9
12
Depth (m)



(c) Site C

Fig. 4.17 Test results obtained from CPTU at the Incheon site

Fig. 4.18 shows a comparison of the strengths measured by laboratory and in-situ test of Incheon soil. The undrained shear strength estimated by the CPTU test was calculated with a cone factor (N_{kt}) of 12. As shown in Fig. 4.18(a), at depths shallower than 8m, the undrained shear strength obtained by the recompression test coincided with that obtained by the CPTU at site A. However, at depths deeper than 8m, the mobilized shear strengths using the CIU triaxial tests were smaller than the strength estimated by the CPTU. This trend was predominant at depths of around 8m and 12m.

On the other hand, the mobilized shear strengths using the CIU and FVT tests can be seen to be nearly equal to each other, except at depths of around 2.5m and 6m, respectively, which may be attributed to sand seams. The vane strength may be considered to be equivalent to the

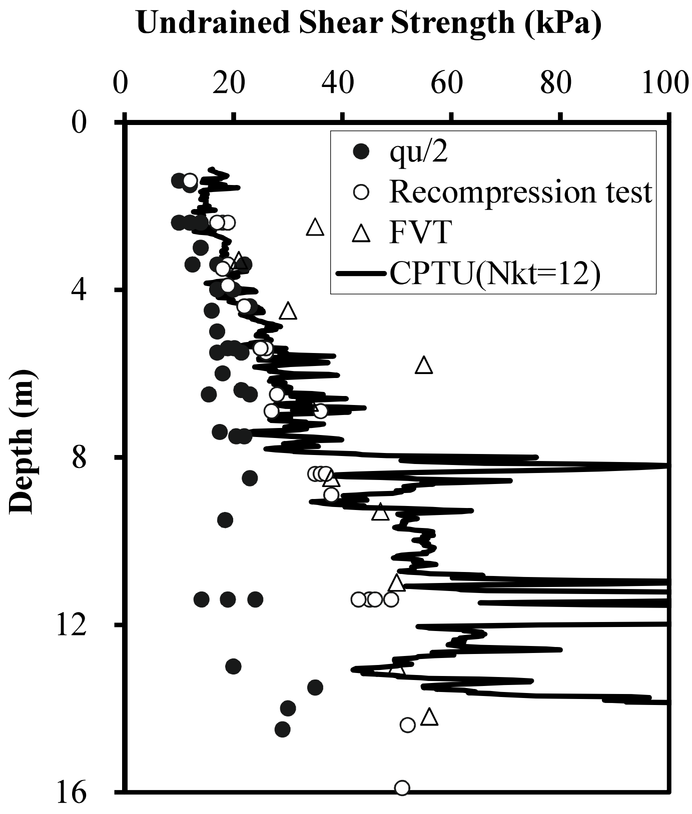
CIU strength, in the sense that both strengths are not influenced by the stress release. Therefore, the FVT can be used to obtain the mobilized shear strength for the clayey silt with low plasticity, such as that obtained by the CIU test, if the CPTU result is used to check whether or not there is a sand seam.

Even though laboratory strength tests were only carried out at site A, CPTU results of site B, C were compared to laboratory test results of site A to judge applicability as undrained shear strength of these sites.

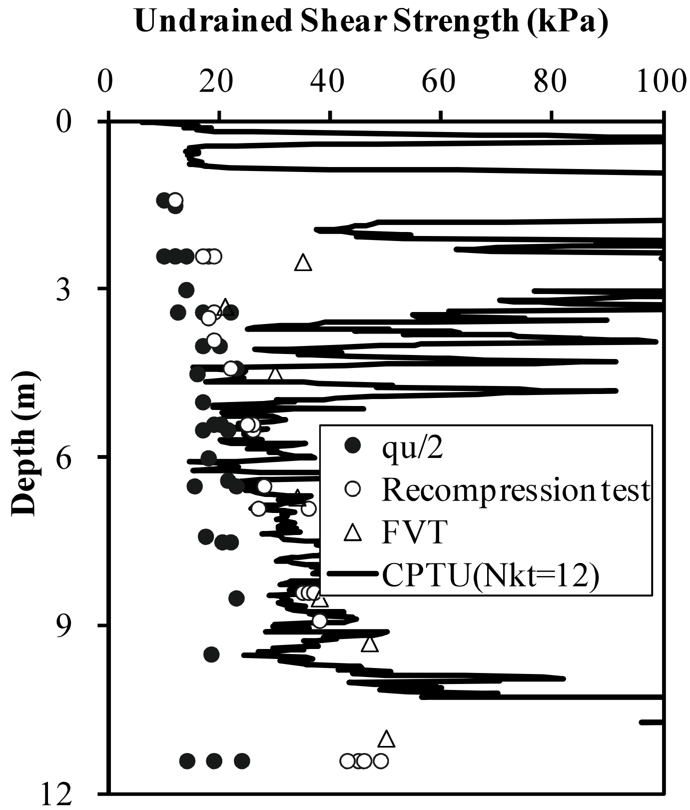
At depths shallower than 6m and at around 10m depth at site B, the s_u values calculated by CPTU results (N_{kt}=12) were larger than those obtained by CIU (recompression test). Especially, the percentage of sand at these depths is approximately larger than 50%.

And, finally, at depths shallower than around 3m and at deeper than around 7m at site C, the s_u values calculated by CPTU results (N_{kt}=12) were larger than those obtained by CIU (recompression test). However, the excess pore pressures at almost the whole depth of the grounds were larger than hydrostatic pressure except for depths shallower than around 3m where the excess pore pressure was somewhat similar to hydrostatic pressure. As a result, if the grounds is likely to be under partially drained or fully drained conditions, it is difficult to apply the estimation method of undrained shear strength using cone factor, N_{kt}, based on φ=0 method considering fully undrained conditions. Therefore, it is necessary to estimate shear strength considering partially drained conditions. This part is treated in more detail in the following section.

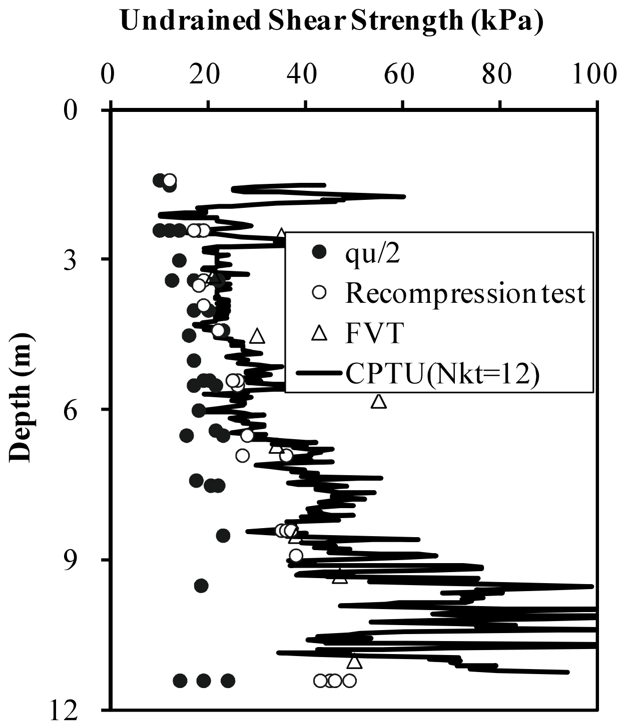
On the other hand, the mobilized shear strengths using the CIU and FVT tests can be seen to be nearly equal to each other except for depths of around 7m and 11m for Gunsan site, respectively, which may be attributed to sand seams, such as Incheon site, as shown in Fig. 4.19



(a) Site A



(b) Site B



(c) Site C

Fig. 4.18 Comparison of strengths measured by laboratory and in-situ tests for Incheon clayey silt with low plasticity

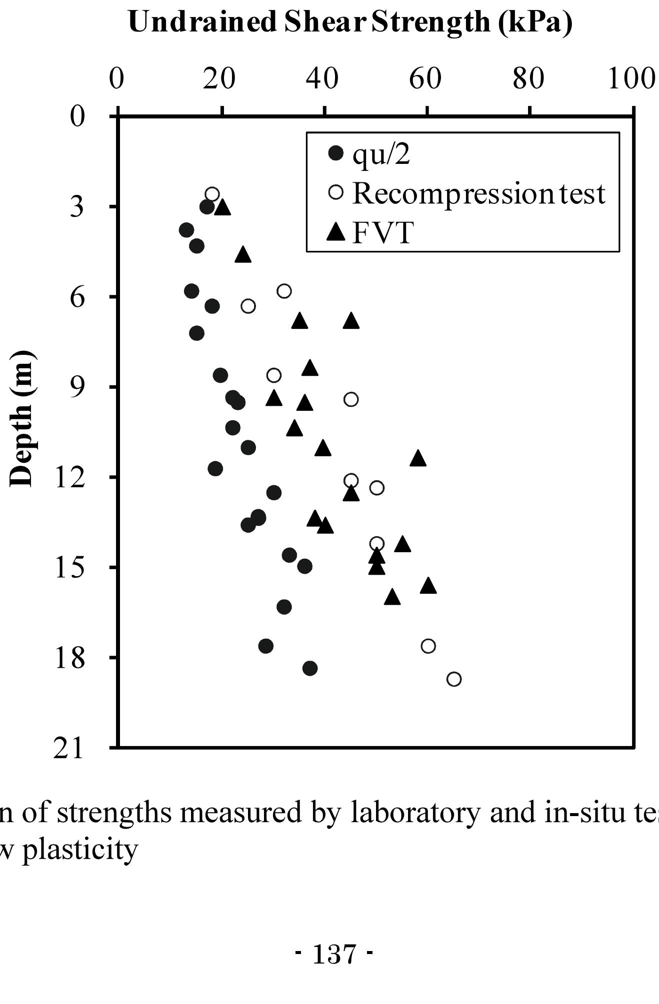


Fig. 4.19 Comparison of strengths measured by laboratory and in-situ tests for Gunsan clayey silt with low plasticity

4.8 CONCLUSIONS

Korean marine clays taken from four coastal areas have been investigated by carrying out a series of laboratory and in-situ tests. The main findings from this research are as follows.

1) The UC test is not suitable to evaluate the undrained shear strength of low plasticity soils. Therefore, when the undrained shear strength of soils with a low plasticity index is evaluated, an effective confining pressure that corresponds to the typical marine clay should be applied to the soil specimen before shearing in order to compensate for the loss of residual effective stress. In this cases, the CIU (recompression test) proposed by Tsuchida and Mizukami (1991), can be quite useful in duplicating the in-situ shear strength of a soil.
2) The vane strength can be equivalent to the CIU strength in the sense that both strengths are not influenced by the stress release. Therefore, FVT can be used to obtain the mobilized shear strength for the clayey silt with low plasticity, such as that obtained by the CIU test, as long as the CPTU result is used to check whether or not there is a sand seam.
3) The s_u/p'_c values for Korean marine clays are considered to be proportional to I_p, and Skempton's equation, which has been widely used in Korea, yields reliable value. However, based on the s_u/p'_c ratios obtained by the CIU (recompression test) in this research, the s_u/p'_c ratio for low plasticity soils, such as Incheon soil, has been underestimated. The s_u/p'_c ratio was found to be independent of I_p, and it was in the range of 0.25 to 0.35, except for the UC test results for the clayey silt, which were similar to the results for Japanese marine clays.
4) When I_p is less than 25, that is, in case of clayey silts with low plasticity such as those of

the Incheon and Gunsan areas, the ratio of $s_{u(FVT)}$ to $q_u/2$ depends on I_p, and the ratio is over unity with large scatter. The reason why the $q_u/2$ values using samples taken from sites having the characteristic of low plasticity are considerably underestimated is the reduction in the negative pore pressure in the specimen acting as a confining pressure.

5) When Bjerrum's and Morris and Williams' correction factor is applied to Korean marine clay, the mobilized undrained shear strength is considerably underestimated, especially when I_p is higher than 40. This trend is noticeable in Morris and Williams' correction factor. The normalized undrained shear strength of Mesri's s_u/p'_c=0.22 result is quite conservative when applied to Korean marine clay.

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CHAPTER 5
DRAINAGE CHARACTERISTICS OF CLAYEY SILTS WITH LOW PLASTICITY

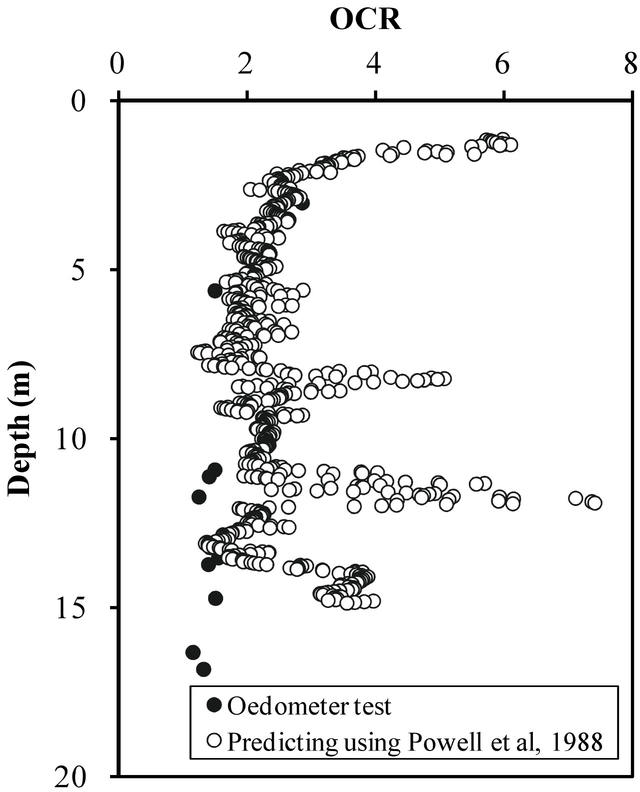
5.1 ASSESSMENT OF THE PARTIALLY DRAINED CONDITIONS

When the CPTU data is interpreted, the existing correlation equations are very useful in estimating the strength and the OCR before directly evaluating the strength and the consolidation conditions by laboratory and field tests. As shown in Fig. 5.1(a), 5.1(b) and 5.1(c), OCRs obtained by oedometer tests were compared with estimates obtained by using the well-known relationship proposed by Powell and Quarterman (1988), which is given by: OCR=κ $((q_t-\sigma_{vo})/\sigma'_{vo})$. The value of κ ranges from 0.2 to 0.5 and its average value is 0.3. Since higher values of κ are recommended for heavily aged overconsolidated clays, the estimated value strongly depends on the κ value. Generally, the estimated OCRs were higher than those obtained by the oeodometer tests, and these trends were noticeable for layers containing a lot of silty or sandy soils. Even though the lowest κ value was applied to estimate the OCR for layers containing much silty or sandy soil, there was no change in the general situation. As shown in Fig. 5.1(d), the estimated OCRs (κ=0.3) were similar to those obtained from the oedometer tests for high plasticity clay from Busan.

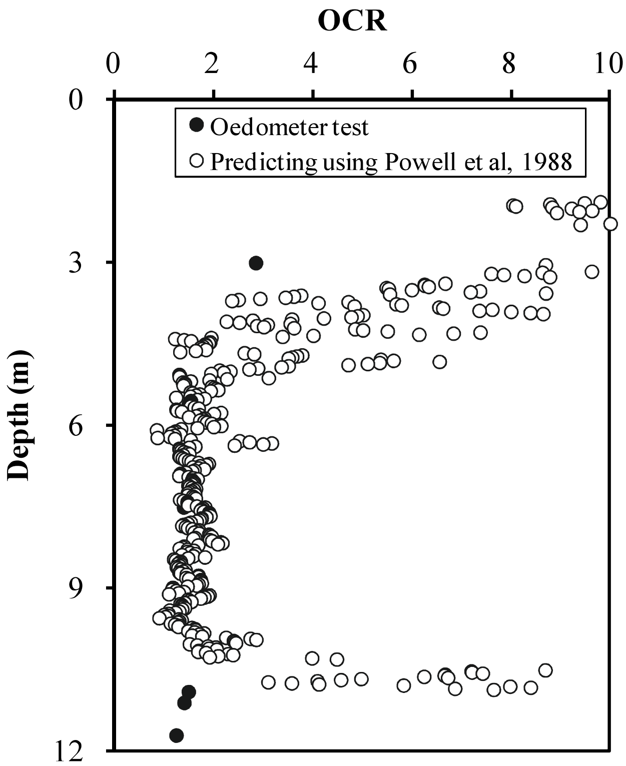
It is worth observing that Powell’s formula provides rather unrealistic predictions, which confirms that the application of existing empirical and semi-empirical approaches that were developed for clays, such as those at the Busan site, generally results in unreliable soil parameter values for Incheon clayey silt. Several researchers have observed that when partial drainage during cone penetration is likely to occur and viscosity effects are neglected, the cone resistance clearly tends to be higher than in fully undrained conditions, therefore providing an overestimation of the relevant geotechnical parameters. These parameters are the OCR values using Powell’s formula, which are in fact higher within the silty unit where the sandy fraction is greater. The preliminary evaluation of such conditions is therefore of paramount importance for the assessment of representative mechanical parameters.

In contractive soils, the common trend is for an increase in the penetration rate at relatively higher values, and the excess pore pressure u_2 and corrected cone resistance q_t also increases. The combination of both q_t and u_2 induces values of the pore pressure ratio, B_q which are largely unaffected within the undrained penetration zone in clayey soils (Randolph, 2004). The derived piezocone parameter (B_q) is therefore used to understand the undrained penetration behavior of contractive clayey soils. In CPTU data analyses, Hight et al. (1994) suggested that the penetration is fully undrained when the B_q values are greater than 0.5 (Schnaid et al., 2004). Schnaid et al. (2004) found that the predominantly undrained testing conditions are still characteristic of normally consolidated silty soils when B_q values lie between 0.3 and 0.5, whereas soils in partially drained conditions generate B_q values between 0 and 0.3. Fig. 5.2 shows the approach that recently proposed by Schnaid et al. (2004) as a development of the pioneering work of Hight et al. (1994). This approach is based on plotting the normalized cone resistance, Q_t, vs. the pore pressure parameter, B_q in combination with the strength incremental ratio s_u/σ'_{vo}, for the CPTU data for the Incheon clayey silt (Tonni and Gottardi, 2009). A cone factor N_{kt} equal to 12 was assumed in order to convert the cone resistance to the undrained shear strength. Fig. 5.2(a), 5.2(b) and 5.2(c) show that a two-third part falls in the range where $B_q < 0.3$, corresponding to the domain in which partial drainage prevails when normally consolidated soils are tested at a standard rate of penetration (2 cm/s). It is worth noting that the calculated strength incremental ratio is, in general, significantly higher than the values estimated using laboratory and in-situ tests for the Incheon clayey silt, and this ratio shows considerable scatter, suggesting that a deviation from this pattern for the Incheon clayey silt is essentially related to the partial drainage phenomena. Moreover, in the original chart proposed by Schnaid et al. (2004), the B_q values range from 0 to 0.3 in partially drained conditions.

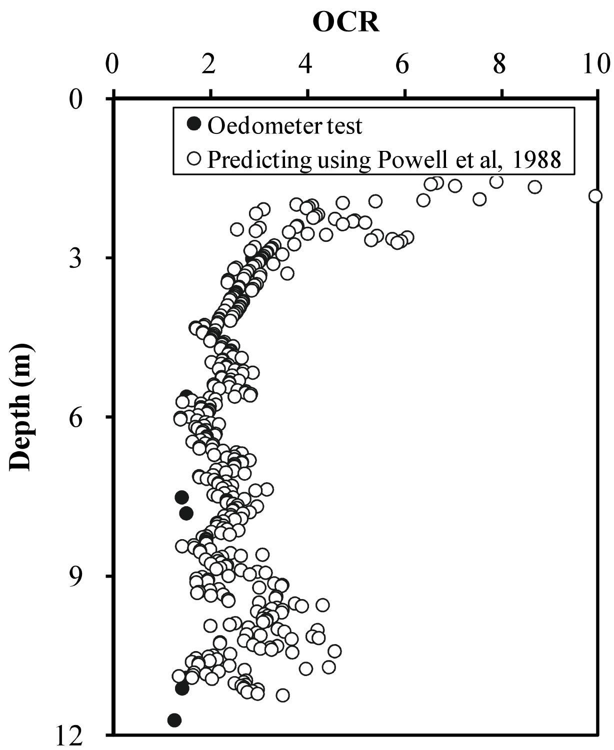
However, when a CPTU test is conducted, the grounds may be considered to be in an over-consolidated state due to delayed consolidation, such as that resulting from an aging or secondary consolidation effect. This can sometimes be likely to generate a negative pore pressure. Therefore, in this study, the lower limit of the partial drainage is eliminated in Fig. 5.2. On the other hand, as shown in Fig. 5.2(d), almost all of the depths fall in the range where $B_q > 0.5$, corresponding to a domain in which fully undrained conditions prevail for high plasticity clay from Busan at a standard penetration rate of 2 cm/s. An assessment of drainage conditions is performed by applying two existing approaches to the specific CPTUs, and this assessment can be quite suitable for the Incheon silty soil with a low plasticity and for the Busan clay with a high plasticity. The preliminary results clearly confirmed the occurrence of partial drainage within the predominantly silty unit. When it is estimated whether or not the ground is in a partially drained condition, the non-dimensional velocity $V= \upsilon D/c_v$ may be used, where υ is the penetration rate, D is the cone diameter, and c_v is the coefficient of consolidation (Schnaid et al., House et al., 2001). The coefficient of consolidation, c_v. may be considered to be the most important factor. Specimens containing sand seams are likely to be disturbed during the preparation process of oedometer tests. Sometimes, the undisturbed specimens that do not contain sand seams are used to conduct the oedometer tests. Therefore, the coefficient of consolidation of clayey silt is likely to be underestimated. Even when the coefficient of consolidation, c_v, is estimated through a dissipation test carried out during piezocone testing, the test is carried out according to a well-known procedure proposed by Teh and Houlsby (1991), which would imply the theoretical initial pore pressure distribution associated with a fully undrained response during penetration. Therefore, the evaluation of the coefficient of consolidation, c_v for clayey silt demands the utmost care when the degree of drainage conditions is estimated by using the non-dimensional velocity $V=\upsilon D/c_v$.



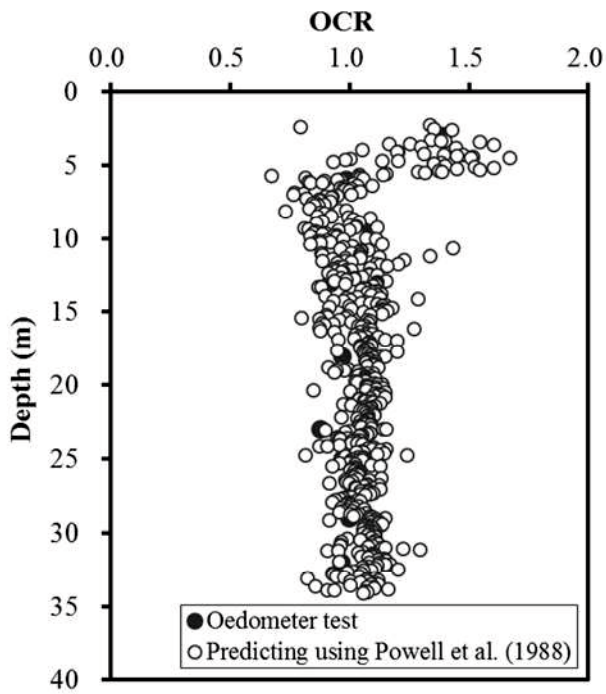
(a) Site A



(b) Site B

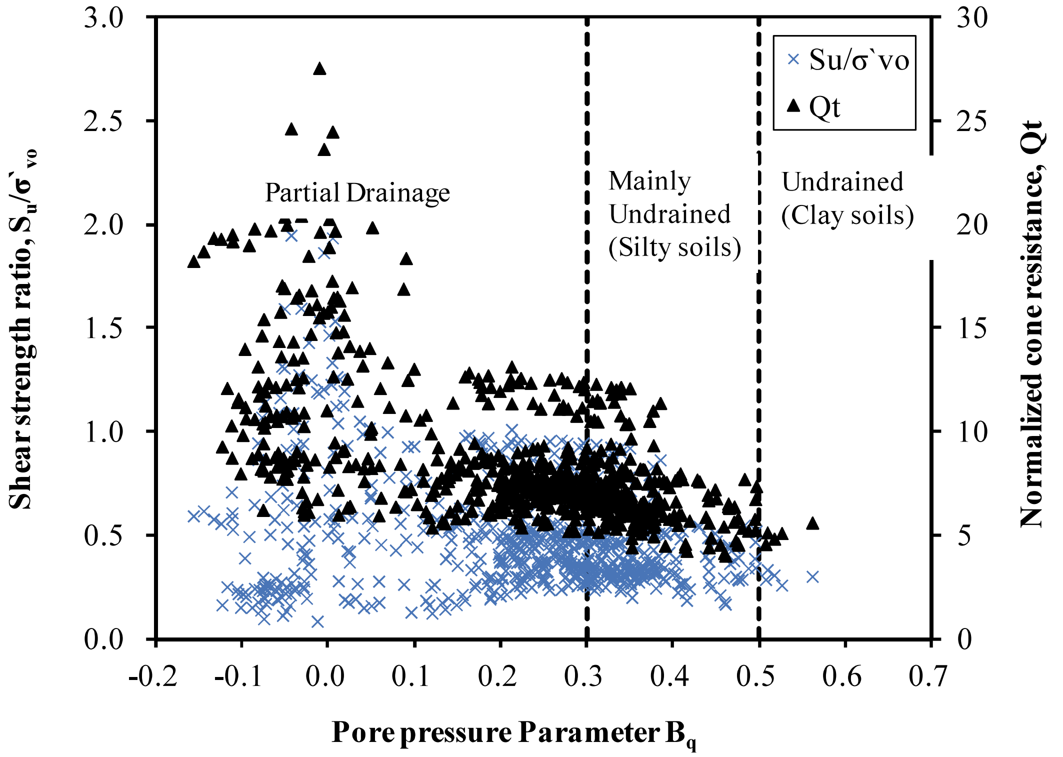


(c) Site C

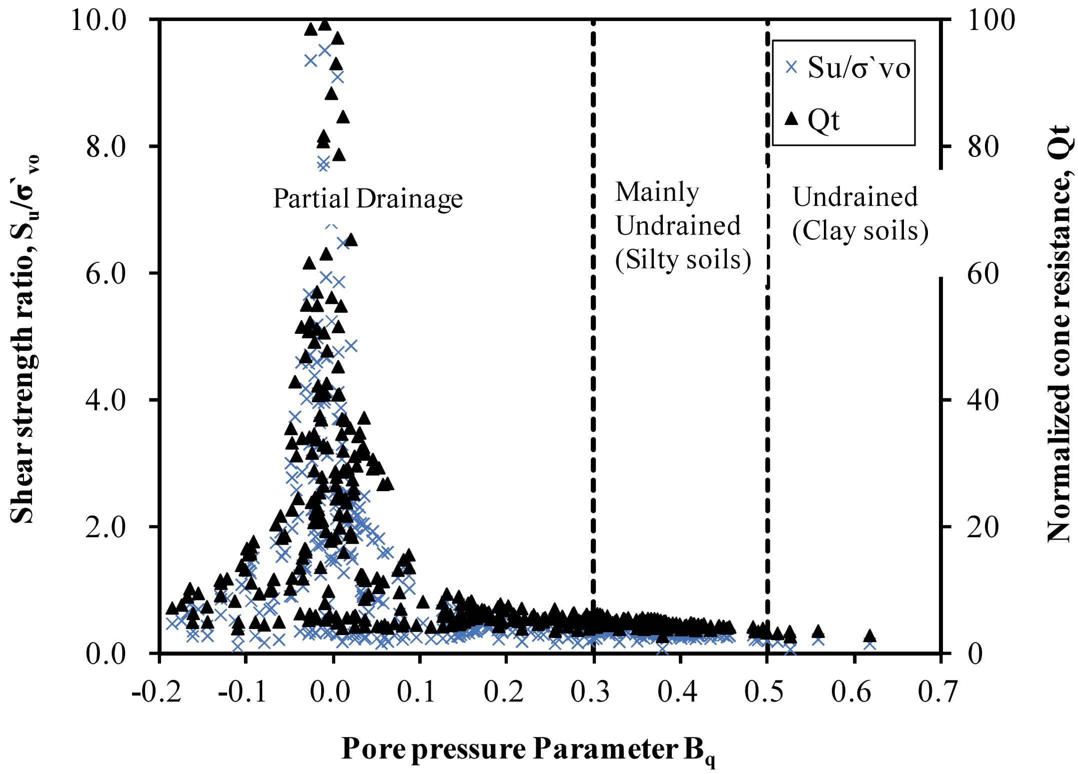


(d) High plasticity clay from Busan

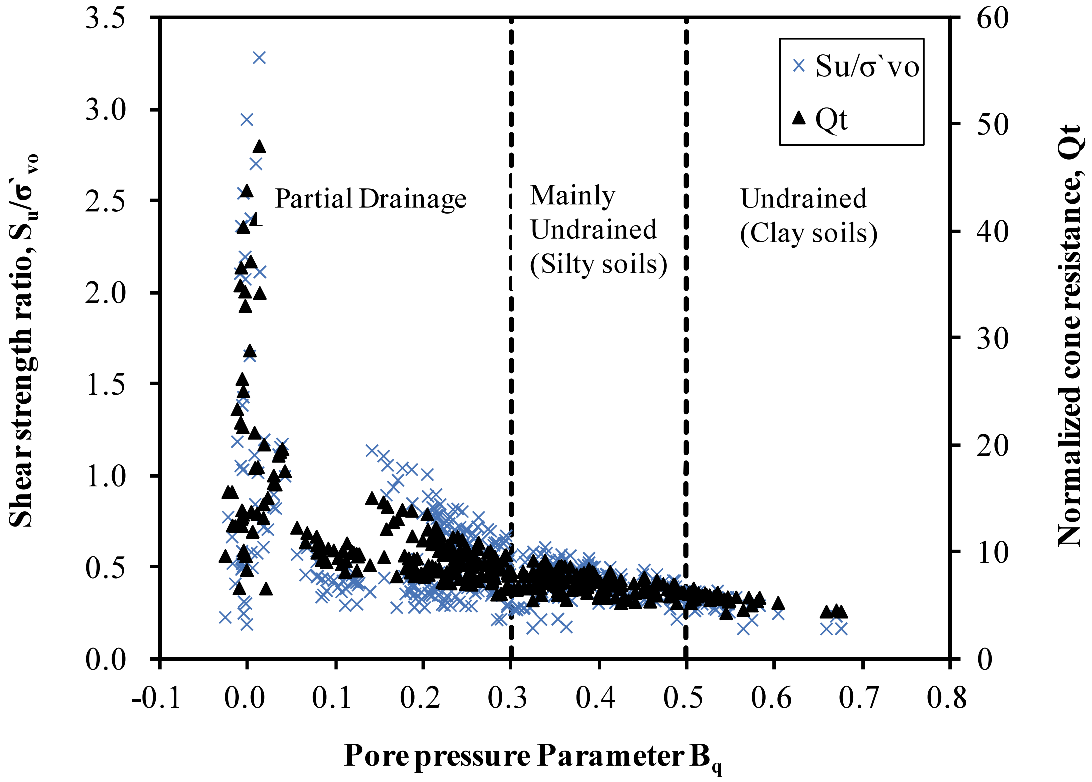
Fig. 5.1 Comparison of over-consolidatin ratio between the oedometer test and the CPTU using Powell's formula



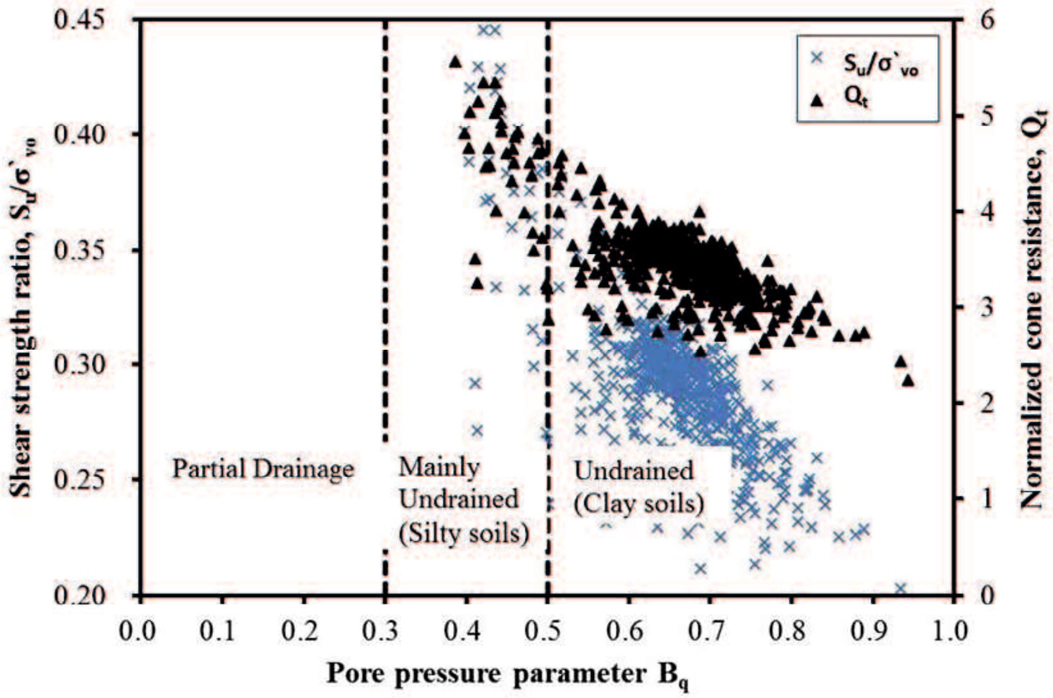
(a) Site A



(b) Site B



(c) Site C



(d) High plasticity clay from Busan

Fig. 5.2 The judgment of drainage conditions at the standard penetration rate, v=2 cm/s based on the CPTU results

5.2 ESTIMATION OF THE UNDRAINED AND PARTIALLY DRAINED SHEAR STRENGTHS

5.2.1 ESTIMATION OF THE UNDRAINED SHEAR STRENGTH BY USING THE RECOMPRESSION TEST AND CPTU

The undrained shear strength ($q_u/2$) of the Incheon subsoil obtained from the UC test was presented in Fig. 5.3. The $q_u/2$ value for clayey silt increases with a strength depth ratio of 1 kPa/m. This value is much smaller than the values from common marine cohesive subsoil. Additionally, this corresponds to a strength incremental ratio (s_u/p'_c) of 0.18. As mentioned earlier, the reason why the UC test has been continuously conducted is the existence of an automatic balance between the factors which overestimate and underestimate the undrained strength. When a soil specimen is retrieved from the ground and is exposed to the atmosphere, part of the in-situ effective stress in the specimen remains in the form of negative pore pressure. This negative pore pressure is called the residual effective stress (p'_r). Even though the specimen is tested under unconfined conditions, the residual effective stress acts as a confining pressure. However, if the value of p'_r in the specimen is reduced due to a disturbance of the specimen during sampling, handling, extruding or trimming, then the $q_u/2$ value will be reduced due to swelling. According to Tanaka et al (1996), the order of p'_r for high quality samples is around 1/5 to 1/6 of the in-situ vertical effective stress (p'_{vo}) for normally consolidated and slightly overconsolidated clays. However, p'_r values measured for clayey silts with low plasticity are much smaller than those measured for ordinary clay, for which p'_r is approximately 1/6 p'_{vo} (Tanaka et al, 2001, Baek et al, 2014). Accordingly, the loss of the residual effective stress (p'_r) is too significant to be compensated for by factors overestimating the strength. Therefore, the validity of the test for clayey soils is not applicable to clayey silts. When the undrained shear strength of soil with low plasticity index is evaluated, an effective confining pressure that corresponds to typical marine clay should be applied to the soil specimen before shearing to compensate for the lost residual effective stress. Therefore, Tsuchida and Mizumaki (1991) proposed a simplified testing method in evaluating s_{umob}, where a specimen is isotropically consolidated for two hours under a mean effective stress, before shearing in undrained condition. The s_{umob} values obtained from the CIU test are multiplied by the correction factor of 0.75 regardless of PI. This factor of 0.75 in the CIU test is derived assuming a strength anisotropy ratio (s_{ue}/s_{uc}) of 0.70 (Tsuchida and Tanaka, 1995). In considering the strain rate effect in the CIU test, based on experience regarding the strain rate effect for Japanese marine clays, a 7% reduction in strength was applied regarding one logarithm scale of the axial strain rate, compared with the CK_oU test results (Tsuchida and Tanaka, 1995). As shown in Fig. 5.3, the s_{umob} values obtained from the recompression test for Incheon clayey silt, unlike the $q_u/2$ values, increased considerably with depth. The s_{umob} values determined by the recompression test for Incheon clayey silt increased with a strength depth ratio of 2.8kPa/m. This corresponds to a strength incremental ratio (s_u/p'_c) of 0.32, which is a common value for typical Korean marine clays.

As shown in Fig. 5.4(a), at depths shallower than 8 m (σ'_{vo} =70kPa), the s_u values calculated by the CPTU results (N_{kt}=12) agree well with the s_u values obtained by CIU (recompression test) at Site A. At depths deeper than 8 m, the s_u values calculated by the CPTU results (N_{kt}=12) were greater than those obtained by CIU (recompression test). The reason behind this might be considered due to the predominant characteristics of partial or full drainage at depths deeper than 8m. At depths shallower than 6 m (σ'_{vo} =50kPa) and at depths of around 10 m (σ'_{vo} =87kPa) at site B, the s_u values calculated by the CPTU results (N_{kt}=12) were greater than those obtained by CIU (recompression test). In particular, the percentage of sand at these depths is approximately greater than 50%. Therefore, this ground is likely to be under partially or sometimes, fully drained conditions during penetration and could be classified as clayey silts. Finally, at depths shallower than around 3 m (σ'_{vo} =26kPa) and depths deeper than around 7 m

(σ'_{vo} =60kPa) at site C, the s_u values calculated by the CPTU results (N_{kt}=12) were greater than those obtained by CIU (recompression test). However, the excess pore pressures for almost the whole depth of the subsoils were higher than the hydrostatic pressure, except for depths shallower than around 3 m, where the excess pore pressure was somewhat similar to the hydrostatic pressure. As a result, if the ground is likely to be under partially drained or fully drained conditions such as the Incheon clayey silts, it is difficult to apply the estimation method of undrained shear strength using the cone factor, N_{kt}, based on the φ=0 conditions considering fully undrained conditions. Therefore, it is necessary to estimate the shear strength considering partially drained conditions. This is treated in more detail in the following section.

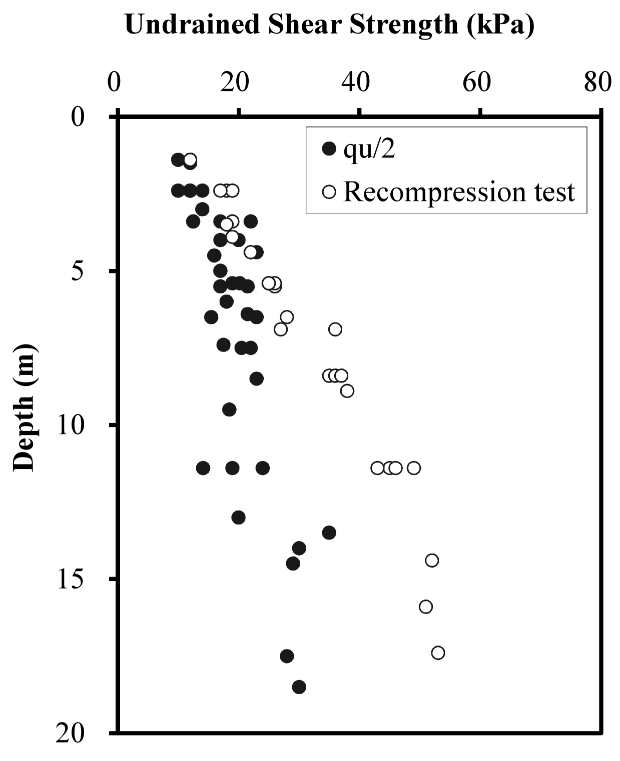
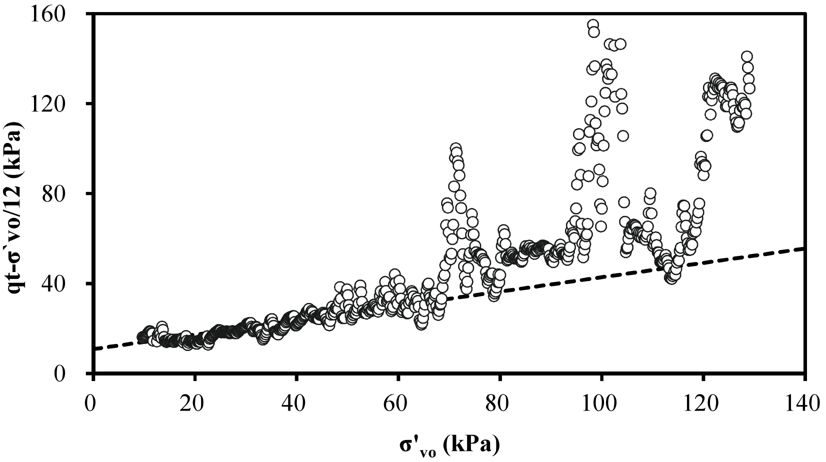
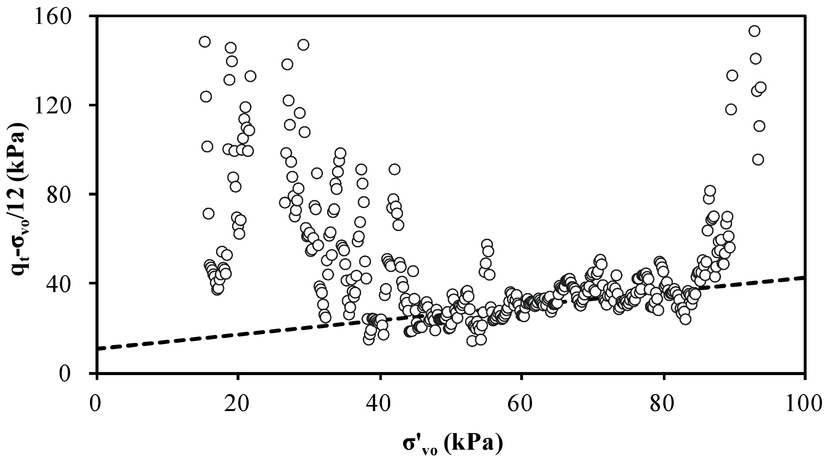


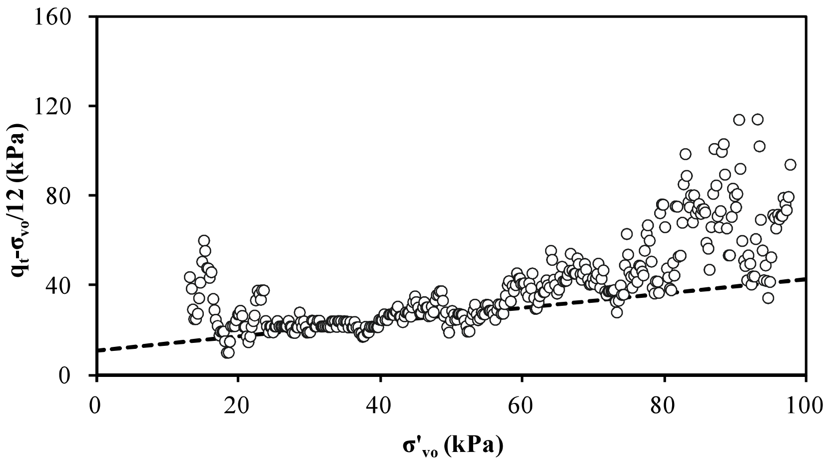
Fig. 5.3 q_u/2 and recompression strengths for Incheon clayey silts



(a) Site A



(b) Site B



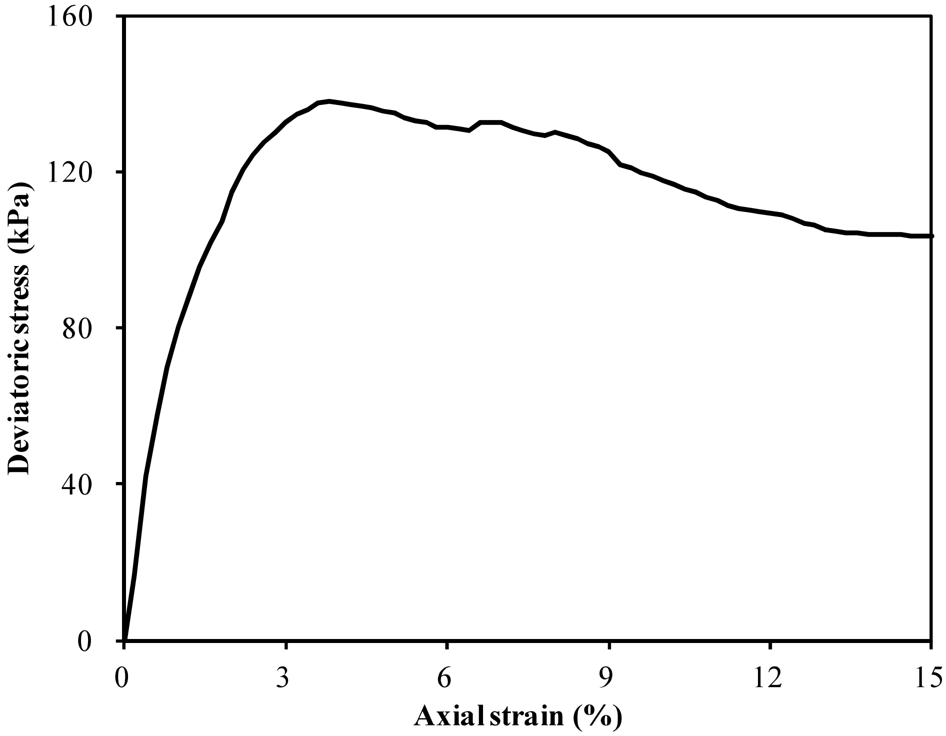
(c) Site C

Fig. 5.4 Relation between the in-situ undrained shear strength (s_{uf}) obtained by CPTU and the effective overburden pressure (σ'_{vo})

5.2.2 ESTIMATION OF SHEAR STRENGTH CONSIDERING PARTIALLY DRAINED CONDITIONS BASED ON THE RECOMPRESSION TEST AND CPTU

As shown in the CPTU results, deposited clayey silts show heavy randomicity regarding their microscopic properties, indicating the existence of many small sandy soil layers. In cases like these clayey silts, an analysis based on total stress conditions might be doubtful. Usually, when embankments are constructed on the cohesive subsoils, since a stability analysis based on the total stress conditions is carried out, an increase of the shear strength in that ground might not be considered at the beginning of construction, considering the safety side which could sometimes be thrown into an uneconomic design. However, when excavating heavily overconsolidated clayey ground, the effective stress in such ground conditions could be gradually reduced, due to the dissipation of negative excess pore-water pressure, from the viewpoint of the long term stability. Therefore, short-term stability analysis could result in an un-safe situation. Generally, since the permeability of clayey silts might be higher than that of ordinary clayey soils, it should be noted that the total stress conditions in clayey silts could be maintained in a short space of time. Therefore, the construction conditions of the ground should be checked, that is, whether the effective stress could gradually increase or not over time. In addition, because information about the physical and mechanical properties in laboratory tests is likely to be obtained from a single point, CPTU can be a powerful tool to avoid wrong judgments about soil distribution characteristics in cases like clayey silts. There is one more thing that should be noted, which is that all of the laboratory tests were carried out under undrained conditions, in this study. Generally, positive excess pore-water pressure might be generated under undrained conditions in slightly overconsolidated clayey soils, including normally consolidated clayey soils and loose sands, while negative pore-water pressure might be generated under undrained conditions in highly overconsolidated clayey soils and dense sands. Therefore, its undrained shear strengths could be higher than those of the fully drained conditions, due to the increase in confining pressures during the shearing phase. In cases like this, in every possible event, the undrained shear strengths obtained from laboratory and in-situ tests should not be applied to the stability analysis, from the viewpoint of long term stability. Fig. 5.5 shows a typical test result from recompression triaxial tests for the Incheon subsoil. All specimen were consolidated under an isotropic consolidation pressure of $2/3p'_{vo}$, and then compressed at an axial strain rate of 0.1%/min. For the Incheon subsoil, the shapes of the stress-strain curve as well as the stress paths are nearly the same at all depths. The stress path goes up directly to the failure envelope, and then the deviator stress decreases and follows the failure envelope.

Fig. 5.6 shows the internal friction angles obtained from the recompression triaxial tests with depth. The values obtained were around 35° indicating higher values than were expected. However, Tanaka et al. (2001) found the internal friction angles of around 42° from recompression triaxial tests for both Ishinomaki intermediate soil and Drammen lean clay. It might be inferred that the internal friction angles of these subsoils show constant trends regardless of the percentage of sandy soils, based on Tanaka et al.'s research and also the present research.



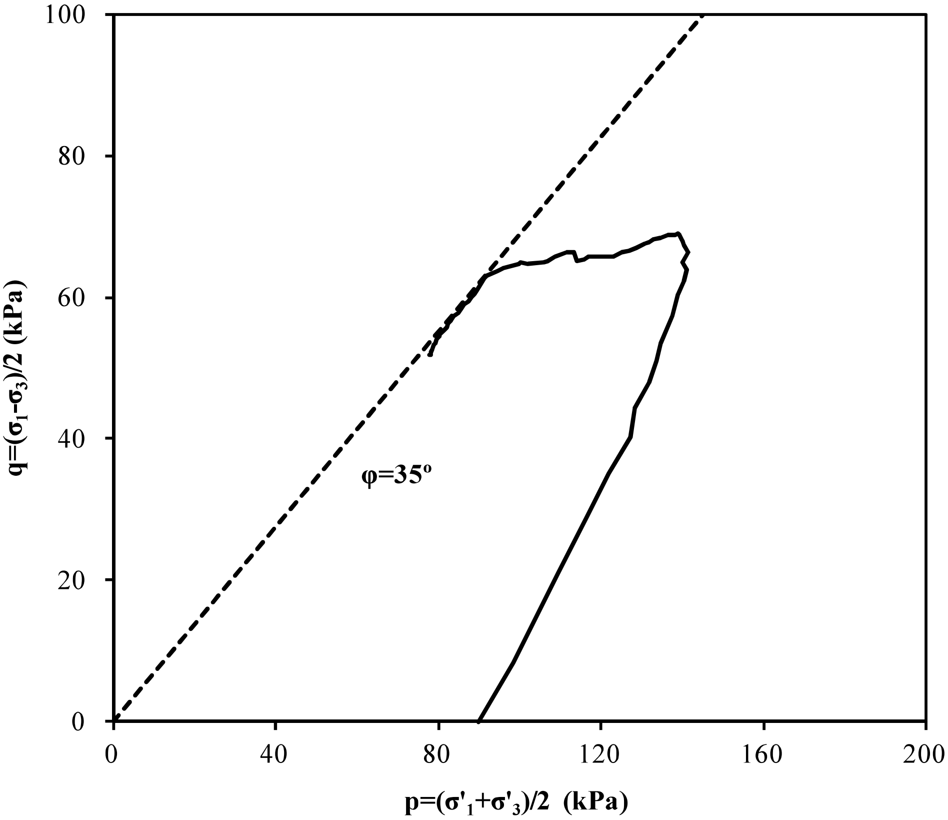


Fig. 5.5 The stress-strain curve and stress path for Incheon clayey silt at depth=14.4m

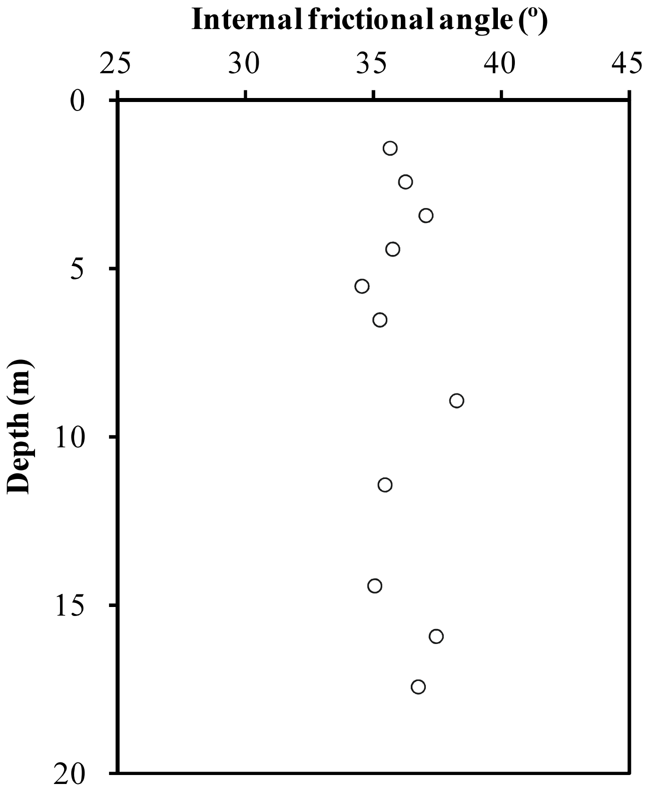


Fig. 5.6 Internal friction angles obtained from recompression triaxial tests with depth for Incheon clayey silts

Meanwhile, as shown in Fig. 5.7, the Incheon site had an over-consolidated state at depths shallower than around 4 m, and the average OCR value was around 3 at these depths. At depths beyond 4 m, the ground could be considered to have a normally consolidated state because the OCRs at these depths were close to 1.0. As shown in Figs. 5.3 and 5.4, the s_{umob} values increased with a strength to depth ratio of 2.8 kPa/m, and an intercept of around 11 kPa was obtained by the recompression test and CPTUs of the Incheon intermediate subsoil. Bjerrum (1973) pointed out that in-situ clay is subjected to various complicated aging effects and exhibits increased strength. The Incheon site showed no historical evidence of experiencing overburden pressures. Therefore, it can be considered that the clays were classified as normally consolidated aged clays (NCA clay).

Hanzawa and Adachi (1983) discussed the following features of the aging effect. Normally consolidated clays have been subjected to either chemical bonding or secondary compression, or their combined action. Where chemical bonding and secondary compression take place together, there exist three possibilities: 1) Case A, where chemical bonding first takes place with secondary compression occurring thereafter; 2) Case B, where chemical bonding and secondary compression simultaneously develop, and 3) Case C, where secondary compression first develops with chemical bonding taking place thereafter. Because of the bond structure developed by chemical bonding, the progress of secondary compression will be prevented in Case A. However, when secondary compression progresses with time, it can cause damage of the soil structure already developed by chemical bonding. The mechanism of Case B will be the same as that in Case A with the final soil structure being dependent on which soil structure is stronger. Because secondary compression continues from sedimentation up to the present time, it would seem that Case C is highly improbable. The undrained shear strength of the clayey soils subjected to these actions has been expressed by the following equations.

$$s_{uf} = s_{un} + k_1 \text{ for chemical bonding} \tag{5.1}$$

$$s_{uf} = s_{un} \times k_2 \text{ for secondary compression} \tag{5.2}$$

where, s_{uf} is the in-situ undrained shear strength, s_{un} is the undrained shear strength in normally consolidated state, and k_1 and k_2 are constants.

Figs. 5.3 and 5.4 might be considered as typical examples of what Hanzawa and Adachi have suggested, except that they do not show a trend of secondary compression at deeper depths. Therefore, the additional strength of the Incheon subsoil might be mainly due to chemical bonding. Further study is necessary to determine these mechanisms at the Incheon site.

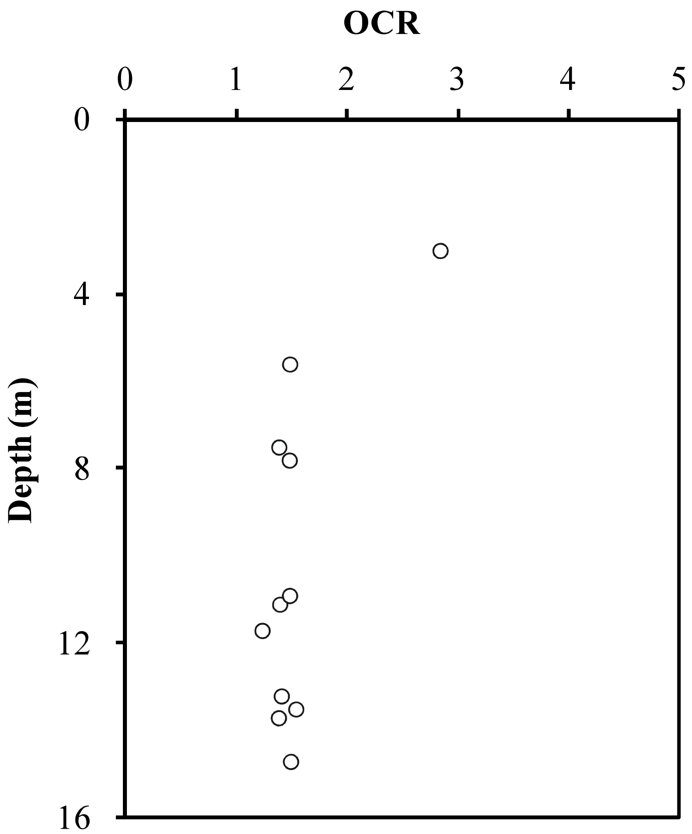


Fig. 5.7 Changes of over-consolidation ratio with depth for Incheon clayey silt (Site A)

Meanwhile, the drained shear strength of normally consolidated cohesive soils is defined by the friction angle φ′, as follows:

$$s=\sigma' \tan \varphi' \tag{5.3}$$

The drained shear strength of overconsolidated clay should ordinarily be greater than the drained strength of the same constituents in a normally consolidated state, mainly due to the chemical bonding and aging effects at the Incheon site. Therefore, even if the overconsolidated subsoils are under fully drained conditions, cohesions (c') are generated under overconsolidated state and reaches its peak resistance at the instant of a global instability. Therefore, the drained intact shear strength of saturated overconsolidated clay is commonly expressed in terms of the intercept c′ and the angle of φ′ of a failure envelope in the Mohr diagram, as

$$s=c' + \sigma' \tan \varphi' \tag{5.4}$$

As shown in Fig. 5.7, since the overconsolidated state might be maintained at depths shallower than around 4 m, the intercept c' of these depths could be generated, due to the chemical bonding and aging effects. The connection point is defined as the effective normal stress of 35kPa at which the failure envelope of the overconsolidated clay joins the envelope for the normally consolidated condition. Therefore, at depths deeper than 4m (σ'_{vo} =35kPa), the fully drained shear strength of the Incheon clayey silt could be expressed as s= σ' tan35°, that is, $c' \fallingdotseq 0$, considered as the normally consolidated state, while at depths shallower than 4 m (σ'_{vo} =35kPa), the failure envelope of the overconsolidated state reaches the envelope for the normally consolidated state maintaining the intercept $c' \fallingdotseq 11$kPa.

Based on this concept, the fully drained shear strength of the overconsolidated Incheon subsoil could be expressed as s=11kPa + σ'tan23°. Of course, the internal friction angle of 23° in the overconsolidated state was estimated based on the point (σ'_{vo} =35kPa) at which both failure envelopes are connected. Therefore, the angle φ' in Eq. (5.4) is not comparable to the friction angle in Eq. (5.3).

Meanwhile, based on these concepts mentioned above, as shown in Fig. 5.8, the shear strengths were estimated considering the undrained, drained and partially drained conditions for Incheon intermediate subsoil, respectively. First of all, the drainage conditions were estimated after inspecting thoroughly whether the s_u values calculated by the CPTU results (N_{kt}=12) were higher than the fully drained shear strength or not. In case of site A, both the undrained and partially drained conditions were jumbled together during penetration at depths shallower than 8 m, showing close to overall undrained conditions.

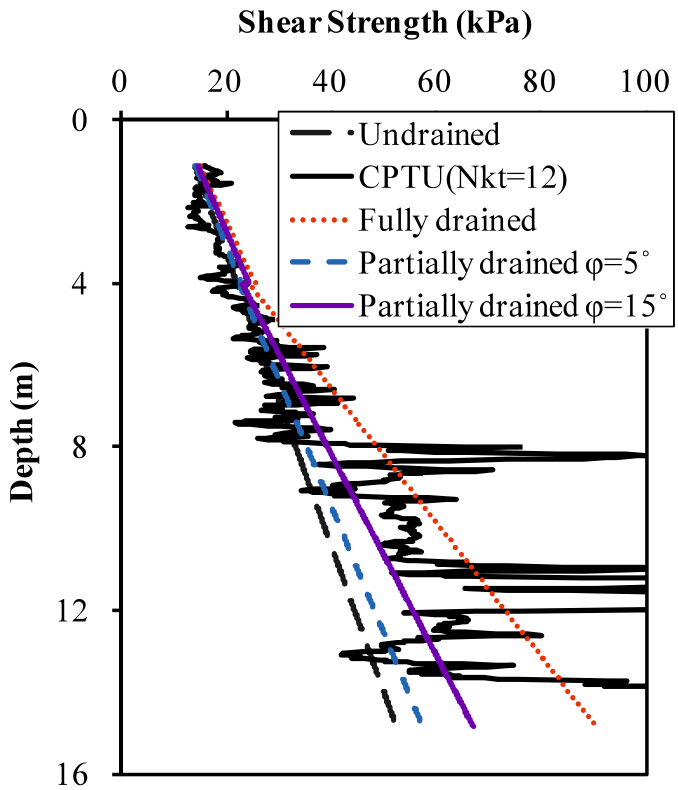
However, the subsoils at some depths might be considered to be under partially drained conditions. Therefore, it showed that the internal friction angles at these depths might be distributed between φ'=5° and φ'=15° considering the partially drained conditions. In addition, at depths deeper than 8 m, the s_u values calculated by the CPTU results (N_{kt}=12) were greater than those obtained by CIU (recompression test), though by and large, these shear strengths tended to be higher than partially drained shear strengths estimated by using φ'=15°. Furthermore, at depths of around 8 m and 12 m, the CPTU based shear strengths showed higher values than the fully drained shear strength. Therefore, it could be considered that partially or fully drained conditions prevail at site A.

In case of site B, at depths shallower than 6 m (σ'_{vo} =50kPa) and at depths of around 10 m (σ'_{vo} =87 kPa), the s_u values calculated by the CPTU results (N_{kt}=12) were greater than the fully drained shear strength. In particular, the percentage of sand at these depths is approximately greater than 50%. Based on these things, these grounds might be likely to be under fully drained conditions rather than undrained or partially drained conditions during penetration. Therefore, it is not suitable to apply the CPTU based undrained shear strength at these depths considering the φ=0 conditions. Additionally, it showed that the internal friction angles at the other depths might be distributed between φ'=5° and φ'=15° considering the partially drained conditions. Therefore, the ground conditions might be also considered to be under partially drained conditions.

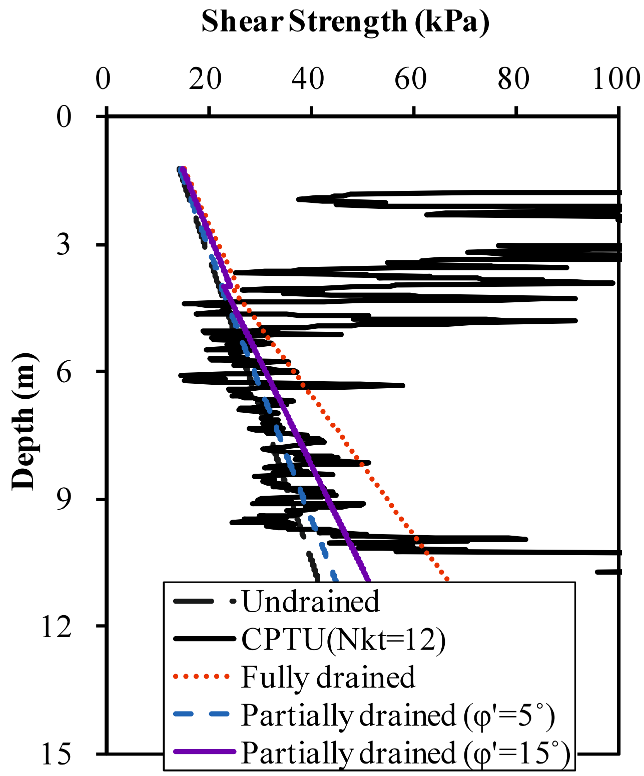
Finally, in case of site C, at depths shallower than around 3 m (σ'_{vo} =26 kPa) and at depths deeper than around 7 m (σ'_{vo} =60 kPa), the s_u values calculated by the CPTU results (N_{kt}=12) were greater than those obtained by CIU (recompression test). Although the excess pore pressures for almost the whole depth of the grounds are higher than the hydrostatic pressure except for depths shallower than around 3 m, where the excess pore pressure is somewhat similar to the hydrostatic pressure, partially drained conditions could be checked based on the pore pressure parameter, $B_q < 0.3$, at depths shallower than 3 m and at depths deeper than around 7 m. However, the results showed that the CPTU based shear strengths were higher than the fully drained shear strength at some depths, which could be considered to be under fully drained conditions. In addition, they showed that the internal friction angles at almost all depths were distributed between $\varphi'=5^{\circ}$ and $\varphi'=15^{\circ}$ considering the partially drained conditions, except for depths shallower than 3 m and for depths deeper than around 7 m.

In summary, if the undrained shear strengths obtained by laboratory and in-situ tests are higher than the fully drained shear strengths, in any event, then these strengths should not be applied to the stability analysis for the intermediate subsoil, from the viewpoint regarding the long term stability. Therefore, at the very beginning, the fully undrained and drained shear strengths should be evaluated from laboratory tests, in that order, and the CPTU based shear strength should be checked as to whether this strength is higher than the fully drained shear strength or not, in cases like Incheon clayey silt with low plasticity, which is likely to be under partially or fully drained conditions during penetration.

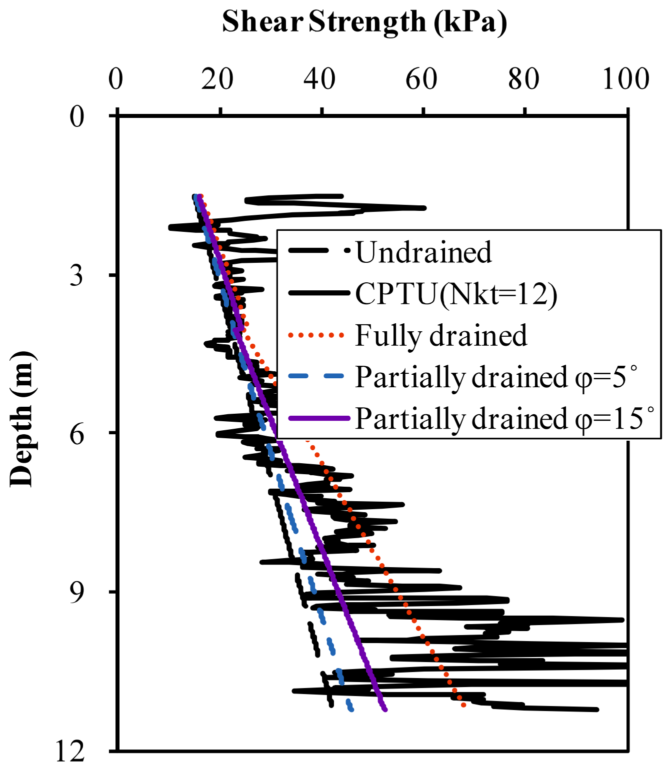
The internal friction angles of Incheon clayey silt in this study are not intended to be obtained from laboratory tests under partially drained conditions. The internal friction angles under partially drained conditions were estimated considering the distribution characteristics of the CPTU based shear strength between the undrained shear strength line and the fully drained shear strength line, which is approximately ranged from $\varphi'=5^{\circ}$ to $\varphi'=15^{\circ}$. This range is treated in the following section based on the characteristics of the compulsory embankment method with low plasticity soils.



(a) Site A



(b) Site B



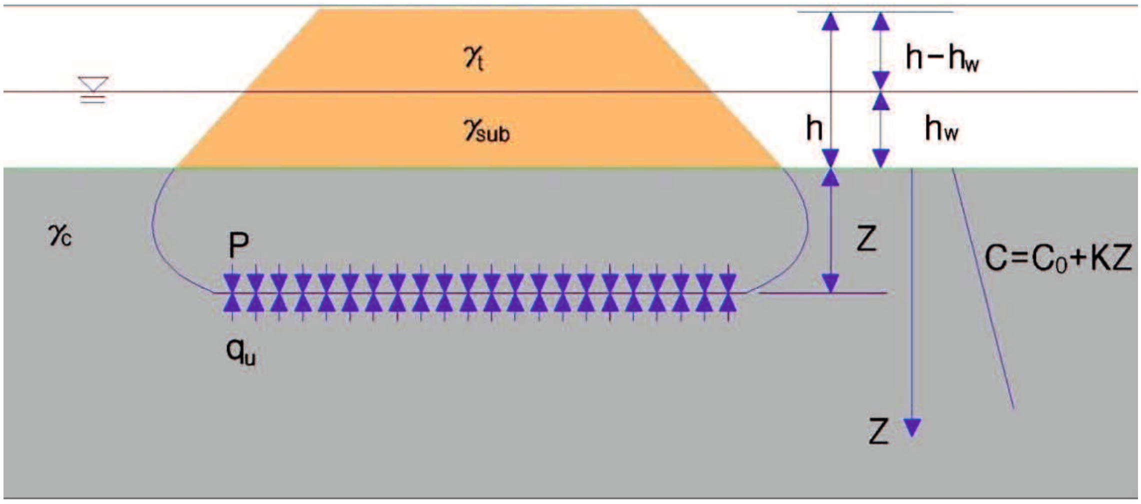
(c) Site C

Fig 5.8 Shear strengths of Incheon clayey silt under undrained, drained and partially drained conditions

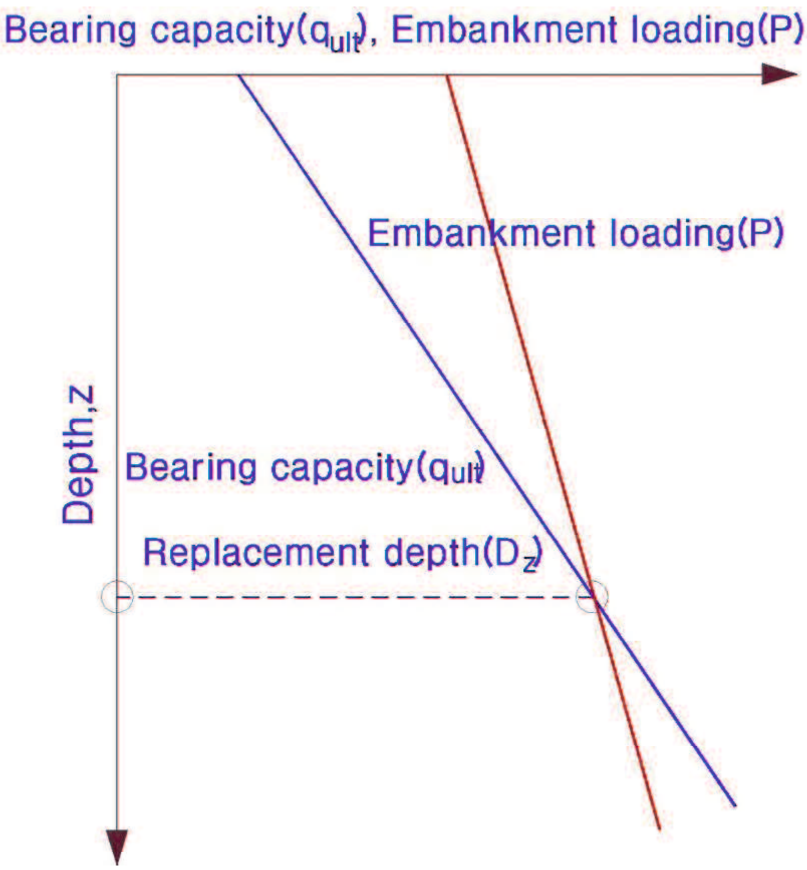
5.2.3 ESTIMATION OF PARTIALLY DRAINED SHEAR STRENGTH BASED ON THE CHARACTERISTICS OF THE COMPULSORY EMBANKMENT METHOD

Recently, a performance based design method has been gradually introduced. In particular, the estimation of soil parameters considering dispersion was included in the Technical standards for port and harbor facilities in Japan. The higher the dispersion is, the larger a correction factor is applied. Therefore, the mobilized undrained shear strength of clayey silts with low plasticity might be much smaller than the in-situ strength, because of the underestimation as well as the increased dispersion of the estimated undrained strength by sampling quality. For such reasons, when determining the soil parameters, it is necessary for an engineer to precisely analyze the experimental results and to perform in-situ and laboratory tests for minimizing the dispersion of in-situ and laboratory test results.

Meanwhile, when a seawall is constructed on marine clayey subsoil, the compulsory replacement method has been widely used, because of its economic feasibility due to a trend for speedy and simple construction. As shown in Fig. 5.9, cohesive subsoils are pushed toward the surrounding layers by loading embankment materials. As shear failure in the ground takes place simultaneously with the construction of the embankment, marine cohesive subsoil could be replaced by high quality materials. In other words, soft clayey subsoil could be replaced by embankment materials, when applying embankment load greater than its ultimate bearing capacity on soft clayey subsoil. Additionally, the replacement depth is estimated by using the bearing capacity equation suggested, for example, by Terzaghi and Meyerhof's methods, based on where the bearing capacity of soft ground equals the embankment load.



(a) Conceptual diagram of Compulsory replacement method



(b) Determination of replacement depth

Fig. 5.9 Compulsory replacement method

When applying the compulsory replacement method on soft clayey subsoils with high plasticity index such as the Busan and Gwangyang sites, it was reported that the estimated replacement depths in the design phase could be approximately equal to the checked depths at completed construction sites. Therefore, the estimation of the replacement depth based on the φ=0 conditions could be valid for high plasticity clayey grounds. However, conversely, as shown in Fig. 5.10, in case of subsoil with low plasticity, it was reported that the checked depths of the replacement were in almost all cases considerably smaller than the estimated depths. The inferred reason is that loading embankment materials on that ground could lead to shear failure under partially drained conditions due to the high permeability characteristics of clayey silts with low plasticity (Kim et al, 2010). Generally, sandy ground is considered as being under drained conditions, while clayey ground is considered as being under undrained conditions in the field of geotechnical design. In comparison, the difficult problem of intermediate subsoils in the design phase is about an engineering judgment of the drainage conditions (undrained or drained conditions).

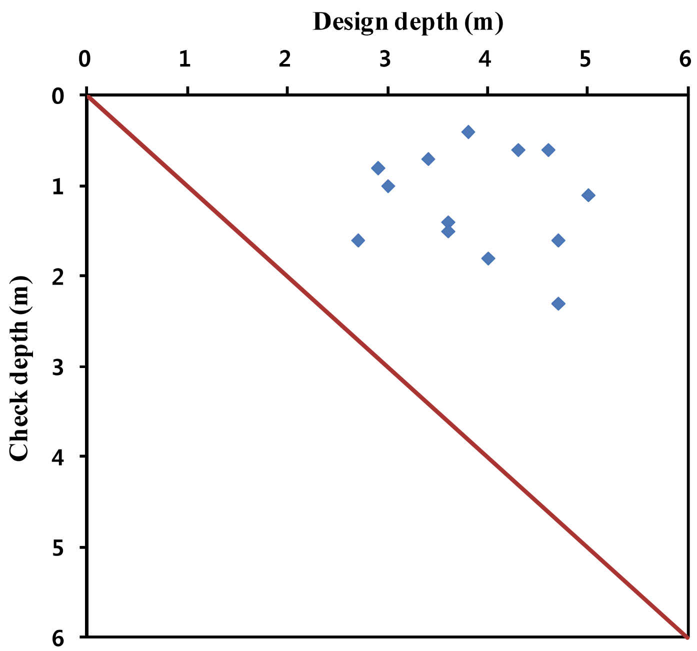


Fig. 5.10 Relationship of the estimated replacement depths in design phase versus the checked depths at completed construction sites with low plasticity

As mentioned earlier, to analyze the relationship between the bearing capacity and the settlement of cohesive subsoils when applying changed permeability to the subsoil, Asaoka et al. (1989) analyzed whether partially drained conditions were present or not, based on conducting FEM analysis focused on the changed permeability of cohesive subsoil; that is, its settlement dramatically increased with increasing embankment loadings on the cohesive subsoil at a range of permeability between 10^{-7} cm/s and 10^{-4} cm/s. They suggested that the partial drainage range is between 10^{-7} cm/s and 10^{-4} cm/s based on FEM analysis.

Therefore, for permeability less than 10^{-7} cm/s, the ground is interpreted to behave in an undrained condition. For permeability exceeding about 10^{-4} cm/s, the ground is interpreted to behave in nearly drained conditions. Soils with permeability between these two limits are behaving in a partially drained condition. Therefore, the degree of drainage is associated with the content of fine-grained particles with interlocking soil, and hence with soil permeability and compressibility properties. Based on a permeability of 9.5×10^{-6} cm/s obtained by an oedometer test at the Incheon subsoils, it can be considered that the clayey silts with low plasticity at this site are likely to be under partially drained conditions.

Meanwhile, according to Ohmaki (1989), as shown in Fig. 5.11, when carrying out UU-tests with low plasticity soils, internal friction angles (φ_u) are frequently obtained from the results. Due to disturbance, stress relief and the saturation reduction of a specimen during sampling, when applying a series of confining pressures, internal consolidation of the sample occurs during UU-tests. Therefore, as shown in Fig. 5.12, the effective stresses of sample soils are increased according to each of the applied stresses.

As shown in Fig. 5.12, under UU-test conditions, as mentioned earlier, the apparent internal friction angles of clayey silts were obtained. The more the plasticity index decreased, the greater the internal friction angles are. To estimate the replacement depth of the Incheon subsoils, back analysis was carried out to evaluate the internal friction angle based on where the designed depths are equal to the checked depths using the equation of bearing capacity, paying attention to the apparent internal friction angle obtained from the UU-test results. As shown in Fig. 5.13, internal friction angles obtained from back analysis and UU-tests tended to increase with decreasing plasticity index. It seems that these values are ranged from $\varphi'=5^{\circ}$ to $\varphi'=15^{\circ}$, similar to ranges of CPTU based internal friction angles. Based on these results, it might be possible to estimate the replacement depth of the clayey silt with low plasticity having the characteristics of partially drained conditions by applying the apparent internal friction angle.

Meanwhile, even though the samples of the intermediate soil were originally under saturated conditions, the degree of saturation decreased due to stress relief and mechanical disturbance during boring, sampling and transferring the samples from the field to the laboratory. Therefore, the internal friction angle could be obtained under UU-test conditions at Incheon site. These apparent internal friction angles could be only obtained under laboratory testing conditions. However, because the cohesive subsoils with low plasticity were originally below the water table, it might be difficult to obtain the internal friction angle of soils under in-situ conditions when applying the compulsory replacement method based on a theoretical viewpoint. Let us take a look at another dimension. Because the intermediate subsoils located at the Incheon site have good drainage due to a lot of sandy layers between subsoils, it might be difficult to estimate the replacement depth of this ground using the $\varphi=0$ conditions. That is, this ground is likely to be under partially drained conditions. Therefore, in case of intermediate soil, the apparent internal friction angle obtained from the UU-test could be used for stability analysis considering the partially drained characteristics.

However, it is important to note that the UU-tests in this study were not carried out under partially saturated conditions. Unsaturated trend of intermediate soils due to sampling process and internal consolidation of samples under confining pressure was considered as partially drained conditions. Add to that, practically, it is difficult to evaluate the mobilized shear strength only through laboratory test of soil samples under partially drained conditions. Therefore, the analysis of the distribution characteristics of the CPTU based shear strength between the undrained shear strength line and the fully drained shear strength line could be viable alternatives to the estimation of the partially drained shear strength. From now on, further in-depth study should be carried out regarding evaluating the partially drained shear strength of the intermediate soil based on the rate effect and drainage characteristics of CPTU data.

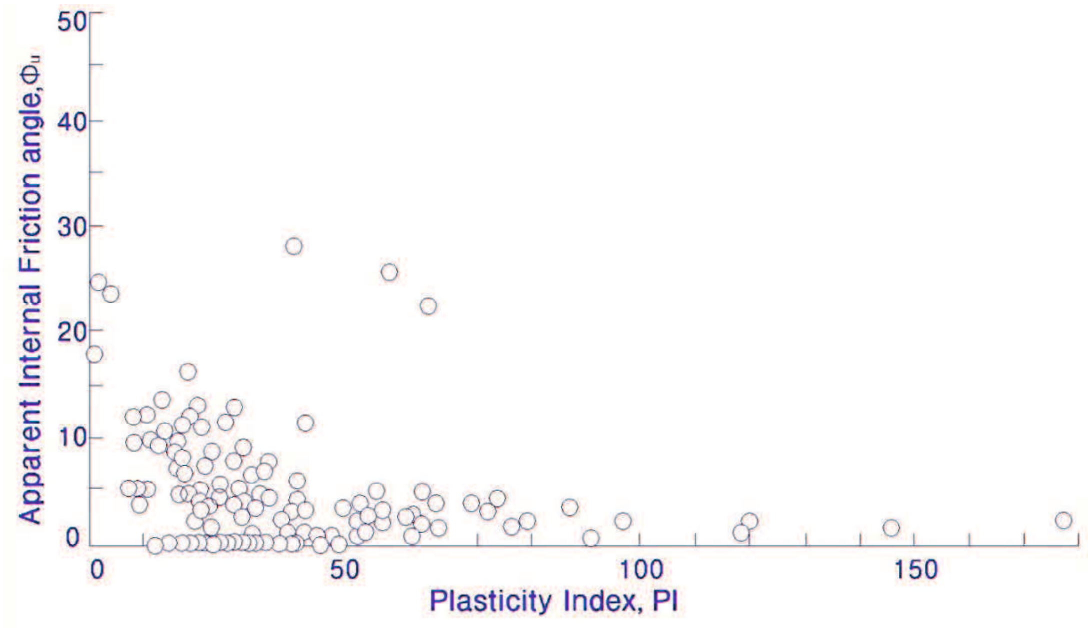


Fig. 5.11 Relation of the plasticity index versus the apparent internal friction angle obtained from UU-tests

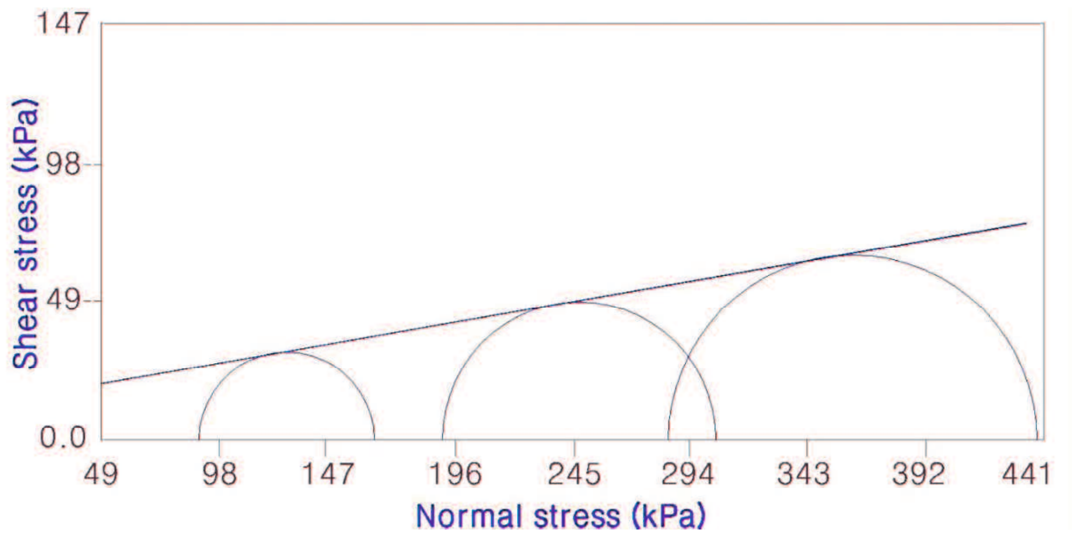


Fig. 5.12 Mohr-Coulomb envelope of the samples with low plasticity for the UU-test

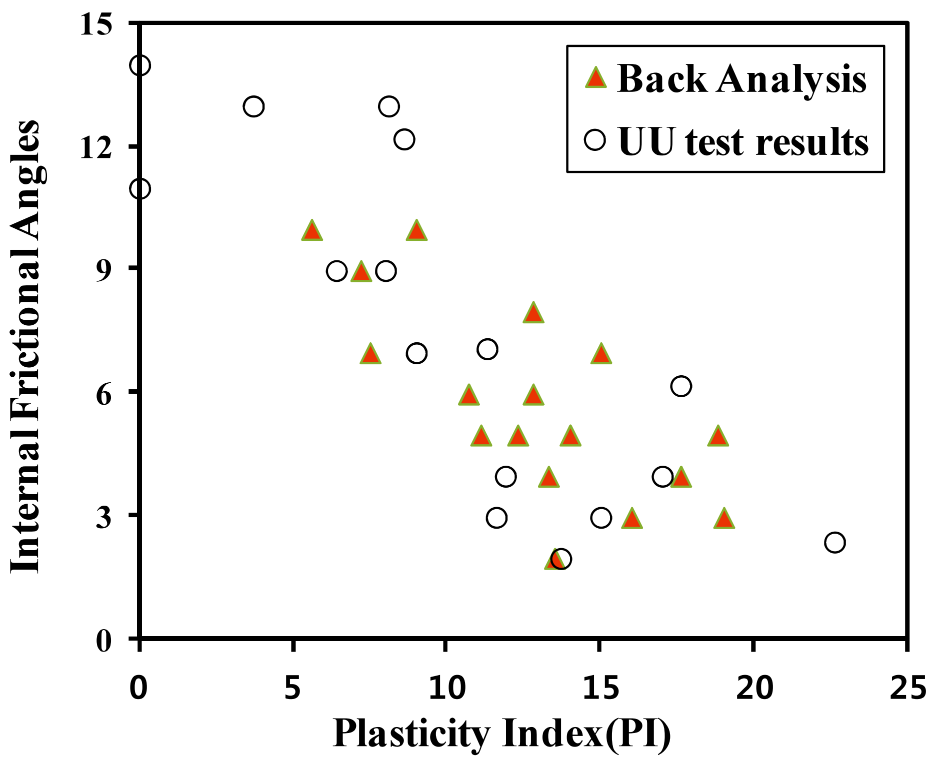


Fig. 5.13 Relationship of the UU test results and back analysis values versus I_P

5.3 CONCLUSIONS

1) In interpreting the CPTU data, the existing correlation equations are very useful for estimating the strength and OCR before directly evaluating the strength and consolidation conditions by the laboratory and field tests. OCRs obtained by the oedometer tests were compared with the estimates obtained using the well-known relationship proposed by Powell and Quarterman (1988), given by: OCR=κ ((q_t-σ_{vo})/σ'_{vo}). Generally, the estimated OCRs were higher than those obtained by the oeodometer tests. These trends were noticeable for the layers which contained a lot of silty or sandy soils. It is worth observing that Powell's formula provides rather unrealistic predictions, therefore confirming that the application of existing empirical and semi-empirical approaches developed for clays generally results in unreliable soil parameter values for Incheon clayey soils with low plasticity. When partial drainage during cone penetration is likely to occur and neglecting viscosity effects, the cone resistance clearly tends to be higher than in fully undrained conditions and provides an overestimation of relevant geotechnical parameter. The preliminary evaluation of such conditions is therefore of paramount importance for the assessment of representative mechanical parameters.
2) The assessment of partial drainage conditions was performed through the application of existing approaches to the CPTU data. When estimating whether the Incheon clayey silt with low plasticity is under partially drained conditions or not, the approach recently proposed by Schnaid et al., can be quite useful; it is based on plotting the normalized cone resistance, Q_t, vs. the pore pressure parameter, B_q in combination with the strength incremental ratio s_u/σ'_{vo}, to the CPTU data. A cone factor N_{kt} equal to 12 was assumed to convert the cone resistance to the undrained shear strength. It is evident

that a two-thirds part falls in the range where $B_q < 0.3$, corresponding to the domain in which the partial drainage prevails when testing normally consolidated soils at a standard rate of penetration (2 cm/s). It is worth noting that the calculated strength incremental ratio is in general significantly higher than values estimated based on laboratory and in-situ tests for Incheon clayey silt with low plasticity; the result shows considerable scatter, therefore suggesting that deviation from this pattern in case of Incheon clayey silt is essentially related to partial drainage phenomena.

3) The definition of intermediate soil based only on physical properties, such as the grain size distribution and soil plasticity, is not adequate for Incheon soil. In this study, the $q_u/2$ value for Incheon soil was much smaller than that for a layer of common marine clay. The strength incremental ratio (s_u/p'_c) of this soil was underestimated relative to that of a high plasticity soil. Partially drained conditions for this soil can be verified by applying two existing approaches to specific CPTUs. Therefore, the classification of intermediate soil should be judged overall on the underestimation of the $q_u/2$ strength of the soil and on the consolidation parameters of the soil, including the partially drained characteristics based on CPTU and the physical properties, such as the grain size distribution and soil plasticity.

4) If the undrained shear strengths obtained by laboratory and in-situ tests are higher than fully drained shear strengths, these strengths should not be applied to the stability analysis for the intermediate soil, from the viewpoint regarding the long term stability. Therefore, at the very beginning, the fully undrained and drained shear strengths should be evaluated from laboratory tests, in that order. The CPTU based shear strength should be checked as to whether this strength is higher than the fully drained shear strength or not, in cases like the Incheon clayey silt with low plasticity, which is likely to be under partially or fully drained conditions during penetration.

5) The internal friction angles of the Incheon clayey silt with low plasticity in this study are not intended to be obtained from laboratory tests under partially drained conditions. The internal friction angles under the partially drained conditions were estimated considering the distribution characteristics of the CPTU based shear strength between the undrained shear strength line and the fully drained shear strength line, which is ranged approximately from $\varphi'=5^{\circ}$ to $\varphi'=15^{\circ}$.
6) To estimate the replacement depth of the Incheon site with low plasticity, back analysis was carried out to evaluate the internal friction angle based on where the design depths are equal to the checked depths using the bearing capacity equation, paying attention to the apparent internal friction angle obtained from the UU-test results. The internal friction angles obtained from the back analysis and UU-test results tend to increase as the plasticity index decreases. It seems that these values range from $\varphi'=5^{\circ}$ to $\varphi'=15^{\circ}$, similar to the ranges of the CPTU based internal friction angles. Based on these results, it might be possible to estimate the replacement depth of the clayey silt with low plasticity as having the characteristics of partially drained conditions by applying the apparent internal friction angle.
7) It is difficult to evaluate the mobilized shear strength only through laboratory testing, considering the partially drained conditions. Therefore, the analysis of the distribution characteristics of the CPTU based shear strength between the undrained shear strength line and the fully drained shear strength line could be viable alternatives to the estimation of the partially drained shear strength. From now on, further in-depth study should be carried out to evaluate the partially drained shear strength of intermediate soil based on the rate effect and drainage characteristics of CPTU tests.

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CHAPTER 6
COMPARISONS OF THE STABILITY ANALYSIS REGARDING FULLY UNDRAINED AND PARTIALLY DRAINED CONCEPTS FOR LOW PLASTICITY SOIL

6.1 INTRODUCTION

Until now, firstly, several points of soil classification using Casagrande's plasticity chart were discussed focusing on clayey silt with low plasticity and revised Polidori's plasticity chart were suggested through verification of applicability using Korean four different marine clayey soils. In order to apply design concepts regarding clayey silt suggested in this research, first of all, subsoil should be classified as silty soil (ML or MH) based on plasticity chart. And then, underestimated trend of Unconfined Compressive strength (UC strength) and incremental strength ratio (s_u/p'_c) should be checked in comparison with advanced laboratory tests (recompression test, etc) and in-situ tests (CPTU, FVT, etc).

In order to check economic feasibility of fully undrained and partially drained concepts proposed in this study, as shown in Fig. 6.1, the compulsory replacement method and Sand Compaction pile (SCP) method were applied to improvement of cohesive subsoils beneath embankment at Incheon site. As mentioned earlier, the Incheon site had an overconsolidated state at depths shallower than around 4m. The average OCR value was around 3 at these depths. At depths beyond 4m, the ground could be considered to have a normally consolidated state because the OCRs at these depths were close to 1.0. The height and width of analytical geometry are 7.5m, 7.5m, respectively, water table equals to ground surface level. When applying compulsory replacement method on Incheon marine subsoil, the replacement depth was estimated by using Meyerhof's bearing capacity equation based on where the bearing capacity of subsoil equals the embankment load. In order to carry out design of soft ground improvement for clayey silt with low plasticity from the view point of fully undrained concept,

$q_u/2$ strength based analysis should be compared with that of recompression strength. The s_{umob} values increased with a strength to depth ratio of 1.0 kPa/m, 2.8 kPa/m obtained by the UC test and recompression test of clayey silt, respectively. Intercepts of 8 kPa, 11 kPa at ground surface levels were obtained by the UC test and recompression test of the Incheon subsoil, respectively.

According to the design code for the construction of port and harbor facilities in Japan (2007), Japanese harbor structures should be designed by the approach based on performance based design in response to worldwide demands. Therefore, an appropriate value of the partial factor, γ_a for the analysis method should be incorporated in each design parameter according to the characteristics of the ground and characteristics of the facilities. Stability analysis was carried out using factor of safety, FS based on traditional concept to enhance content understanding, when all partial factors are 1.0. In general, Factor of safety can be set at 1.30 or higher for permanent situations, but in cases where the reliability of the constants used in verification can be considered high based on actual data for the same ground, and in cases where monitoring work is carried out by observing the displacement and stress of the ground during construction, values from of larger than 1.10 and less than 1.30 can be used (Japanese Port Association (2007). In Korea, factor of safety, FS ≥ 1.1-1.2 have been applied to perform stability analysis under the short-term condition based on the importance of structure, ground inhomogeneity, reliability of shear strength data and the construction condition. In this case, stability analysis of embankment on marine soil was carried out based on the minimum factor of safety, FS ≥ 1.2.

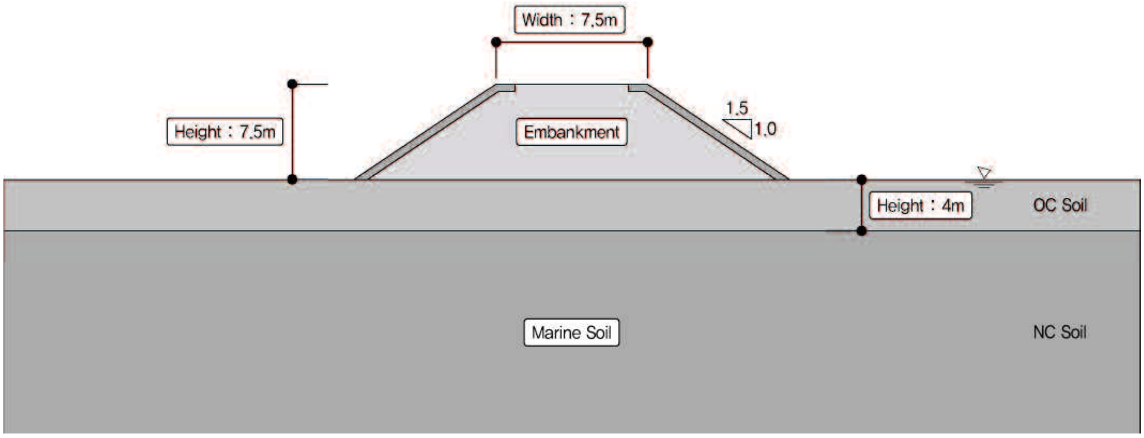


Fig 6.1 Typical cross section of embankment on marine subsoil at Incheon site

6.2 COMPARISONS OF THE STABILITY ANALYSIS RESULTS BY APPLYING COMPULSORY REPLACEMENT METHOD TO LOW PLASTICITY SOIL

6.2.1 STABILITY ANALYSIS BY USING FULLY UNDRAINED CONCEPT

When a seawall is constructed on marine clayey subsoil, the compulsory replacement method has been widely used, because of its economic feasibility due to a trend for speedy and simple construction. Soft subsoil is pushed toward the surrounding layers by loading embankment materials. As shear failure in the ground takes place simultaneously with the construction of the embankment, soft clay could be replaced by high quality materials, when applying embankment load greater than its ultimate bearing capacity on soft clayey subsoil. The replacement depth was estimated by using Meyerhof's bearing capacity equation based on $\varphi=0$ conditions, that is, bearing capacity factor, $N_c = 5.14$ is applied. The depth dependent equation of $q_u/2$ strength, that is, $s_u = 8+1.0\times$depth (kPa) was applied to estimate replacement depth. As shown in Fig. 6.2, the replacement depth was estimated at 6.9m by using $q_u/2$ strength. The minimum factor of safety, FS = 1.058 based on Bishop's simplified method was evaluated by the stability analysis of a slope under the same conditions (Fig. 6.3). If the increase of shear strength obtained by consolidation effect taken place in the construction process is appropriately considered to evaluate the stability of subsoil beneath the embankment, FS of more than or equal to 1.2 could be obtained from the view point of short-term stability. SLOPE/W 2007 software was used to analyze the slope stability, which is developed by GEO-SLOPE International Ltd.

Meanwhile, the depth dependent equation of recompression test was evaluated at su = 11 + 2.8 $\times$ depth (kPa). The replacement depth was estimated at 3.6m by using recompression strength and this value was almost half of $q_u/2$ based value (Fig. 6.4). The minimum factor of safety, FS = 1.314 based on Bishop's simplified method was evaluated by the stability analysis of a slope under the same conditions (Fig. 6.5). As mentioned earlier, in case of silty soil with low plasticity, it was reported that the checked depths obtained at completed construction sites were in almost all cases considerably smaller than the estimated depths in the design phase. It is inferred that the underestimation trend of $q_u/2$ strength in clayey silt with low plasticity is certainly one major reason why this phenomenon takes place.

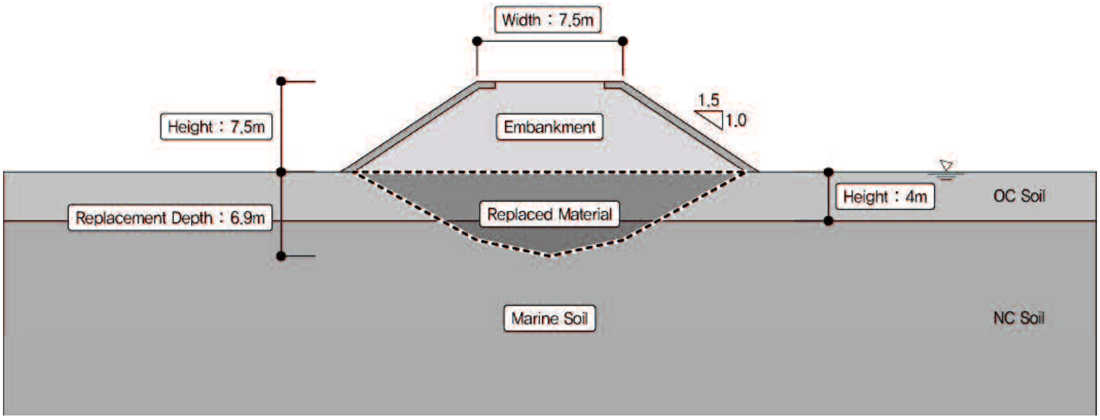


Fig 6.2 Replacement depth estimated by using $q_u/2$ strength based on φ=0 conditions

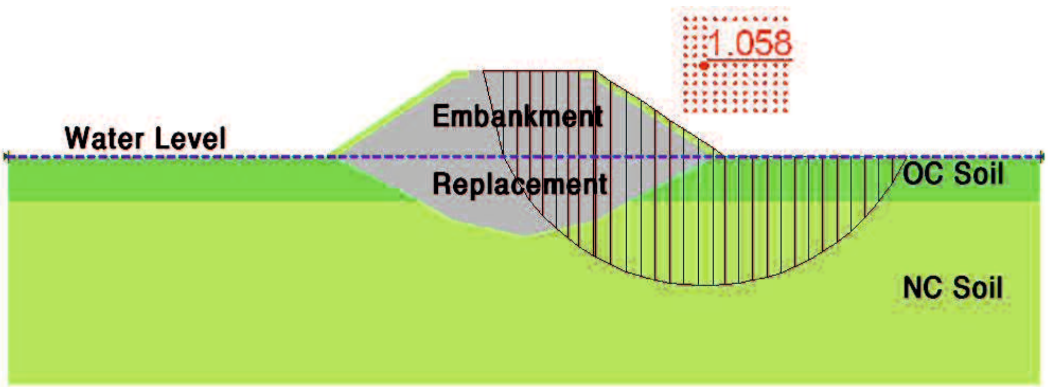


Fig 6.3 Minimum factor of safety when applying compulsory replacement method (UC test)

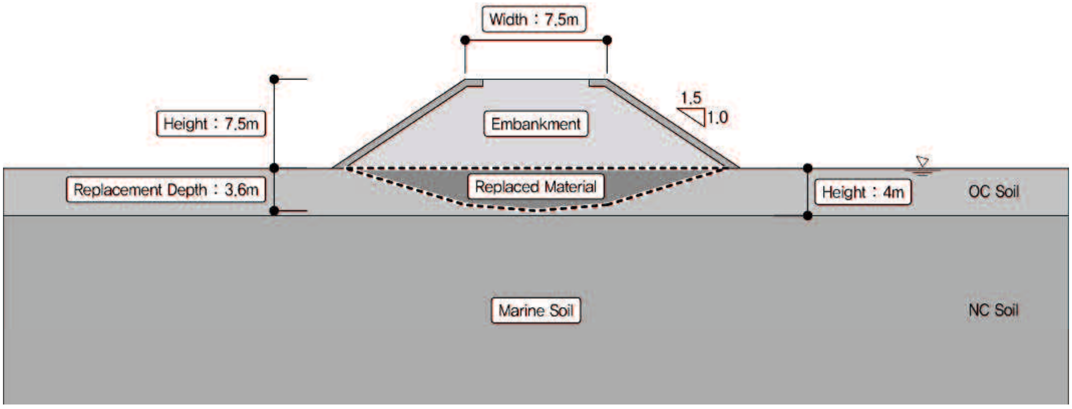


Fig. 6.4 Replacement depth estimated by recompression strength based on φ=0 conditions

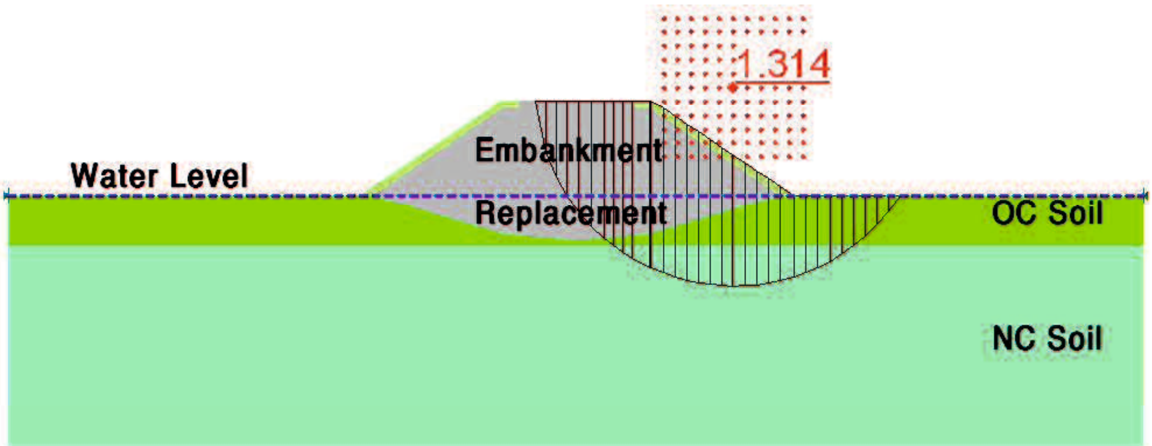


Fig 6.5 Minimum factor of safety when applying compulsory replacement method (Recompression test)

6.2.2 STABILITY ANALYSIS BY USING PARTIALLY DRAINED CONCEPT

In order to apply partially drained concept in designing soft ground improvement, first of all, the assessment of partial drainage conditions should be performed through the application of existing approaches to the CPTU data (comparison oedometer's OCR with that estimated by Powell and Quarterman' approach and Schnaid et al.'s approach, that is, plotting the normalized cone resistance, Q_t, vs. the pore pressure parameter, B_q in combination with the strength incremental ratio s_u/σ'_{vo}, to the CPTU data). Secondly, fully undrained shear strength line should be estimated through carrying out recompression test or in-situ tests (CPTU, FVT). At this point, $q_u/2$ strengths are only used to check underestimation trend of strength for intermediate subsoil in comparison with recompression strength. Thirdly, drained internal friction angle (°) with increasing depth should be estimated by analyzing stress path curve of recompress test to establish fully drained shear strength line, if possible, it is desirable that conventionally Consolidated Undrained triaxial test with checking pore water pressure (CU-TEST) is carried out to evaluate drained internal friction angle. Fourthly, the undrained shear strength obtained by recompression test and in-situ tests (CPTU, FVT) should be compared with fully drained shear strength. If the undrained shear strengths are higher than fully drained shear strengths, these strengths should not be applied to the stability analysis of the intermediate subsoil, from the viewpoint regarding the long term stability. Therefore, at the very beginning, the fully undrained and drained shear strengths should be evaluated from laboratory tests, in that order. Fifthly, the internal friction angles under the partially drained conditions can be estimated considering the distribution trend of the CPTU based shear strength between the undrained shear strength line and the fully drained shear strength line.

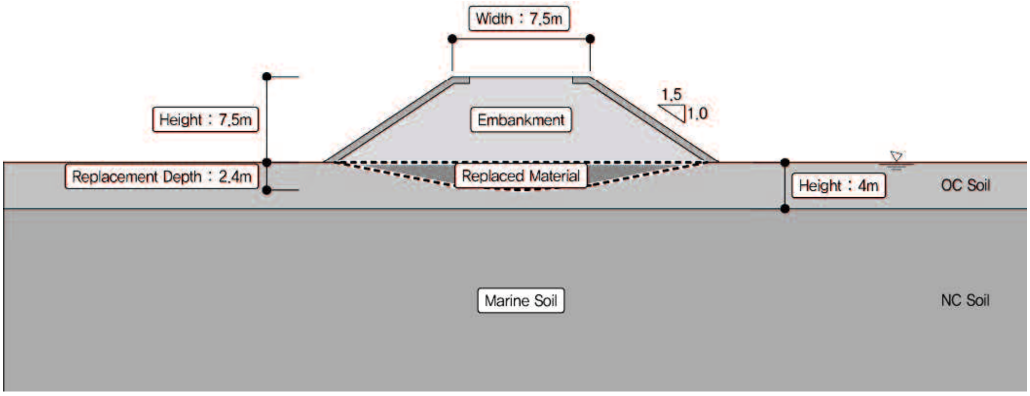
As shown in Fig. 6.6, the replacement depths were estimated by applying a series of internal friction angles to consider its sensitivity under partially drained conditions. The drained shear strength of overconsolidated clay should ordinarily be greater than the drained strength of the same constituents in a normally consolidated state, mainly due to the chemical bonding and aging effects at the Incheon site. Since the overconsolidated state might be maintained at depths shallower than around 4 m, the intercept c' of these depths could be generated, due to the chemical bonding and aging effects. The connection point is defined as the effective normal stress of 35kPa at which the failure envelope of the overconsolidated clay joins the envelope for the normally consolidated condition. Therefore, at depths deeper than 4m (σ'_{vo} =35kPa), the fully drained shear strength of the Incheon clayey silt could be expressed as $s= \sigma' \tan 35^{\circ}$, that is, $c' \fallingdotseq 0$, considered as the normally consolidated state, while at depths shallower than 4 m (σ'_{vo} =35kPa), the failure envelope of the overconsolidated state reaches the envelope for the normally consolidated state maintaining the intercept $c' \fallingdotseq 11$kPa. Based on this concept, the fully drained shear strength of the overconsolidated Incheon soil could be expressed as $s=11\text{kPa}+ \sigma' \tan 23^{\circ}$. Of course, the internal friction angle of 23° in the overconsolidated state was estimated based on the point (σ'_{vo} =35kPa) at which both failure envelopes are connected. As shown in Fig. 6.6(a), the replacement depth was estimated at 2.4m when applying $\varphi'=3°$ to evaluate bearing capacity of intermediate subsoil under partially drained conditions. This value was 30% smaller than the case applied recompression strength. Since the ground at depths shallower than 4m is under overconsolidated state, the failure envelope of the overconsolidated state reaches the envelope for the normally consolidated state maintaining the intercept $c' \fallingdotseq$ 11kPa. Since $\varphi'=23°$ represents fully drained internal friction angle at 100% degree of consolidation under overconsolidated state, $\varphi'=3°$ applied to estimate replacement depth denotes partially drained internal friction angle at approximately 13% degree of consolidation. On the other hand, since $\varphi'=35°$ represents fully drained internal friction angle at 100% degree of consolidation under normally consolidated state, $\varphi'=3°$ denotes partially drained internal friction angle at approximately 9% degree of consolidation. Since coefficient of consolidations,

c_v at overconsolidated layers generally represent higher than that of normally consolidated layers at the same loading conditions, the internal friction angles at overconsolidated layers can be larger than those of normally consolidated layers. Although, in this example, the same values at each layer were applied to stability analysis as a matter of accommodation, different internal friction angles at each layer can be applied through the analysis of dissipation trend of excess porewater pressure in CPTU data.

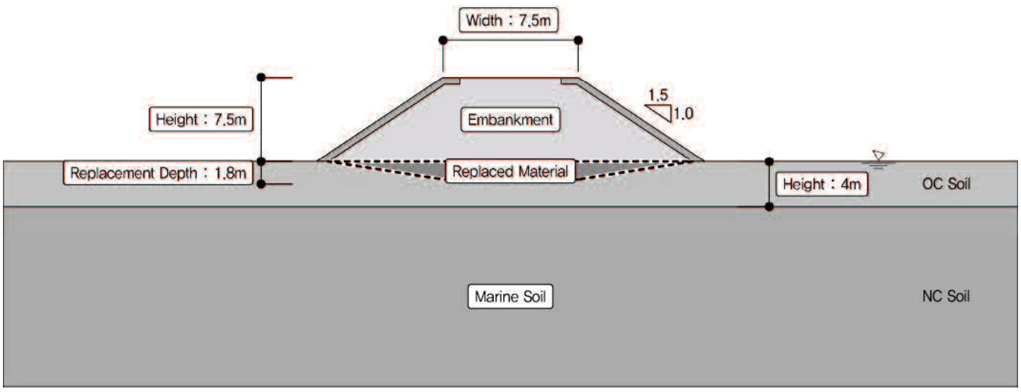
As shown in Fig. 6.6(b), the replacement depth was estimated at 1.8m when applying $\varphi'=5°$ to evaluate bearing capacity of intermediate subsoil under partially drained conditions. This value was 50% smaller than that of recompression strength. $\varphi'=5°$ at overconsolidated and normally consolidated layers denote partially drained internal friction angle at approximately 22% and 14% degree of consolidation, respectively.

As shown in Fig. 6.6(c), the replacement depth was estimated at 0.8m when applying $\varphi'=7°$ to evaluate bearing capacity of intermediate subsoil under partially drained conditions. This value was 80% smaller than that of recompression strength. $\varphi'=7°$ at overconsolidated and normally consolidated layers denote partially drained internal friction angle at approximately 30% and 20% degree of consolidation, respectively.

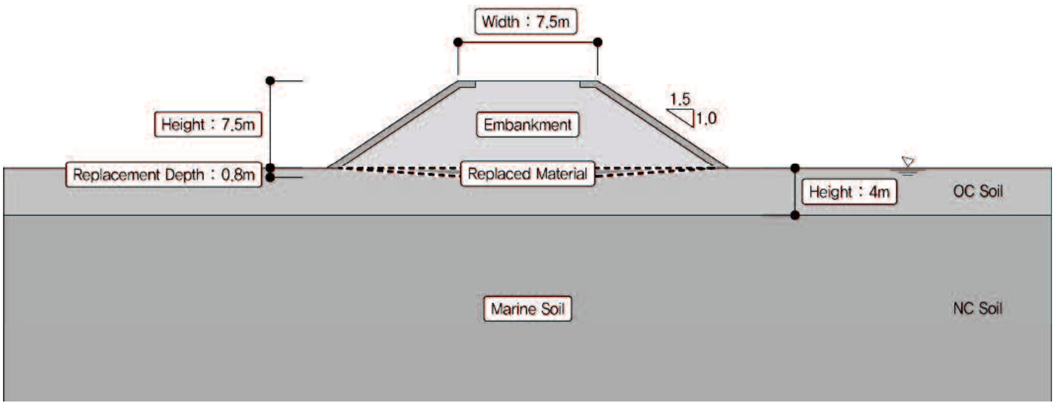
Fig. 6.7 showed the minimum factor of safety, FS > 1.2 based on Bishop's simplified method at each case, separately, from the view point of short-term stability. As reviewed earlier, the estimated replacement depths were drastically changed with a slight change of internal friction angles. Accordingly, selecting appropriate strengths in stability analysis is the most important thing. In chapter 5, according to back analysis from field measurement data at completed construction sites applied compulsory replacement method and the analysis of CPTU based strengths, the internal friction angles under partially drained conditions ranged from $\varphi'=5°$ to $\varphi'=15°$. At the present stage, it is very difficult to select appropriate φ' value to design improvement of cohesive subsoils under partially drained conditions beneath gravity type structures such as breakwaters, quaywalls, and revetments. Therefore, when applying partially drained concept in designing soft ground improvement, it is recommended that the lowest φ′=3-5 ° be applied to stability analysis from the view point of safety side.



(a) The estimated replacement depth under φ'=3°

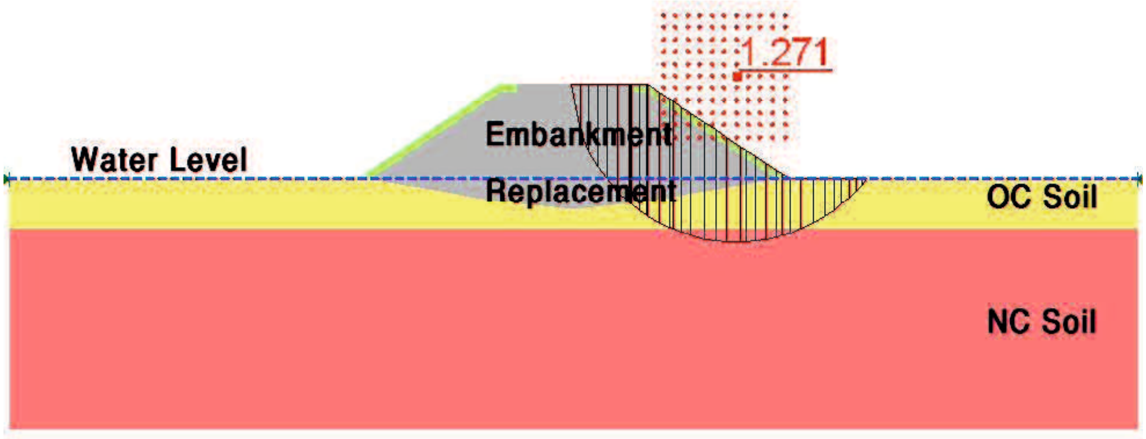


(b) The estimated replacement depth under φ'=5°

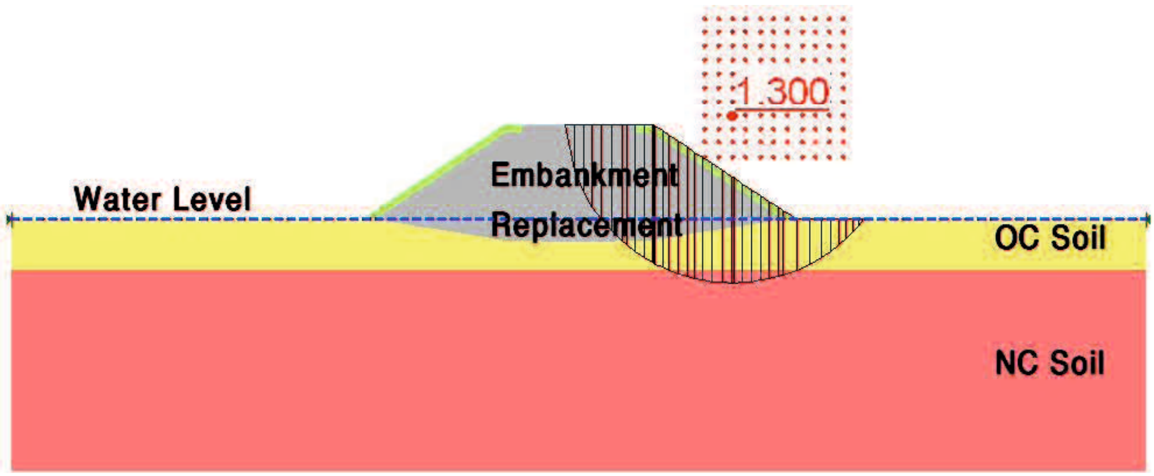


(c) The estimated replacement depth under φ'=7°

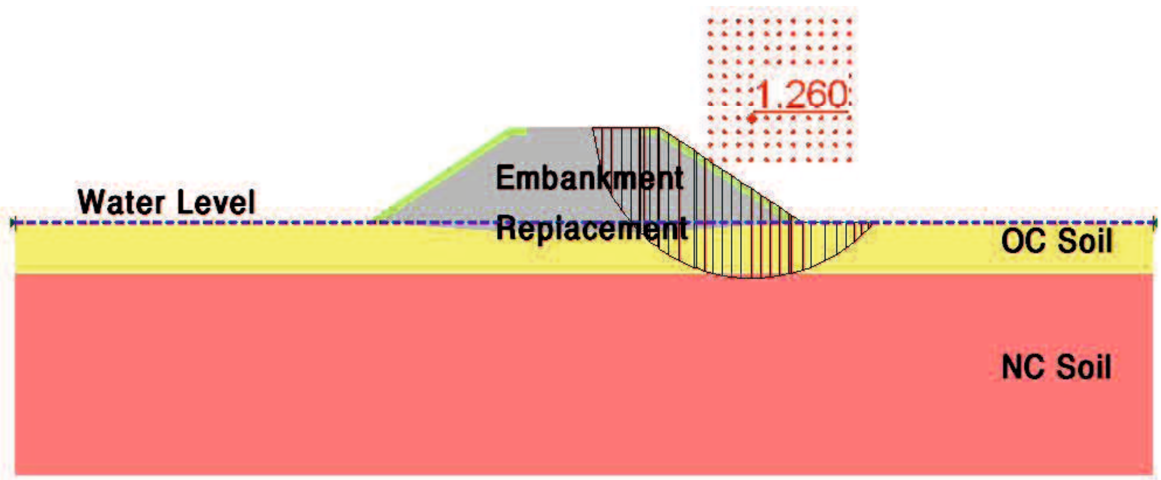
Fig. 6.6 The estimated replacement depth when applying φ' under partially drained conditions



(a) Result of stability analysis under φ'=3°



(b) Result of stability analysis under φ'=5°



(c) Result of stability analysis under φ'=7°

Fig. 6.7 Minimum factor of safety when applying φ' under partially drained conditions

6.3 COMPARISONS OF THE STABILITY ANALYSIS RESULTS BY APPLYING SAND COMPACTION PILE METHOD TO LOW PLASTICITY SOIL

6.3.1 STABILITY ANALYSIS BY USING FULLY UNDRAINED CONCEPT

The Sand Compaction Pile (SCP) method has been developed and frequently adopted for many construction projects in Korea, in which sand is fed into a ground through a casing pipe and is compacted by either vibration, dynamic impact or static excitation to construct a compacted sand pile in a soft soil ground. The principal concept for application to clay ground is reinforcement of composite ground consisting of compacted sand piles and surrounding clay, which is different from that of the Sand Drain method in which sand piles without any compaction are constructed principally for drainage function alone. The replacement ratio, a_s, is defined as the ratio of the sectional area of the sand pile to the hypothetical cylindrical area. The SCP improved ground can be divided into 'low', 'medium' and 'high' for a replacement area ratio of less than 0.4, 0.4 to 0.7, and higher than 0.7. The magnitude of stress concentration ratio ($n = \Delta\sigma_s/\Delta\sigma_c$ where $\Delta\sigma_s$: mean increment of vertical stress due to external forces in a sand pile at a slip surface, $\Delta\sigma_c$: increment of vertical stress due to external forces in clay part between sand piles at a slip surface) usually used in the practical design can be divided into n = 3, 2, 1 for a replacement area ratio of less than 0.4, 0.4 to 0.7, and higher than 0.7. As shown in Fig. 6.8, the shear strength of sand piles is considered to be incorporated alone with an average internal friction angle of sand piles, φ_m. The internal friction angle, φ_m, of 'sandy ground' is estimated by incorporating the composite ground consisting of sand piles and clay, and is expressed as $\varphi_m = \tan^{-1} (\mu_s \cdot a_s \cdot \tan \varphi_s)$ where φ_s = internal friction angle of sand in pile (°), μ_s = stress concentration coefficient in a sand pile ($\mu_s = \Delta\sigma_s/\Delta\sigma_z = n / [1 + (n-1)\, a_s]$), $\Delta\sigma_z$: mean increment of vertical stress due to external forces at a slip surface.

As shown in Fig. 6.9, the replacement ratio, a_s of SCP method was estimated at 23% through analyzing slope stability (FS=1.208) when applying $q_u/2$ strength. As shown in Fig. 6.10, the replacement ratio, a_s of SCP method was estimated at 9.3% through analyzing slope stability (FS=1.201) when applying recompression strength. This value was 60% smaller than that of recompression strength. Therefore, if this advanced approach is applied to the evaluation of undrained shear strength of clayey silts with low plasticity, construction cost can be drastically reduced in this kind of subsoil.

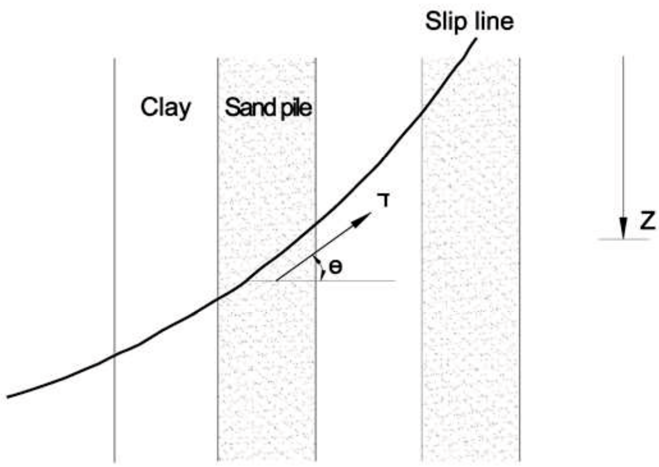
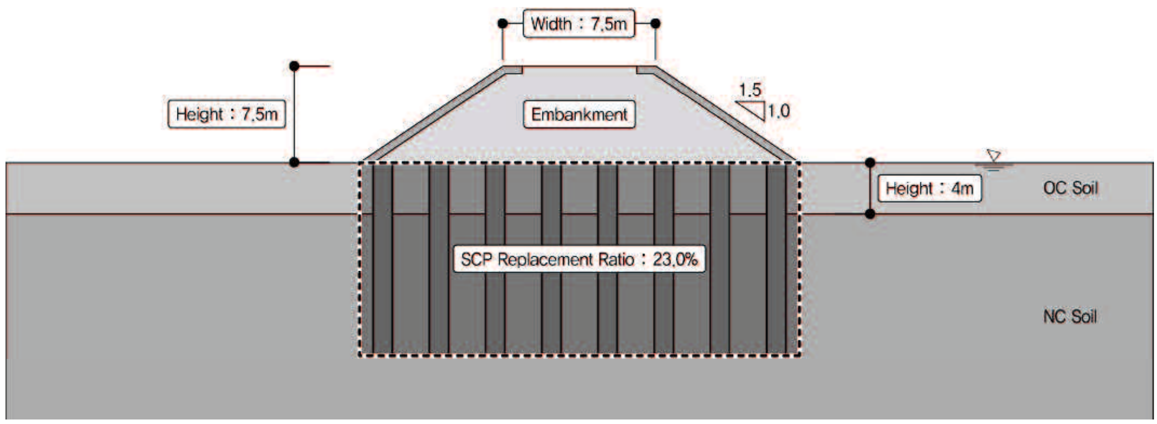
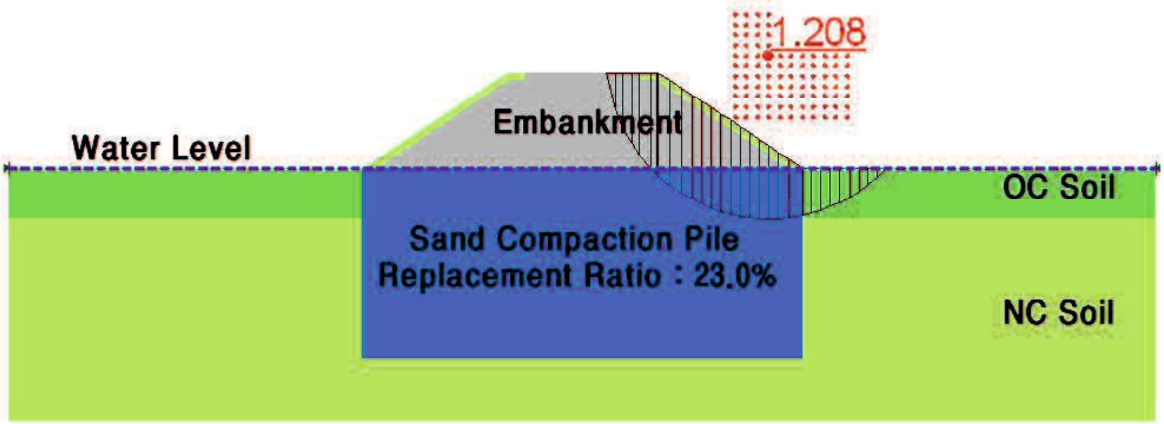


Fig. 6.8 Shear strength of composite subsoil

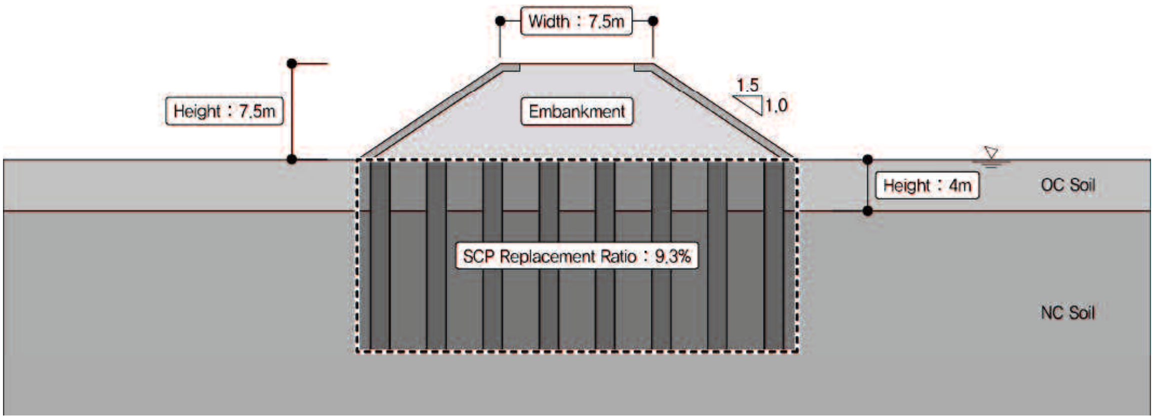


(a) The subsoil improved the replacement ratio of SCP, a_s = 23%

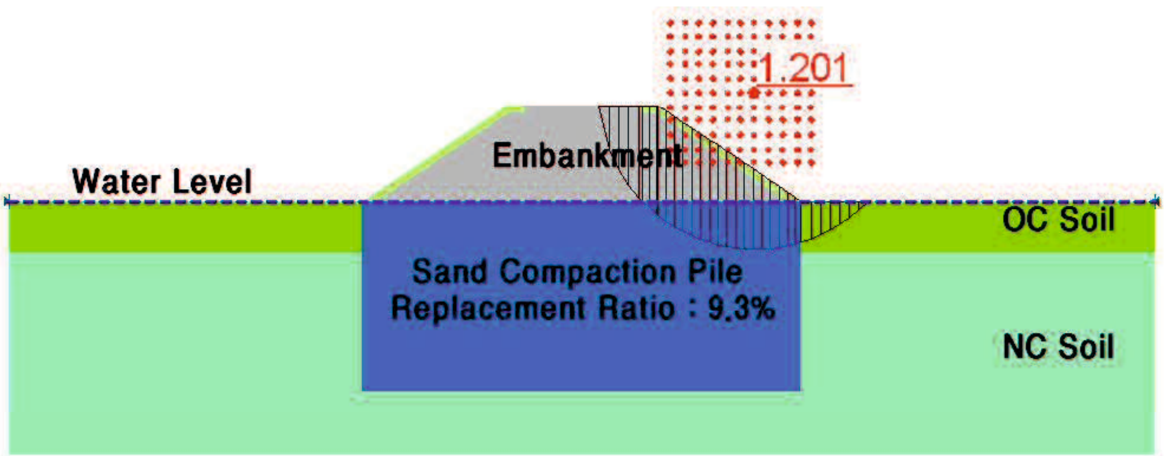


(b) Result of stability analysis under $q_u/2$ strength

Fig. 6.9 The subsoil improved the replacement ratio of SCP, a_s = 23% when applying $q_u/2$ strength and its minimum factor of safety



(a) The subsoil improved the replacement ratio of SCP, a_s = 9.3%



(b) Result of stability analysis under recompression strength

Fig. 6.10 The subsoil improved the replacement ratio of SCP, a_s = 9.3% when applying recompression strength and minimum factor of safety

6.3.2 STABILITY ANALYSIS BY USING PARTIALLY DRAINED CONCEPT

The replacement ratios, a_s of SCP method were estimated by applying a series of internal friction angles to consider its sensitivity under partially drained conditions. As shown in Fig. 6.11(a), 6.11(b), the replacement ratio, a_s of SCP method was estimated at 6.1% when applying $\varphi'=3°$ to secure slope stability (FS = 1.203) in the subsoil under partially drained conditions, from the view point of short-term stability. This value was 35% smaller than the case applied recompression strength.

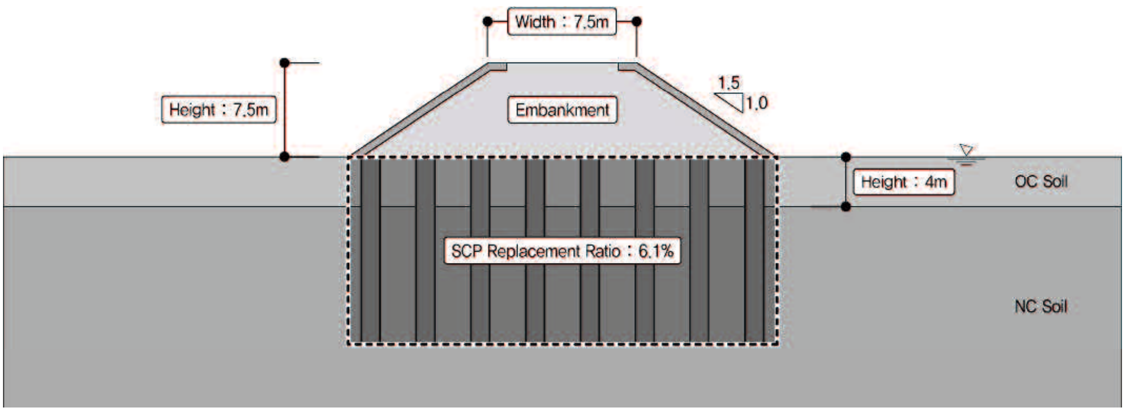
As shown in Fig. 6.12(a), 6.12(b), the replacement ratio, a_s of SCP method was estimated at 3.9% when applying $\varphi'=5°$ to secure slope stability (FS = 1.207) in the subsoil under partially drained conditions, from the view point of short-term stability. This value was 58% smaller than the case applied recompression strength.

As shown in Fig. 6.13(a), 6.13(b), the replacement ratio, a_s of SCP method was estimated at 1.4% when applying $\varphi'=7°$ to secure slope stability (FS = 1.210) in the subsoil under partially drained conditions, from the view point of short-term stability. This value was 85% smaller than the case applied recompression strength.

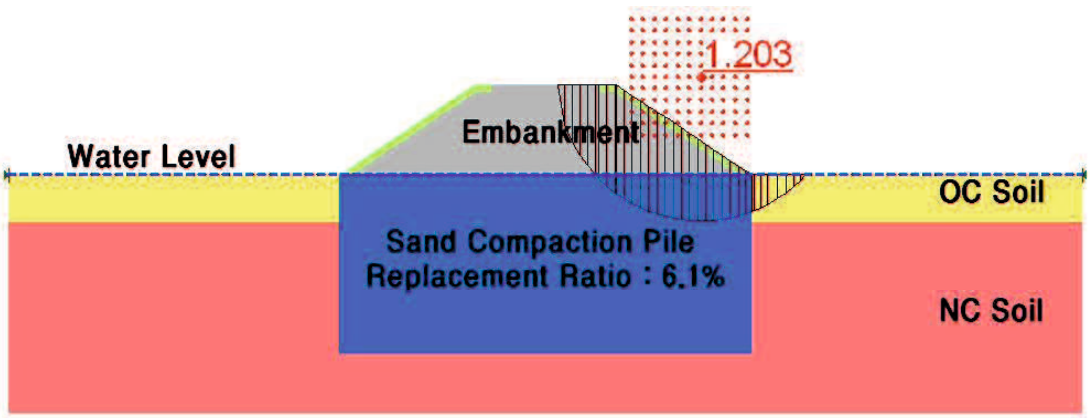
It is known that, practically, in Korea, the lowest replacement ratio of marine subsoil applied SCP method beneath gravity type structures is approximately 10%, because more than this value is believed to behave as composite ground reinforced by sand compaction pile. As shown in Fig. 6.14, each of minimum factors of safety was more than 1.0 when applying a series of internal friction angles under partially drained conditions in the subsoil beneath embankment. If the increase of shear strength obtained by consolidation effect taken place in the construction process is appropriately considered to evaluate the stability of subsoil beneath the embankment, FS of more than or equal to 1.2 could be obtained from the view point of short-term stability. In such a case, subsoil improvement method might not be necessary to secure stability of subsoil allowing the replacement of subsoil at shallow depths taken place by local shear failure.

Until now, in order to evaluate the applicability of each design concept (Fully undrained and partially drained conditions) regarding intermediate subsoil in this research, the differences between the improvement ratios (replacement depths) of each design concept were analyzed through the subsoil improved by compulsory replacement and SCP methods, respectively.

From now on, further in-depth study should be carried out regarding the applicability of each design concept focusing on the evaluation of partially drained strength (φ') of the clayey silt with low plasticity through detailed analysis of dissipation trend in CPTU data.

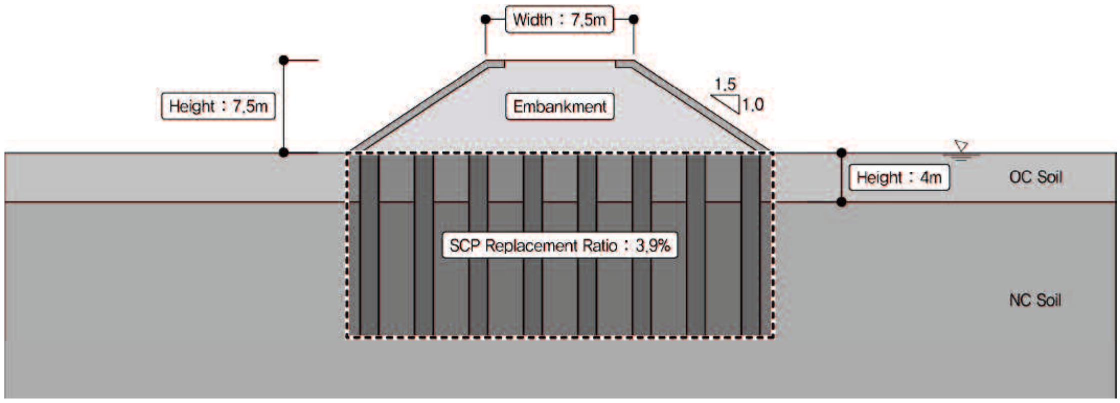


(a) The subsoil improved the replacement ratio of SCP, a_s = 6.1%

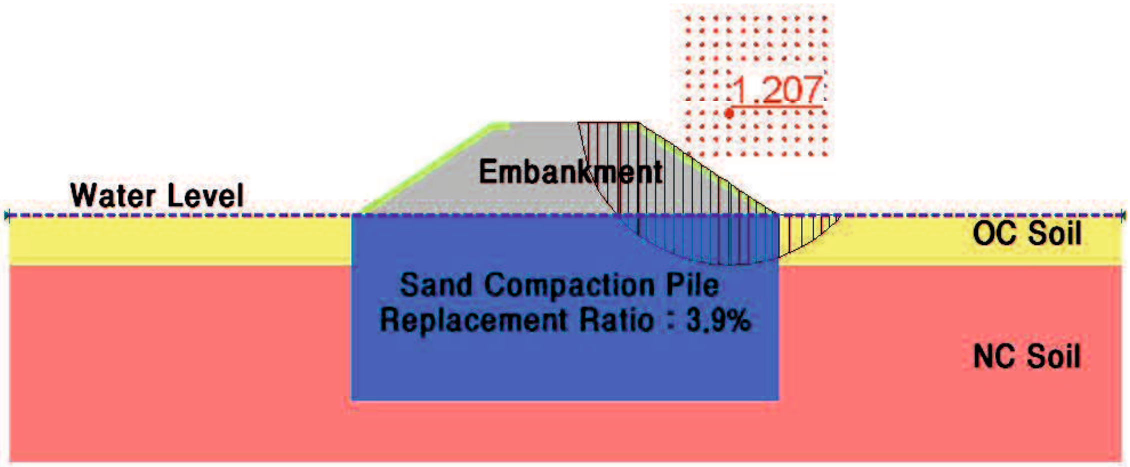


(b) Result of stability analysis under φ'=3°

Fig. 6.11 The subsoil improved the replacement ratio of SCP, a_s = 6.1% when applying φ'=3° under partially drained conditions and its minimum factor of safety

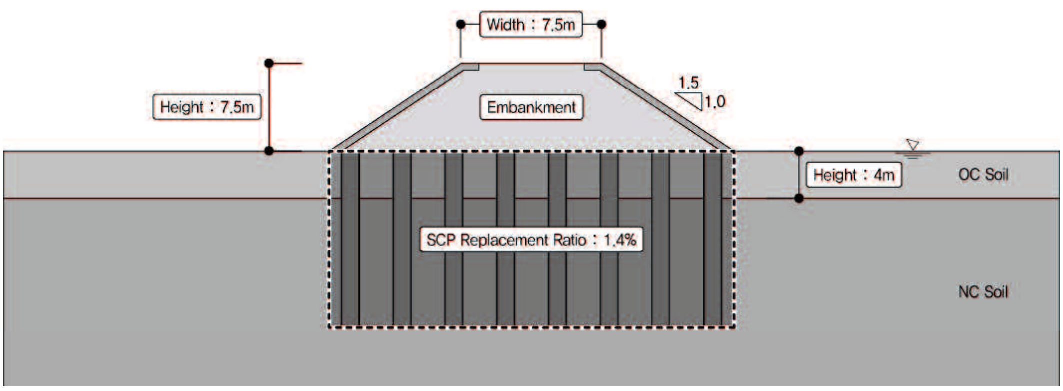


(a) The subsoil improved the replacement ratio of SCP, a_s = 3.9%

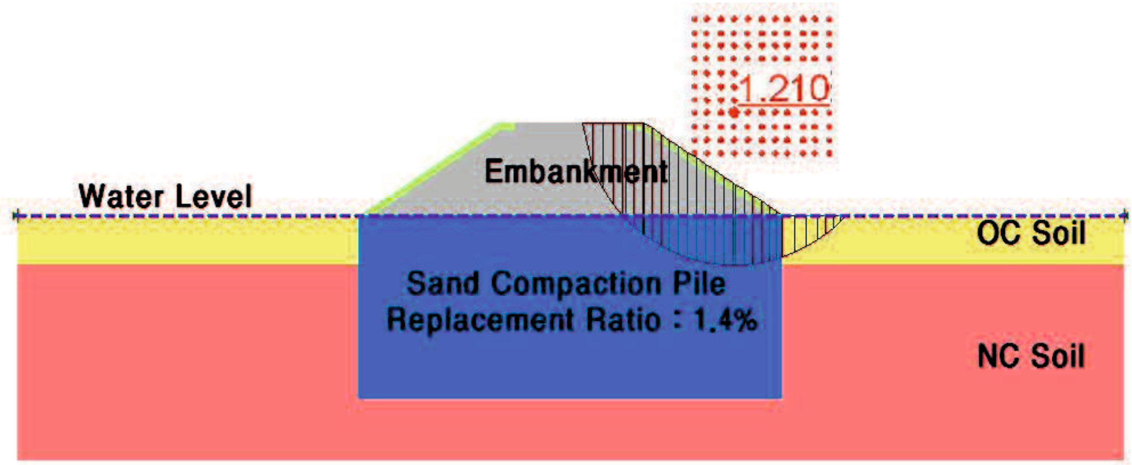


(b) Result of stability analysis under φ'=5°

Fig. 6.12 The subsoil improved the replacement ratio of SCP, a_s = 3.9% when applying φ'=5° under partially drained conditions and its minimum factor of safety

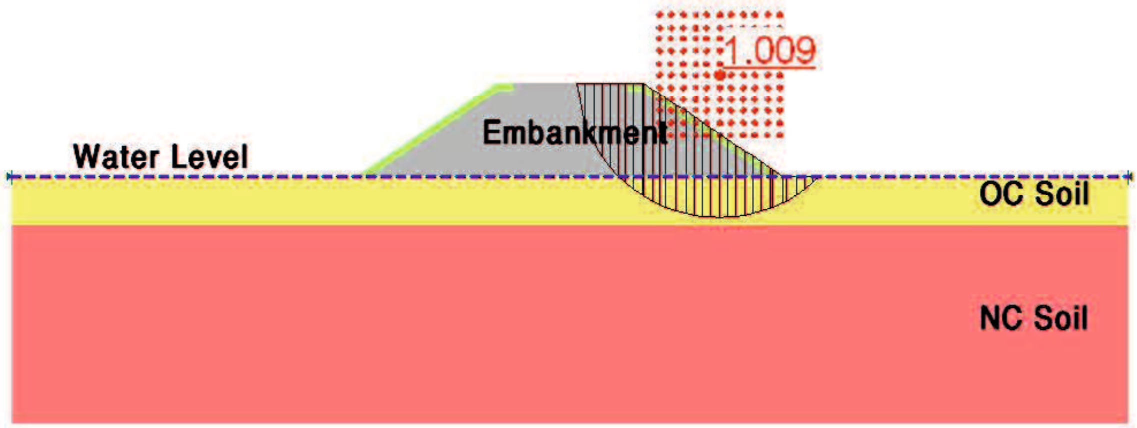


(a) The subsoil improved the replacement ratio of SCP, a_s = 1.4%

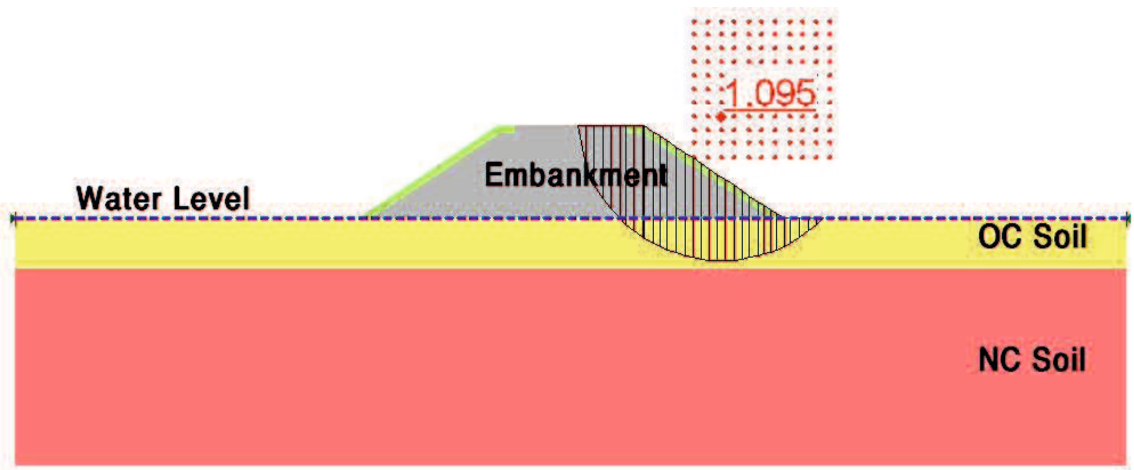


(b) Result of stability analysis under φ'=7°

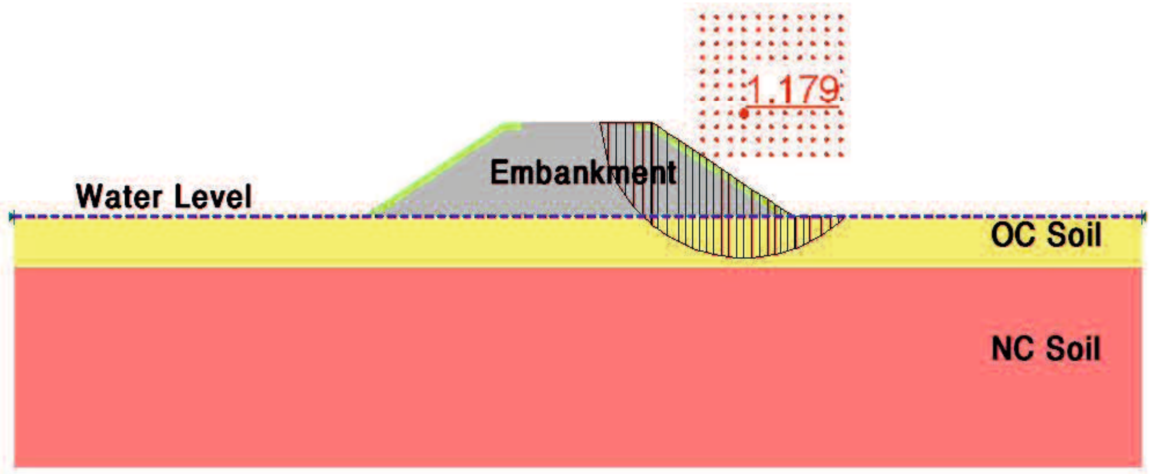
Fig. 6.13 The subsoil improved the replacement ratio of SCP, a_s = 1.4% when applying φ'=7° under partially drained conditions and its minimum factor of safety



(a) Result of stability analysis under φ'=3°



(b) Result of stability analysis under φ'=5°



(c) Result of stability analysis under φ'=7°

Fig. 6.14 Minimum factor of safety of non-improved subsoil when applying φ' under partially drained conditions

6.4 CONCLUSIONS

In order to check economic feasibility of fully undrained and partially drained concepts proposed in this study, the compulsory replacement method and Sand Compaction pile (SCP) method were applied to improvement of cohesive subsoils beneath embankment at Incheon site. In order to apply these design concepts, first of all, subsoil should be classified as silty soil (ML or MH) based on plasticity chart. And then, underestimated trend of Unconfined Compressive strength (UC strength) and incremental strength ratio (s_u/p'_c) should be checked in comparison with advanced laboratory tests (recompression test, etc) and in-situ tests (CPTU, FVT, etc).

When applying compulsory replacement method to improve subsoil beneath embankment, replacement depth evaluated by using recompression strength was almost half of $q_u/2$ based value. In case of silty soil with low plasticity, it was reported that the checked depths obtained at completed construction sites were in almost all cases considerably smaller than the estimated depths in the design phase. It is inferred that the underestimation trend of $q_u/2$ strength in clayey silt with low plasticity is certainly one major reason why this phenomenon takes place. Underestimation of the undrained strength given by UC test may create a false demand for soil treatment work to support superstructures on such soils, causing wastage of a huge amount of money. The same trend holds true for Sand Compaction Pile method.

In order to apply partially drained concept in designing soft ground improvement, first of all, the assessment of partial drainage conditions should be performed through the application of existing approaches to the CPTU data (comparison oedometer's OCR with that estimated by Powell and Quarterman' approach and Schnaid et al.'s approach, that is, plotting the normalized cone resistance, Q_t, vs. the pore pressure parameter, B_q in combination with the strength incremental ratio s_u/σ'_{vo}, to the CPTU data). Secondly, fully undrained shear strength line should be estimated through carrying out recompression test or in-situ tests (CPTU, FVT). At this point, $q_u/2$ strengths are only used to check underestimation trend of strength in intermediate subsoil in comparison with recompression strength. Thirdly, drained internal friction angle (°) with increasing depth should be estimated by analyzing stress path curve of recompress test to establish fully drained shear strength line, if possible, it is desirable that conventionally Consolidated Undrained triaxial test with checking pore water pressure (CU-TEST) is carried out to evaluate drained internal friction angle. Fourthly, the undrained shear strength obtained by recompression test and in-situ tests (CPTU, FVT) should be compared with fully drained shear strength. If the undrained shear strengths are higher than fully drained shear strengths, these strengths should not be applied to the stability analysis of the intermediate subsoil, from the viewpoint regarding the long term stability. Therefore, at the very beginning, the fully undrained and drained shear strengths should be evaluated from laboratory tests, in that order. Fifthly, the internal friction angles under the partially drained conditions can be estimated considering the distribution trend of the CPTU based shear strength between the undrained shear strength line and the fully drained shear strength line.

The replacement depths were estimated by applying a series of internal friction angles to consider its sensitivity under partially drained conditions. The estimated replacement depths (replacement ratio, a_s) were drastically changed with a slight change of internal friction angles. Accordingly, selecting appropriate strengths in stability analysis is the most important thing. In chapter 5, according to back analysis from field measurement data at completed construction sites applied compulsory replacement method and the analysis of CPTU based strengths, the internal friction angles under partially drained conditions ranged from $\varphi'=5°$ to $\varphi'=15°$. At the present stage, it is very difficult to select appropriate φ' value to design improvement of cohesive subsoils under partially drained conditions beneath gravity type structures such as breakwaters, quaywalls, and revetments. Therefore, when applying partially drained concept in designing soft ground improvement, it is recommended that the lowest $\varphi'=3\text{-}5°$ be applied to stability analysis from the view point of safety side.

From now on, further in-depth study should be carried out regarding the applicability of each design concept focusing on the evaluation of partially drained strength (φ') of the clayey silt through detailed analysis of dissipation trend in CPTU data.

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CHAPTER 7
GENERNAL CONCLUSIONS

In this thesis, characteristics of clayey silt with low plasticity located at western site of Korean peninsula were analyzed focusing on the evaluation of fully undrained and partially drained shear strengths. Several points of soil classification using Casagrande's plasticity chart were discussed and revised Polidori's plasticity chart were suggested through verification of applicability using Korean four different marine clayey soils.

Firstly, it was carried out to investigate the applicability and problems of soil classifications based on the plasticity charts for natural marine soils taken from four different Korean coastal areas, and also through comparison with the plasticity charts in Casagrande and Polidori.

Secondly, a series of laboratory and in-situ tests such as CPTU, FVT were carried out to evaluate undrained strength of clayey silts with low plasticity at Incheon and Gunsan site. The applicability of UC test in determining the undrained strength of these soils was examined. And, an appropriate method to evaluate the design parameters of low I_p soils was discussed through the test results and the literature review. In designing soft ground improvement, the strength incremental ratio (s_u/p'_c) is one of the most important factors to evaluate appropriate time of staged embankment loading. Skempton's equation has been widely used to evaluate strength incremental ratio in Korea. Therefore, field applicability of this equation for clayey soil with low plasticity was analyzed comparing with in-situ test and laboratory test results.

Thirdly, the assessment of partial drainage conditions was performed through the application of existing approaches to the CPTU data (comparison oedometer's OCR with that estimated by Powell and Quarterman' approach and Schnaid et al.'s approach, that is, plotting the normalized cone resistance, Q_t, vs. the pore pressure parameter, B_q in combination with the strength incremental ratio s_u/σ'_{vo}, to the CPTU data). And, internal friction angles under partially drained conditions were estimated based on where the design depths are equal to replacement depths checked at completed construction sites using back analysis. These values were compared with apparent internal friction angles obtained from UU-test (unconsolidated undrained triaxial test) and CPTU data. Because the undrained or partially drained shear strength should not be larger than the fully drained shear strength, in every possible case, based on the long term stability, the estimation of the shear strength under partially drained conditions has been discussed.

Finally, in order to check economic feasibility of fully undrained and partially drained concepts proposed in this study, the compulsory replacement method and Sand Compaction pile (SCP) method were applied to improvement of clayey silt beneath embankment at Incheon site, respectively.

In chapter 3, applicability and problems of soil classifications based on both plasticity charts for natural marine soils taken from four different Korean coastal areas were reviewed. The main findings from this research are as follows.

Almost all the soils taken from the four different Korean coastal areas have been classified as low plasticity or medium inorganic clay (CL) by Casagrande's plasticity chart, while using the data obtained from grain size analysis, the excessive silt concentration in the soil (50%-64%) supports its specification as an ML type soil assigned according to Polidori's soil classification.

Almost all the soils were classified as the silt according to Polidori's plasticity chart, which is based on the relationship between the Atterberg limits and the clay fraction percentage, mostly from using artificial soil mixtures, while ignoring the content of the silt fraction, that is, the role of silt in the plasticity chart. Because the liquid limit and plastic limit of fine-grained soils could change according to both the percentages of the silt and clay fractions, the silt fraction could be almost as important as the clay fraction in the relation between the Atterberg limits and the relevant fraction.

Based on the experimental data in this study, and because no distinction exists above the A-line where the natural soils taken from the four different coastal areas lie on the empirical plasticity chart proposed by Casagrande and because these soils were classified as silt by Polidori's plasticity chart, Polidori's revised plasticity chart based on the borderline of CF=30% proposed by this study appears to be more appropriate for classifying between silt or clay. To understand the reasons for the difference in the borderline of the CF in soil classifications in the present study and Polidori's soil classifications, further experimental data and studies are necessary.

In chapter 4, Korean marine clays taken from four coastal areas have been investigated by carrying out a series of laboratory and in-situ tests. The main findings from this research are as follows.

It can be concluded that UC test is not suitable for evaluating the undrained shear strength of low plastic soils. Therefore, in evaluating the undrained shear strength of soil with a low plasticity index, effective confining pressure that corresponds to typical marine clay should be applied to a soil specimen before shearing in order to compensate for the lost residual effective stress. In this cases, the CIU (recompression test) proposed by Tsuchida and Mizukami, can be quite useful in duplicating the in-situ shear strength of a soil.

The vane strength could be equivalent to CIU strength, in the sense that both strengths are not influenced by the stress release. Therefore, FVT could be used as the mobilized shear strength for the intermediate soil such as that obtained by CIU test, if FVT results are combined with CPTU to check whether the sand seam is or not.

The undrained shear strength normalized by the yield consolidation pressure, s_u/p'_c, is in the range of 0.25 to 0.35, independently of the plasticity index, I_p except for s_u/p'_c using $q_u/2$ values in case of soils having a low plasticity, such as Incheon and Gunsan clayey silts.

When I_p is less than 25, that is, in case of clayey silts such as those of the Incheon and Gunsan areas, the ratio of $s_{u(FVT)}$ to $q_u/2$ depends on I_p, and the ratio is over unity with large scatter. The reason why the $q_u/2$ values using samples taken from sites having the characteristic of low plasticity are considerably underestimated is the reduction in the negative pore pressure in the specimen acting as a confining pressure.

When Bjerrum's and Morris and Williams' correction factor is applied to Korean marine clay, the mobilized undrained shear strength is considerably underestimated, especially when I_p is higher than 40. This trend is noticeable in Morris and Williams' correction factor. The normalized undrained shear strength of Mesri's $s_u/p'_c=0.22$ result is quite conservative when applied to Korean marine clay.

In chapter 5, partial drainage characteristics of clayey silts with low plasticity were analyzed using laboratory and in-situ tests. The main findings from this research are as follows.

In interpreting the CPTU data, the existing correlation equations are very useful for estimating the strength and OCR before directly evaluating the strength and consolidation conditions by the laboratory and field tests. OCRs obtained by the oedometer tests were compared with the estimates obtained using the well-known relationship proposed by Powell and Quarterman (1988), given by: $OCR=\kappa\ ((q_t-\sigma_{vo})/\sigma'_{vo})$. Generally, the estimated OCRs were higher than those obtained by the oeodometer tests. These trends were noticeable for the layers containing a lot of silty or sandy soils. It is worth observing that Powell's formula provides rather unrealistic predictions, therefore confirming that the application of existing empirical and semi-empirical approaches developed for clays generally results in unreliable soil parameter values for Incheon clayey silt with low plasticity. When partial drainage during cone penetration is likely to occur and neglecting viscosity effects, the cone resistance clearly tends to be higher than in fully undrained conditions and provides an overestimation of relevant geotechnical parameter. The preliminary evaluation of such conditions is therefore of paramount importance for the assessment of representative mechanical parameters.

The assessment of partial drainage conditions was performed through the application of existing approaches to the CPTU data. When estimating whether the Incheon clayey soil is under partially drained conditions or not, the approach recently proposed by Schnaid et al., can be quite useful; it is based on plotting the normalized cone resistance, Q_t, vs. the pore pressure parameter, B_q in combination with the strength incremental ratio s_u/σ'_{vo}, to the CPTU data. A cone factor N_{kt} equal to 12 was assumed to convert the cone resistance to the undrained shear strength. It is evident that a two-thirds part falls in the range where $B_q < 0.3$, corresponding to the domain in which the partial drainage prevails when testing normally consolidated soils at a standard rate of penetration (2 cm/s). It is worth noting that the calculated strength incremental ratio is in general significantly higher than values estimated based on laboratory and in-situ tests for Incheon clayey silt; the result shows considerable scatter, therefore suggesting that deviation from this pattern in case of Incheon clayey silt is essentially related to partial drainage phenomena.

If the undrained shear strengths obtained by laboratory and in-situ tests are higher than fully drained shear strengths, these strengths should not be applied to the stability analysis for clayey silt, from the viewpoint regarding the long term stability. Therefore, at the very beginning, the fully undrained and drained shear strengths should be evaluated from laboratory tests, in that order. The CPTU based shear strength should be checked as to whether this strength is higher than the fully drained shear strength or not, in cases like the Incheon clayey silt with low plasticity, which is likely to be under partially or fully drained conditions during penetration.

The internal friction angles of the Incheon clayey silt with low plasticity in this study are not intended to be obtained from laboratory tests under partially drained conditions. The internal friction angles under the partially drained conditions were estimated considering the distribution characteristics of the CPTU based shear strength between the undrained shear strength line and the fully drained shear strength line, which is ranged approximately from

$\varphi'=5^{\circ}$ to $\varphi'=15^{\circ}$.

To estimate the replacement depth of the Incheon site with low plasticity, back analysis was carried out to evaluate the internal friction angle based on where the design depths equal to the checked depths using the bearing capacity equation, paying attention to the apparent internal friction angle obtained from the UU-test results. The internal friction angles obtained from the back analysis and UU-test results tend to increase as the plasticity index decreases. It seems that these values range from $\varphi'=5^{\circ}$ to $\varphi'=15^{\circ}$, similar to the ranges of the CPTU based internal friction angles. Based on these results, it might be possible to estimate the replacement depth of clayey silt with low plasticity as having the characteristics of partially drained conditions by applying the apparent internal friction angle.

It is difficult to evaluate the mobilized shear strength only through laboratory testing considering the partially drained conditions. Therefore, the analysis of the distribution characteristics of the CPTU based shear strength between the undrained shear strength line and the fully drained shear strength line could be viable alternatives to the estimation of the partially drained shear strength. From now on, further in-depth study should be carried out to evaluate the partially drained shear strength of clayey silt with low plasticity based on the rate effect and drainage characteristics of CPTU tests.

In chapter 6, the comparison of stability analysis (compulsory replacement method and Sand Compaction Pile method) regarding fully undrained and partially drained concepts for clayey silt with low plasticity were carried out to check economic feasibility of these design concepts.

The replacement depths were estimated by applying a series of internal friction angles to consider its sensitivity under partially drained conditions. The estimated replacement depths (replacement ratio, a_s) were drastically changed with a slight change of internal friction angles. Accordingly, selecting appropriate strengths in stability analysis is the most important thing. According to back analysis from field measurement data at completed construction sites applied compulsory replacement method and the analysis of CPTU based strengths, the internal friction angles under partially drained conditions ranged from $\varphi'=5^{\circ}$ to $\varphi'=15^{\circ}$. At the present stage, it is very difficult to select appropriate φ' value to design improvement of clayey silts under partially drained conditions beneath gravity type structures such as breakwaters, quaywalls, and revetments. Therefore, when applying partially drained concept in designing soft ground improvement, it is recommended that the lowest $\varphi'=3\text{-}5^{\circ}$ be applied to stability analysis from the view point of safety side.

From now on, further in-depth study should be carried out regarding the applicability of each design concept focusing on the evaluation of partially drained strength (φ') of the clayey silt with low plasticity through detailed analysis of dissipation trend in CPTU data.

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