EVALUATION OF MECHANICAL ANISOTROPY OF SOILS CONSIDERING PARTICLE ORIENTATION AND PRINCIPAL STRESS RATIO

(粒子配向性と主応力比を考慮した土の力学異方性の評価)

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ABSTRACT

Soils existed in nature are commonly formed in many kinds of depositional manner because of the different effects from formation history. From this point, anisotropy, defined from initial soil particle's fabric and applied stress state to soil, is an important key properties of soil, play a crucial role in the designing and constructing the geotechnical structure. Profound understanding about anisotropy of soils can be used to solve many geotechnical engineering problems related to designing and constructing geotechnical structures.

The thesis describes the effects of anisotropy on behavior of both saturated and unsaturated soils. Triaxial apparatus incorporated local small strain measured technique and bender element test was used to investigate anisotropy on shear strength and shear modulus.

Inherent anisotropy is referred to the initial fabric resulting from sedimentation process. Under different deposition process, soils with different particle shape can create various fabrics. An inclined container with changeable its walls was used to prepare sand specimen at different deposition angles of 0°, 22.5^o, 45°, 67.5^o and 90°. Two specimen preparation methods namely air pluviation and dry vibration method was used to simulate different deposition processes in nature. Toyoura sand with different mica content of 0%, 1%, 2.5%, 5% and 10% was used as the testing materials presented as different shapes. Toyoura sand has the sub-angular shape while mica has the angular shape. Mica 5% and 10% were conducted at different void ratio to investigate effects of void ratio on anisotropy shear strength and shear modulus. A two dimensional optical microscope was used to evaluate the fabric structure of sands.

The test results suggest that the particle shape has the significant effects on the anisotropy behavior of soil. The shear strength of sand without mica content decreases when deposition angle changes from horizontal (0°) to vertical (90°) directions. However, the initial shear modulus increases with increase of deposition angle. The shear strength and the initial shear modulus become small with increase of mica content because of high compressibility of mica. The sand mixed mica sand shows the different behaviors from those of pure sand on the shear anisotropy. The anisotropy on shear strength vanishes because of particle breakage of mica. However, the degree of anisotropy on the initial shear modulus increases concomitantly the increase of mica content. These results suggested that the increase of angularity increase the degree of anisotropy on initial shear modulus. The test results of mica 5% and mica 10% at different void ratio demonstrated that the initial shear modulus decrease with increase of void ratio. However, the degree of anisotropy is independent from the changing of void ratio. On the comparison between saturated and unsaturated sands, unsaturated

sand produces a slightly higher degree of anisotropy on the initial shear modulus than saturated sand because of the application of matric suction. Two equations were suggested based on the measurement of particle orientation to assess the degree of inherent anisotropy.

Induced anisotropy is related to the soil structure created from an applied anisotropic stress state during ground formation. The influence of stress isotropic and anisotropy on the small strain shear modulus of cohesive soils was investigated using bender element test and monotonic compression test. Triaxial anisotropic consolidation tests were performed on Yoneyama silt collected from Niigata prefecture. The various value of ratio $K=\sigma_h/\sigma_v$ of 0.35, 0.43, 0.6, 0.8 (Compression anisotropy consolidation), 1.0 (isotropic consolidation), 1.5, 2.0, 3.0, 3.5 (extension anisotropy consolidation) calculated as the ratio between horizontal stress σ_h and vertical stress σ_v was used to define different stress stated applied during consolidation.

The obtained results on induced anisotropic topic show that the initial shear modulus of saturated soil clearly exhibits anisotropy as stress ratio K changes. On those results, shear moduli in extensional consolidation (K>1) is smaller than those of compressional condition (K<1). However, this kind of anisotropy is indistinct in the case of unsaturated sand.

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LIST OF SYMBOLS

α	Depositional angle
η	q/p’
θ	Averaged value of particle orientation
μ	Mean (average value)
Δt	Delay time in bender element test
σ_h; σ_3	Principal effective stress in horizontal direction
σ_v; σ_1	Principal effective stress in vertical direction
ε_a	Axial strain
ε_r	Radial strain
ε_s	Shear strain
ρ_t	Wet density
ρ_s	Dry density
A	Fitting parameter to calculate initial shear modulus.
AEV	Air entry value
B	Pore pressure coefficient
BE	Bender element test
COV	Coefficient of variation
E_h	Young modulus in horizontal plane
E_v	Young modulus in vertical plane
e	Void ratio
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
F (e)	Void ratio function
I_p	Plasticity inded
LSS	Local small strain test
K	Stress ratio; σ_h/σ_v
K_0	Stress ratio with Earth pressure at rest
L_{tt}	Travel length using tip-to-tip method in bender element test
q	Deviator stress
q_{max}	Maximum shear strength
p’	Effective stress used for saturated soil
p_a	Atmospheric pressure of 100kPa
p_{net}	Effective stress used for unsaturated soil
G_0	Initial shear modulus

G_{sec}	Secant shear modulus
G_{hh}	Shear stiffness on horizontal direction
G_{vh}	Shear stiffness on vertical direction
OCR	Over consolidation ratio
R	Roundness
R_s	Radius of curvature of the contractive skin or radius of the
	Maximum pore size
RSD	Relative standard deviation
r_i	Average radius of the curvature of surface features relative
r_{inner_max}	Diameter of the largest inscribed sphere of particle
r_{outer_min}	Diameter of the smallest circumscribed sphere of particle
S	Sphericity
s	Suction, u_a-u_w
SD	Standard deviator
T_s	Surface tension of contractive skin
Ua	Pore air pressure
Uw	Pore water pressure
V.M	Value of vector magnitude
V_s	Volume of sand
$V_{s(hh)}$	Shear wave velocity on horizontal direction
$V_{s(vh)}$	Shear wave velocity on vertical direction
V_w	Volume of water
W_L	Liquid limit

CHAPTER 1. INTRODUCTION

1.1. Background of research

Soils in nature commonly exists in many kind of particle shapes and formed in many kind of deposition manners. Behavior of soils has been demonstrated exhibit more or less degree of anisotropy, even applied the isotropic stress state. From this point, Casagrande and Carillo (1944) firstly defined two type of anisotropy in two components as follows.

(1) Inherent anisotropy develops from formation history through different sedimentation process. Due to the different particle shapes combined with another factors such as air, water or the folding process (uplift and subsidence), the soils structure was created in various initial fabric results from deposit process. The inherent anisotropy exhibits clearly in the soil with particle with weak cohesion. Particle shape and the mode of sedimentation play the crucial role for the inherent anisotropy (Oda, 1972; Hoque and Tatsuoka, 1998; Guo, 2008; Georgiannou, 2008; Tong, 2014; Kodicherla, 2018; Li et al., 2018)

(2) On the other hand, induced anisotropy produces in a soil when an applied anisotropic stress state during ground formation.

The soil depositional fabric is the mainly factor attributed to inherent anisotropy of sand, while the re-arrangement soil fabric or the displacement of the layered structure of clay minerals is attributed mainly for induced anisotropy of clay. The profound understanding about the anisotropy of soils becomes more and more necessary because of the demand on the increasing the geotechnical structures. The anisotropy affects deformation and strength characteristic of soils. Oda et al. (1978), Ochiai and Lade (1983), Wong and Arthur (1985) demonstrated that the shear strength is greater when the soil deposits in horizontal direction. At very small strain, the anisotropy affects to the initial shear modulus in both initial structure and stress state. Jovicic and Coop (1998), Gasparre et al. (2007), Hight et al. (2007); Teng et al. (2013) and Teachavorasinskun and Lukkanaprasit (2008) indicated that the ratio of shear modulus between horizontal and vertical direction was to be found unequal to 1.0.

The distinction between saturated and unsaturated is essential due to the different of their behaviors. The unsaturated soil usually has more than two phases (soil, water and air) while the saturated soil is included only two phases namely soil and water. The occurrence of unsaturated soil on geotechnical engineering is reported in many cases. Fredlund (1993) pointed out that almost soils nearby the surface of earth tends to be dehydrated, undergone the negative pressure and change the condition to unsaturated. Moreover, the soils are possible desaturation resulted from excavating, remolding and re-compacted. The research on unsaturated is necessary because of some reasons are as follows.

(1) The increasing on the building of the structure near the ground surface.

(2) The development on the research related to saturated soil has more remarkable achievements compared to unsaturated soil. As a results, results on researches of saturated are usually applied for unsaturated soi. However, some problems have been reported that only happen on unsaturated soil. For example, the collapsing of the structures constructed on unsaturated soil because of the dehydration process changing the mechanical properties of soils.

(3) Recently, the effects of global warming and climate change cause the increasing the areas of unsaturated on the surface of earth. The high plasticity soils, undergone the variation of environment, were producing a new material, which called as expansive soil. The expansive soil is produced when the high plasticity saturated soil is evaporated to change to unsaturated condition. The shrinkage of soil resulted from expansive process could bring many serious situations for the construction project such as inclination or collapse. Due to the lack of research related to unsaturated soil, the results obtained from saturated research have been usually used for the construction built on unsaturated soil. However, using those results, the questions on safety and economic during constructing and using remains a major challenge for both many researchers and commercial companies.

1.2. Scopes, object and limitation of this research

In this research, a triaxial apparatus with an integrated local small train measuring device and bender elements was used to measure the shear modulus and shear strength in terms of anisotropy. Sub-angular Toyoura sand mixed different flaky mica contents of 0%, 1%, 2.5%, 5% and 10% was used to investigation the effects of inherent anisotropy. While sandy silt namely Yoneyama was employ to investigate effects of induced anisotropy by applied different stress history during consolidation.

The conducted works of this research are summarized as follows.

(1) Topic related to inherent anisotropy

Shear strength and shear modulus of sands deposited at various angles α in two preparation methods were investigated in both saturated and unsaturated condition. Because of the relations between anisotropy and particle shape, particle characteristics included sphere *S* and roundness *R* was measured to explain for behavior of soil. The effects of particle breakage on anisotropy are also discussed.

During sedimentation, the particle orientation tends to arrange in direction that the longitudinal axis of particle perpendicular to direction of gravity. Qualifying the fabric structure of sand particle is carried out to obtain a quantitative evaluation of inherent anisotropy using a two dimensional optical microscope. Based on the obtained results from experiment and mathematic, this study proposed a new method to estimate the shear modulus using the particle orientation data.

(2) Topic related to induced anisotropy

Shear modulus of cohesive soil was investigated in terms of two mainly consolidation status namely anisotropy condition and isotropy condition. The stress ratio K defined as the ratio between horizontal effective stress σ'_3 and vertical effective stress σ_1' was used to created specimen at different stress state. Anisotropy consolidation was conducted at compression condition at K<1 (0.35, 0.43, 0.6, 0.8) and extension at K>1 (1.5, 2.0, 3.0, 3.5). K=1 was conducted for isotropy consolidation test.

Two series of triaxial test, incorporated local small strain test and bender element test, were conducted to investigate the effects of induced anisotropy on deformation characteristics as below:

(1) Specimen was consolidated under consolidation at anisotropic and isotropic condition at effective stress 300kPa. Then, bender element test and local small strain test were carried out to obtain the initial shear modulus. All of tests were conducted at both saturated and unsaturated soil

(2) Specimen was consolidated under consolidation at anisotropic and isotropic condition with the variation of effective stress 50kPa to 100kPa, 200kPa, 300kPa, 400kPa, 500kPa and 600kPa. Then, to investigate the effects of overconsolidation on initial shear modulus G_0, effective stress was decreased from 600kPa to 500kPa, 400kPa, 300kPa, 200kPa, 100kPa and 50kPa. The bender element test was conducted after effective stress reach the targeted value. This series of test was applied only for saturated soil.

1.3. Organization of this research

The thesis consists of nine chapters, which the detail content of each chapters can be as follows.

Chapter 1 Presents generally a background of anisotropy and the factors effects to anisotropy in geotechnical engineering. This chapter also brings an overview about the effects of anisotropy to shear strength and shear modulus in both saturated and unsaturated soil. The necessary of the conducting research on unsaturated soil, the scope and objective of this research are also introduced in this chapter.

Chapter 2 Presents a literature review of the factors caused the anisotropy. The effects of anisotropy on many aspects of geotechnical properties such as shear modulus, shear strength, bearing capacity, liquefaction of both saturated and unsaturated sand. The effects of some factors such as void ratio, particle shape, confining stress on shear modulus are also reviewed in this chapter.

Chapter 3 Describes the testing apparatus employed in this research. The working principle and calibration of devices was introduced in this chapter.

Chapter 4 Introduces the different testing materials employed for the topic related to inherent anisotropy. The advanced triaxial apparatus incorporated bender element test and local small

strain measured technique to obtain the initial shear modulus. This chapter also presents the testing method included specimen preparation methods, testing procedure for sands to investigate the effects of anisotropy of Toyoura sand at different mica contents on shear strength and shear modulus. This chapter highlights two testing preparation methods namely air pluviation and dry vibration to investigate the effects of different sediment process on inherent anisotropy. The procedure on measurement of particle orientation

Chapter 5 Describes the testing method and material used for cohesive soil to investigate the recent stress history on initial shear modulus. The method of production the induced anisotropy defined as the anisotropy stress state applied during consolidation is presented.

Chapter 6 Discusses the results of the inherent anisotropy on shear strength and shear modulus of sandy soil. The effects of void ratio on shear strength and shear modulus is also presented in this chapter. Moreover, some comparisons related to influence of mica contents, difference between bender element and local small strain method, difference between saturated and unsaturated are also presented in this chapter.

Chapter 7 Discusses the results obtained from induced anisotropy topic. The effects of anisotropy consolidation on shear modulus of both saturated and unsaturated soil is introduced. Moreover, effects of various stress state under different anisotropy and isotropy conditions.

Chapter 8 gives the summary and draws conclusions. Some recommendations and the future works of research are suggested.

CHAPTER 2. LITERATURE REVIEW

2.1. Definition of anisotropy

Anisotropy was distinguished into two main components: inherent anisotropy and induced anisotropy by Casagrande and Corrilo (1944). As illustration in Figure 2.1, the inherent anisotropy was considered as the fabric of soil particle formed from sedimentation process. The induced anisotropy was defined as the soil structure applied from anisotropy stress state during ground formation. While inherent anisotropy is attributed from particle shape and contact orientation developed during sedimentation, induced anisotropy is created from the difference of applied stress.

Figure 2.1 Definition two types of anisotropy in soil

2.2. Inherent anisotropy

Soils in nature are commonly existed in many kind of shapes and deposited in many different manners because of the different factors from the formation history. Under the effects of gravity, elongated particle deposited through air and water tends to align preferentially the longest axis perpendicular to the direction of gravity. Inherent anisotropy is actually reported to be appeared in both sand and cohesive soil.

Based on the definition about two types of anisotropy, the research related to inherent anisotropy normally investigation the effects of particle shape and the soil depositional process on the behavior of soil. Oda (1972) showed that shape of grains and depositional manner has the relationship to inherent anisotropy. In followed works, Oda et al. (1985) separated three

approaches the anisotropy of granular materials: (1) the orientation of each particle; (2) the distribution of individual particle to the other particles; (3) the orientation of voids space distributed in soil. Similarly, Stokoe et al., (1985), Mitchell and Soga (2005), Wang and Mok (2008) and Wang and Gao (2013).

2.2.1. Inherent anisotropy on shear strength

The inherent anisotropy on shear strength has been investigated in 1970s. Oda (1972) cut specimen at horizontal direction and vertical direction and demonstrated that the shear strength decrease when deposited direction changes from zero to 90°. Author and Menzies (1972), Guo (2008), Rodriguez and Lade (2014), Tong et al. (2014), and Le at al. (2018) reveal that inherent anisotropy plays as a function of shear strength in non-cohesive soil.

2.2.2. Inherent anisotropy on shear modulus

The ratio between shear stiffness on horizontal direction (G_{hh}) and that value on vertical direction (G_{vh}) is the most common parameter to investigate the effect of soil fabric. Normally, the G_{hh} and G_{vh} are measured by means of $V_{s(hh)}$ and $V_{s(vh)}$ as the concept illustrated in Figure 2.2. Due to the longitudinal axis of particle tends to arrange into horizontal direction, which induces the ratio G_{hh}/G_{vh} >1.0 (Chang et al., 1989; Lo Presti and O'Neill, 1991; Bellotti et al., 1996; Pennington et al., 1997; Jovičić et al., 1998; Fioravante and Capoferri, 2001; Santamarina et al., 2001; Chaudhary et al., 2004; Gasparre et al., 2007; Wang and Mok., 2008, Escribano and Nash, 2015).

Figure 2.2 Bender elements installed in triaxial specimen and different types of shear waves applied (after Pennington *et al*., 1997)

2.2.3. Inherent anisotropic on other aspects of geotechnical engineering

Inherent anisotropy shows the significant on the other aspects of geotechnical engineering. Yu et al. (2013) presented that dry soil fabric deposited at different angles of 0°, 45° and 90° did not show influence on the acceleration during earthquake. However, at the saturation condition, the soil deposited at 90° shows the most vulnerable on the liquefaction and settlement and the particle alignment in the 0° contributed the highest liquefaction resistance. Ueda et al. (2019) used a geotechnical centrifuge modelling with 300mm high and found that the inherent anisotropy did not show effects on the response acceleration at the lower part. However, the influence shows clear influence on the near-surface of centrifuge. Moreover, the settlement induced by liquefaction at 90° was greater two times than that at 0°. Similarly, anisotropy on liquefaction behavior or seismic resonse has been reported by Mitchell et al. (1976), Mulilis et al. (1977), Seed at al. (1983), Qin et al. (2016) and Qin et al. (2019).

Considering the effects of inherent anisotropy on bearing capacity, Li et al. (2011) used a scaled strip foundation place on the ground with soils deposited at different directions (Figure 2.2). Their observation demonstrated that the soil deposited at 90° obtains a greater influence to the response of the ground and structure displacement, acceleration and excess pore pressure compared to soil deposited at 0°. As a result, they recommended that the initial fabric of soil should be considered in the design phase. Oda and Koishikawa (1979), Azami et al. (2010) and Chaloulos et al. (2019) also presented that a sand with a horizontal deposition plan show a smaller footing settlement, and higher bearing capacity than that of vertical deposition.

Figure 2.3 Centrifuge model for fabric anisotropy
(a) soil deposit and (b) model of strip foundation, Li et al. (2011)

2.3. Induced anisotropy

Effects of induced anisotropy on soil has been commonly conducted by applied different stress state history on the small strain. The term "different stress state" can be considered from different stress on horizontal direction σ_h and vertical direction σ_v. It can be defined as the coefficient $K = \sigma_h/\sigma_v$ or stress ratio q/p'. Natural clay is normally deposited in vertical direction under gravity and generally consolidated under K_0 condition. Figure 2.3 implies that even deposited in K_0 consolidation, applied stress-state in soil are commonly anisotropic.

The changing of stress state resulted from tectonic movement, landslide, excavation in foundation and tunneling constructing, the ratio K can be different from K_0. This natural or artificial processes resulted the loading or unloading turn the clay to re-consolidation condition, which change the initial consolidation state. (Finno et al., 1992; Bozzano et al., (2014); Shi et al., 2015; Cai et al., 2018). Kamei (1996) illustrated (Figure 2.4) that the adding of footing or removing the soil by excavating can change the stress state on K_0-consolidation compression loading (a) or extension loading (b). Considering to the stability problems, the compression loading and extension loading are corresponded respectively on the active zone and passive zone.

Figure 2.4 Explanation of K_0 consolidation

Figure 2.5 Typical stress system for a normally consolidated cohesive soil in the filed, Kamei. (1999).

2.3.1. Induced anisotropy on shear strength

The effects of induced anisotropy on shear strength of cohesive soil have been normally investigated in the aspects of undrained shear strength. Hansen and Gibson (1949) was firstly introduced the stress induced anisotropy of undrained shear strength c_u by developed a number of theoretical expressions for the variation of c_u. Next, Bishop (1966), Duncan and Seed (1966a and 1966b) and Nakase and Kamei (1983) concluded almost the same that shear strength obtained from compression and extension test increases with decrease of plastic index I_p. Similarly, Bjerrum (1972 and 1973) proved that shear strength is depended by stress condition induced by extension and compression in triaxial test.

The initial stress state on the undrained shear strength has been revealed that the anisotropy strength might be higher, equal or smaller than isotropic shear strength. Mayne (1985) suggested that the anisotropic shear strength, on the average, are about 87% and 60% of the isotropic strength for compression and extension, respectively. Cai et al (2018) demonstrated that the increment of $K=\sigma'_1/\sigma'_3$ leads to an increment of the shear strength and the decrease of negative pore water pressure. Similar tendency was observed by Kantesaria and Sachan (2020), On the other side, Toyota et al. (2014) suggested a $K=\sigma'_3/\sigma'_1$ consolidation process on Yoneyama silty clay and obtained a greater undrained shear strength in the case of smaller K. Shi et al. (2015) proved that the volumetric strain during anisotropic consolidation increases with the decrease of $K=\sigma'_3/\sigma'_1$. Xu et al. (2019) indicated that the decrease of K results the increase

of maximum deviator stress.

In the case of sand, Kato et al. (2011) pointed out that undrained shear strength of anisotropically consolidation soil (K=0.33) at the shear strain less than 5% exhibited the softening behavior compared to monotonic increase in stress of isotropically consolidation soil. However, at strain exceeded 10%, both of two type consolidation sand showed the same behavior on stress-strain and excess pore pressure.

2.3.2. Induced anisotropy on shear modulus

Atkinson (1993) classified the range of soil into three stages: very small strain (less than 0.001%), small strain (strain from 0.001% to 1%) and large strain (more than 1%).

Soil behavior is highly non-linear and has influence on selection parameter for designing and constructing the construction in geotechnical field. However, at small shear strain less than 0.001%, soil is assumed that it behaves elastically and linearly. The initial shear modulus, represented for elastic behavior area of soil and usually denoted as G_0, is a very important parameter to settle many geotechnical engineering problems such as designing and constructing the underground structure, predicting the deformation of ground caused by settlement or earthquake. G_0 is commonly determined through field seismic measurement of shear wave velocity V_s or estimated indirectly by correction results of shear strength obtained from penetration test. In laboratory test, some methods have been employed to obtained G_0 such as bender element test (Viggiani and Atkinson, 2005 Jamiolkowski *et al*., 2005), resonant column tests (Iwasaki et al., 1978; Kumar et al., 2007; Khan et al., 2008), small strain triaxial tests (Hoque and Tasuoka, 1998), and and small strain torsional shear tests (Iwasaki et al., 1978).

In early work, Hardin and Richart (1963) and Hardin and Drnevich (1972) suggested an empirical expression for G_0, accounted for the variation by effective stress, void ratio at the isotropic stress state, as follows:

$$G_{0(isotropic)} = AF(e)\left(\frac{\sigma_m^{'}}{p_a}\right)^n \qquad (2.1)$$

Where p_a is considered as the reference pressure and usually used the value of the atmospheric pressure of 100kPa. *A* and *n* are fitting parameter varied by soil types; *F(e)* is the function of void ration *e*. Equation (2.1) implies that the initial shear modulus relates to effective stress and packing density. However, the influence of anisotropy on G_0 is neglected.

An empirical expression for determination of G_0 was developed from equation (2.1) and proposed a relation among different factors to the shear modulus by Hardin and Blandford (1989), Jamiolkowski et al (1995) and Stokoe et al. (1999), as followed:

$$G_0 = S_{ij}.F(e).OCR^k.\sigma_r^{1-ni-nj}.\sigma'^{ni}.\sigma'^{nj}(MPa) \qquad (2.2)$$

Where

- S_{ij} is the dimensionless coefficient for a given soil. Hardin and Blandford (1989) proposed that S_{ij} referred to the anisotropic of a reference fabric and to be constant with changing of stress state. Mitchell and Soga (2005) suggested that S_{ij} is a material constant.
- *F(e)* is the function of void ratio.
- Parameter *i* and *j* are denoted as the direction of the wave propagation/polarization or applied stress direction, respectively.
- *OCR* is the overconsolidation ratio and accounts for some effects stress history.
- *k* is an empirical stress exponent depended on plasticity index as the suggestion of in
- n_i *and* n_j is the empirical coefficient that accounts for the effect of the principal effective stress in vertical and horizontal direction, respectively. Stokoe *et al*. (1985), Hardin and Blandford (1989) and Bellotti *et al*. (1996) reported that n_i and n_j are both equal to 0.25. Roesler (1979) found that although n_i is smaller than n_j, but the $n_i + n_j$ was equal to 0.5.

As shown in equation (2.2), anisotropy is considered as an important parameter affected to initial shear modulus. Triaxial embedded bender element seems to the most common apparatus which used to investigate the effects of anisotropy on shear modulus. As mention above (section 2.1.2), the shear modulus obtained from horizontal direction is higher than that result measured from vertical direction. Considering to induced anisotropy, many equations were proposed to improve the equation (2.1). Yu et al. (1984) pointed out the importance of stress ratio $K=\sigma'_1/\sigma'_3$ on the expression, as below.

$$G_0 = CP_a^{0.49}\sigma_a^{0.26}\sigma_p^{0.25}\left(1-0.18K_n^2\right) \qquad (2.3)$$

Where K_n is the normalized stress ratio and calculated as the ratio in equation (2.4). C is the factor following to Hardin's equation, σ_a and σ_p are the stress along the direction of shear wave propagation of shear wave transmitter and the stress along the direction of soil grain vibration, respectively.

$$K_n = \frac{\left(\frac{\sigma_1}{\sigma_3}-1\right)}{\left(\left(\frac{\sigma_1}{\sigma_3}\right)_{\max}-1\right)} \qquad (2.5)$$

Jafarian et al. (2018) suggested that the initial shear modulus on anisotropic is always greater than that result at isotropic condition, as followed expression.

$$G_{0(anisotropy)} = \left[\frac{3K^{0.589}}{2+K}\right] G_{0(isotropy)} \tag{2.6}$$

Where $G_{0(isotropy)}$ is the initial shear modulus at isotropy condition mentioned in expression 2.1. $K=\sigma'_1/\sigma'_3$ is stress ratio and varied from 1.0 to 2.8.

Rampello et al. (1997) revealed that the small-strain shear modulus is higher on anisotropic soil than that on isotropic consolidation. Mitaritona et al. (2013) considered the effects of both inherent anisotropy and induced anisotropy by investigating the ratio G_{hh}/G_{vh} under different stress-path $\eta=q/p'$ ranged from $\eta = 0$ (isotropic) to $\eta \neq 0$ (anisotropy). They demonstrated that clay shows the ratio G_{hh}/G_{vh} increases from 1.0 at isotropic condition to 1.2 at $\eta = 0.8$. Jafarian et al. (2018) conducted the resonant column and cyclic loading test on at both isotropic and anisotropic under extension condition and found that G_0 increase with the increase of K from 1.0 to 2.8, as shown in Figure 2.7. Similar tendency was observed by Chien and Oh (2002), Hao and Lok (2008).

Figure 2.6 Effects of stress anisotropy on initial shear modulus, Jafarian et al. (2018)

CHAPTER 3. TESTING APPARATUS

3.1. Introduction

This chapter describes the testing apparatus used in this research. The triaxial apparatus was employed to investigate the anisotropy on shear strength and shear modulus. Shear strength of soil can be measured using conventional triaxial test. Initial shear modulus G_0 was obtained from two the testing apparatus with an integrated local small strain (LSS) measuring device and bender elements (BEs). Unsaturated test was carried out using the axis-translation technique using ceramic disc. All of the tests were carried out in the same and stable temperature about 20°C ± 1°C.

3.2. Testing apparatus

3.2.1. General view

Laboratory tests on both saturated and unsaturated soils were conducted in a triaxial apparatus with an integrated local small strain (LSS) measuring device and bender elements (BE), as illustrated in Figure 3.1. To reduce the effects of friction when applying the shearing, the load cell was install inside the cell acrylic.

The outstanding feature of this testing apparatus are described as below:

- The local deformation of specimen can be measured directly using the local small strain devices
- The global volume changes via drained volume using the volumeter or volume change measurement devices (section 3.5.5)
- Bender element was mounted in both top cap and pedestal to measure initial shear modulus
- A ceramic was mounted in pedestal to carry out the unsaturated test.

Shear strength test was carried out using the conventional triaxial apparatus without LSS devices. However, when measuring the shear strength of unsaturated soil, the volume change was measured using the LSS devices.

The triaxial apparatus consists five main components: support frame or frame, deformation measured devices, stress control system, drained control system and data acquisition system.

Support frame is used to hold the gap sensor in local small strain test. The size of specimen used was about 50mm in diameter and 125mm in height. Specimen is contacted directly to top-cap (on the top) and pedestal (on the bottom) during carrying out test time.

Deformation measured system mentioned here included three devices: local small strain measured devices or the proximity transducers, dial gauge or external displacement gauge and volumeter. The overall axial displacement of specimen is measured by dial gauge mounted externally on triaxial cell and operated by electrical motor, whereas the proximity transducers

mounted internally measured the local axial strain and radial strain and controlled by setting up

Figure 3.1 Schematic diagram of triaxial apparatus.

in computer program. The proximity transducers are also considered as the volume changed measurement system. Volume change can be determined via the direct measurement of specimen dimension. Volume change also can be measured by using the volumeter, monitored through the pore water volume change, which recorded by gap sensor.

Table 3.1 Properties of drained measurement device

Devices	Type of gauge	Capacity	Accuracy level
External displacement	Dial gauge	20mm	0.001mm
Axial force	Load cell	500N/2000N (1)	0.125N
Pore pressure and cell pressure	Water pressure	1MPa	0.3kPa
Drained measured devices	Non-contact sensor	60cm^3	0.001cm^3

(1): The load cell of 500N was used in LSS test of sand. It has the higher sensitive level than load cell of 2000N.

3.2.2. Overview of measurement and control system

The following control system was developed and assembled to improve the accuracy and reduce the error during testing.

(a) Measurement system

The data was measured automatically by connecting a CONTECT GP (IEEE-488) interface board for DOS/V and a data logging system UCAM manufactured by Kyowa Dengyo Co., Ltd. All of the test were automatically controlled by computer program namely *Tripwin 8*. This program was created by Toyota Hirofumi in Nagaoka University of Technology.

(b) Control system

The control axial load, cell pressure and back pressure were performed from CONTEC A/D conversion board DOS/V. The axial load was controlled using a stepping motor and a clutch. The cell pressure and back pressure were controlled by an electric regulator.

(c) Stepper motor

A steeper motor, played the role as the control system, was used to measure the small strain with high accuracy. A stepping motor is a synchronous electronic motor that operates in a synchronization with pulse powered. It is also called by “pulse motor”. With the simple circuit configuration, this motor controls the rotation control (rotation angle, rotation speed) of the motor with the number of pulses at a fixed angle (basic step angle) and its frequency. The stepping motor rotates by a certain angle each time a pulse is sent. “Step angle” is used as the term to call this rotation angle. The higher the resolution is corresponded to the smaller step angle. Figure 3.2 shows the relationship between the pulse and the step angle. The motor advances α degrees each time one pulse is sent. It can be seen that if 3 pulses are sent, the motor advances by 3α degrees

Figure 3.2 Description of pulse motor

MOTOR DRIVE PRESSURE REGULATOR

Figure 3.3 Schematic of electric regulator

Figure 3.4 Pictures of front side and back side of electric regulator

(d) Electric regulator

The cell pressure and back pressure were controlled by an electric regulator. As illustrated in Figure 3.1, the pressure was connected to the compressor using plastic tubes. On the detail description, Figure 3.3 and 3.4 show the electric regulator. This electric regulator controls ON / OFF only, and the speed is controlled manually.

3.2.3. Local small strain measurements device

(a) Properties and principle working

Figure 3.5 describes schematic diagram of the local small strain (LSS) who intergrated into triaxial apparatus. Two separated targets were glue directly on the membrane to measure the vertical axial strain. A couple of proximity transducers were fastened on two columns near specimen so as not to contact to target. The distance between target and proximity transducer can be adjusted from outside of triaxial cell and the distance between 2 target was about 80mm. Another proximity producer was installed in the middle part of the specimen using a clamping device.

These sensors are worked based on the eddy current principle. As illustrated in Figure 3.6, a

high frequency current flowed through the coil inside the sensor head generates a frequency magnetic field. If there is a measurement object (metal) in this magnetic field, the magnetic flux causes the passage of magnetic flux and the eddy current in the vertical direction to flow on the object surface, and the impedance of the sensor coil changes. Using this principle, the distance is measured by the change of the oscillation state. This is based on the fact that the amplitude and phase change when the distance between the metal target and the sensor changes (black part). Figure 3.7 shows picture on the small strain devices installed on the testing apparatus during testing.

Figure 3.5 Schematic of triaxial apparatus

Figure 3.6 Schematic of principle working of gap sensor

a) Axial strain measured devices

b) Radial strain measurement devices

Figure 3.7 Pictures of small strain measurement devices

(a) Axial strain gap sensor and (b) Radial strain gap sensor

Figure 3.8 Description of reliable gauge for calibration of gap sensor

(b) Calibration

In order to reach a precise test, calibration of measurement and operation check of equipment are conducted in advance. The calibration of measurement is necessary to evaluate and improve the accuracy and reliability of measured devices. As shown in Figure 3.8, a special calibration device was designed and fabricated to simulate the measurement the strain during testing. The calibration was performed by changing the distance between the metal target and non-contact sensor. The micrometer is used as the reliable gauge for changed distance reference. If the displacement of value of gap sensor show the linear relationship to voltage, it can be said that the accuracy of the sensor is high.

Figure 3.9 (a) and (b), respectively, present an example about the calibrated results on gap sensor need to be modified and after modifying. The step for each measured time is 0.05mm. The calibration result is more accuracy with the smaller step for each time adjusted. On the gap sensor before modifying, the red points implied the ideally measured level of sensor, which the displacement and the voltage are linear relationship. However, the green points show the scatter relation between the displacement and voltage. Therefore, this sensor need to be modify to obtain a higher accuracy, as shown in Figure 6.17(b).

Figure 3.9 An example of gap sensor calibration

(a) Before modifying gap sensor and (b) After modifying gap sensor

3.2.4. Bender element

(a) Properties and principle working

In laboratory test, the shear wave velocity using bender elements is employed to measure stiffness at very small strain (Atkinson, 1993). Bender element (BE) has been commonly used to identify the initial shear modulus G_0 from shear wave velocity V_s. Bender element consists a pair of piezo electronic transducers working based on the principle: when exciting an electrical source with a small voltage, this electrical energy is converted to mechanical energy at the "transmitter". This wave signal is propagated through specimen and detected by the "receiver" and converted again to electrical energy. The time different between the transmitted and received wave considered as the propagation time Δt of shear wave velocity V_s. The initial shear modulus can be obtained based on the equation as followed.

$$G_o = \rho_t V_s^2 = \rho_t \frac{L_{tt}^2}{\Delta t^2} \tag{3.1}$$

Where shear wave velocity can be estimated via the travel distance L_{tt} and travel time Δt.

In this research, the BEs with the dimension of 12mm length, 2.5mm height, and 1mm thickness was mounted on the top cap (works at the wave transmitter) and the pedestal (as the receiver). A computer is used to record data in both transmitter element and receiver element. Figure 3.10 depicts the schematic diagram of BE employed to the triaxial apparatus. Figure 3.11 shows the detail devices of BE and can be described as follows.

- I-a Function generator: Plays role to create the electrical energy, which sending the transmitter wave. The properties of function generator are shown in Table 3.2
- I-b Top cap: Mounted bender element part, which converted electrical to mechanical energy.
- II-a Pedestal: Mounted bender element part, which converted mechanical energy to electrical signal
- II-b Data logger: Record the receiver wave and then transfer to computer. The properties of data logger are shown in Table 3.3.
- III Computer: Use to record both transmitted wave and received wave signal.

Table 3.2 Properties of function generator

Model name	WF1943B	Function	
Oscillation frequency range	0.01μHz-15MHz	Continuous oscillation	available
Number of channel	1	Sweep	Frequency/phase
Waveform weight resolution	14bit	Inter modulation	FM, PM
Frequency setting	0.01μHz	Inter addition	Not available

resolution			
Maximum output voltage	20Vp-p/; 10Vp-p/50Ω	User unit function	Available
Power supply	AC100/115/230V	LOAD	Available
Power consumption	Lower than 65VA	memory	Available
External dimension	216mm (W) x 132.5m(H) x 290 (L)		

Table 3.3 Properties of data logger

Model name	SL1000	Module	
Range of Operated temperature	5°C - 40 °C	Maximum sample rate	1MS/s
Power supply	100-120VAC/220-240VAC 50/60Hz	Resolution	16bit
External dimension	319mm(W)×154mm(H) ×350mm(D)	Band	300kHz
		Number of channel	2

(b) Calibration

Calibration for BE devices is conducted to obtain to determine the delay time induced by the alignment of the polarities of transmitter and receiver as suggestion of Kuwano et al., (1998) and Yamashita et al., (2009). As presented in Figure 3.12, the calibration was conducted by putting the BE transmitter (CAP) and BE receiver (pedestal) contacted directly. Figure 3.13 shows an example of the delay time Δt of the entire measurement apparatus.

Figure 3.10 Schematic of bender element

Figure 3.11 Pictures of bender element

Figure 3.12 Calibration of Δt of bender element

Figure 3.13 Explanation of Δt in bender element calibration

3.2.5. Drained volume changed measurements

(a) Properties and principle working.

Figure 3.14 (a), (b), and (c) present a schematic diagram and photograph of the drained volume changed measurement device. This device is included a floating metal target and a gap sensor. The principle working is the same to the proximity transducers devices, which using the eddy current working. During testing, Because of cell pressure (isotropic consolidation) or shearing, the amount of degassed water in specimen is changed and led to the increase or decrease the volume of degassed water in device. The fluctuation of amount of water changes the distance between the floating target and sensor.

(a) Schematic of drained measure system

(b) General view

(c) Measurement amplifier

Figure 3.14 Volume changed measurement devices

(b) Calibration

A reliable gauge is used to confirm the accuracy of the drained volume measure device. As shown in Figure 3.15, reliable gauge (burette) with full of degassed water is connected directly to this device. The system is connected to a degassed water tank (see Figure 3.1). Before conducting calibration, the water is provided to degassed water tank and removed air using the pump. Next, the degassed water is poured to the volumeter and the tube connected to burret. By opening the vale, a specified amount of water is flown to this device and changed the distance between metal target and sensor. The changed volume is recorded by gap sensor and burrete gauge in voltage unit and cc unit respectively.

Figure 3.16 presents a typical example of calibration on two time: (a) is corresponded in case of providing the degassed water from device to burette and (b) from burette to device. It can be seen that the volume change measured by volumeter and burrete is linear, implied a good working condition of gap sensor.

Figure 3.15 Calibration of volume changed measurement device

Figure 3.16 An example on the calibration results of volume change measurement device

3.2.6. Ceramic disc

(a) Principle

A ceramic disk has a small pores of relatively uniform size and it was usually glued by epoxy resin around pedestal to prevent the air leakage. The disk, usually play a role as a membrane between air and water as illustration in Fig. 3.17. In unsaturated test, the high air entry porous

disk is necessary to separate the pore – air and pore water so that different pressure can be kept. Air cannot pass through the disk due to the ability of the contractile skin to resist the flow of air.

The ability of the ceramic disk to withstand the flow of air result from the surface tension, T_s developed by the contractile skin. The contractile skin acts like a thin membrane joining the small pores of radius, R_s, on the surface of ceramic disk. The difference between the air pressure above the contractile skin and the water pressure below the contractile skin is defined as the matric suction. The maximum matric suction could be maintained on the surface of disk is considered as Air Entry Value (AEV) as shown in Fig. 3.17.

The AEV of ceramic disk can be indicated by Kelvin's equation as:

$$\left(u_a - u_w\right)_d = \frac{T_s}{R_s} \tag{3.2}$$

Where:

$(u_a - u_w)_d$: Air Entry Value of Ceramic disk

T_s: Surface tension of the contractile skin (T_s = 72.75 mN/m at 20°C)

R_s: Radius of curvature of the contractile skin or radius of the maximum pore size.

The AEV of ceramic disk is mainly controlled by the radius Rs of largest pore in disk. The equation 3.1.1. shows that the pore size in disk is smaller, the AEV is greater.

Figure 3.17 Principle working of ceramic disc

(b) Air entry value of ceramic disc

Air entry value (AEV) of the ceramic disc generally represented the maximum capacity when suction applied into the saturated porous. Therefore, AEV should be checked before triaxial test to evaluate the capacity of ceramic disc when pressure applied during drying step.The pedestal included a ceramic disc was kept in drying at 40°C for at least one day to eliminate the water in its pore. After that, the ceramics disc was saturated using the same method mention in Section 4. 22. Then, a specified pressure is applied above the pressure for at least 2 hours. The schematic of this process is illustrated in Figure 3.18. As shown in Figure 3.19, AEV is defined as the value where the drained value is increased strongly under application of matric suction.

Figure 3.18 Schematic of AEV checking

Figure 3.19 Air entry value of ceramic disc

3.2.7. Microscope device

In this study, a microscope device was used to quantify the orientation of individual particle deposited from different sediment process. The properties of this device is shown in Table 3.4. The schematic is shown in Figure 3.20.

Table 3.4 Description of microscope devices

Model	DINOAM4815ZT	Focal length rate	30fps
External Dimension	105mm(H)×32mm(D)	sensor	1.3Mpixels，1/4inch CMOS
Weight	104g	Correction	Automatic zoom/ Automatic correction
Resolution	1280×1024, 130milion pixels	Magnification ability	20～220 times

Figure 3.20 2-D optical microscope devices for particle orientation evaluation

CHAPTER 4. TESTING METHOD OF SANDY SOIL

4.1. Introduction

This chapter mainly describes the method for making specimen and testing procedure applied on sands. Toyoura sand represented as sub-angular shape was mixed mica presented as angular shape at different contents of 0%, 1%, 2.5%, 5%, 10%. A cylindrical specimen with 125mm height and 50mm diameter was shaped from a block soil. This block soil was created at different depositional angle α of 0°, 22.5°, 45°, 67.5° and 90° using a container with a movable wall. Using this container, the sand was deposited in different angle, referred to the inherent anisotropy. The testing methods namely air pluviation and dry vibration was used to make specimen. To investigate the soil fabric related to deposit angle of sand particle, a 2-D optical microscope was used to estimate the particle orientation. This conducted work was applied only on Toyoura sand using air pluviation and dry vibration method.

The testing method for cohesive soil are described in chapter 5.

4.2. Testing materials

4.2.1. Toyoura sand

Toyoura sand, which is Japanese standard sand, was used as one of the testing material in this research. It is collected from Yamaguchi prefecture, Japan. As shown in Figure 4.1, Toyoura sand is a sub-angular to angular poorly graded fine quartz rich sand. The sand particle is somewhat flat and elongated because the length (*L*) to with (*W*) ratio *L/W* distributes from 1.0 to 1.6 and the average value of *L/W* is about 1.5, as shown in Figure 4.2.

4.2.2. Mica

Mica is a natural material. Mica commonly contains muscovite existed abundantly in metamorphic rock and biotie appeared in igneous, metamorphic and sedimentation rock. Mica is reported as one of the most common mineral species at the earth's surface (Gidigasu, 1976). Mica has a flaky particle shape, as shown in Figure 4.3.

In this study, to create "mica sand", mica was added to Toyoura sand with different proportions of 0%, 1%, 2.5%, 5% and 10% by dry weight as calculated in equation 4.1.

$$mica(\%) = \frac{weight_of_\text{mica}}{total_weight} \tag{4.1}$$

The presence of platy mica particles in nature and its effects on soils' mechanical behavior have been discussed in many studies. Basically, the present of mica in soil causes the negative influence and give many challenges for road construction, foundation and earth dams. Soil has been proved that optimum moisture content, compressibility, plastic limit and frost susceptibility increase, whereas compatibility plastic index, bearing capacity and shear strength decrease

(McCarthy and Leornard, 1963; Tubey and Webster, 1978; Harris et al., 1984; Zang et al., 2019). If soils contain with mica more than 10%, it is not recommended for use in pavement layers (Tubey, 1964). Ishihara (2008) and Langlord et al. (2010) demonstrated that mica is related to the slope slide in Bangladesh and Norway, respectively. Okamura et al. (2015) observed that mica accounts for 20% of the soil's minerals in Manamaiju, Nepal, where soil was liquefied during earthquake.

In recent experimental works, Lee et al. (2007) demonstrated that the shear wave velocity and shear strength decrease with increase of mica content. Georgiannou (2008) proved that when using different sample preparation methods in the same 2.5% micaceous sand, undrained and drained strengths of sands differ because of the anisotropic structures of the sand grains. Schmidt (2008) carried out cyclic triaxial tests on mica-contained sand and concluded that the liquefaction resistance is more vulnerable with increase of mica content.

Figure 4.1 Scanning optical microscope view of Toyoura sand

Figure 4.2 Grain shape ratio distribution between length and width of Toyoura sand

Figure 4.3 Scanning optical microscope view of mica

4.2.3. Particle breakage

(a) Particle breakage in previous research

Lee and Farhoomand (1967), Hardin (1985), Murphy (1970), Billiam (1972) demonstrated that there are several factors affected the amount of particle breakage such as particle size, particle angularity, particle gradation and stress magnitude. Particle breakage tends to be increased with the increase of particle size because larger particle size contains more flaws or defects. Particle breakage is also increased with the increase of particle angularity. The higher angular particle more easily breaks due to the fact that applied stresses can concentrate at angular contact point of particle. Lade et al. (1996), McDowell et al. (1996), Ueng and Chen (2000), Hyodo et al. (2002), Yao et al. (2008) and Kikumoto et al. (2010) showed that the phenomenon of particle breakage has been observed in many previous research and revealed to have a significant influence of the soil behavior.

From those effects, a number of indexes has been suggested to evaluate the particle breakage. Lee and Farhoomand (1967), Marsal (1967), Lade et al. (1996), Nakata et al. (1999), Miura et al (2003) defined the particle breakage index based on the difference of the particle gradation before and after testing. Nakata et al. (2001) used the area change of soil for particle breakage index. Mandelbrot (1983), Turcotte (1986), Tyler and Wheatcraft (1992) and McDowell and Bolton (1998) introduced the fractal theory to define the particle breakage index.

(b) Evaluation particle breakage for Toyoura sand and mica

Sieving test were performed to confirm the occurrence of particle breakage before and after triaxial testing. Because of the difficulties on separation pure mica from mica sand after mixing for triaxial testing, triaxial test employed mica 100% was conducted using the same testing materials as mica sand (mentioned in chapter 4) to investigate the present of mica particle crushing. Test was conducted at effective stress of 150 kPa and completed at deviator stress of 100kPa and 700kPa for mica 100% and Toyoura sand, respectively. After finishing triaxial test, mica 100% was collected and kept drying. Next, the sieving test was conducted to compare the particle distribution curve.

Figure 4.4 shows the comparison on particle size distribution curves of Toyoura sand before and after testing. It can be seen that the crushability of mica occurred considerably at high deviator stress of 100kPa. However, the particle size distribution curve of Toyoura sand of applied stress 700kPa is almost the same to that of Toyoura sand without triaxial testing, implied that the maintenance of particle shape of Toyoura sand.

Figure 4.4 Particle gradation curve before and after testing of Toyoura sand mica

4.2.4. Particle shape characteristic for Toyoura sand and mica

(a) Sphericity and roundness

The parameter sphericity *S* and roundness *R* are used to characterize the particle shape of Toyoura sand and mica. Sphericity *S* is considered as the overall morphology of particle and related to length, height and width of particle and calculated by the ratio between two surface areas of the given particle and the sphere with the same volume. Roundness *R* refers to the angularity and roughness mentions the condition of the surface texture of particle. The sphericity *S* and roundness *R* of Toyoura sand and mica can be estimated by using the same method that widely employed in previous studies (Mitchell and Soga, 2005; Cho et al., 2006; Guo and Su, 2007). Sphericity *S* is qualified as the ratio of "diameter of the largest inscribed sphere" *(r_{inner_max})* to the "diameter of the smallest circumscribed sphere" *(router_min)* as equation (4.2).

$$S = \frac{r_{inner_\max}}{r_{outer_\min}} \tag{4.2}$$

Roundness *R* is calculated as the relationship between "the average radius of the curvature of surface features relative "(r_i) among the number of inscribing circles *N* and "the radius of maximum sphere that can be inscribed in the particle" as described in equation (4.3)

$$R = \frac{\sum r_i / N}{r_{\text{inner_max}}} \tag{4.3}$$

The values of *S* and *R*, which calculated, which are calculated from 2-D scanning optical

microscope images and particle shape analysis, are presented in Table 4.1 from about 50 individual particles for sand and mica. Value of sphericity *S* (0.702) and roughness *R* (0.697) of Toyoura sand were higher than those results of mica (*S* =0.598; *R* = 0.697). These values imply that particle shape of mica is more angular than Toyoura sand. Moreover, maximum void ratio e_{max} and minimum void ratio e_{min} obtained by JGS 0161-2009 and value $e_{max}-e_{min}$ increase with the increase of mica content. Yoshimura and Ogawa (1993), Miura et al. (1997), Cho et al. (2006) and Altuhafi et al. (2016) demonstrated that the void ratio increase with the when the particle shape becomes more angular.

Table 4.1. Physical properties of testing materials

	Toyoura sand	Mica	Mica 1%	Mica 2.5%	Mica 5%	Mica 10%
Specific gravity, ρ_s (g/cm3)	2.650	2.865	2.650	2.653	2.660	2.670
Maximum void ratio, e_{max}	0.990	-	1.036	1.165	1.284	1.599
Minimum void ratio, e_{min}	0.597	-	0.61	0.721	0.728	0.812
e_{max} - e_{min}	0.393	-	0.426	0.444	0.556	0.787
Sphericity, *S*	0.702	0.598	-	-	-	-
Roundness, *R*	0.697	0.536	-	-	-	-

(b) Thickness

Regarding to the measurement on the thickness of particle, the thickness particle of Toyoura sand can be estimated from the particle gradation test because of the uniform shape of Toyoura sand in three-dimensional directions. However, mica is a flat particle. Therefore, it is necessary to estimate the thickness of mica in another method. This study used an electron microscope to measure the thickness of mica. Figure 4.5 shows the measurement results in which the thickness of mica ranges somewhere from 4.0μm to 33.09μm. These thickness values mention that mica is very flat compared to Toyoura sand.

4.3. Specimen preparation

4.3.1. Specimen preparation methods in previous research

Specimen preparation method influences the orientation and fabric of soil particles. Thus, the specimen preparation method plays a crucial role affected the mechanical properties of sand in various aspects. Many methods were employed to remold and reconstitute sand specimen for laboratory testing such as dry/water pluviation, moist/wet tamping method. Different methods are considered as the different soil fabrics, which affected to the behavior of soil (Ladd, 1974; Mulilis et al., 1977; Vaid et al., 1999). Li et al. (2018) showed that different specimen preparation methods resulted in different liquefaction resistance. Yang et al. (2008) found that the specimen

deposited under dry vibration method more anisotropic structure than specimen prepared by moist tamping method.

Figure 4.5 Surface and thickness of mica

4.3.2. Specimen preparation flowchart

Figure 4.6 shows the flowchart in detail step for making specimen. Because of the mode of sedimentation of soil has the significant influence on the inherent anisotropy of soil, making specimen plays an important role. The methods making specimen adopted in this research for sandy soil namely air pluviation (AP) and dry vibration (DV) are presented in this chapter. AP method was applied only to Toyoura sand because mica particle hover easily in the air. Whereas AP and DV method were applied for all of sand.

4.3.3. Assemble container

To investigate the inherent anisotropy in this research, a rectangular container consisted of six separated metal pieces was designed and fabricated for preparing specimen. Six separated metal plates are connected together by fastening screws and can accommodate up to ten layers of testing material with 25mm height in each layer. The container dimensions are 250mm (height) x 250 mm (length) x 100 mm (width). This contained is located on a frame, step by step as shown in Figure 4.7. Using this container and frame, each depositional angle α of 0°, 22.5°, 45°, 67.5° and 90° can be created by changing the inclination and the position of the container

walls, as illustrated in Figure 4.8.

4.3.4. Air pluviation method

AP method was used to make with different depositional angle with uniform relative density and constant fabric. Oda et al. (1978) demonstrated that natural depositional process can be simulate using AP method. Miura and Toki (1984) compared the fabric of characteristic and strength properties between in-situ undisturbed soil and reconstituted specimen and found that the pluviation of sand through air method can reproduce the artificial anisotropy specimen similar to the natural deposited soil.

Air pluviation can be considered as the simulation the deposition process of wind. As shown in Figure 4.9, the testing material was poured into container through a sieve with mesh size 425μm. Tatsuoka et al. (1982) and Vaid and Nugussey (1984) demonstrated that the height of falling affected to relative density D_r. Therefore, to control the constant value of relative density, a weight hang to another metal mesh net was kept to be constant at about 425mm. During pouring material into container, the surface of material inside container was keep to be flat so that the soil grain cannot be collapsed and the depositional angle of each particle was preserved. The container with full the full of sand was immersed in water for 02 hours. Then, water was removed slowly and kept dehydration naturally more than 6 hours. Remained water was pumped at -10 kPa. The container was removed by each wall and sand block can stand by itself. Cyclinder specimen with dimension 125 mm in height and 50 mm in diameter was trimmed from sand block. Finally, membrane was inserted to cover specimen before setting it to triaxial apparatus.

4.3.5. Dry vibration

Dry vibration method uses to simulate for the shaking of ground such as earthquake region or the folding (uplift and subsidence) created by the tectonics movement.

Dry vibration method was conducted by firstly using a funnel with 8mm opening diameter and 280mm falling height to pour the sand into container. Next, a rubber hammer was used to hit the container side walls at the same energy with the number of hitting as shown in Table 4.2. As presented in Figure 4.10, this process was repeated until 10 sand layers, this process was repeated until 10 layers, each width 25 mm depth, were accumulated into this container. The same procedure with air pluviation from submersing container until dismantling the walls of container was conducted.

Significant difference in void ratio *e* (before shearing) was observed with the changing of the mica content. The void ratio becomes higher with increased mica contents under the same vibration energy. Miura et al. (1997), Cho et al. (2006), Altuhafi et al. (2016) suggested that the void ratio increases when the particle shape becomes more angular.

Figure 4.6 Flowchart of specimen preparation

Step 1:Assemble the container in specified angle

Step 2: Prepare the frame

Step 3: Assemble container and frame

Figure 4.7 Assemble frame for making specimen

Figure 4.8 Inclined container and specimen at different deposition angles

Figure 4.9 Schematic chart showing air pluviation method

Figure 4.10 Schematic of dry vibration method

Table 4.2. Time of hitting in dry vibration method

Material	Hitting time	Void ratio, e [(1)] (before shearing)	Description on relative density
Toyoura sand	1000	About 0.700	Medium relative density specimen
Mica 1%	1000	About 0.700	Medium relative density specimen
Mica 2.5%	1000	About 0.805	Medium relative density specimen
Mica 5%	1000	About 0.885	Medium relative density specimen
Mica 5%	0	About 0.970	Loose relative density specimen
Mica 5%	2000	About 0.745	Dense relative density specimen
Mica 10%	1000	About 1.000	Medium relative density specimen
Mica 10%	2000	About 0.840	Dense relative density specimen

4.4. Microscope for quantifying inherent anisotropy using particle orientation

After removing side wall of containers, a microscope work was conducted to measure the particle orientation. This attempt conducted to qualified the fabric structure of sand particle. In this study, a two dimensional (2-D) optical microscope was used to measure and evaluate particle orientation. This work only applied for soil block of Toyoura sand employed AP and DV methods at depositional angle α of 0°, 45°, 90°. Mica sand cannot be observed because of the limitation of 2-D optical microscope.

As illustrated in Figure 4.11, image for analysis were obtained from nine positions located on the side of the soil block. The number of measured particles were approximately 300 grains for each case of depositional angle of 0°, 45° and 90°. The definition of measurement angle θ, specified from 0° to 180°, is shown in Figure 4.11. The horizontal axis X is the reference axis.

Angle of each particle θ was specified from 0° to 180°. The value of θ was identified as the angle between the reference axis X and the particle's longitudinal direction measured using 2-D images. The average value of angle and vector magnitude $V.M$ (%), as equation 4.1 and 4.2 proposed by Curray (1956), were used to characterize the distribution of the particle orientation.

$$\theta = \frac{1}{2}\tan^{-1}\left(\frac{\sum_{i=1}^{n}\sin 2\theta_1}{\sum_{i=1}^{n}\cos 2\theta_1}\right) \tag{4.4}$$

$$V.M = \frac{100}{n}\sqrt{\sum_{i=1}^{n}\left(\sin 2\theta_i\right)^2 + \sum_{i=1}^{n}\left(\cos 2\theta_i\right)^2} \tag{4.5}$$

Figure 4.11 Procedure for measuring particle orientation angle θ

In equation (4.4), θ varies from 0° to 180°; n is the number of observation particles in each group. The value of $V.M$ in equation (4.5) presents the intensity of preferred orientation of particle. Value of $V.M$ ranges from 0% for a completely random and unsystematic distribution of particle orientation to 100% for the same direction of all particles.

Figure.4.12 Testing flowchart of saturated sand

Figure.4.13 Testing flowchart of unsaturated sand

4.5. Testing method

4.5.1. Testing flowchart

The flowchart of testing method for saturated soil and unsaturated soil are shown in Figure 4.12 and Figure 4.13, respectively. The detailed of each step is described from section 4.5.2 to 4.5.11.

4.5.2. Shaping specimen

After putting naturally dehydrated at least 6 hours, the container with the full of sand was pumped at negative of -10 kPa to remove the remained water. After dismantling the container, the soil block can stand by itself. The cylindrical specimen with 125mm height and 50mm in diameter is trimmed from block soil. The process of trimming specimen is mentioned as following:

① Dismantling each wall of container
② Conducting microscope as mentioned in Section 4.4.
③ Locating the soil block to trimmer, using a knife to cut specimen at dimeter of 50mm (Figure 4.14)
④ Inserting the membrane to a stretch,
⑤ Putting specimen to triaxial apparatus

Figure 4.14 Trimmer and knife for cutting specimen

4.5.3. Saturation for ceramic disc in unsaturated test

The producer of experiment on unsaturated soil is started with saturated of ceramics disc before setting of specimen to pedestal. A pedestal included ceramic disc was setting on the triaxial

basement. A cell acrylic is also assembled to triaxial apparatus to make the triaxial chamber air leakage prevention. The double negative pressure of -100 kPa are established in both inside ceramic disc and inside cell acrylic for removing air inside ceramic disc for 1 hour and 30 minutes. After that the degassed water is supplied for 1 hour from the bottom top of pedestal and water will flow up to the ceramic disc due to negative pressure in triaxial cell acrylic. Next, degassed water is provided to cell acrylic which the level of degassed water is higher than level of pedestal. This step is conducted for 1 hour. Finally, the vacuum pump was switched off and degassed water in cell acrylic was removed before starting testing.

4.5.4. Setting up specimen

The pedestal was fastened to triaxial apparatus. The saturation process for ceramic disc was conducted before setting specimen to pedestal. After trimming specimen to reach the cylindrical shape with 125mm in height and 50mm in diameter, specimen is inserted to pedestal. The grease is applied to the bottom of pedestal to prevent leakage.

4.5.5. Specimen saturation process

Initially, suction at negative pressure of -20kPa was applied to the specimen. The double negative pressure at -100kPa inside specimen and -80kPa outside specimen was used to remove air in pore of specimen. Next, degassed water was supplied from bottom to top of specimen in order to make specimen become full saturation.

The both negative pressures inside and outside specimen were decreased gradually to value of 0kPa while another cell pressure was increased to 20kPa to ensure that effective pressure always kept at constant value.

B value suggested by Skepmton (1954) was use to check the degree of saturation as follows: Firstly, the top and bottom drainage valves of specimen were opened then the cell pressure was increase to 50kPa (it is names as $P_{(50)}$). The response of pore pressures was recorded as $u_{w(50)}$ at bottom drainage valve and $u_{a(50)}$ at top drainage valve. After pore pressure and drainage volume reached constantly, the cell pressure was increase to 100kPa (it is name $P_{(100)}$) with the undrained condition and the response of pore pressure at bottom and top valves were recorded as $u_{w(100)}$ and $u_{a(100)}$ relatively. B value was calculated as the equation:

$$B = \frac{\Delta u}{\Delta P} \tag{4.6}$$

Where $\Delta P = P_{(100)} - P_{(50)}$ is defined as the increment of cell pressure (play role as the total pressure) and $\Delta u = u_{w100)} - u_{w(50)}$ and $u_{a100)} - u_{a(50)}$ are increment of pore pressures which are considered as the response of water pressure in pore of specimen. The specimen is considered in fully saturation if B – value is greater than 0.95.

4.5.6. Installing the transducer proximities

After completion of saturation process, cell water was taken out to measure the size of specimen in diameter and height carefully while the effective was kept constantly at 30kPa by using negative pressure provided by vacuum pump. The transducers for LSS were installed in triaxial cell so as not to affect to the condition of specimen. The targets are attached directly on specimen for local strain measurement. When setting the target attached to specimen for measure axial strain, a support frame is used to keep target so as long the condition of specimen is not changed (Fig.4.15). A clamp device (included a gap sensor and a target) is attached directly to middle part of specimen to measure the radial strain as illustration in Fig.4.16. The distance between target and gap sensor is adjusted to ensure that the gap sensor can work under the best measured ability.

Figure. 4.15 Install local transducer proximity to measure axial strain

Figure. 4.16 Install local transducer proximity to measure radial strain

4.5.7. Backpressure

The purpose of this step is that to increase the degree of saturation. The back pressure of 200kPa was applied through the top and the bottom of drained tube.

4.5.8. Consolidation

The consolidation process has been conducted after completion of saturation process. This process is started by opening the both of top and bottom drainage valves of specimen at effective pressure increase from 50kPa to 150kPA. During consolidation process, the volume of drainage water, axial displacement and local displacement (both axial and radial strain) are recorded by Gap sensors, dial gauge, proximity transducers, respectively. The consolidation time usually takes about 90 minutes when volume of drainage water reaches constant value.

4.5.9. Bender element test

BEs with dimensions of 2.5mm length, 12mm width, and 1mm thickness are mounted in both top cap and pedestal. The BE test conditions were based on the standard of Japanese

Geotechnical Society (2011) and can be described as following:

(1) Input frequency of 15kHz, 20kHz and 30kHz at the voltage ±10V were used to generate a single sinusoidal wave in the transmitter. Each frequency was conducted 10 times.

(2) The start-to-start method was used to identify the propagation time, as shown in Figure 4.17. The tip-to-tip L_{tt} (about 12cm) method was performed as the wave propagation length.

Figure 4.17 Time propagation determination method in BE test

4.5.10. Drying for unsaturated test

The purpose of drying step is to dehydrate the water in specimen and make specimen from saturated to unsaturated condition. By applying air pressure from top of specimen at a certain specific pressure (u_a), water in specimen is drained throughout ceramic disc at the bottom of specimen, while the air pressure is maintained inside specimen. This step may take for a few days, until drainage volume becomes constantly.

4.5.11. Monotonic compression test

Shearing compression is carried out by applied axial load monotonically under drainage condition and the pore air pressure still keep at a specific value (defined at drying step) to establish the constant suction. Because the suction is defined as $u_a - u_w$ and the pore water pressure u_w is equal to 0 due to the the water us drainage out during compression, so that the value of suction is equal to the value of pore air pressure u_a. to The strain rate of this stage is 0.0025mm/minutes in case of local small strain test and 0.1mm/min in case of shear strength test. Monotonic compression is completed when shear strain reach 0.1% (LSS test) and 10% (shear strength test).

4.6. Testing conditions

Table from 4.3 (a) to 4.3 (h), 4.4 (a) to 4.4(h) and 4.5(a) to 4.5(h) present a summary of the testing conditions applied in this study for saturated shear strength, saturated shear modulus and unsaturated shear modulus test, respectively, on the topic related to inherent anisotropy. As mention in Section 4.3.5, dry vibration method can create specimen at different void ratio by changing the energy hitting. Mica 5% and mica 10% were generate different void ratio ranged from loose to dense specimen. The term “medium density” applied for mica 5% and 10% in table 4.3 (d), 4.4(d) and 4.5 (d) are described for specimen used the same vibration energy to the other cases of Toyoura sand, mica 1% and mica 2.5%. The term “loose density” applied for mica 5% and 10% in table 4.3(e), 4.4(e) and 4.5(e) is mentioned for specimen created without vibration energy. Whereas, the term “dense density” applied for mica 5% and 10% in table 4.3(f), 4.4 (f) and 4.5 (h) is used for specimens produced by increasing the vibration energy.

4.7. Void ratio calculation method

The void ratio mention in topic related to inherent anisotropy is defined as the void ratio before shearing. The void ratio is calculated as follows:

(1) Determine the drained volume during shearing
(2) After testing, dismantle and measure the total weight m_a of soil + water
(3) Put the soil + water after testing to drying machine, weight the m_b of dry soil; the volume of water V_w and volume of sand V_s are calculated based on m_a and m_b.
(4) The volume of void ratio e before shearing can be calculated using the drained volume during shearing, m_a, V_w and m_w

Table 4.3 (a). Testing conditions of saturated sand on shear strength test of AP Toyoura sand

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
Shear strength test	Toyoura sand	0	0.636	0.05	AP
		0	0.649	0.05	AP
		0	0.649	0.05	AP
		0	0.67	0.05	AP
		0	0.60	0.05	AP
		22.5	0.634	0.05	AP
		22.5	0.652	0.05	AP
		22.5	0.665	0.05	AP
		22.5	0.65	0.05	AP
		45	0.637	0.05	AP

		45	0.672	0.05	AP
		67.5	0.635	0.05	AP
		67.5	0.618	0.05	AP
		67.5	0.645	0.05	AP
		90	0.634	0.05	AP
		90	0.646	0.05	AP
		90	0.664	0.05	AP

Table 4.3 (b). Testing conditions of saturated sand on shear strength test of DV Toyoura sand

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
Shear strength test	Toyoura sand	0	0.690	0.05	DV
		0	0.693	0.05	DV
		0	0.703	0.05	DV
		22.5	0.709	0.05	DV
		45	0.402	0.05	DV
		67.5	0.707	0.05	DV
		90	0.696	0.05	DV

Table 4.3 (c). Testing conditions of saturated sand on shear strength test of DV mica 1%

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
Shear strength test	Toyoura sand	0	0.70	0.05	DV
		0	0.719	0.05	DV
		0	0.701	0.05	DV
		0	0.724	0.05	DV
		22.5	0.687	0.05	DV
		22.5	0.70	0.05	DV
		22.5	0.70	0.05	DV
		22.5	0.70	0.05	DV
		45	0.702	0.05	DV
		45	0.687	0.05	DV
		67.5	0.705	0.05	DV
		67.5	0.701	0.05	DV
		67.5	0.705	0.05	DV

		67.5	0.714	0.05	DV
		67.5	0.710	0.05	DV
		90	0.659	0.05	DV
		90	0.679	0.05	DV
		90	0.693	0.05	DV
		90	0.717	0.05	DV
		90	0.702	0.05	DV
		90	0.726	0.05	DV

Table 4.3 (d). Testing conditions of saturated sand on shear strength test of DV mica 5% at medium density

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
Shear strength test	Mica 5%	0	0.882	0.05	DV
		0	0.901	0.05	DV
		0	0.907	0.05	DV
		22.5	0.881	0.05	DV
		45	0.877	0.05	DV
		67.5	0.885	0.05	DV
		90	0.884	0.05	DV

Table 4.3 (e). Testing conditions of saturated sand on shear strength test of DV mica 5% at loose density

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
Shear strength test	Toyoura sand	0	0.983	0.05	DV
		45	0.987	0.05	DV
		90	1.003	0.05	DV

Table 4.3 (f). Testing conditions of saturated sand on shear strength test of DV mica 5% at dense density

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
Shear strength test	Mica 5%	0	0.743	0.05	DV
		0	0.725	0.05	DV
		45	0.746	0.05	DV
		90	0.736	0.05	DV
		90	0.734	0.05	DV

Table 4.3 (g). Testing conditions of saturated sand on shear strength test of DV mica 10%

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate[2] (%/min)	Preparation method
Shear strength test	Mica 10%	0	1.023	0.05	DV
		0	1.000	0.05	DV
		0	1.078	0.05	DV
		22.5	1.031	0.05	DV
		22.5	1.023	0.05	DV
		22.5	1.046	0.05	DV
		22.5	1.026	0.05	DV
		45	1.036	0.05	DV
		45	1.017	0.05	DV
		45	1.075	0.05	DV
		67.5	1.017	0.05	DV
		67.5	1.017	0.05	DV
		90	1.04	0.05	DV
		90	1.035	0.05	DV
		90	1.068	0.05	DV

Table 4.3 (h). Testing conditions of saturated sand on shear strength test of DV mica 10%

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate[2] (%/min)	Preparation method
Shear strength test	Mica 10%	0	0.845	0.05	DV
		45	0.826	0.05	DV
		45	0.851	0.05	DV
		45	0.852	0.05	DV
		90	0.847	0.05	DV

Table 4.4 (a). Testing conditions of saturated sand on BE and LSS test of AP Toyoura sand

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate[2] (%/min)	Preparation method
		0	0.630	0.0025	AP
		0	0.624	0.0025	AP

		0	0.608	0.0025	AP
		0	0.625	0.0025	AP
		0	0.624	0.0025	AP
		0	0.642	0.0025	AP
		0	0.651	0.0025	AP
BE and LSS test	Toyoura sand	0	0.620	0.0025	AP
		0	0.635	0.0025	AP
		22.5	0.645	0.0025	AP
		22.5	0.652	0.0025	AP
		22.5	0.665	0.0025	AP
		22.5	0.633	0.0025	AP
		22.5	0.636	0.0025	AP
		22.5	0.614	0.0025	AP
		45	0.634	0.0025	AP
		45	0.642	0.0025	AP
		45	0.646	0.0025	AP
		45	0.618	0.0025	AP
		67.5	0.618	0.0025	AP
		67.5	0.637	0.0025	AP
		67.5	0.649	0.0025	AP
		67.5	0.923	0.0025	AP
		67.5	0.624	0.0025	AP
		90	0.638	0.0025	AP
		90	0.646	0.0025	AP
		90	0.620	0.0025	AP
		90	0.612	0.0025	AP

Table 4.4 (b). Testing conditions of saturated sand on BE and LSS test of DV Toyoura sand

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
		0	0.702	0.0025	DV
		22.5	0.706	0.0025	DV
BE and LSS test	Toyoura sand	45	0.706	0.0025	DV
		67.5	0.706	0.0025	DV
		90	0.709	0.0025	DV

Table 4.4 (c). Testing conditions of saturated sand on BE and LSS test of mica 2.5%

Name of test	Material	Deposition angle, *α* (°)	Void ratio *e*	Axial strain rate(2) (%/min)	Preparation method
BE and LSS test	Mica 2.5%	0	0.812	0.0025	DV
		45	0.810	0.0025	DV
		90	0.807	0.0025	DV

Table 4.4 (d). Testing conditions of saturated sand on BE and LSS test of mica 5% (medium density)

Name of test	Material	Deposition angle, *α* (°)	Void ratio *e*	Axial strain rate(2) (%/min)	Preparation method
BE and LSS test	Mica 5% (medium density)	0	0.878	0.0025	DV
		0	0.877	0.0025	DV
		22.5	0.896	0.0025	DV
		22.5	0.890	0.0025	DV
		45	0.880	0.0025	DV
		45	0.890	0.0025	DV
		67.5	0.885	0.0025	DV
		90	0.888	0.0025	DV
		90	0.884	0.0025	DV

Table 4.4 (e). Testing conditions of saturated sand on BE and LSS test of mica 5% (loose density)

Name of test	Material	Deposition angle, *α* (°)	Void ratio *e*	Axial strain rate(2) (%/min)	Preparation method
BE and LSS test	Mica 5% (loose density)	0	0.961	0.0025	DV
		0	0.994	0.0025	DV
		0	0.974	0.0025	DV
		45	0.959	0.0025	DV
		45	0.971	0.0025	DV
		45	0.983	0.0025	DV
		90	0.950	0.0025	DV
		90	0.955	0.0025	DV
		90	1.003	0.0025	DV

Table 4.4 (f). Testing conditions of saturated sand on BE and LSS test of mica 5% (dense density)

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
BE and LSS test	Mica 5% (dense density)	0	0.823	0.0025	DV
		0	0.814	0.0025	DV
		0	0.731	0.0025	DV
		0	0.743	0.0025	DV
		0	0.725	0.0025	DV
		45	0.792	0.0025	DV
		45	0.761	0.0025	DV
		45	0.746	0.0025	DV
		90	0.782	0.0025	DV
		90	0.776	0.0025	DV
		90	0.736	0.0025	DV
		90	0.734	0.0025	DV

Table 4.4 (g). Testing conditions of saturated sand on BE and LSS test of mica 10% (medium density)

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
BE and LSS test	Mica 10% (medium density)	0	1.023	0.0025	DV
		0	1.000	0.0025	DV
		0	0.990	0.0025	DV
		0	0.994	0.0025	DV
		0	1.097	0.0025	DV
		0	1.103	0.0025	DV
		0	1.078	0.0025	DV
		0	1.096	0.0025	DV
		22.5	1.031	0.0025	DV
		22.5	1.023	0.0025	DV
		22.5	1.046	0.0025	DV
		22.5	0.976	0.0025	DV
		45	1.004	0.0025	DV
		45	1.026	0.0025	DV
		45	1.096	0.0025	DV
		45	1.08	0.0025	DV

		45	1.075	0.0025	DV
		67.5	1.008	0.0025	DV
		67.5	0.965	0.0025	DV
		90	1.008	0.0025	DV
		90	1.103	0.0025	DV
		90	1.068	0.0025	DV
		90	1.082	0.0025	DV

Table 4.4 (h). Testing conditions of saturated sand on BE and LSS test of mica 10% (dense density)

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Preparation method
BE and LSS test	Mica 10% (dense density)	0	0.854	0.0025	
		0	0.851	0.0025	DV
		0	0.846	0.0025	DV
		45	0.845	0.0025	DV
		45	0.845	0.0025	DV
		45	0.826	0.0025	DV
		45	0.851	0.0025	DV
		45	0.864	0.0025	DV
		90	0.849	0.0025	DV
		90	0.829	0.0025	DV
		90	0.847	0.0025	DV
		90	0.837	0.0025	DV

Table 4.5 (a). Testing conditions of unsaturated sand on BE and LSS test of AP Toyoura sand

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Suction (kPa)	Degree of saturation S_r	Preparation method
		0	0.622	0.0025	50	About 13%	AP
		0	0.629	0.0025	50		AP
		0	0.647	0.0025	50		AP
		0	0.631	0.0025	50		AP
		0	0.665	0.0025	50		AP
		0	0.624	0.0025	50		AP

BE and LSS test	Toyoura sand	0	0.639	0.0025	50		AP
		22.5	0.636	0.0025	50		AP
		45	0.648	0.0025	50		AP
		45	0.634	0.0025	50		AP
		45	0.618	0.0025	50		AP
		67.5	0.620	0.0025	50		AP
		67.5	0.637	0.0025	50		AP
		90	0.368	0.0025	50		AP

Table 4.5 (b). Testing conditions of unsaturated sand on BE and LSS test of DV Toyoura sand

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Suction (kPa)	Degree of saturation S_r	Preparation method
BE and LSS test	Toyoura sand	0	0.678	0.0025	50	About 13%	DV
		0	0.705	0.0025	50		DV
		45	0.703	0.0025	50		DV
		90	0.701	0.0025	50		DV

Table 4.5 (c). Testing conditions of unsaturated sand on BE and LSS test of DV mica 2.5%

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(1) (%/min)	Suction (kPa)	Degree of saturation S_r	Preparation method
BE and LSS test	Mica 2.5%	0	0.810	0.0025	50	About 13%	DV
		45	0.803	0.0025	50		DV
		90	0.812	0.0025	50		DV

Table 4.5 (d). Testing conditions of unsaturated sand on BE and LSS test of DV mica 5% (medium density)

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Suction (kPa)	Degree of saturation S_r	Preparation method
BE and LSS test	Mica 5%	0	0.877	0.0025	50	About 13%	DV
		22.5	0.896	0.0025	50		DV
		45	0.880	0.0025	50		DV
		67.5	0.887	0.0025	50		DV
		90	0.884	0.0025	50		DV

Table 4.5 (e). Testing conditions of unsaturated sand on BE and LSS test of DV mica 5% (loose density)

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Suction (kPa)	Degree of saturation S_r	Preparation method
BE and LSS test	Mica 5%	0	0.973	0.0025	50	About 13%	DV
		45	0.969	0.0025	50		DV
		90	0.956	0.0025	50		DV
		90	0.954	0.0025	50		DV

Table 4.5 (f). Testing conditions of unsaturated sand on BE and LSS test of DV mica 5% (dense density)

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Suction (kPa)	Degree of saturation S_r	Preparation method
BE and LSS test	Mica 5%	0	0.748	0.0025	50	About 13%	DV
		0	0.731	0.0025	50		DV
		45	0.761	0.0025	50		DV
		90	0.776	0.0025	50		DV

Table 4.5 (g). Testing conditions of unsaturated sand on BE and LSS test of DV mica 10% (medium density)

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate(2) (%/min)	Suction (kPa)	Degree of saturation S_r	Preparation method
BE and LSS test	Mica 10%	0	0.987	0.0025		About 13%	
		0	1.101	0.0025			
		22.5	1.044	0.0025			
		45	1.078	0.0025	50		DV
		45	1.023	0.0025	50		DV
		67.5	1.005	0.0025	50		DV
		90	0.999	0.0025	50		DV
		90	1.102	0.0025	50		DV

Table 4.5 (h). Testing conditions of unsaturated sand on BE and LSS test of DV mica 10% (dense density)

Name of test	Material	Deposition angle, α (°)	Void ratio e	Axial strain rate[(2)] (%/min)	Suction (kPa)	Degree of saturation S_r	Preparation method
BE and LSS test	Mica 10%	0	0.850	0.0025	50	About 13%	DV
		45	0.844	0.0025	50		DV
		90	0.827	0.0025	50		DV

[(1), (2)]: Axial rate during monotonic compression test.

CHAPTER 5. TESTING MATERIAL AND TESTING METHOD OF COHESIVE SOIL

5.1. Introduction

This chapter describes the detailed method for testing applied for cohesive soil. Yoneyama sandy silt was employed as the lateritic soil. A series of triaxial test included local small strain test and bender element test was conducted on both saturated and unsaturated sois. The induced anisotropy is referred to the history of stress state applied during consolidation. Various value of K, defined as the ratio between horizontal stress σ_h and vertical stress σ_v, were consider as the different stress stated during consolidation.

5.2. Testing material

Cohesive soil was used for this topic. Yoneyama silt, represented as the lateritic and located in Niigata prefecture, Japan (Figure 5.1), was employed to investigate the effects of induced anisotropy on initial shear modulus G_0 of saturated and unsaturated cohesive soils. The grain size distribution curve, presented in Figure 5.2, shows that Yoneyama sandy silt has a wider distributed grain size compared to Toyoura sand. In addition, the percentage of particle size less 0.075mm about 90% implies that, sand particle occupies about 10%. The physical properties of Yoneyama sandy silt is shown in Table 5.1.

Table 5.1. Physical properties of Yoneyama silts soil

ρ_s (g/cm3)	W_L (%)	I_p (%)	Clay (%)	Silt (%)	Sand (%)
2.746	52.4	23.6	25.0	67.03	7.97

Figure 5.1 Location of Yoneyama sandy silt

Figure 5.2 Grain size distribution of Yoneyama sandy silt and referenced Toyoura sand

5.3. Specimen preparation

The general view for making specimen is shown in Figure 5.3. 1300g of dry and clean Yoneyama and 800g distilled water were mixed together and kept at temperature room at least 15 hours. When soil was completely absorbed water, the mixture was poured into an acrylic cylinder. As illustrated in Figure 5.4, this slurry was removed air under vacuum state. Next, pre-consolidation of 45kPa was applied to to the cylinder under one-dimensional direction. The vertical stress was selected as the necessary value to keep the soil block can stand by itself during specimen. The soil block was pushed out of cylinder under a specific pressure. Using a wire saw and a trimmer, a specimen of 50mm in diameter and 125mm in height was trimmed from soil block. A mold with two separated pieces (as Figure 5.5) was used to put specimen on the pedestal. Finally, a membrane was covered specimen before starting the test.

Figure 5.3 Procedure of specimen preparation of cohesive soil

Figure 5.4 Removing air and applying one-dimension consolidation pressure

Figure 5.5 Specimen trimming process of cohesive soil

(a) Trimmed tools, (b) Specimen before trimming and (c) Specimen after trimming

Figure 5.6 Testing procedure of saturated soil

Figure 5.7 Testing procedure of unsaturated soil

Figure 5.8 3t method in pre-consolidation

5.4. Procedure of testing at effective stress *p'* of 300kP

The testing procedure from saturation for specimen until shearing are presented in Figure 5.6 and 5.7 for saturated soil and unsaturated soil, respectively. The detailed of each step are described as follows:

5.4.1. Isotropic consolidation process

After setting the specimen to pedestal, the specimen was saturated using the vacuum saturated procedure (Rad and Clough, 1984), as mentioned in Section 4.4, Chapter 4. Next, the proximity transducers were installed while vacuum was provided to keep suction inside specimen about -20kPa.The specimen was consolidated isotropically under an effective stress of *p'* = 50 kPa. The *3t* method proposed by Kamei et al., (1987) was used to determine the consolidation time. As shown in Figure 5.8, consolidation time was about 2 hours. Back pressure of 200 kPa was applied in the case of saturated soil and skipped for unsaturated soil.

5.4.2. Drained q-loading process

Keeping the effective stress of 50 kPa, the specimen was sheared under drained condition with axial strain rate of 0.005mm/min. This strain rate was selected to ensure that the pore pressure is not increased during loading. The shear stress was applied until reach a certain *K* defined as the stress ratio of anisotropic condition σ'_3/σ'_1. Value of *K* was selected as presented in Table

5.2. Generally, value of K was summaries as follows.

- *K*=1: Represented the isotropic consolidation
- *K*<1: Played the role as the anisotropic consolidation under compression condition.
- *K*>1: Implied the anisotropic consolidation under extension condition.

In the extension and compression testing, the top platen was fixed during test, while the bottom platen of triaxial apparatus move in an upward direction and downward direction for compression and extension test, respectively.

Table 5.2 (a). The testing condition of saturated soil in q-loading process

Cell pressure Cell P (kPa)	Back pressure (kPa)	Effective stress, p' (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, q (kPa)	Stress ratio, K
265.6	200	50.0	18.8	65.6	65.6	-46.9	3.5
264.3	200	50.0	21.4	64.3	64.3	-42.9	3
260.0	200	50.0	30.0	60.0	60.0	-30.0	2.00
256.3	200	50.0	37.5	56.3	56.3	-18.8	1.50
250.0	200	50.0	50.0	50.0	50.0	0.0	1.00
246.2	200	50.0	57.7	46.2	46.2	11.5	0.80
240.9	200	50.0	68.2	40.9	40.9	27.3	0.60
234.7	200	50.0	80.6	34.7	34.7	46.0	0.43
230.9	200	50.0	88.2	30.9	30.9	57.4	0.35
228.1	200	50.0	93.8	28.1	28.1	65.6	0.30

Table 5.2 (b). The testing condition of unsaturated soil in q-loading process

Cell pressure (kPa)	Back pressure (kPa)	Effective stress, p' (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, q (kPa)	Stress ratio, K
65.6	0	50.0	18.8	65.6	65.6	-46.9	3.5
64.3	0	50.0	21.4	64.3	64.3	-42.9	3
60.0	0	50.0	30	60.0	60.0	-30.0	2
56.3	0	50.0	37.5	56.3	56.3	-18.8	1.5
50.0	0	50.0	50	50.0	50.0	0.0	1
46.2	0	50.0	57.7	46.2	46.2	11.5	0.8

40.9	0	50.0	68.2	40.9	40.9	27.3	06.
34.7	0	50.0	80.6	34.7	34.7	46.0	0.43
30.9	0	50.0	88.2	30.9	30.9	57.4	0.35
28.1	0	50.0	93.8	28.1	28.1	65.6	0.3

5.4.3. Drying process

The deviator stress q (Table 5.2 (b) was maintained while matric suction, $s = u_a$-u_w of 400 kPa was applied to specimen for drying. Effective stress $p_{net} = p$-u_a of 50 kPa was kept constantly using the axis-translation technique. Because of the application of air pressure from the top-cap, the water inside specimen was drained out through ceramic disk for u_w. This process was continued about 4 days until drained volume from ceramic disc become less than 0.2cm^3/day.

Table 5.3. The testing condition of unsaturated soil in drying process

Cell pressure (kPa)	Suction u_a-u_w (kPa)	Effective stress, p_{net} (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, q (kPa)	Stress ratio, K
465.6	400	50.0	18.8	65.6	65.6	-46.9	3.5
464.3	400	50.0	21.4	64.3	64.3	-42.9	3
460.0	400	50.0	30	60.0	60.0	-30.0	2
456.3	400	50.0	37.5	56.3	56.3	-18.8	1.5
450.0	400	50.0	50	50.0	50.0	0.0	1
446.2	400	50.0	57.7	46.2	46.2	11.5	0.8
440.9	400	50.0	68.2	40.9	40.9	27.3	06.
434.7	400	50.0	80.6	34.7	34.7	46.0	0.43
430.9	400	50.0	88.2	30.9	30.9	57.4	0.35
428.1	400	50.0	93.8	28.1	28.1	65.6	0.3

5.4.4. K consolidation process

The stress ratio K was kept constant with the same value to q-loading process. Effective stress p' or p_{net} was increased at 100 kPa, 200 kPa and 300 kPa. The deviator stress q was increased corresponding to the changing of effective stress as show in Table 5.4 to 5.6.

Table 5.4 (a) The testing condition of saturated soil in *K*-consolidation process at *p'* of 100kPa

Cell pressure (kPa)	Back pressure (kPa)	Effective stress, *p'* (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, *q* (kPa)	Stress ratio, *K*
265.6	200	100	37.5	131.3	131.3	-93.8	3.5
264.3	200	100	42.9	128.6	128.6	-85.7	3
260	200	100	60.0	120.0	120.0	-60.0	2
256.3	200	100	75.0	112.5	112.5	-37.5	1.5
250	200	100	100.0	100.0	100.0	0.0	1
246.2	200	100	115.4	92.3	92.3	23.1	0.8
240.9	200	100	136.4	81.8	81.8	54.5	06.
234.7	200	100	161.3	69.4	69.4	91.9	0.43
230.9	200	100	176.5	61.8	61.8	114.7	0.35
228.1	200	100	187.5	56.3	56.3	131.3	0.3

Table 5.4 (b) The testing condition of unsaturated soil in *K*-consolidation process at p_{net} of 100kPa

Cell pressure (kPa)	Suction u_a-u_w (kPa)	Effective stress, *p'* (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, *q* (kPa)	Stress ratio, *K*
531.3	400	100	37.5	131.3	131.3	-93.8	3.5
528.6	400	100	42.9	128.6	128.6	-85.7	3
520.0	400	100	60.0	120.0	120.0	-60.0	2
512.5	400	100	75.0	112.5	112.5	-37.5	1.5
500.0	400	100	100.0	100.0	100.0	0.0	1
492.3	400	100	115.4	92.3	92.3	23.1	0.8
481.8	400	100	136.4	81.8	81.8	54.5	06.
469.4	400	100	161.3	69.4	69.4	91.9	0.43
461.8	400	100	176.5	61.8	61.8	114.7	0.35
456.3	400	100	187.5	56.3	56.3	131.3	0.3

Table 5.5 (a) The testing condition of saturated soil in *K*-consolidation process at *p'* of 200kPa

Cell pressure (kPa)	Back pressure (kPa)	Effective stress, *p'* (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, *q* (kPa)	Stress ratio, *K*
462.5	200	200	75.0	262.5	262.5	-187.5	3.5
457.1	200	200	85.7	257.1	257.1	-171.4	3
440.0	200	200	120.0	240.0	240.0	-120.0	2
425	200	200	150.0	225.0	225.0	-75.0	1.5
400.0	200	200	200.0	200.0	200.0	0.0	1
384.6	200	200	230.8	184.6	184.6	46.2	0.8
363.6	200	200	272.7	163.6	163.6	109.1	06.
338.7	200	200	322.6	138.7	138.7	183.9	0.43
323.5	200	200	352.9	123.5	123.5	229.4	0.35
312.5	200	200	375	112.5	112.5	262.5	0.3

Table 5.5 (b) The testing condition of unsaturated soil in *K*-consolidation process at p_{net} of 200kPa

Cell pressure (kPa)	Suction u_a-u_w (kPa)	Effective stress, *p'* (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, *q* (kPa)	Stress ratio, *K*
662.5	400	200	75.0	262.5	262.5	-187.5	3.5
657.1	400	200	85.7	257.1	257.1	-171.4	3
640.0	400	200	120.0	240.0	240.0	-120.0	2
625.0	400	200	150.0	225.0	225.0	-75.0	1.5
600.0	400	200	200.0	200.0	200.0	0.0	1
584.6	400	200	230.8	184.6	184.6	46.2	0.8
563.6	400	200	272.7	163.6	163.6	109.1	0.6
538.7	400	200	322.6	138.7	138.7	183.9	0.43
523.5	400	200	352.9	123.5	123.5	229.4	0.35
512.5	400	200	375.0	112.5	112.5	262.5	0.3

Table 5.6 (a) The testing condition of saturated soil in *K*-consolidation process at *p'* of 300kPa

Cell pressure (kPa)	Back pressure (kPa)	Effective stress, p' (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, q (kPa)	Stress ratio, K
593.8	200	300	112.5	393.8	393.8	-281.3	3.5
585.7	200	300	128.6	385.7	385.7	-257.1	3
560.0	200	300	180.0	360.0	360.0	-180.0	2
537.5	200	300	225.0	337.5	337.5	-112.5	1.5
500.0	200	300	300.0	300.0	300.0	0.0	1
476.9	200	300	346.2	276.9	276.9	69.2	0.8
445.5	200	300	409.1	245.5	245.5	163.6	06.
408.1	200	300	483.9	208.1	208.1	275.8	0.43
385.5	200	300	529.4	185.3	185.3	344.1	0.35
368.8	200	300	562.5	168.8	168.8	393.8	0.3

Table 5.6 (b) The testing condition of unsaturated soil in *K*-consolidation process at p_{net} of 300kPa

Cell pressure (kPa)	Suction u_a-u_w (kPa)	Effective stress, p' (kPa)	Vertical stress, σ'_1 (kPa)	Horizontal stress, σ'_2 (kPa)	Horizontal stress, σ'_3 (kPa)	Deviator stress, q (kPa)	Stress ratio, K
793.8	400	300	75.0	262.5	262.5	-187.5	3.5
785.7	400	300	85.7	257.1	257.1	-171.4	3
760.0	400	300	120.0	240.0	240.0	-120.0	2
737.5	400	300	150.0	225.0	225.0	-75.0	1.5
700.0	400	300	200.0	200.0	200.0	0.0	1
676.9	400	300	230.8	184.6	184.6	46.2	0.8
645.5	400	300	272.7	163.6	163.6	109.1	0.6
608.1	400	300	322.6	138.7	138.7	183.9	0.43
585.3	400	300	352.9	123.5	123.5	229.4	0.35
568.8	400	300	375.0	112.5	112.5	262.5	0.3

5.4.5. Bender element test

Bender element test was conducted based on the standard of Japanese Geotechnical Society

(2011) and can be described as following:

(1) Frequency of 10 kHz, 15 kHz and 20 kHz at the voltage ±10V were used to produce a single sinusoidal wave in the transmitter.

(2) The start-to-start method was used to determine the travel time Δt. The tip-to-tip L_{tt} method was used to identify the wave travel length.

5.4.6. Shearing

Monotonic compression test was conducted under at 0.005%/min drained condition and constant suction for saturated soil and unsaturated soil, respectively. The test was completed until shear strain ε_s reach 0.1%.

Figure 5.9 Stress path during testing at effective stress 300kPA

5.5. Testing procedure of testing at various stress state

To investigate the effects of stress state under anisotropic consolidation on initial shear modulus, a series of experiment is conducted under different effective stress. Generally, the testing procedure is almost similar conditions to the testing procedure mentioned in Section 5.5. However, the effective stress p' are increased from 50kPa to 600kPa, referred to consolidation state, then decreased from 600kPa to 50kPa, represented as the swelling condition. The detail step is shown in Figure 5.10. Finally, monotonic compression is conducted at strain rate of 0.005%/min and effective stress of 50kPa. The stress path of testing procedure is illustrated in

Figure 5.11.

Figure 5.10 Testing procedure of various effective stress

Figure 5.11 Stress path of various effective stress

CHAPTER 6. INHERENT ANISOTROPY OF SANDY SOIL

6.1. Introduction

This chapter describes the evaluation on anisotropy of shear strength properties and shear modulus for different type of sands at both saturated and unsaturated conditions.

The first section of this chapter presents the testing program which different sands was employed at different depositional angles. The detailed of testing procedure and properties of materials was introduced in Chapter 4. Next, shear strength for saturated test was conducted using two methods on measuring volume change during shearing. One method is that the volume change was measured directly from drained volume during shearing. The remained one is conducted using the local small strain technique. For unsaturated sand, local small strain (LSS) technique was incorporated into triaxial apparatus (mentioned in Chapter 3 and Chapter 4) to measure directly the volume change during shearing. The discussion on initial shear modulus obtained from bender element test and local small strain test is presented in separated sections between saturated and unsaturated test. The particle orientation was quantified using the *2-D* optical microscope. Finally, this Chapter proposes a method to assess initial shear modulus based on the particle orientation.

6.2. Testing conditions program

Several triaxial compression test were performed using Toyoura sand mixed mica at different content of 0%, 1%, 2.5%, 5%, 10%. The testing procedure was written in detail in chapter 3 is summarized as below.

A specimen was saturated using the vacuum saturation procedure suggested by Rad and Clough (1984). Cell water was removed while suction of 20 kPa was maintained inside specimen. The proximity transducers for LSS test were installed before cell water was provided again. B-value greater than 0.97 was check based on Skempton (1954). Then, in isotropic consolidation process, effective stress of 150 kPa was applied. The BE test was conducted to obtain the shear wave velocity. Matric suction, $s = u_a - u_w$ of 50 kPa was applied from top cap to the specimen for drying under a constant $p_{net} = p - u_a$ in the axis-tranlation technique using a ceramic disc. This step was skipped for unsaturated sand. The BE test was conducted before shearing. Finally, monotonic compression was carried out under drained condition for saturated sand and constant suction for unsaturated sand at the axial strain rate of 0.0025%/min for the LSS test and 0.05%/min for shear strength test.

6.3. Effects of inherent anisotropy on shear strength

6.3.1. Shear strength obtained on drained volume change of saturated sand

Figure 6.1-6.5 respectively show the deposition angle *α* as a function of stress-strain and the volumetric strain against the shear strain relation of saturated Toyoura sand (AP and DV), mica 1%, mica 5% and mica 10% using DV in the same energy vibration. Deviator stress *q* and shear strain ε_s are calculated as followed:

$$q = \sigma_1^{'} - \sigma_3^{'} \tag{6.1}$$

$$\varepsilon_s = \frac{2}{3}\left(\varepsilon_a - \varepsilon_r\right) \tag{6.2}$$

Toyoura sand and mica 1% show that the soil initially exhibited small contractive behavior at shear strain ε_s less than 1% and followed by the dilation with continued shearing. On the other side, mica 5% and mica 10% sands are much more contractive. Moreover, the effects of deposition angle *α* on stress-strain and volumetric strain are clear in Toyoura sand. The deviator stress become greater at the same strain when *α* is smaller. The dilative behavior is apparent in the volumetric strain. Nonetheless, when mica contents are higher than 5%, the differences of stress-strain and volumetric strain behavior become disappear.

It can be explained that the increase of mica content causes the increasing of void ratio. Mica 5% and mica 10% sands tend to behave as loose soil. Lee et al. (2007) demonstrated that the presents of mica postpones the appearance of peak strain. Therefore, results obtained in this research show that, the peak strength appear in stress-strain relation in case of Toyoura sand and mica 1%. In contrast, mica 5% and mica 10% do not obtain the peak strength even the shear strain reached 10%. Shin and Santamarina (2013) proved that the effects of mica is more significant than those of angularity shape on the mechanical behavior of granular mixture. Seethalakshmi and Sachan (2019) used SEM on mica and indicated that because of the crushability of mica, the peak strength decreases and pore pressure behavior changes from dilative to contractive. Therefore, mica 5% and mica 10% exhibit the higher contractive behavior than Toyoura sand, even mica has a greater angular shape.

Figure 6.6 (a) presents the relations between shear strength q_{max} and depositional angle α. Due to the occurrence of mica delays the q_{max} on mica 5% and mica 10%, those results are selected as deviator stress at strain of 10%. It is evident that the peak deviator stress of Toyoura sand in AP method obtained the highest strength because of the smallest void ratio as mentioned in Chapter 4. The shear strength decreases when the deposition angle change from horizontal direction at α of 0° to the vertical direction at α of 90°. However, the peak strength obtained at α=67.5° was almost the same to that α=90°. Oda et al. (1978) and Tatsuoka et al. (1986) indicated that the lower strength appears in the direction of the “potential” strain localization plane base on the Morh-Coulomb failure criterion. The mica content reduces the drained shear

strength.

Figure 6.6(b) shows that the variation of shear strength with the changing of deposition angle become independently with the present of mica. However, using only Toyoura sand in DV method, the inherent anisotropy on shear strength is not apparent independent of the mica content because the same slope line was observed in the relation between normalized q_{max} and deposition angle α. These results imply that the breakage particle of mica causes the disappearance of anisotropy and AP method can produce a greater degree of inherent anisotropy than DV method can.

Figure 6.1 Drained shear properties of saturated Toyoura sand (AP method): (a) stress–strain and (b) volumetric strain.

Figure 6.2 Drained shear properties of saturated Toyoura sand (DV method): (a) stress–strain and (b) volumetric strain.

Figure 6.3 Drained shear properties of saturated mica 1% (DV method): (a) stress–strain and (b) volumetric strain.

Figure 6.4 Drained shear properties of saturated mica 5% (DV method): (a) stress–strain and (b) volumetric strain.

Figure 6.5 Drained shear properties of saturated mica 10% (DV method): (a) stress–strain and (b) volumetric strain.

Figure 6.6 Drained shear properties of saturated sand:
(a) maximum shear strength and (b) inherent anisotropy of shear strength.

6.3.2. Shear strength obtained on proximity transducer of saturated sand and unsaturated sand

Figure 6.7 and 6.8, respectively, show the stress-strain and the volumetric strain against shear strain relations in saturated sand under drained conditions and unsaturated sand under constant suction using the proximity transducers to measure volume change. This method is defined as the “local measurement method” using LSS devices compared to the “global measurement method” using external displacement and external volume gap sensor.

The shear stress shows the same tendency on both saturated sand and unsaturated sand, where the shear strength becomes smaller with increased α. However, the volumetric behaviors are the same and independent with the variation of α. Thus, the anisotropic shear strength behavior was not observed on both saturated and unsaturated sand.

Figure 6.9 presents the comparison of peak strength versus deposition angle α of saturated sand between two volume change measurement method. It can be seen that both global measurement method and local displacement method obtain the same tendency: The shear strength becomes smaller when changes from a horizontal to a vertical deposition angle. However, greater shear strength appears on the global measurement method compare to local measurement method. Similarly, the same observation is obtained on unsaturated sand, as illustrated in Figure 6.10. Having said that, the difference between the saturated sand and unsaturated sand is induced by the application of matric suction of 50kPa. Thus, the strength of unsaturated sand can be evaluated by adding suction effect to the strength of the saturated sand.

Figure 6.7 Drained shear properties of saturated Toyoura sand (AP method) using local measurement method: (a) stress–strain and (b) volumetric strain.

Figure 6.8 Drained shear properties of unsaturated Toyoura sand (AP method) using local measurement method: (a) stress–strain and (b) volumetric strain.

Figure 6.9 Comparison shear strength of saturated Toyoura sand (AP method) in two measurement method

Figure 6.10 Comparison anisotropy shear strength of unsaturated and saturated sand

6.4. Inherent anisotropy on shear modulus

6.4.1. Inherent anisotropy on secant shear modulus

In this section, initial shear modulus G_0 was obtained in different mica contents using the same energy vibration using DV method. That result on AP method is also presented in this section.

Figure 6.11 and 6.12, respectively, present the secant shear modulus as calculated using the relation of $G_{sec}=\Delta q/3\Delta\varepsilon_s$ versus shear strain in saturated and unsaturated sand. There are five graphs presented as follows: (a) Toyoura sand using AP method, (b) Toyoura sand using DV method, (c) mica 2.5% using DV method, (d) mica 5% using DV method, and (e) mica 10% using DV method. The same tendency is obtained in all case: at ranges of ε_s <0.001%, G_{sec} is highest at α=90° and smallest at α=0° . Those are interesting results because this trend is reversed to the decreasing q_{max} trend. When the shear strain become larger (more than 0.01% and less than 0.1%), the G_{sec} trend of Toyoura sand is opposite to the tendency G_{sec} less than 0.001%. Yamashita et al. (2003 and 2005) made specimen as pluviation direction of 0° (presented as *V*-specimen) and 90° (considered as *H*-specimen) and obtained the same observation. The elastic shear moduli E_v on V-specimen are slightly smaller than E_H on H-specimen.

The presence of mica has the significant influence on the behavior of soil. At the very small strain (ε_s <0.001%), mica-mixed sands show the same tendency to Toyoura sand using AP method. However, the gaps between G_{sec} *of* 0° *and* G_{sec} *of 9*0° of mica-mixed sand show the wider distance compared to Toyoura sand. At the shear strain larger than 0.01%, G_{sec} becomes independently with deposition angle α.

Figure 6.11 Secant shear modulus of saturated sand in LSS test (ε_s = 0.0025%/min; p'= 150 kPa): (a) Toyoura sand (AP), (b) Toyoura sand (DV), (c) mica 2.5% (DV), (d) mica 5% (DV), and (e) dry vibrated mica 10% (DV).

Figure 6.12 Secant shear modulus of unsaturated sand in LSS test (ε_s = 0.0025%/min; $p'=p_{net}$= 150 kPa; s =50 kPa): (a) Toyoura sand (AP), (b) Toyoura sand (DV), (c) mica 2.5% (DV), (d) mica 5% (DV), and (e) mica 10% (DV).

6.4.2. Inherent anisotropy on initial shear modulus G_0

(a) Results on LSS test

Figure 6.13 depicts how the initial shear modulus varies through the deposition angle α. Initial shear modulus G_0 was obtained from secant shear modulus at shear strain about 0.001%. The results on both saturated and unsaturated sand show the similar changing trend: G_0 becomes

larger when α increase from 0° to 90°. However, because of the application of matric suction, G0 of unsaturated sand is higher than that of saturated sand. As mentioned on Section 6.4, the effects of drained shear strength of both saturated sand and unsaturated sand tend to decrease with the increase of deposition angle.

Figure 6.13 Comparison initial shear modulus between saturated sand and unsaturated sand in LSS test (ε_s = 0.0025%/min; p'=p_{net}= 150 kPa; s =50 kPa): (a) Toyoura sand (AP), (b) Toyoura sand (DV), (c) mica 2.5% (DV), (d) mica 5% (DV), and (e) mica 10% (DV).

(b) Results on BE test

Figure 6.14 (a) presents a comparison of the typical received wave obtained on BE test on saturated sand and unsaturated sand of Toyoura sand prepared using AP method. Because of the application of matric suction, the point selected as the arrived time on unsaturated sand become faster than that on unsaturated sand. Therefore, the shear wave velocity V_s is higher than V_s of saturated sand.

Figure 6.14 (b) presents the shear wave velocity V_s against different frequency of 15kHz, 20kHz and 30kHz at three representative deposition angles α on both saturated sand and unsaturated sand. V_s becomes higher with both increase of frequency and α. Moreover, the shear wave velocity V_s of unsaturated sand are higher than V_s of saturated sand.

Figure 6.14 An example of received waves on saturated sand and unsaturated sand in BE test at frequency of 15kHz.

Figure 6.15 (a), (b), (c), (d) and (e), respectively, show the comparison between saturated sand and unsaturated sand on the initial shear modulus G_0 versus the deposition angle α in BE test. Similar tendency to those of saturated sands were observed in unsaturated sands. For comparison between saturated sands and unsaturated sands, the G_0 of unsaturated sand is greater than that of saturated sand because of the application of matric suction of 50kPa.

(c) Comparison results between LSS and BE test

Figure 6.16 (a), (b), (c), (d), respectively, present the comparison on the initial shear modulus obtained from LSS test and BE test for Toyoura sand using air pluviation method, Toyoura sand using drying vibration method, mica 2.5%, mica 5%, mica 10%. Toyoura sand using air pluviation method and dry vibration method shows the same results on two testing methods. However, with the appearance of mica, G_0 obtained from BE is higher than those results obtained from LSS test. The same observation was obtained from unsaturated sand, as shown in Figure 6.17.

Figure 6.15 Comparison initial shear modulus between saturated sand and unsaturated sand in BE test (ε_s = 0.0025%/min; p'=p_{net}= 150 kPa; s =50 kPa) (a) Toyoura sand (AP), (b) Toyoura sand (DV), (c) mica 2.5% (DV), (d) mica 5% (DV), and (e) mica 10% (DV).

Figure 6.16 Comparison initial shear modulus of saturated soil between LSS test and BE test: (a) Toyoura sand (AP), (b) Toyoura sand (DV), (c) mica 2.5% (DV), (d) mica 5% (DV), and (e) mica 10% (DV).

Figure 6.17 Comparison initial shear modulus of unsaturated soil between LSS test and BE test: (a) Toyoura sand (AP), (b) Toyoura sand (DV), (c) mica 2.5% (DV), (d) mica 5% (DV), and (e) mica 10% (DV).

6.4.3. Effects of mica contents on initial shear modulus

Effects of mica contents on the initial shear modulus in saturated sands are shown in Figure 6.18 (a) and (b) from LSS and BE test, respectively. The initial shear modulus decreases strongly with the range of mica content from 0% to 2.5% compared to between the range of mica content from 2.5% to 10%, especially for LSS test results. The similar tendency was observed in unsaturated sand, as shown in Figure 6.18 (c) from LSS test and (d) from BE test. However, the initial shear modulus of unsaturated sand is higher than those results of saturated sand because of suction effects.

Figure 6.18 Mica content effects on initial shear modulus: (a) saturated sand, LSS test; (b) saturated sand, BE test; (c) unsaturated sand, LSS; and (d) unsaturated sand, BE test.

6.5. Evaluation of the degree of anisotropy

6.5.1. Degree of anisotropy in previous research

In previous research, the degree of anisotropy was expressed as G_{hh}/G_{hv} or E_h/E_v, which G_{hh} and E_h, respectively, defined as the shear modulus and Young modulus in horizontal plane. G_{hv} and E_v is shear modulus and Young modulus in horizontal plane in vertical plane, respectively. Bellotti et al. (1996), Pennington et al. (1997), Yamashita et al. (2005), Hight et al. (2007), and Teng et al. (2013) revealed that shear moduli in horizontal plane is greater than those in vertical plane, implies the ratio G_{hh}/G_{hv} or E_h/E_v is higher than 1.0. In unsaturated soil, Ng and Yung (2008) demonstrated that the increase of matric suction from 0 kPa to 50 kPa caused the increase of the shear modulus ratio ($G_{0(hh)}/G_{0(hv)}$) from1.03 to 1.05.

6.5.2. Degree of anisotropy on shear modulus

The degree of anisotropy related to the initial shear modulus is defined by the slope of the relation between the normalized initial shear modulus ($G_{0(\alpha=0^o)}/G_{0(\alpha=90^o)}$) and deposition angle. The steeper slope implies a greater the degree of anisotropy. Figure 6.19 presents the degree

of anisotropy of the initial shear modulus, calculated as normalized initial shear modulus and deposition angle. Results show a similar tendency for all graphs presented in 6.43 (a), (b), (c) and (d). The degree of anisotropy is lowest in Toyoura sand (BE test), which reaches value of 1.10 and 1.09 for unsaturated sand and saturated sand respectively. ($G_{0(\alpha=0o)}/G_{0(\alpha=90^o)}$) of mica 10% exhibits the highest degree of anisotropy.

Figure 6.19 Inherent anisotropy of initial shear modulus of LSS and BE test: (a) saturated sand, LSS test; (b) saturated sand, BE test; (c) unsaturated sand, LSS; and (d) unsaturated sand, BE test.

On those results, the similar trend was observed on both LSS and BE test, where degree of anisotropy is depended on deposition angle α. However, greater inherent anisotropy, which steeper slope between G_{nor} and α were obtained in LSS test than those in BE test. There are two main reasons for this difference.

Firstly, the possible reason seems to be induced by the degree of the homogeneity of the specimen. Choi et al. (2010) employed a rainer system with a porous plate to make specimen in a 1.0m split mold. They conducted three uniformity test included measurement of the relative

density of 20 small molds along the height of split mold using the cone resistance from cone penetration test along the depth and the shear wave velocity from BE test. The obtained results showed that the relative density increase along the depth. Tabaroei et al. (2017) demonstrated that the density differences were ranged from 2.3% to 5%, even in horizontal direction. As mention in Section, even using the small scale container, 250 in height and width, was used to make specimen, it is difficult to make a perfectly uniform specimen. Moreover, the different length applied in BE test (120mm) and in LSS test (80mm) mentioned that there is the difference on the approached condition of specimen in the point of void ratio. In early studies (Hardin and Richart, 1963; Hardin and Drnevich, 1972), an empirical expression for the initial shear modulus G_0, accounted for the variation in mean effective stress and void ratio at the isotropic stress state, was suggested as follows:

$$G_0 = AF(e)\left(\frac{\sigma'_m}{p_a}\right)^n \tag{6.3}$$

Where A and n are fitting parameters varies by soil types; σ'_m is the confining stress; p_a is usually taken value as the atmospheric pressure of 100kPa; $F(e)$ is the function of void ratio e .

The equation (6.3) indicates that the initial shear modulus is varied with the changing of the void ratio e. As the results, the influence of the heterogeneous specimen can create the difference between LSS and BE tests. The second reason for this difference might be that the initial shear modulus is measured at different strain level, which is about 2 x 10^{-4}% in LSS test and less than 10^{-4}% in BE test. Thus, G_0 differs between LSS and BE tests.

6.6. Discussion

Particle shape and fabric might create the anisotropic mechanical behavior in sands. As mentioned in Chapter 4, Toyoura sand shape is somewhat elongated rather than spherical. The aspect ratio *L/W* between length *(L)* and width *(W)* ranges from 1.0 to 1.6; the average value of *L/W* is about 1.5. The skeleton structure of particles' (fabric) of both Toyoura sand and mica is shown schematically in Figure 6.20. The number of contact point of Toyoura sand is small and the rigid area considered as the elastics region for small stiffness (LSS test and BE test) is large at α =90°. The particle structure of α =0° is more stable than that of α =90° against compressional loading. Therefore, the greater shear strength was obtained for α=0°. However, the number of contact points in the vertical direction is greater in the case of α=0° than in the case of α=90°. Therefore, the rigid area of particle without contact points becomes larger in the case of α=90°. Results show that the higher initial shear modulus was measured in the case of α=90° than in the case of α=0°.

Consider to the behavior of mica mixed sand, the particle shape has the effects on the behavior of mica sands. Mica has the flat and flaky shape and les sphericity, less roundness, and a greater aspects ratio than that of Toyoura sand. Using the same concept with Toyoura sand, as

illustrated in Figure 6.20 (b), the degree of anisotropy in mica show a higher result than that in Toyoura sand. However, because of the high compressive material induced by particle breakage, the degree of anisotropy on shear strength in mica sand becomes smaller than that in Toyoura sand.

As mention in Chapter 3, the presence of mica causes the larger void ratio compared to Toyoura sand. The void ratio effects on initial shear modulus have been proposed in equation 6.3. Therefore, the initial shear modulus becomes smaller with increased mica content.

Figure 6.20 Effects of particle shape on soil skeleton: (a) Toyoura sand, (b) mica sand

6.7. Quantifying inherent anisotropy using particle orientation

In order to obtain a profound understanding about inherent anisotropy, it is necessary to qualify the soil fabric anisotropy. In this study, a computer controlled 2-D optical microscope was used to measure and evaluate particle orientation. As displayed in Figure 6.21, this work was conducted only on the block of Toyoura sand prepared using AP and DV methods. The images for analysis were obtained at 9 positions located on the front view, as displayed in Figure 6.22. The number of measured particles were approximately 300 grains for each case. The definition for measured angles is shown in Figure 6.21. Angle θ of each particle was identified from 0° to 180°. The value θ of was specified as the angle between the reference axis X and the longitudinal direction of particle measured using two dimensional images. Figure 6.23 shows an example of images taken from microscope work and measurements of the inclined angle of sand particles.

To characterize the distribution of the particle orientation from specimen preparation in this research, the average quantitative value the average quantitative value of the depositional angle θ and vector magnitude *V.M* are determined with the equation suggested by Curray (1956). In equation (6.4), θ varies from 0° to 180° and *n* is the number of the observation (particles) in each group. The value of *V.M* in equation (6.5) presents the intensity of the preferred orientation of particle and ranges from zero for a completely random and unsystematic distribution of particle orientation to 100% for the same direction of all particles.

$$\theta = \frac{1}{2}\tan^{-1}\left(\frac{\sum_{i=1}^{n}\sin 2\theta}{\sum_{i=1}^{n}\cos 2\theta}\right) \tag{6.4}$$

$$V.M = \frac{100}{n}\sqrt{\left(\sum_{i=1}^{n}\sin 2\theta_i\right)^2 + \left(\sum_{i=1}^{n}\cos 2\theta_i\right)^2} \tag{6.5}$$

As listed in Table 6.2, the value of V.M obtained from both AP method and DV method is entirely higher than zero (greater than 9%), implying that both methods can create somewhat the inherent anisotropy. In the comparison between two methods, it is obvious that AP produces greater inherent anisotropy than DV because the average depositional θ is closer to α and value of V.M obtained from AP is much than that result in DV method.

Figure 6.24 (a) – 6.24(c) respectively present the distributions of sand particle orientation prepared using AP and DV methods at different deposition angles of α=0°, α=45°, and α=90°. The orientation of sand particle has the tendency to stand with higher frequency around deposition angle α. This tendency is more clearly for AP method than for DV method. Figure 25 indicates the relations between the initial shear modulus or normalized initial shear modulus and the average particle orientation particle. The specimen preparation method affects significantly

the orientation of sand particle. The average θ of AP method is distributed widely. However, in DV method, average θ show a narrow range in angle. Because of the difference on void ratio between AP and DV method, the initial shear modulus obtains a large difference between two methods. Therefore, the normalized method on void ratio e need to be carried out to remove its effects.

Figure 6.26 shows the relation between initial shear modulus G_0 normalized by void ratio *e* and the average angle θ in both saturated and unsaturated Toyoura sand. Effects of void ratio generated by AP and DV methods were removed using the void ratio function suggested by Iwasaki and Tatsuoka (1977).

$$F(e) = \frac{(2.17 - e)^2}{(1 + e)} \tag{6.6}$$

Figure 6.26 presents the relation between G_0 and θ, which is independent of testing methods (LSS and BE tests) and specimen preparation methods (AP and DV method). A good tendency was obtained from these data arrangements. These tendencies can be expressed for saturated and unsaturated Toyoura sand, as equation (6.7) and (6.8), respectively.

$$\frac{G_0}{F(e)} = 0.22\theta + 84.9 \tag{6.7}$$

$$\frac{G_0}{F(e)} = 0.29\theta + 91.2 \tag{6.8}$$

Two equations show that: unsaturated sand has a higher coefficient of 0.29 than 0.22 of saturated sand. Therefore, initial shear modulus of unsaturated sand has a slightly greater degree of anisotropy than saturated sand. Equations (6.7) and (6.8) are derived a new method to evaluate the anisotropy of initial shear modulus using the particle orientation data. This finding is suggested from data of Toyota sand. Thus, it is necessary to confirm the applicability to the other sands. However, the possible application of this achievement are as follows:

(1) In case the average particle orientation angle can be measured in an undisturbed sample, the initial shear moduli of every direction can be evaluated using these equations.
(2) In case the particle orientation angle cannot be measured, these equations can be applied to evaluate the anisotropy of initial shear moduli by using deposition angle of sands. Those are available for accuracy designing and construction of earth structures considering economic and safety factors.
(3) By using those equations, a series of complicated shearing tests cost money and time is no longer needed by using the suggested method.

Table 6.1. Summary vector magnitude *V.M* and average depositional angle *q* in air pluviation and dry vibration method

Specimen preparation method	Depositional angle, α (°)	Number of measured particles, *n*	Vector magnitude, *V.M* (%)	Average depositional angle, *q* (°)
Air pluviation	0°	336	25.12	15.60
	45°	325	36.00	54.70
	90°	300	29.41	79.6
Dry vibration	0°	275	24.29	42.4
	45°	354	19.73	57.5
	90°	325	19.11	54.2

Figure 6.21. Procedure for measuring particle orientation angle θ

Figure 6.22 Description of position in microscope view

Figure 6.23 An example of the typical picture from microscope work

Figure 6.24 Distribution of particle orientation of Toyoura sand prepared using AP and DV methods: (a) α=0°, (b) α=45°, and (c) α=90°

Figure 6.25 Relation between initial shear modulus and average θ for Toyoura sand: (a) G_0 for saturated sand, (b) G_{0_nor} for saturated sand, (c) G_0 for unsaturated sand, and (d) G_{0_nor} for unsaturated sand

Figure 6.26 Initial shear modulus normalized by void ratio versus average θ

6.8. Effects of void ratio on shear strength and shear modulus

6.8.1. Effects of void ratio on shear strength considering inherent anisotropy

Figure 6.26 to 6.45, respectively, show stress-strain and volumetric strain against shear strain relations of saturated Toyoura sand (AP and DV), mica 1%, mica 5% and mica 10% at different void ratio. The void ratio was measured before monotonic compression. Mica 5% and Mica 10% were conducted in different void ratio by changing the energy vibration as mentioned in chapter 4. All of results show that the repeatability was obtained, where stress-strain and volumetric strain against shear strain relations repeated with the soil at the soil same void ratio.

Figure 6.37 to Figure 6.45 shows the effects of void ratio on shear strength behavior of mica 5% and mica 10%. With dense relative density (at the void ratio *e* about 0.720-0.750 and 0.820-0.850 for mica 5% and mica 10%, respectively), the peak or maximum deviator stress was appeared. Moreover, the sand behaves as contractive initially and then dilative until the end of shearing. However, the loose and medium density of mica 5% (*e* about of 0.880-0.920 and more than 0.98 for medium and loose specimen, respectively) and loose density of mica 10% (e about 1.000) are much more contractive on the volumetric strain versus shear strain relation. Nevertheless, the peak strength was not obtained for both loose and dense mica 5% and loose mica 10%.

Figure 6.46 summaries the shear strength of mica 5% and 10% in different void ratio. The decrease of void ratio induces the increase of shear strength. However, the degree of anisotropy on shear strength is independent from variation of void ratio

Figure 6.47 shows the comparison of shear strength among different mica contents. It can be seen that, even using the same energy vibration, the mica content has the considerable effects on both void ratio and shear strength. The void ratio tends to increase with the increase of mica

content. Thus, the shear strength becomes smaller when mica content changes from 0% to 10%. Lee et al. (2007) proposed a “Ordering and Bridging” concept based on the size ratio L_{mica}/D_{sand} between the length of mica and the diameter of sand. Their research concluded that the void ratio generally increases with the increasing of mica content. However, at the ratio L_{mica}/D_{sand} = 0.33 and 0.66, small mica particles can decrease the void ratio by filled the voids between particle. At the ratio $L_{mica}/D_{sand} \geq 1$, the long mica particle generates the “ordering and bridging” caused the increasing of void ratio. Also, the occurrence of mica delays the appearance of the peak strain.

Figure 6.27 Repeatability of drained shear properties of saturated Toyoura sand (AP method) at 0 degree：(a) stress–strain and (b) volumetric strain.

Figure 6.28 Repeatability of drained shear properties of saturated Toyoura sand (AP method) at 22.5 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.29 Repeatability of drained shear properties of saturated Toyoura sand (AP method) at 45 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.30 Repeatability of drained shear properties of saturated Toyoura sand (AP method) at 67.5 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.31 Repeatability of drained shear properties of saturated Toyoura sand (AP method) at 90 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.32 Repeatability of drained shear properties of saturated Toyoura sand (DV method) at 0 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.33 Repeatability of drained shear properties of saturated mica 1% (DV method) at 0 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.34 Repeatability of drained shear properties of saturated mica 1% (DV method) at 22.5 degrees: (a) stress–strain and (b) volumetric strain.

Figure 6.35 Repeatability of drained shear properties of saturated mica 1% (DV method) at 45 degrees: (a) stress–strain and (b) volumetric strain.

Figure 6.36 Repeatability of drained shear properties of saturated mica 1% (DV method) at 67.5 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.37 Repeatability of drained shear properties of saturated mica 1% (DV method) at 90 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.38 Repeatability of drained shear properties of saturated mica 5% (DV method) at 0 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.39 Repeatability of drained shear properties of saturated mica 5% (DV method) at 22.5 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.40 Repeatability of drained shear properties of saturated mica 5% (DV method) at 45 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.41 Repeatability of drained shear properties of saturated mica 5% (DV method) at 90 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.42 Repeatability of drained shear properties of saturated mica 10% (DV method) at 0 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.43 Repeatability of drained shear properties of saturated mica 10% (DV method) at 0 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.44 Repeatability of drained shear properties of saturated mica 10% (DV method) at 22.5 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.45 Repeatability of drained shear properties of saturated mica 10% (DV method) at 22.5 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.46 Repeatability of drained shear properties of saturated mica 10% (DV method) at 0 degree: (a) stress–strain and (b) volumetric strain.

Figure 6.47 Effects of void ratio on shear strength of saturated sand

Figure 6.48 Effects of void ratio on shear strength of saturated sand

6.8.2. Effects of void ratio on shear modulus considering anisotropy

The previous studies demonstrated that, void ratio generally causes the decrease of shear modulus. Hardin and Richart (1963) and Hardin and Drnevich (1972) proposed an empirical expression for the initial shear modulus G_0, accounted for the variation in mean effective stress and void ratio at the isotropic stress state, was suggested as follows:

$$G_0 = AF(e)\left(\frac{\sigma_m^{'}}{p_a}\right)^n \tag{6.3}$$

Where where σ'_m is the confining stress; p_a is usually taken the value as the atmospheric pressure of 100kPa and defined as the reference pressure; *A* and *n* are fitting parameters varied by soil types; *F(e)* is the function of void ratio *e* and determined from different suggestions. The equation 6.3 implies that the initial shear modulus G_0 is significant affected by changing of void ratio *e*.

This study used two methods to evaluate the initial shear modulus namely: local small strain (LSS) test and bender element test.

The small-strain value obtained from LSS test are estimated through deformation values measured directly from triaxial test. Particularly, local axial strain ε_a (%) and radial strain are obtained from transducer displacement proximity. The secant shear modulus is calculated as follows.

$$q = \sigma_1^{'} - \sigma_3^{'} \tag{6.4}$$

$$\varepsilon_s = \frac{2}{3}\left(\varepsilon_a - \varepsilon_r\right) \tag{6.5}$$

$$G = \frac{q}{3\varepsilon_s} \tag{6.6}$$

Where, *q*: deviator stress (kPa),

σ'_a : vertical effective stress (kPa),

σ'_r : horizontal effective stress (kPa),

ε_s: shear strain (%),

ε_a: axial strain at small strain (%),

ε_r: radial strain at small strain (%),

G: secant shear modulus (Mpa).

Using the BEs method, small strain shear modulus G_o can be determined based on the elastic continuum mechanics:

$$G_0 = \rho_t V_s^2 = \rho_t \frac{L_{tt}^2}{\Delta t^2} \tag{6.7}$$

Where elastic shear velocity Vs can be found via the propagation distance L_{tt} and and Δt

propagation time. Bulk density ρ_t can be obtained after experiment.

Figure 6.49 and 6.50, respectively, show the comparison the obtained initial shear modulus on saturated and unsaturated sand of mica 5% and 10% between LSS and BE in terms of variation of void ratio. Initial shear modulus G_0 changed when void ratio decrease from 0.73 to 1 in both saturated and unsaturated sand. However, results obtained from BE test were greater than those results obtained on LSS test.

Figure 6.51 and 6.52, respectively, summarize the comparison the initial shear modulus between unsaturated sand and saturated sand of mica 5% and mica 10%. Obtained results show the same tendency on both LSS and BE test, initial shear modulus of unsaturated sand is higher than those results of saturated sand.

Figure 6.49 Effects of void ratio on initial shear modulus of mica 5% (DV method): (a) saturated sand and (b) unsaturated sand.

Figure 6.50 Effects of void ratio on initial shear modulus of mica 10% (DV method): (a) saturated sand and (b) unsaturated sand.

Figure 6.51 Effects of void ratio on initial shear modulus of mica 5% (DV method): (a) bender element test and (b) local small strain test.

Figure 6.52 Effects of void ratio on initial shear modulus of mica 10% (DV method): (a) bender element test and (b) local small strain test.

6.9. Evaluation repeatability of testing results

In this research, some tests were conducted more than one time to confirm the reliability of the testing results. Therefore, it is necessary to evaluate the repeatability on those obtained results. Repeatability is defined as the closeness of the agreement among the results of the tests using the same conditions.

In geotechnical engineering field, some researchers proposed coefficient of variation (*COV*) as the parameter to estimate the repeatability. In probability and statistics theory, *COV* is known as relative standard deviation (*RSD*) and defined as the ratio between standard deviation (SD) σ to the mean μ (average value). A smaller *COV* implies higher repeatability. The detailed calculation is shown in equations (6.14), (6.15) and (6.16).

$$COV = \frac{SD}{mean} = \frac{\sigma}{\mu} \tag{6.14}$$

$$\sigma = \frac{100}{N}\sqrt{\frac{\sum_{i=1}^{N}\left(x_i - \mu\right)^2}{N-1}} \tag{6.15}$$

$$\mu = \frac{1}{N}\left(\sum_{i=1}^{N} x_i\right) \tag{6.16}$$

Where:

COV, σ and μ are coefficient of variation, standard deviation and mean, respectively.

N: number of observations in the sample

x_i: observed values of the sample items.

μ: the mean value of the observation.

Figure 6.53 summaries the shear strength results obtained in different tested times in Toyoura sand (AP and DV method), mica 1%, mica 5% and mica 10%. Results show that the tendency is not changed compared to Figure 6.27

Table 6.4 and 6.5 show the *COV* values of shear strength and shear modulus obtained from BE in saturated sand. Both results had a *COV* less than 5%, demonstrating that test obtained a high repeatability.

Table 6.2. Values of coefficient of variation for geotechnical engineering field

Parameter	Researcher	COV
Unit weight	Harr (1984), Duncan (2000) Schweiger (2005)	1-10%
Friction angle (φ')	Harr (1984), Kalhawy (1992) Schweiger (2005)	2-15%
Undrained strength ratio (S_u/σ'_v)	Lacasse and Nadim (1997), Duncan (2000)	20-50%
Undrained shear strength (S_u)	Harr (1984), Kulhawy (1992), Lacasse and Nadim (1997), Duncan (2000)a	13-40%
Compression index (C_c)	Harr (1984), Kulhawy (1992), Duncan (2000)	10-37%

Figure 6.53 Summary the maximum shear strength in different tested time

Table 6.3. The repeatability of shear strength test of saturated sand

Materials	α	Number of test	Standard deviator, σ	Mean, μ	Coefficient of variation, COV (%)
AP Toyoura sand	0	5	7.04	655.23	1.07
	22.5	4	8.24	631.11	1.31
	45	2	12.98	560.25	2.32
	67.5	2	9.25	536.99	1.72
	90	1	4.74	541.75	0.88
DV Toyoura sand	0	3	19.19	532.82	3.6
	22.5	1	-	-	
	45	1	-	-	
	67.5	1	-	-	
	90	1	-	-	
Mica 1%	0	4	7.87	483.75	1.63
	22.5	4	6.40	465.23	1.38
	45	2	1.88	459.02	0.41
	67.5	5	3.92	438.41	0.90
	90	5	10.89	449.76	2.42
Mica 5%	0	7	9.26	367.04	2.52
	22.5	2	3.71	354.11	1.05
	45	3	14.43	344.95	4.18

	67.5	1	-	-	
	90	3	12.67	344.46	3.68
Mica 10%	0	2	3.70	295.71	1.25
	22.5	4	2.68	295.54	0.91
	45	2	6.80	304.89	1.25
	67.5	2	1.75	304.78	0.58
	90	2	2.31	306.64	0.75

Table 6.4. The repeatability of bender element test of saturated sand

Materials	α	Number of test	Standard deviator, σ	Mean, *μ*	Coefficient of variation, *COV* (%)
AP Toyoura sand	0	2	0.0	130.0	0.00
	22.5	5	1.0	133.7	0.75
	45	3	0.1	137.2	0.32
	67.5	3	0.5	138.9	0.39
	90	3	1.5	146.8	1.05
Mica 10%	0	4	1.1	88.0	1.27
	22.5	4	1.0	92.3	1.12
	45	2	1.6	94.6	1.72
	67.5	1	-	102.3	-
	90	1	-	109.8	-

CHAPTER 7. INDUCED ANISOTROPY OF COHESIVE SOIL

7.1. Introduction

This chapter presented the results obtained from a series of triaxial test performed on Yoneyama sandy silt for investigation the effects of induced anisotropy on small strain characteristic. The effects of anisotropy on shear modulus were investigated using the testing conditions (testing method) introduced in Chapter 5. Specimen was a consolidated under different value of stress ratio *K* defined as the horizontal effective stress σ'_h and vertical effective stress at compression anisotropic condition (*K*=0.35; 0.43; 0.6; 0.8), extension anisotropic condition (*K*=1.5; 2.0; 3.0;3.5) and isotropic condition (*K*=1). Moreover, the effects of void ratio, over consolidation ratio, effective stress on initial shear modulus G_0 also are discussed.

7.2. Effects of induced anisotropy on shear modulus

7.2.1. Saturated soil

(a) Bender element test

Figure 7.1 presents the effects of *K* consolidation on the initial shear modulus G_0 obtained from BE test of saturated Yoneyama clay. All of the test were conducted under the conditions as mentioned in Chapter 5. Test results show a good repeatability. *K* of 1.0, 0.8, 0.6, 0.43 and 0.35 were selected to conduct the *K* consolidation. Therefore, the specimen was compressive during consolidation. It can be seen that initial shear modulus obtains the greatest value at *K* of 0.43 and smallest value at *K* of 1.0 implies the effects of anisotropy on G_0.

On the other side, Figure 7.2 shows the relationship between G_0 and *K* in terms of extension consolidation. Initial shear modulus G_0 increases with increase of *K* from 1.0 to 2.0, G_0 obtained the highest value of 128.7 MPa at *K* = 2.0. Then, G_0 decreases gradually to 108.9 MPa at *K* =3.5.

To evaluate the degree of anisotropy induced by anisotropy consolidation, the normalized method defined by the slope of the relation between the normalized initial shear modulus ($G_{0\,(K)}/G_{(K=1)}$ and *K*. Figure 7.3 shows the degree of anisotropy, expressed as normalized initial shear modulus ($G_{0\,(K)}/G_{(K=1)}$ and stress ratio. The obtained results show that G_0 of *K* = 0.43 and *K* =3.5 shows the highest value and smallest value, respectively, at 1.14 and 0.88.

(b) Monotonic compression test

Figure 7.4 presents the effects of compressive *K* consolidation on initial shear modulus G_0 obtained from LSS test in compression anisotropy consolidation. It is apparent that secant shear modulus of isotropic stress state at shear strain less than 0.002 % (*K* = 1.0) obtained the lowest value compared to those results at anisotropic stress state under compressive *K* condition. On the other side, at the same shear strain, secant shear modulus of *K* = 0.43 reaches the highest

value. However, this tendency becomes reversely when shear strain rate is larger than 0.01%.
Figure 7.5 shows the comparison of initial shear modulus among the different extension *K* consolidation obtained from LSS test. Experiment is conducted for case of *K* of 1.0; 1.5; 2 and 3. Secant shear modulus at shear strain smaller than 0.002% show the smallest result of K=3.0, while that result of *K*=2.0 obtained the highest value.
Figure 7.6 summarizes the initial shear modulus in relationship with variation of *K*. The obtained results can be described as follows:

- G_0 of anisotropy consolidation with compression consolidation increases with decrease of *K* from 1.0 to 0.43. However, when *K* decreases from 0.43 to 0.35, G_0 tend to be smaller. The reason might be the failure plane appeared at *K*=0.35.
- G_0 of anisotropy consolidation with extension consolidation history increases with the increase of *K* from 1.0 to 2.0.

(c) Comparison shear modulus obtained from BE and monotonic compression test
The comparison on initial shear modulus G_0 between LSS and BE test is shown in Figure 7.7. G_0 obtained from BE test is slightly higher than those results obtained from LSS test. At the isotropic *K*=1, that difference is higher compared to anisotropic consolidation, even *K* smaller or larger than 1.0.
Figure 7.8 presents the comparison between LSS and BE in terms of initial shear modulus of anisotropic consolidation normalized to that result of isotropic consolidation in relationship to the stress ratio *K*. Detail values mentioned in Table 7.1 demonstrated that the G_0 of anisotropic consolidation (*K*<1.0) are ranged from 5% to 17% larger than those from isotropic condition. On the other side, when *K*>1.0 or anisotropic under extension consolidation, the differences are increased from 1.0 to 1.06 with the increases of *K*=1.0 to 2.0. Then, the difference decreases to 0.88 when *K* is equal to 3.5.

Table 7.1. Degree of anisotropy induced by anisotropy consolidation

Stress ratio, *K*	0.35	0.43	0.6	0.8	1.0	1.5	2.0	3.0	3.5
$(G_{0\,(K)}/G_{(K=1)}$ LSS test		1.17	1.09	1.06	1.0	1.01	1.06	0.92	
$(G_{0\,(K)}/G_{(K=1)}$ BE test	1.14	1.16	1.07	1.05	1.0	1.02	1.04	0.91	0.88

(d) Effects of stress direction on initial shear modulus
Figure 7. 9 and 7.10, respectively, show the relationship between initial shear modulus normalized effective stress in horizontal and vertical direction. The value n_i and n_j are the

empirical coefficient that account for effects of effective stress in vertical σ_v and horizontal, respectively. By changing the value of n_i and n_j in equation (7.1), the effects of stress direction can be evaluated. The results show that with n_i=0.55 and n_j=0.45, the initial shear modulus of *K* less than 1 becomes equally, while the those of results of *K* more than 1 still increases. However, with n_i = 0.3, n_j=0.7, initial shear modulus is the same with the increase of *K* from 1.0 to 3.5, meanwhile initial shear modulus is varied with the increase of K from 0.35 to 1.0.

These results imply that the initial shear modulus of saturated soil clearly exhibits anisotropy as stress ratio changes. Moreover, at *K* less than 1, the effect of vertical stress on initial shear modulus is more significant than that effects of horizontal stress. However, at *K* more than 1, those effects on initial shear modulus obtain the opposite tendency, compared to *K* less than 1.

$$G_0 = S_{ij}.F(e).OCR^k.\sigma_r^{1-ni-nj}.\sigma'^{ni}.\sigma'^{nj}(MPa) \qquad (7.1)$$

Figure 7.1 Effects of compression *K* consolidation on initial shear modulus of saturated Yoneyama clay of BE test

Figure 7.2 Effects of extension *K* consolidation on initial shear modulus of saturated Yoneyama clay of BE test

Figure 7.3 Degree of anisotropy on G_0 from BE test of saturated soil

Figure 7.4 Secant shear modulus of saturated soil in compression *K* consolidation of LSS test

Figure 7.5 Secant shear modulus of saturated soil in extension *K* consolidation of LSS test

Figure 7.6 Effects of K consolidation on G_0 in LSS test of saturated soil

Figure 7.7 Comparison G_0 obtained from LSS and BE test of saturated soil

Figure 7.8 Comparison the degree of anisotropy between LSS and BE test of saturated soil

Figure 7.9 Normalized the initial shear modulus G_0 to effective stress at K<1 (saturated soil)

Figure 7.10 Normalized the initial shear modulus G_0 to effective stress at K>1 (saturated soil)

7.2.2. Unsaturated soil

(a) Bender element test

Figure 7.9 shows the effects of K consolidation on initial shear modulus G_0 obtained from BE test of unsaturated soil. The results show that G_0 is independent from K, which G_0 is almost maintained about 190Mpa.

Figure 7.10 presents the initial shear modulus G_0 of anisotropy consolidation normalized to that results of isotropy consolidation. The ratio $G_{0_K}/G_{0_(K=1)}$ is almost 1.0, as presented in Table 7.2.

(b) Monotonic compression test

The secant shear modulus in relation to different consolidation condition is shown in Figure 7.11. The obtained results present that shear modulus at shear strain less than 0.01% is independent from stress ratio K. However, at shear strain larger than 0.005%, shear modulus tends to be increase with the increase of K from 0.35 to 3.0.

(c) Comparison shear modulus obtained from BE monotonic compression test

The initial shear modulus obtained from BE and LSS test shows almost the same results, as presented in Figure 7.12. As a result, the degree of anisotropy defined as the G_0 of different stress ratio K (anisotropy condition) compared to G0 of K of isotropy condition is almost equal to 1.0, as shown in Table 7.2.

Table 7.2. Degree of anisotropy induced by anisotropy consolidation of unsaturated soil

Stress ratio, K	0.35	0.43	0.6	0.8	1.0	1.5	2.0	3.0	3.5
$(G_{0\,(K)}/G_{(K=1)}$ LSS test	0.97	1.00			1.00			1.02	
$(G_{0\,(K)}/G_{(K=1)}$ BE test	0.99	1.01			1.00	1.01	1.00	1.03	

7.2.3. Comparison effects of anisotropy consolidation between saturated test and unsaturated test

Figure 7.13 to 7.15 present the comparison between saturated soil and unsaturated soil in BE test, LSS test and degree of anisotropy, respectively. The obtained results can be described as follows:

- Initial shear modulus of unsaturated soil is greater than those results of saturated soil because of application of matric suction in both LSS and BE test.
- Anisotropy shows stronger effects on saturated soil than unsaturated soil.

The similar void ratio between saturated soil and unsaturated soil, as presented in Figure 7.17, demonstrated that the application of matric suction does not have the influence on the variation of void ratio

Figure 7.11 Effects of compression *K* consolidation on initial shear modulus of unsaturated Yoneyama clay of BE test

Figure 7.12 Degree of anisotropy on G_0 from BE test of saturated soil

Figure 7.13 Secant shear modulus of unsaturated soil in different K consolidation of LSS test

Figure 7.14 Comparison G_0 obtained from LSS and BE test of unsaturated soil

Figure 7.15 Comparison effects of K consolidation on G_0 (BE test) between saturated soil and unsaturated soil

Figure 7.16 Comparison effects of K consolidation on G_0 (LSS test) between saturated soil and unsaturated soil

Figure 7.17 Comparison degree of anisotropy on G_0 (BE test) between saturated soil and unsaturated soil

Figure 7.18 Comparison the variation of void ratio between saturated soil and unsaturated soil

7.3. Effects of induced anisotropy on initial shear modulus at various stress state

7.3.1. Effects of various stress state on initial shear modulus

Figure 7.17 to 7.20 show the effects of various stress state under both isotropic consolidation (*K*=1) and anisotropic consolidation (*K*=0.35; 0.43 and 0.6). As presented in Chapter 5, after applying *q*-loading step to reach the desired *K*, specimen was kept at a constant *K* (defined in q-loading step), and effective stress *p'* was increase from 50kPa to 100kPa, 200kPa, 300kPa, 400kPa, 500kPa and 600kPa for consolidation stress state. Then *p'* was decreased from 600kPa to 500kPa, 400kPa, 300kPa, 200kPa, 100kPa and 50kPa for swelling stress state. BE test was conducted after 24 hours from p' reached the targeted value.

Some conclusions can be given from the obtained results as follows:

- The level of increase of initial shear modulus on consolidation (effective stress increases from 50kPa to 600kPa) is higher than those on swelling (effective stress decreases from 600kPA to 50kPa).
- Results of initial shear modulus G_0 obtained from swelling test is higher than those results obtained from consolidation, implies that OCR has the significant effects to the initial shear modulus.

7.3.2. Variation of void ratio

Figure 7.21 to 7.24 show the changing of void ratio under consolidation and swelling condition of different stress state history. It can be seen that the void ratio increases/decreases with the decreases/increases of effective stress, respectively.

Figure 7.25 - 7.28 present the variation of void ratio with the comparison between that value at selected effective stress and void ratio of effective stress of 50kPa. Under consolidation condition, void ratio of effective stress 50kPa is smaller 20% than void ratio of effective stress of 600kPa. However, under swelling condition, void ratio increases about only 3% with the decreases of effective stress of 600kPa to 50kPa.

In the comparison among different *K*, the variation of void ratio is very scatter and not linear. However, the variation of void ratio among different *K* is quite similar. Thus, anisotropic consolidation does not play the important role in the variation of void ratio.

7.3.3. Normalized void ratio and OCR on initial shear modulus

As reviewed in Chapter 2, the dependence of initial shear modulus on void ratio is generally summarized in some void ratio function as follows.

$$F(e) = \frac{(a-e)^2}{1+e} \tag{7.2}$$

Where *a* = 1.46; 1.5; 2.170; 2.174; 2.27; 2.973; 4.4 and 7.32

$$F(e) = e^{-x} \tag{7.3}$$

Where *x* = 1.1; 1.3; 1.27; 1.33; 1.43; 1.5 and 1.52

$$F(e) = (1+e)^{-x} \tag{7.4}$$

Where *x* = 2.4 and 3

Depend on the value of void ratio, the choice of void ratio function can be different. This research uses the function in equation (7.2) with *a*=2.973 because the void ratio varies from 0.78-0.98. Figure 7.29 presents the relationship between the initial shear modulus normalized void ratio in relationship with effective stress. Even applying the normalizing void ratio, the difference on the initial shear modulus between consolidation and swelling is still existed. However, with the adding the effects of *OCR*, the normalized initial shear modulus between consolidation and swelling becomes similar. Figure 7.30 shows an example of initial shear modulus normalized both void ratio and *OCR* using void ratio function of equation (7.2) of *K*=0.35. The results show that initial shear modulus becomes independently between swelling and consolidation. Thus, void ratio and *OCR* play important roles in the changing of effective stress.

7.3.4. Comparison initial shear modulus normalized void ratio and OCR among different stress

ratio *K*

Figure 7.31 shows the relationship between initial shear modulus normalized void ratio and *OCR* with variation of anisotropy effective stress among *K*=1.0; 0.6; 0.43 and 0.35. The void ratio function (7.2) with a=2.973 was used. Value of *k =0.2* was chosen based on the plastic index from I_p from the suggestion of Hardin and Drnevich (1972), as shown in Table 7.2. Even the void ratio and *OCR* are normalized, the difference of initial shear modulus among different stress ratio *K*, implies that void ratio and OCR do not play the important role on the variation of initial shear modulus. The obtained results show that G_{nor} of *K*=0.43 obtained the highest value and followed by those results of *K*=0.35. While G_{nor} of *K*=1.0 reached the lowest value.

To confirm the reliability of tendency, the void ratio function $F(e)=e^{-1.5}$ is used for normalizing initial shear modulus from void ratio and *OCR*. Figure 7.32 shows the same trend obtained in Figure 7.31.

Table 7.3. Description value of k base on plastic index I_P (Hardin and Drnevich, 1972)

I_P	k
0	0
20	0.18
40	0.30
60	0.41
80	0.48
≥100	0.50

Figure 7.19 Effects of effective stress on G_0 at K of 0.35 (saturated soil)

Figure 7.20 Effects of effective stress on G_0 at K of 0.43 (saturated soil)

Figure 7.21 Effects of effective stress on G_0 at K of 0.6 (saturated soil)

Figure 7.22 Effects of effective stress on G_0 at K of 1.0 (saturated soil)

Figure 7.23 Effects of effective stress on void ratio e at *K* of 0.35 (saturated soil)

Figure 7.24 Effects of effective stress on void ratio e at *K* of 0.43 (saturated soil)

Figure 7.25 Effects of effective stress on void ratio e at *K* of 0.6 (saturated soil)

Figure 7.26 Effects of effective stress on void ratio e at *K* of 1.0 (saturated soil)

Figure 7.27 Variation of void with effective stress at *K* of 0.35

Figure 7.28 Variation of void with effective stress at *K* of 0.43

Figure 7.29 Variation of void with effective stress at *K* of 0.6

Figure 7.30 Variation of void with effective stress at *K* of 1.0

Figure 7.31 Example of initial shear modulus G_0 normalized void ratio at K of 0.35

Figure 7.32 Example of initial shear modulus G_0 normalized void ratio and OCR at K of 0.35

Figure 7.33 Initial shear modulus G_0 normalized void ratio e and OCR (case 1)

Figure 7.34 Initial shear modulus G_0 normalized void ratio e and OCR (case 2)

CHAPTER 8. CONCLUSIONS AND RECOMENDATIONS

The effects of anisotropy on mechanical properties of soil were conducted in this study. Two type of anisotropies, namely inherent anisotropy and induced anisotropy were separately investigated using granular material and cohesive materials, respectively.

Inherent anisotropy related to the initial soil particle's fabric was created using a container with movable walls. Toyoura sand with different mica contents: 0%, 1%, 2.5%, 5% and 10% was deposited at different angles of 0°, 22.5°, 45°, 67.5° and 90°. Two specimen preparation method namely air pluviation (AP) and dry vibration (DV) were used to make specimen. Conventional triaxial tests was used to obtain shear properties. Small strain stiffness (shear modulus) was acquired from both bender element (BE) and local small strain (LSS) tests. Two-dimensional optical microscopic measurements and mathematical estimation were used to assess the orientation of sand particle.

Induced anisotropy, referred to the applied stress state during consolidation history of soil, was investigated using cohesive soil. Stress ratio *K*, defined as the ratio between horizontal effective stress to vertical effective stress, was used to created different history of consolidation. *K* of 0.35, 0.43, 0.6, 0.8 was used for compression anisotropy consolidation, whereas *K* of 1.5, 2.0, 3.0 and 3.5 was used as the extension anisotropy consolidation. *K* of 1.0 was employed as the isotropic condition. Two series of testing were conducted as follows.

(1) Specimen was consolidated at different condition of K at effective stress of 300kPa. BE and LSS tests were carried out to obtained initial shear modulus
(2) Specimen was consolidated at different *K* of 1.0; 0.6; 0.43 and 0.35 at different effective stress in both consolidation and swelling condition.

The conclusions of this study can be summarized as follows.

8.1. Conclusions

8.1.1. On inherent anisotropy conducted works

(1) The air pluviation method can produce the particle orientation, which create a greater degree of anisotropy than dry vibration
(2) The shear strength decrease with the deposition angle changes from a horizontal (0°) to vertical direction (90°). However, the initial shear modulus exhibits higher initial shear modulus with the deposition angle of 90°, which is completely the reverse relationship to shear strength.
(3) Both shear strength and shear modulus decrease with increase of mica contents. On those results, anisotropy of shear strength disappears by content of mica. However, anisotropy of shear modulus increase with increase of mica contents.
(4) Shear strength and shear modulus of unsaturated soil are higher than those results of

saturated sand. Moreover, degree of anisotropy of both shear strength and shear modulus of unsaturated sand is higher than degree of anisotropy of unsaturated sand The reason for this difference can be considered on the application of matric suction.

(5) The increase of mica contents results the increase of void ratio. As a results, initial shear modulus becomes small with higher mica content. However, at mica 10%, even using G_0 normalized void ratio, the degree of anisotropy of mica 10% is still higher than the lower mica content.

(6) Anisotropy of initial shear modulus can be evaluated using the average particle orientation angle of sand. From that point, two equations were suggested to evaluate the effects of particle orientation on initial shear modulus G_0 normalized the function of void ratio $F(e)$:

$$\frac{G_0}{F(e)} = 0.22\theta + 84.9 \tag{8.1}$$

$$\frac{G_0}{F(e)} = 0.29\theta + 91.2 \tag{8.2}$$

Where θ is the average deposition angle of sand particle.

Although these equations are suggested based on the conducted works on Toyoura sand. Thus, it is necessary to confirm the applicability of this proposal to other sands. However, the following applications are possible.

- When choosing value of initial shear modulus for designing and constructing, by changing value of deposition angle θ, the range of variation on G_0 can be obtained. From that variation, the economic and safety factors can be evaluated.
- If the evaluation of deposition angle is possible, a series of complicated shear tests is no longer need compared to using proposed method. This allows for avoidance of the waste of time of money.

8.1.2. On induced anisotropy conducted works

(1) Initial shear modulus of anisotropy consolidation is higher than that result of isotropic consolidation

(2) Initial shear modulus of anisotropy consolidation in compression condition is higher those results of extension consolidation.

(3) The initial shear modulus increase with the increase of effective stress and void ratio

(4) The initial shear modulus exhibits greater value on swelling than those results on consolidation because of effects of overconsolidation.

(5) The effects of stress ration $K=\sigma'_h/\sigma'_v$ to initial shear modulus is independent from influence of void ratio and OCR to initial shear modulus.

8.2. Recommendations

The recommendations for further research are as follows.

(1) It is necessary to consider the effects of inherent anisotropy and induced anisotropy when choosing initial shear modulus G_0. Initial shear modulus at both vertical and horizontal direction should be considered to understand the range of variation. From that point, the safety factor can be evaluated based on that variation.

(2) The appearance of mica need to be consider carefully due to the reduction of shear strength and shear modulus. Also, the safety factor needs to be paid attention with the appearance of mica contents due to the high degree of anisotropy

(3) There are some differences on the behaviors of soil between saturated sand and unsaturated in terms of inherent anisotropy. Therefore, it is necessary to separate two kind of soils when selecting parameter for designing and constructing.

8.3. Further research

Based on the obtained results in this study, the author suggests some conducted works for further research as follows.

(1) On the topic related to inherent anisotropy, the current research has been investigated the effects of particle orientation on mechanical anisotropy properties. Two materials with different shapes was used. The parameters of sphericity and roundness were considered while particle roughness is still neglected. Therefore, it is necessary to investigate the influence of three parameters on inherent anisotropy behavior of soil.

(2) The assessment of particle orientation has been conducted in 2-D optical measurement. Thus, the particle orientation can be approached in 3-D measurement.

(3) Finally, the combination between inherent and induced anisotropy is not considered in this study. This topic should be considered due to the relationship between particle orientation and how the soil behaves under different consolidation conditions.

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