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Date: 2020/03/10

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SMALL STRAIN DEFORMATION CHARACTERISTICS IN ANISOTROPICALLY CONSOLIDATED COHESIVE SOIL

1. Introduction

This study investigated the deformation characteristics at small strain (about 0.001%) of lateritic soil which is classified as cohesive soil. According to Mitchell and Soga (2005), a soil is classified as cohesive if the amount of fines (silt and clay-sized material) exceeds 50% by weight. This type of soil is widely distributed in Africa and used for multi earthworks. Heavy damages have been reported during and after civil engineering constructions. These damages are often caused by a lack of understanding of the ground behavior. In order to accurately predict the soil response underneath geotechnical structures, it is important to take account the deformation at very small strain (<0.001%) and the anisotropic effect. The deformation characteristics at very small strain is mainly quantified by the initial shear modulus G_0. This parameter is evaluated by triaxial apparatus equipped with bender elements (BEs) and local small strain (LSS) measurement for dynamic and static method, respectively. The soil's anisotropy, which has paid a little attention, refers to the directional property, in which properties are different in different directions. In order to simulate the anisotropy in laboratory, the specimen is consolidated at different stress ratio K (σ'_3/σ'_1). The factors influencing G_0 such as the anisotropy and the overconsolidation ratio OCR have been elucidated by applying the well-known empirical equation of Hardin and Blandford (1989). The parameters of this equation applicable to lateritic soil is also assessed in this research.

The purpose of this study is to provide deep understanding of fundamental mechanical properties of lateritic soil for construction and consulting companies in Africa and to assess the parameters of the Hardin's equation for application to lateritic soil. This equation is in the form of:

$$G_0 = S_{ij}\, F(e)\, (OCR)^k\, Pa^{(1-ni-nj)}\, (\sigma'_i)^{ni}\, (\sigma'_j)^{nj} \qquad 1.1$$

where S_{ij}, n and k are material contant depending on the soil fabric, F(e) is the void ratio function, OCR is the overconsolidation ratio which is defined as the ratio between the maximum effective stress and the current stress, Pa is the atmospheric pressure, σ'_i and σ'_j are the effective stress in the i and j planes

2. Littérature review

The initial shear modulus G_0 is the basic parameter for evaluating the soil response induced by seismic or dynamic loading and for description of the non-linear behaviour of soil in a wide range of strain. Atkinson (1993) demonstrated that the strain range in the ground of many structures (retaining wall, foundation…) is less than 0.1%. Soils have highly non-linear mechanical behavior. Hence, he postulated three regions: very small strain, small strain, and large strain. The stiffness of geomaterial is higher at small strains than at the large strains. Although the anisotropy has important effects on G_0 for non-cohesive soil (Oda et al., 1985; Tatsuoka et al., 1979; Yu and Richart, 1984; Belloti et al., 1996), the stress induced anisotropy has minor influence on G_0 for cohesive soils (Sagae et al., 2006; Jovcic, 1997; Teachavorasinkun and Lukkanaprasit, 2008). Hardin and Blandford (1989) demonstrated that the initial shear modulus G_0 is influenced many factors such as current stress state, overconsolidation ratio, density, void ratio, microstructure, and so on. Several equations have been proposed by researchers to evaluate G_0. The maximum stiffness of geomaterial can be measured by dynamic and static methods. Laboratory measurements includes resonant column, torsional shear, bender elements, and triaxial shear with local strain measurement (Schneider et al., 1999).

In the case of BE test, the corresponding shear stiffness (G_0) is calculated by measuring the shear wave velocity (v_s) and bulk density (ρ) of geomaterial by the formula as:

$$G_0 = \rho \times {v_s}^2 \qquad 2.1$$

Shear wave velocity is determined by ratio between distance of bender elements (d) and the travel time of shear wave (Δt).

$$v_s = \frac{d}{\Delta t} \qquad 2.2$$

The start to start method illustrated in Fig. 1 is adopted to calculate the shear wave travel time (Δt) corresponding to the time difference between the starting of the input signal and the first arrivals of the received signals through visual inspection (Yang and Gu, 2013).

The local small strain (LSS) measurement is nowadays widely used by researchers to assess more accurately soil response and minimize errors such as bedding error, sample tilting, apparatus compliance. G_0 is calculated by the following formula:

$$G_0 = \frac{q}{3\Delta\varepsilon_s} \qquad 2.3$$

where q is the deviatoric stress and ε_s corresponds to the shear strain

The comparison of G_0 obtained from BE and LSS test indicates a good agreement with each other (Jovcic, 1997; Kiku and Yoshida, 2000; Tatsuoka et al., 1995). G_0 from BE is sometime found to be higher than that from LSS (Yamashita et al., 2003; Kung, 2007).

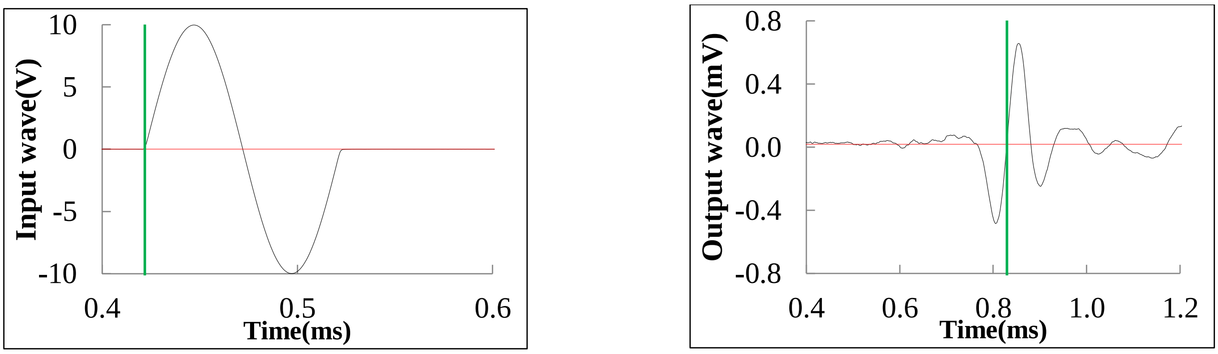


Fig.1. Determination method of shear wave travel time by start to start method

3. Soil characteristics

The lateritic soil is named as Yoneyama silt and was sampled in costal area in Niigata. Table 1 shows some physical characteristics of the soil. From Fig. 2, silt and clay contents are dominant in the soil. The sand content is less than 10%. The soil is then termed as cohesive soil.

Table 1: physical characteristics of Yoneyama silt

Parameters	Data
Specific gravity (g/cm^3)	2.746
Liquid limit (%)	52.4
Plastic limit (%)	29.9
Plasticity index (%)	22.5
Sand content (%)	8.5
Silt content (%)	65.5
Clay content (%)	26

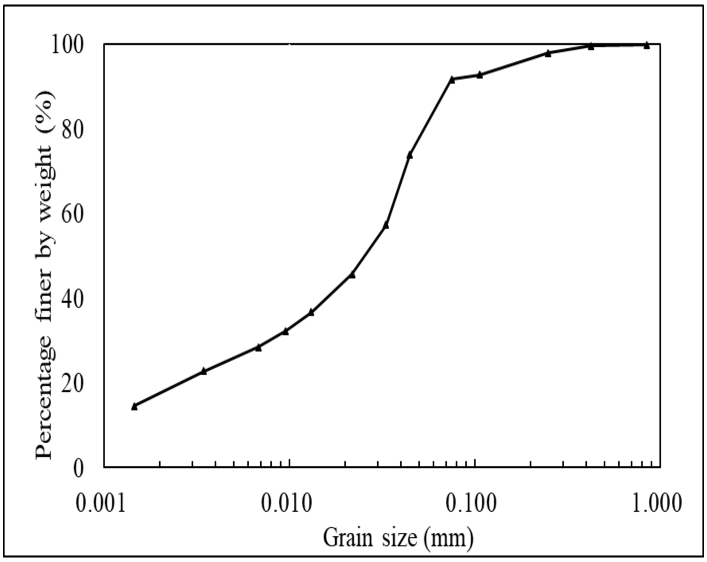
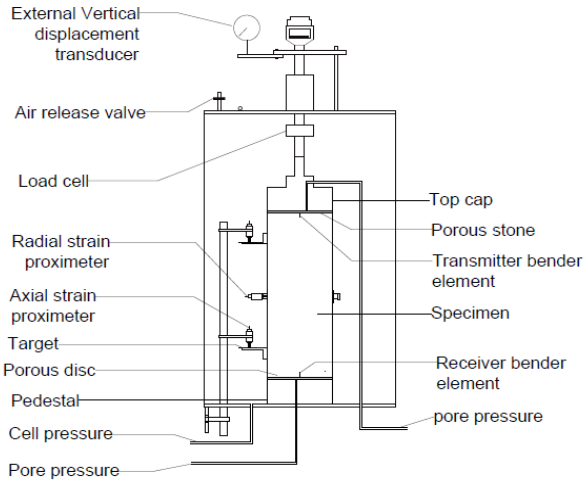


Fig.2. Grain size distribution of Yoneyama silt

4. Experiment procedures and methodologies

The triaxial apparatus is represented schematically in Fig. 3. The BEs are installed on top cap (transmitter function) and pedestal (receiver function) and the travel time is determined by start-to-start method. The LSS technique was incorporated into the triaxial apparatus. The vertical axial displacement is measured using 2 separated targets directly glued on the membrane of the specimen. The change in the diameter is calculated through the lateral proximity transducer installed on the middle part of the specimen.



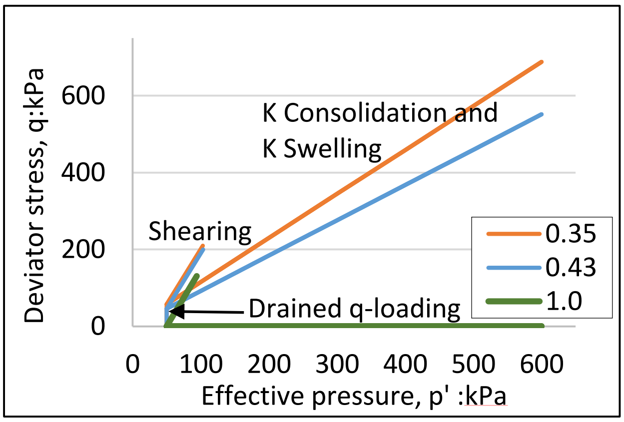


Fig.3. triaxial apparatus methodology

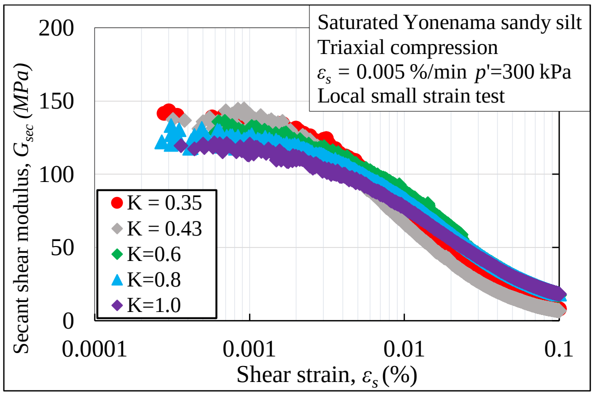
Fig.4. Representative stress path of the testing

Two testing methodologies are applied in this study. In the first methodology, shearing is applied at effective stress p' of 300 kPa. This methodology investigated anisotropy of G_0 at the same p' and the aging effect, which is skipped in this resume. Figure 4 illustrated the second methodology in which the overconsolidation ratio OCR effect was investigated. During the specimen preparation, soil and de-aired water are mixed and stirred to produce slurry. One-dimensionally pre-consolidation pressure of 70 kPa was applied on this slurry in a cylindrical mould. The specimen set in the triaxial cell was saturated using the vacuum saturation procedure and consolidated isotropically under p'= 50 kPa with back-pressure of 200 kPa. Drained q-loading was completed under constant p' of 50 kPa up to a certain K. Five cases of K are defined in this study: 0.35, 0.43, 0.6, 0.8, and 1.0. K consolidation was performed with maintaining K constant and p' increases gradually (50, 100, 200, 300, 400, 500, and 600 kPa). K swelling was conducted by gradual decrease of p' up to 50 kPa. BE test is performed 24h after p' reached a target value. LSS test was performed with the monotonic loading at shear strain rate of 0.005 %/min.

5. Experimental results and discussion

5. 1 Stress ratio effect on G_0

Figure 5 shows linear behaviour of G_0 at very small strain and decreasing in stiffness with the increase of strain for all cases of stress ratio K. Highest value of G_0 is observed at the stress ratio K=0.43. Furthermore, the increase of the stress ratio K lead to a slight decrease in the initial stiffness. Similar results have been observed in BE test (Fig. 6). G_0 decays by about 10% and 17% in BE and LSS tests, respectively (Fig. 7). Therefore, greater degree of anisotropy was observed in G_0 obtained from LSS test than that from BE test. Results also elucidated the increase of G_0 with p' during K consolidation and K swelling stage (Fig. 8). This may be related to the decrease of the void ratio. Furthermore, at a constant value of p', there is small variation of G_0 with K as shown in the previous results.



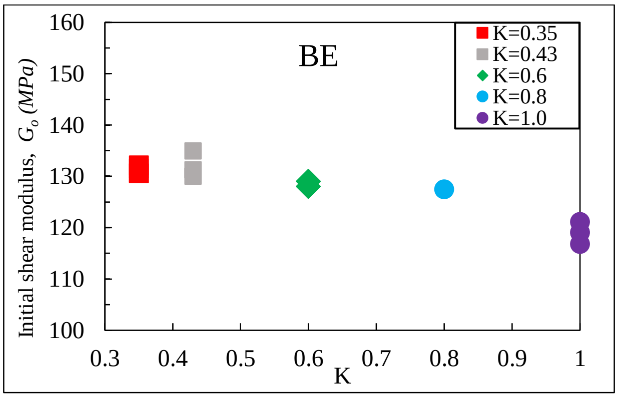
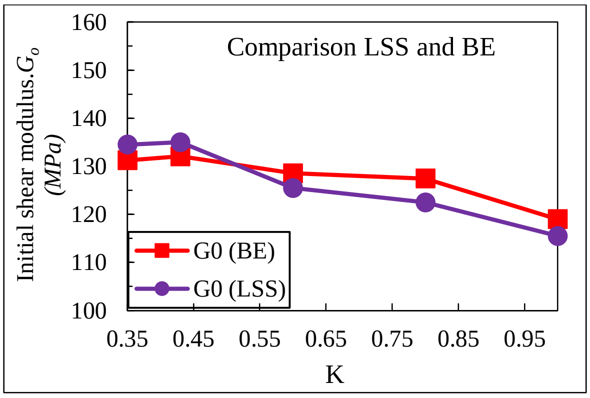


Fig.5. Relation between K and G_0 (LSS test) Fig.6. Relation between K and G_0 (BE test)



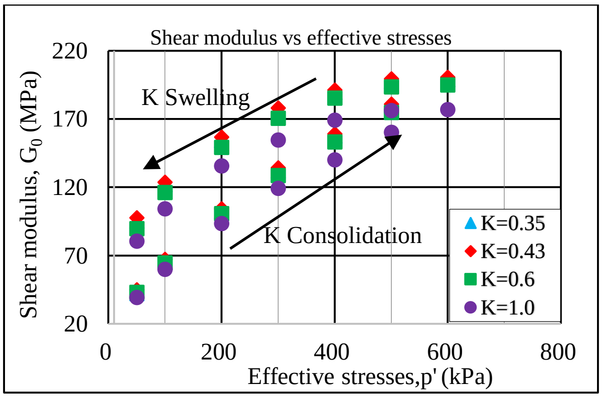


Fig.7. Comparison of G_0 from BE and LSS test Fig.8. Relation between K, p' and G_0

5. 2 Assessment of the parameters of Hardin's equation in lateritic soil

The factors affecting G_0 is considered by applying the proposed equation by Hardin and Blandford (1989), which is illustrated in equation 1.1. Different parameters of this equation have been defined in the literature for different type of soil.

Tables 2 and 3 indicate the variation of the parameters Sij, n and K summarized from the literature reviews. These parameters are found to be dependent on the soil fabric; k, the exponent of the OCR is mostly controlled by the soil plasticity.

Table 2. Values of n and S_{ij} summarized from the literature

Type of soil	n	Sij	References
Six undisturbed Italian clay	0.2-0.29	520-810	Jamiolkowski et al., 1995
Ottawa Sand	0.5	6900	Hardin and black (1968)

Table 3. Values of k, exponent of OCR summarized from the literature

Type of soil	PI (Plasticity index	k	References
Sand	<40 >40	0 0.5	Hardin and black (1966)
London clay	10-40	0.2-0.25	Viggiani and Atkinson (1995)
Boston blue clay	22.7	0.15	Santagata (2005)

In this study, n_i and n_j are assumed to be equal to **n = 0.25**. From the experimental results, the parameters of the Hardin's equation are assessed in the case of lateritic soil as follow: **S_{ij} =730, k=0.24.** This result is similar to the finding of other researchers (Jamiolkowski et al., 1995; Viggiani and Atkinson, 1995). Figure 9 shows that the predicted value of G_0 (pred G_0) calculated from the Hardin's equation agrees well with the true value of G_0 (true G_0) from BE test. Several void ratio functions have been found in the literature. Hardin and Richart (1963) proposed the most frequently applied void ratio function to estimate the initial shear modulus:

$$f(e) = \frac{(2.17-e)^2}{1+e} \tag{5.1}$$

$$f(e) = \frac{(2.97-e)^2}{1+e} \tag{5.2}$$

Lo Presti (1989) and Jamiolkowski et al. (1995) also proposed an empirical void ratio function on the following form.

$$f(e) = e^{-x} \tag{5.3}$$

Therefore, one of the mostly applied function corresponding to the equation 5.1 was used in this study. In the case of lateritic soil, the equation of Hardin can be rewritten as the following form:

$$G_0 = 730\,\frac{(2.17-e)^2}{1+e}\,(OCR)^{0.24} Pa^{(1-0.5)} \sigma_i{}^{0.25} \sigma_j{}^{0.25}$$

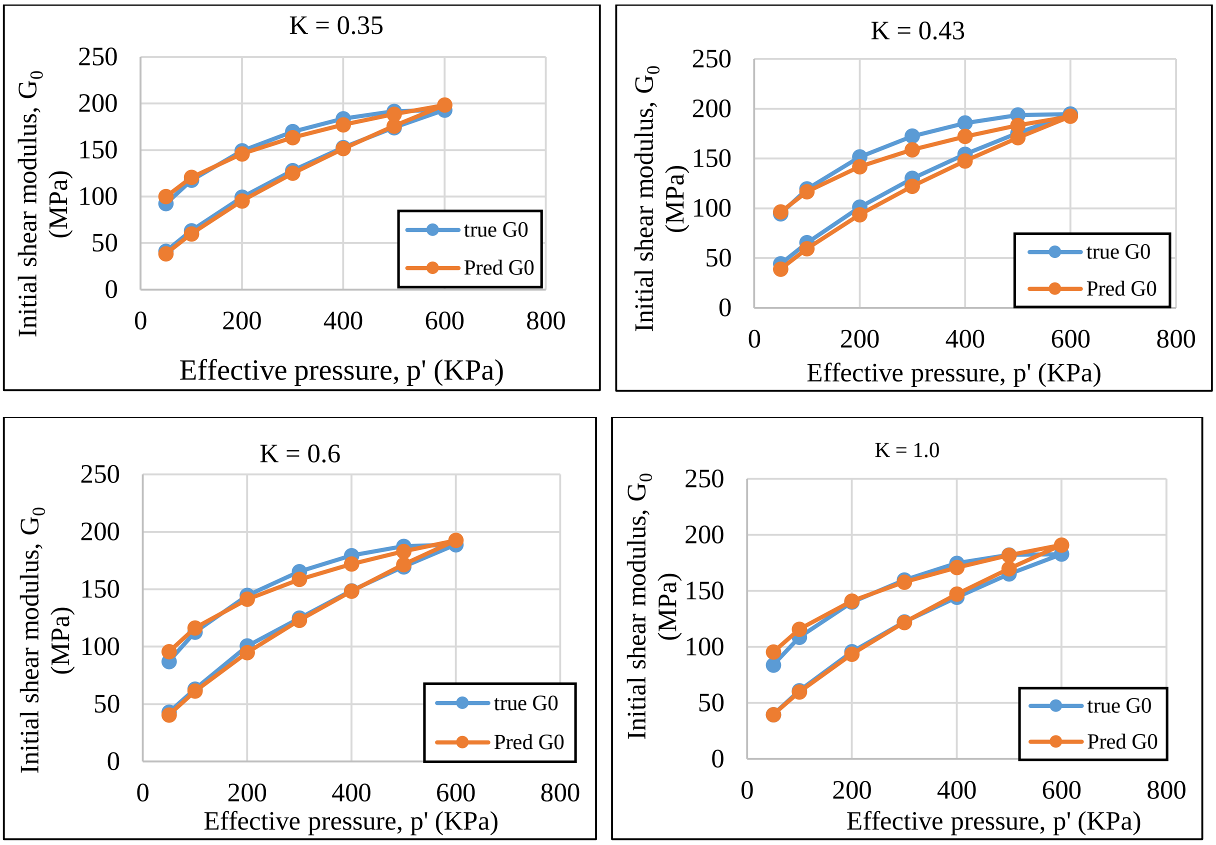


Fig.9. True and predicted G_0

6. Conclusion

The conclusions of this study can be summarized as follows:

- ➢ G_0 decreases with the increase of K under constant p'. Thus, there is effect of K on G_0.
- ➢ The degradation of G_0 with increased K is about 10% and 17% in BE test and LSS test, respectively. Greater degree of anisotropy is observed in G_0 obtained from LSS than that from BE.
- ➢ G_0 can be predicted for the lateritic soil using the following Hardin's equation:
  G_0= 730 F(e) (OCR)$^{0.24}$ Pa$^{(1\text{-}0.5)}$ (σ'$_i$)$^{0.25}$ (σ'$_j$)$^{0.25}$

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