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# Application of the dilatometer test for estimating undrained shear strength of Busan New Port clay

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ABSTRACT

The dilatometer test (DMT) is an economical and fast tool that evaluates the stratigraphy and properties of soil. In this study, a series of DMTs, field vane tests (FVTs), and $K_0$ consolidated-undrained triaxial compression ($CK_0UC$) tests were performed to develop an empirical correlation for the undrained shear strength ($s_u$) of clayey soils from the Busan New Port site. The stress-normalized mobilized $s_u(\mu s_u/\sigma'_v)$ for both laboratory tests and FVTs is determined to be around 0.22, which is consistent with previous studies. The exponent m for overconsolidation ratio (OCR), which is in the relation of $s_u/\sigma'_v = S \cdot OCR^m$, is determined to be 0.83 by using the SHANSEP technique. Two different methods of estimating $s_u$ are reviewed to make use of either the horizontal stress index ($K_D$) or the bearing factor ($N_c$). Because the DMT results indicate that the $N_c$ is linearly proportional to the material index ($I_D$), representing the characteristics of soil, the empirical $s_u$ estimating formula with $I_D$ is newly suggested in this study. According to the values for the mean absolute percentage error (MAPE), the empirical correlation with $I_D$ shows a slightly better accuracy than that using $K_D$. However, a comparison between the values for the $s_u$ that are measured and estimated by using two different methods shows good agreement in general.



## 1. Introduction

The undrained shear strength ($s_u$) of soil is a key parameter that influences the design of the earth structures in soft soils. The $s_u$ can be measured directly by using the field vane test (FVT), triaxial test, and simple shear test. In addition, it can be indirectly predicted through field penetration tests, such as the piezo cone penetration test (CPTu) and flat dilatometer test (DMT). These field penetration tests, which are fast and economical, can adequately reflect in-situ conditions. However, the measured results must be interpreted by using suitable empirical correlations to evaluate the $s_u$ because the data obtained from these tests reflects complex interactions between the test equipments, various soil properties, and the in-situ conditions (Chung et al., 2010; Jamiolkowski et al., 1988; Lunne et al., 1997; Marchetti et al., 2001; Mayerhof, 1956). In addition, because the empirical correlations vary with regional soil characteristics, proper care must be taken to perform an effective evaluation of $s_u$.

Since Marchetti (1980) first introduced the DMT as a soil testing technique in 1980, the DMT has been extensively used in many geotechnical investigations (Marchetti, 2006; Robertson, 2009). The DMT measures the values for the horizontal pressures $p_0$ (life-off pressure) and $p_1$ (1.1 mm deflection pressure) at specific horizontal displacements of the soil. Three DMT indices, including the material index ($I_D$), the horizontal stress index ($K_D$), and the dilatometer modulus ($E_D$), are evaluated by using the measured $p_0$ and $p_1$ values. A number of empirical correlations based on these DMT indices have been suggested to evaluate various geotechnical properties of soils, including the unit weight ($\gamma_t$), coefficient of lateral earth stress at rest ($K_0$), overconsolidation ratio (OCR), undrained shear strength ($s_u$), constrained modulus (M), small strain shear modulus ($G$), and soil classification (Baldi et al., 1989; Hryciw, 1990; Iwasaki et al., 1991; Jamiolkowski et al., 1985; Kamei and Iwasaki, 1995; Lacasse and Lunne, 1989; Marchetti, 1980; Powell and Uglow, 1988; Roque et al., 1988). Since the DMT generates less disturbance in the soil due to the shape of blade and works on a small strain level than through other penetration tests such as the SPT (standard penetration test) and CPT, the stress history and deformation of clayey soil can be better evaluated through DMT (Baligh and Scott, 1975; Cruz et al., 2006; Iwasaki et al., 1991; Kamei and Iwasaki, 1995; Lacasse and Lunne, 1989; Lutenegger 1988; Marchetti, 1980; Marchetti et al., 2001; Monaco et al., 2007; Powell and Uglow, 1988). Because the stress history (or overconsolidation) is closely related to the undrained shear

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strength (Ladd and Foott, 1974), the $s_u$ obtained through a DMT is reliable (Byeon et al., 2006).

The purpose of this study is to evaluate the mobilized $s_u$ ($\mu s_u$) of Busan New Port clay using the DMT results. Therefore, a series of in-situ tests, including the DMT and FVT, were performed at the Busan New Port site. In addition, undisturbed samples were retrieved with depths to measure the basic index properties and to conduct $CK_0UC$ ($K_0$ consolidated-undrained triaxial compression) tests. The DMT indices, including the material index ($I_D$), dilatometer modulus ($E_D$), and horizontal stress index ($K_D$), were evaluated by taking the confining stress into account. The $\mu s_u$ was calculated using the measured $s_u$ through FVTs and $CK_0UC$ tests with the recommended correction factors due to the discrepancy between the measured $s_u$ from in-situ or laboratory tests and the mobilized $s_u$ in the actual field. Previous $\mu s_u$ estimating formulas using the DMT results have been reviewed, and the new empirical formula for estimating $\mu s_u$ of Busan New Port clay is suggested in this study.

## 2. Undrained shear strength ($s_u$) based on DMT

The $s_u$ varies according to the shearing mode, anisotropy, strain rate, boundary conditions, in-situ testing device, confining stress level, and other factors (Kamei and Iwasaki, 1995; Ladd, 1991; Mitchell and Soga, 2005). Therefore, the $s_u$ is not unique for a given soil, but depends on the type of testing (Mayne, 2008). The $s_u$ is typically normalized with the vertical effective stress ($\sigma'_v$) to obtain the following form:

$$(s_u/\sigma'_v)_{NC} = S \tag{1}$$

where subscript NC=normally consolidated state; and $S$=undrained strength ratio at NC, which is given in Table 1. Since the $s_u/\sigma'_v$ depends on the overconsolidation ratio (OCR), the stress history and normalized soil engineering properties (SHANSEP) technique has been developed for use in practice (Ladd and Foott, 1974; Mitchell and Soga, 2005):

$$s_u/\sigma'_v = (s_u/\sigma'_v)_{NC} \cdot OCR^m = S \cdot OCR^m \tag{2}$$

where $m$=fitting parameter, typically 0.8 (Ladd and Foott, 1974; Ladd et al., 1977). OCR of clays in Eq. (2) can be estimated with $K_D$ (Marchetti, 1980):

$$OCR = (0.5 \cdot K_D)^{1.56} \tag{3}$$

Additionally, Marchetti (1980) proposed $S$=0.22 and $m$=0.8 in Eq. (2) by comparing the results of field vane tests. Therefore, the substitution of $S$ and $m$ values of Marchetti (1980), and Eq. (3) into Eq. (2) yields the following $s_u$ estimating formula based on the DMT results (Kamei and Iwasaki, 1995; Lacasse and Lunne, 1989; Marchetti, 1980; Powell and Uglow, 1989):

$$s_u/\sigma'_v = 0.22 \cdot (0.5K_D)^{1.25} \tag{4}$$

However, it is important to note that the $S$ in Eq. (2) can vary depending on the testing type or shearing mode as shown in Table 1 (Kamei and Iwasaki, 1995).

The bearing capacity theory can be also used with the DMT results for an alternate method to estimate $s_u$, and this method is similar to the $s_u$ estimating formulas based on the cone tip resistance (Ebrahimian et al., 2012; Mayne 2008). An inverted form of the equation for the bearing capacity is (Roque et al., 1988):

$$s_u = \frac{p_1 - \sigma_h}{N_c} \tag{5}$$

where $\sigma_h$=total horizontal stress; and $N_c$=bearing factor. Based on the comparison with the triaxial compression test results, Roque et al. (1988) suggested that the $N_c$ ranges from 5 to 9 according to the soil types. It is interesting to note that Eq. (4) uses $p_0$ value to estimate $s_u$, while Eq. (5) employs $p_1$ value. Because of the difficulty in a direct estimation of $\sigma_h$ in Eq. (5), the coefficient of lateral earth stress at rest ($K_0 = \sigma'_h/\sigma'_v$, where $\sigma'_h$=horizontal effective stress) has been used to evaluate $\sigma_h$ (Ku and Mayne, 2013). Following the work of Marchetti (1980), $K_0$ of clays can be expressed as a function of $K_D$ (Eq. (6)). Therefore, $\sigma_h$ in Eq. (5) can be estimated using Eq. (7).

**Table 1**
Undrained strength ratios for Boston Blue Clay (after Mayne 2008).

| Type of testing | $S$ in Eq. (1) |
|---|---|
| Triaxial compression | 0.33 |
| Direct simple shear | 0.20 |
| Triaxial extension | 0.16 |
| Plane strain compression | 0.33 |
| Plane strain extension | 0.19 |
| Field vane test | 0.21 |
| Self-boring pressure meter | 0.42 |

$$K_0 = (K_D/1.5)^{0.47} - 0.6 \tag{6}$$

$$\sigma_h = \sigma'_v \cdot \left[(K_D/1.5)^{0.47} - 0.6\right] + u_0 \tag{7}$$

where $u_0$=hydrostatic pore water pressure.

## 3. Site description

Busan New Port is located in the southeast coastal area of the Korean peninsula as shown in Fig. 1. The clayey soils in this region are typically referred to as Busan clay. Previous studies (Chung et al., 2002, 2007, 2005) have reported that the Busan clay layer is extended to a maximum depth of 70 m. The clay layer tested at the Busan New Port is deposited from EL-2 m up to EL-50 m and can be divided into two layers at EL-30 m (i.e., a soft upper clay layer and a stiff lower clay layer). The New Port was constructed by carrying out ground improvement with prefabricated vertical drains (PVDs) and the gravel surcharge load of 13 m in height. This surcharge load corresponds to a vertical effective stress increment of ~260 kPa.

Fig. 2 shows the profiles of typical properties of the Busan New Port clay. The natural water content ($w_n$) ranges from 35% to 75%, and it is almost equal to or slightly lower than the liquid limit ($w_L$) for the entire depth. The plastic limit ($w_p$) is within a relatively narrow range, from 20% to 35%. These values increase as the depth increases down to EL-25 m and decrease with a depth below EL-25 m (Chung et al., 2002; Kim, 2008; Tanaka et al., 2001). The total unit weight ($\gamma_t$) remains relatively constant up to EL-30 m, and then it increases with depth, which may reflect two different layers. The percent passing of # 200 sieve (0.075 mm in sieve size) ranges from 83% to 99%. The activity (=plasticity index/clay fraction) of New Port clay ranges from 0.6 to 1.1. Except some depths with sharp increases in cone tip resistance ($q_t$) (or sharp decreases in measured pore water pressure behind the cone tip ($u_2$)) reflecting the existence of layers with high silt or shell content, both $q_t$ and $u_2$ increase with an increase in depth (Chung et al., 2007). It is notable that relatively low $q_t$ values ($<$5 MPa) and high $u_2$ values ($>u_0$) infer that the tested soil layers are clayey soils (Mayne, 2007).