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Analyze the Deformation Characteristics of Cohesive Soil at Various Stress Conditions

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ABSTRACT

Large-scale deep excavations have become more common in recent years because of the growing need for significant subsurface space usage. The process of gradual stress release during excavations causes upward ground movement, which is known as rebound or heaving deformation. The rebound amount has been estimated using conventional tests and empirical approaches. In the laboratory, the deformation coefficient, which is used to predict rebound, has been determined through monotonic compression or cyclic loading tests. However, these deformation coefficients are not appropriate for capturing the rebound phenomenon, where the soil swells in the extension direction. This study examined the deformation characteristics of undisturbed and reconstituted samples to evaluate rebound phenomenon for underground constructions.For this objectives, triaxial extension tests using K0 consolidated specimens were considered as a most reliable test. In practice, triaxial tests with local small strain (LSS) measurement, bender element (BE), and shear strength tests were conducted under K0 and K consolidations. Initial and secant shear modulus (G_0, Gsec) were obtained under compression and extension loading.Triaxial tests, incorporating with bender element (BE) tests and local small-strain (LSS) tests, were conducted on undisturbed and reconstituted specimens extracted from a metro construction site (depths: 13.50–14.50 meters and 30.50–31.50 meters). The following findings are obtained from the study. G0 is largely different by depths and layers. Therefore, G0 should be measured using in-situ methods. To compare Gsec between undisturbed and reconstituted specimens, Gsec was normalized by each G0, referred to as normalized Gsec. The degradation of normalized Gsec from extension loading of reconstituted samples is approximately consistent with that of undisturbed ones. These findings contribute to accurate estimation of rebound amount using disturbed soil samples, which are easily and inexpensively obtained compared to undisturbed soil samples. As a result, reconstituted soil can be replaced from undisturbed soil in the small strain test for rebound evaluation. The use of Gsec from compressional loadings will lead to overestimate rebound amount than that from extensional loadings.

Introduction

The rebound phenomenon occurs due to the release of stress in the ground following excavation (Lambe and Whitman, 1991), and its underestimation can significantly affect the stability and deformation of underground structures. However, accurately predicting rebound deformation remains a challenging issue because conventional approaches based on empirical equations and laboratory tests cannot fully capture the tensile expansion associated with soil rebound (Lambe, 1967). Therefore, it is necessary to develop reliable methods for estimating rebound deformation and to improve the design criteria used for deep excavation projects.The deformation characteristics of soil at small strains, particularly its response to unloading caused by excavation, are essential for evaluating ground rebound. Rebound deformation is generally estimated using soil deformation coefficients, which are influenced by the initial stress state, excavation depth, and surrounding ground conditions (Li et al., 2023). The coefficient of earth pressure at rest (K_0), first conceptually introduced by Terzaghi (1920) and later empirically defined by Jaky (1944), is particularly important because it governs the horizontal effective

stress in horizontally semi-infinite ground. Recent studies, such as Huang et al. (2018), have investigated soil behavior under different stress paths, including unloading after K_0-consolidation, to evaluate the tangent modulus. These approaches allow soil stiffness parameters to be determined under stress conditions that are more representative of the in-situ state (Berre, 1973), indicating that unloading tests are important for understanding the soil response to excavation-induced stress release.

The key parameters considered in this study are the initial shear modulus (G_0) and secant shear modulus (G_{sec}), which represent the soil's resistance to shear deformation at small strain levels. Atkinson (2017) reported that shear strains in the ground around various underground structures are generally less than 0.1%, highlighting the importance of small-strain soil behavior in ground deformation analysis. Since soils exhibit strongly nonlinear mechanical behavior (Puzrin and Burland, 1998), different testing techniques are required to characterize stiffness over a wide range of strain levels, as shown in Fig. 1. In this study, bender element (BE) tests were used to determine stiffness at very small strains (less than 0.001%), while local small-strain (LSS) devices were used to measure stiffness in the small-strain range of 0.001% to 1%. The combination of these techniques provides a more comprehensive understanding of soil stiffness across different strain levels and helps to improve the evaluation of rebound behavior.

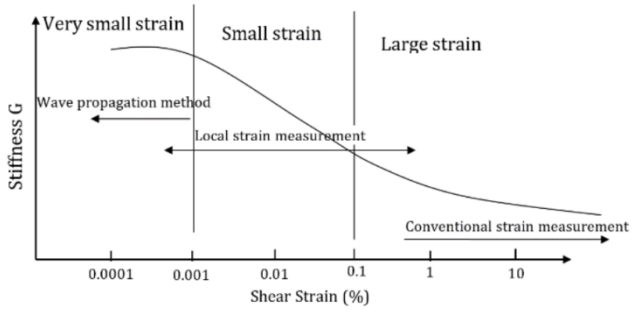

Fig. 1. Stiffness degradation curve of soil from laboratory tests (Atkinson, 1991; Likitlersuang et al., 2013).

2 Testing material and specimen preparation

2.1 Undisturbed specimen

Cylindrical soil samples with an initial height of 200 mm and a diameter of 75 mm were collected from two different depths in the field: 3Tr-13 (13.50–14.50 m) and 3De-30 (30.50–31.50 m). A Denison (rotary double-tube) sampler was employed to preserve the in-situ soil conditions and minimize disturbance to the natural soil structure, density, and moisture content of the cohesive soils.

The undisturbed samples collected from a depth of 13.50–14.50 m are shown in Fig. 2. The soil, referred to as tuffaceous sandy clay, consists of a mixture of sand, clay minerals, and volcanic ash (tuff). Its ochre colour may be attributed to the presence of oxides. The volcanic material may significantly influence the mechanical and geotechnical behaviour of the soil compared with conventional clays. The samples collected from the deeper layer, 3De-30 (30.50–31.50 m), are shown in Fig. 3 and primarily consist of sandy-clay particles with fragments of seashells. This soil displays dark Gray to blackish colours.

The physical properties and particle-size distributions of the two soils are presented in Table 1 and Fig. 4, respectively. Although both soils are classified as sandy clay, they show different particle-size distributions.

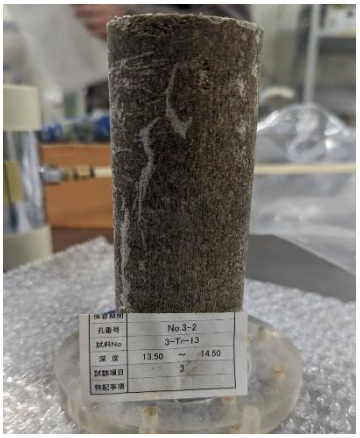

Fig. 2. Sample3Tr-13.

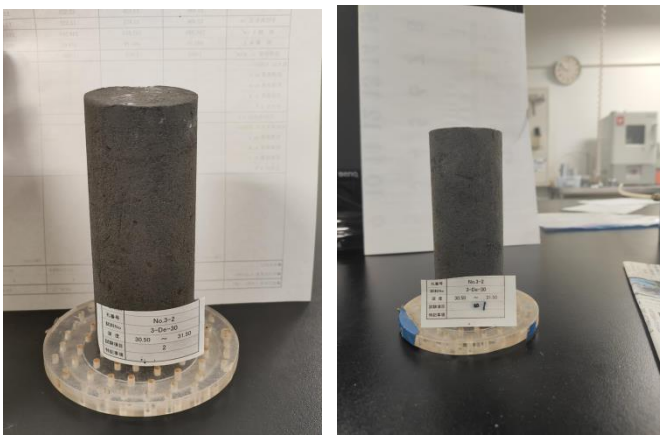

Fig. 3. Sample 3De-30.

Table 1. Physical properties of soils.

Properties	3Tr-13	3De-30
Specific gravity	2.789	1.995
Sand (%)	85.12	55.03

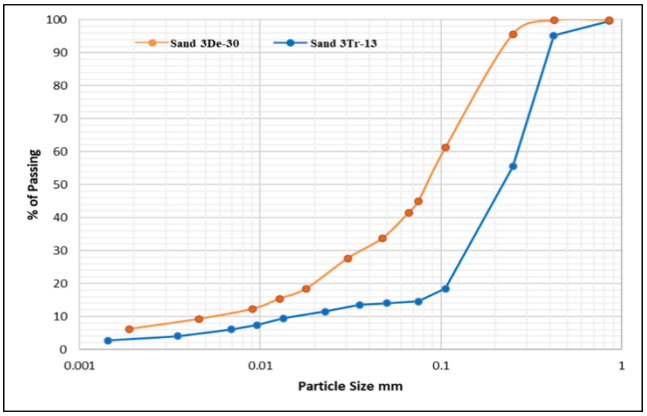

Fig. 4. Particle size distribution curves.

2.2 Reconstitutedspecimenpreparation

After testing, the undisturbed soil was crushed and passed from sieveof 0.425 mm to obtain a reconstituted mixture: For sample 3Tr-13 almost85% sand and 25% clay also for sample 3De-30 almost55% sand and 45% clay. To prepare the reconstituted sample I used small mold. Initially I set the small mold in tri-axial chamber as shown in Fig. 5.After that,by using funnel I pored the sandy soil

sample inside the rubber membrane and small mold. Finally,I released the small mold from outside of the rubber membrane.

Fig. 5. Reconstituted specimen preparation (Small mold).

3 Testing apparatus

A series of K_0-consolidated drained (CD) triaxial tests was conducted using a triaxial apparatus equipped with bender elements (BEs) and a local small-strain (LSS) measuring device to investigate the deformation characteristics of the soil.

The detailed descriptions of these measurement

3.1 Bender element test

The bender element (BE) test is a non-destructive method used to determine the initial shear modulus (G_0) of soil by measuring the shear wave transmitted through a soil specimen, following the standardized procedure specified in JGS 0544 (Japanese Geotechnical Society, 2011). BEs consist of piezoelectric ceramic bimorphs that deform when an electrical voltage is applied, thereby generating a shear wave that propagates through the specimen. Two BEs are installed at opposite ends of the soil specimen, with one serving as the transmitter and the other as the receiver. Each BE has dimensions of 2.5 mm in length, 12 mm in width, and 1 mm in thickness. Figure 6 shows a representative example of the input and received waves obtained from the BE tests.

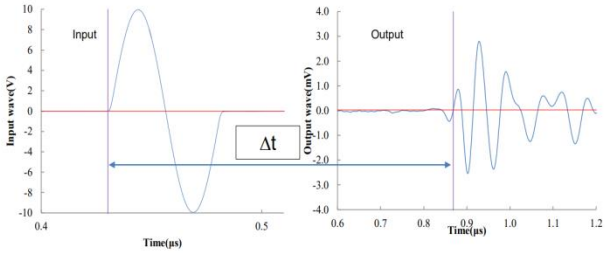

Fig. 6. Transmitted wave and received wave in BEtest.

3.2 Local small strain test

Accurate evaluation of soil stiffness at small strain levels (0.001%–0.1%) is essential for understanding the deformation behavior of soil. In this study, local small-strain (LSS) devices were integrated into the triaxial apparatus to directly measure the local axial and radial strains of the specimen. Two targets, spaced 80 mm apart, were attached to the membrane of the specimen, while the transducers were mounted on a separate pole with an adjustable target-to-transducer distance. An additional transducer was installed at the center of the specimen to measure its radial deformation. The transducers operate based on the eddy-current principle and measure the distance between the metal target and the sensor head (Le et al., 2020). Figure 7 shows the LSS setup and the devices used during the testing.

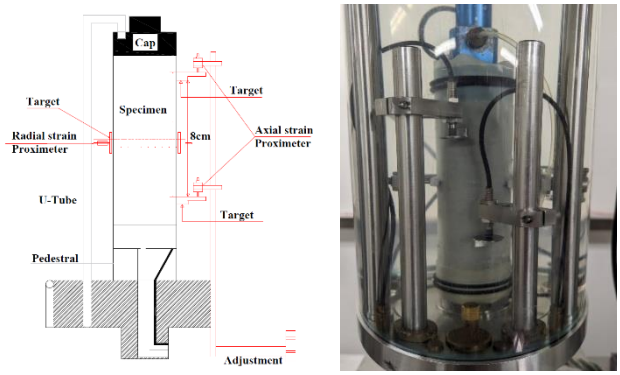

Fig. 7. Local small strain devices.

4 Testing procedure

A A cylindrical specimen with a height of 125 mm and a diameter of 50 mm was placed in the triaxial cell. Initially, a suction of −20 kPa was applied, and the specimen was saturated using the double-vacuum method (Rad and Clough, 1984). The degree of saturation was confirmed by a B-value (Skempton, 1954) greater than 0.95. While maintaining the suction at −20 kPa, the cell water was removed, and the dimensions of the specimen were carefully measured. The LSS transducers were then installed inside the triaxial cell without disturbing the specimen, after which the cell was filled with de-aired water. The suction was gradually reduced while increasing the cell pressure to maintain a constant isotropic effective stress of 20 kPa. Finally, a back pressure of 200 kPa was applied to further improve the saturation of the specimen.

For K_0 consolidation, the effective axial stress (σ_a') was gradually increased at an axial strain rate of 0.005%. The target axial effective stress was set to 100 kPa for 3Tr-13, and for 3De-30, corresponding to the estimated in-situ overburden pressure. The lateral stress was automatically controlled to restrain lateral deformation of the specimen and maintain the K_0 condition. After σ_a' reached the target value, consolidation was continued at the same K_0 value for 24 hours. A BE test was then performed to determine the initial shear modulus. Subsequently, the specimen was sheared under both compression and extension loading while maintaining a constant cell pressure to reproduce the stress-release conditions associated with ground rebound. The secant shear modulus (G_{sec}) was calculated from the LSS measurements obtained at an axial strain rate of 0.0025%/min.

5 Testing results and discussion

5.1 K_0consolidation stress path

Figure 8 (a)presents the K_0 consolidation stress path for 3Tr-13 samples with σ'_a = 100 kPa. The deviator stress$q$and the mean effective stress$p'$are defined by the following equations:

$$q = \sigma_a' - \sigma_r' \qquad (1)$$

$$p' = \frac{\sigma_a' + 2\sigma_r'}{3} \qquad (2)$$

whereσ_a' and σ_r' represent the axial and the radial effective stresses, respectively.

The undisturbed specimen (green line) exhibits a lower stress ratio ($K = \sigma_r'/\sigma_a'$ = 0.602) and mean effective stressof p'=70 kPa, while the reconstituted specimen shows a higher stress ratio (K = 0.614) and p'=72 kPa. The lower K in the undisturbed sample indicates greater resistance to deformation,attributedto more preserved initial soilstructure than that in the reconstituted sample.

undisturbed soil.Table 2 indicates all samples' K_0values and p'at the end of K_0 consolidation.

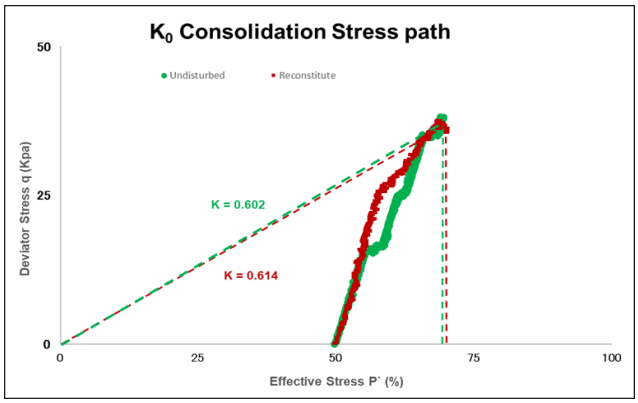

Fig. 8 (a). Stress path of K_0 consolidation.

Table 2. Summary of K_0 values and p'.

Soil samples	3Tr-13		3De-30	
	K_0	p' (kPa)	K_0	p' (kPa)
Undisturbed-Compression	0.602	70	0.67	82
Undisturbed-Extension	0.601	71.5	0.65	83.5
Reconstituted-Compression	0.614	72	0.72	86
Reconstituted-Extension	0.612	71	0.71	84.5

5.2 Comparisonthe shear strength results for compression and extension Loading test

Fig. 9 (a) and Fig. 9 (b) presents the shear strength test results of undisturbed and reconstituted soil specimens obtained from two different layers 3Tr-13: (depth:13.50 to 14.50m) 3De-30: (depth:30.50-31.50m). It shows the relationship between shearstrain (eps. s (%)) on the x-axis and deviator stress (kPa) on the y-axis for the two specimens. The general trend observed is that for both specimens, the shear stress initially increases with increasing shear strain, reaches a peak value (which represents the shear strength of the soil), and then decreases as the shear strain continues to increase.

Specifically, the "3De-30" specimen exhibits higher shear strength compared to the "3Tr-13" specimen, as evident from the higher peak shear stress values in both extension and compression tests. This difference in shear strength can be attributed to the variation in soil properties with depth, such as compaction, soil type, and overburden pressure, which can influence the soil's shear resistance. For both undisturbed and reconstituted samples, the 3De-30 results in higher shear strengths compared to the 3Tr-13. This suggests that the 3De-30 is more effective in preserving or enhancing the soil's shear strength characteristics. The reconstituted 3Tr-13 sample exhibits the lowest shear strength and appears to strain-soften more rapidly after reaching its peak deviator stress.

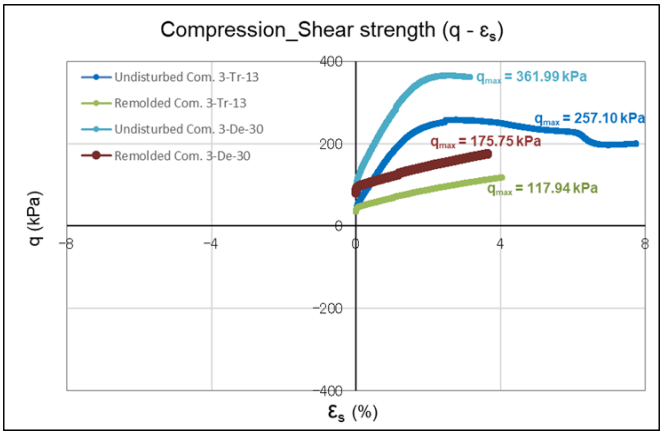

Fig. 9(a).Comparison between compression and shear strength (for sample 3Tr-13 and 3De-30).

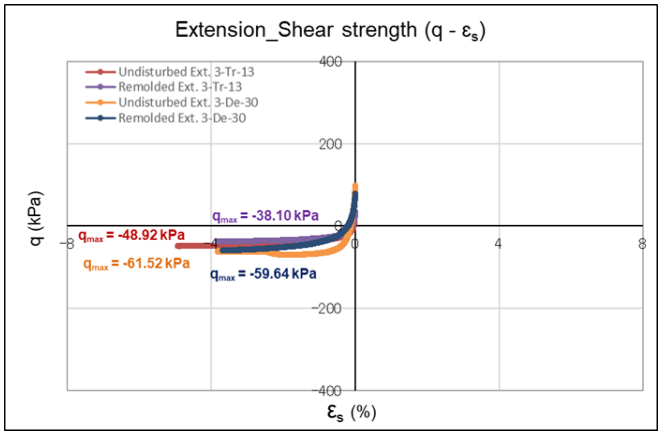

Fig. 9 (b). Comparison between extension and shear strength(for sample 3Tr-13 and 3De-30).

5.3 Local small strain and bender test results in compression and extension loading for sandy specimens (3Tr-13 and 3De-30).

Fig. 10 (a) and Fig. 10 (b) presents local small strain (LSS) and bender element results of undisturbed and reconstituted sandy specimens obtained from the Tokyo Metro area at two different layers, (depth; 13.50-14.50m) denoted as 3Tr-13 and (depth;30.50-31.50m) denoted as 3De-30 in compressional and extensional loading condition. The specimens from the deeper depth (3De-30) in the extension test demonstrate higher shear modulus G0 values which is 106Mpa compared to the shallower specimen for both compression and extension loading. The shallower specimen also has a higher shear modulus in extension conditions around 61Mpa.

The curves exhibit nonlinear behavior, with increasing shear strain leading to a gradual reduction in stiffness for both loading conditions and depths. This suggests that the undisturbed sandy specimens exhibit depth-dependent behavior, with the shallower sandy soil being stiffer and stronger than the deeper sandy soil. The horizontal lines represent the initial shear modulus G0 values obtained from bender element tests, which are known to be reliable for measuring the stiffness of soils at very small strains.

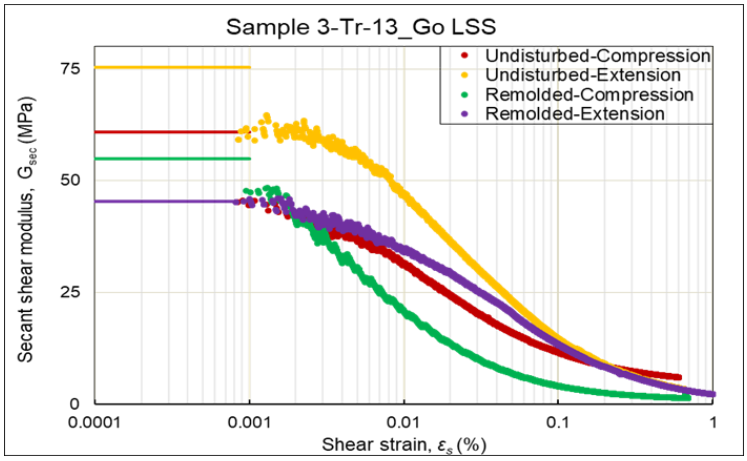

Fig. 10(a). Comparison the results of undisturbed and remolded Specimen for sample 3Tr-13.

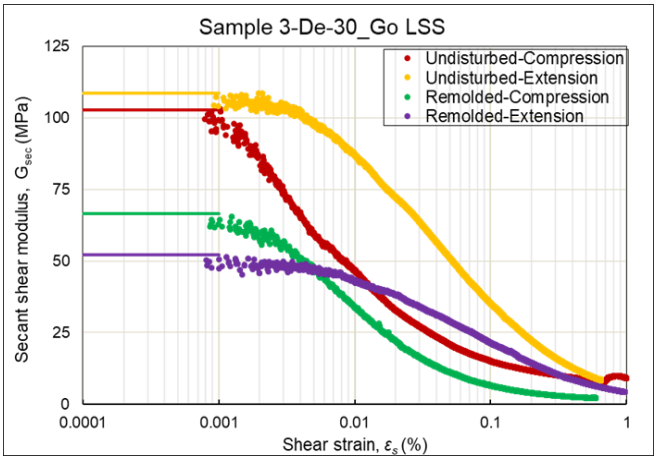

Fig. 10(b). Comparison the results of undisturbed and remolded Specimen for sample 3De-30.

5.4 Comparison of normalized reconstituted and undisturbed results for sandy specimens.

Fig. 11 (a) and Fig. 11 (b) presents the normalized results of reconstituted and undisturbed soil samples with two different layers (3Tr-13: depth 13.50 to 14.50m) and 3De-30: depth: 30.50 to 31.50m) in compression and extension loading. The normalized secant shear modulus (G/Gmax) is plotted against the shear strain (%). At very low shear strains (below 0.001%), the differences in secant shear modulus between the reconstituted and undisturbed specimens minimally, indicating that at small strains, the soil behavior is less influence and the value is almost equivalent allowing reconstituted sandy soil to substitute for undisturbed sandy soil in small strain applications.

The shear modulus decreases with increasing shear strain; however, the rate of decrease varies among the samples. The Undisturbed specimens 3Tr-13 and 3-De-3 exhibits a more rapid decline in shear modulus compared to the other samples, suggesting a higher susceptibility to degradation under shear loading. At very small strains the secant shear modulus of reconstituted and undisturbed sandy soil is equivalent, allowing reconstituted sandy soil to substitute for undisturbed sandy soil in small strain applications to study the rebound phenomenon. As the shear strain increases, the normalized shear modulus decreases for all specimens, following a nonlinear trend. This behavior is typical of soil specimens under compression loading, where the soil stiffness reducesas the strain increases.

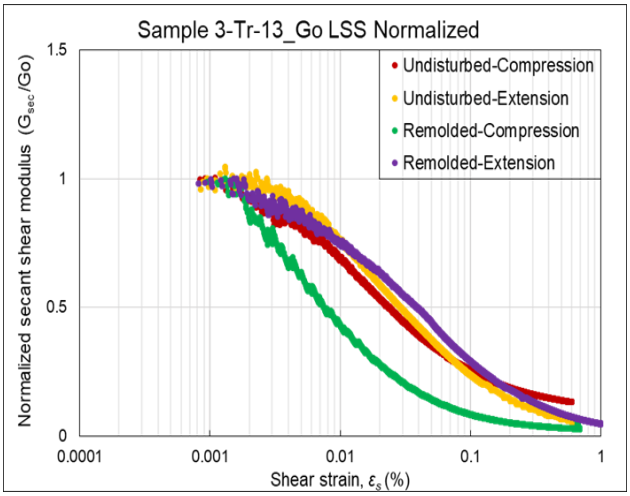

Fig. 11(a). Normalized results of the undisturbed and reconstituted specimensfor sample 3Tr-13.

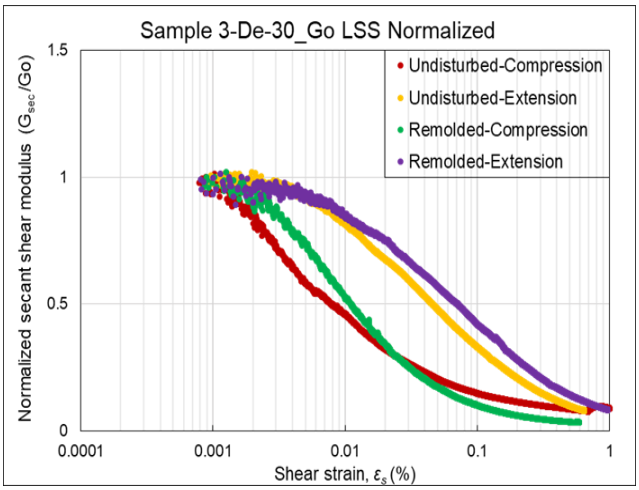

Fig. 11(b). Normalized results of the undisturbed and reconstituted specimens for sample 3De-30.

6 Conclusions

The findings obtained in this study for two different layers (two sandy layers) of undisturbed and reconstituted specimens 3Tr-13 (depth: 13.50 to 14.50m)and 3De-30 (depth: 30.50 to 31.50m) are as follows.

1. For both layers of sandy soils, large difference appears in Gsec trend between compressional and extensional loadings (Extension > Compression). Therefore, extensional loading tests are necessary to evaluate the rebounding behavior.
2. Extensional *Gsec* trend is similar between the undisturbed and the reconstituted samples. Therefore, the use of reconstituted sample is acceptable as a cost-effective testing procedure.
3. The values of G_0 are largely dependent on ground layers. Therefore, G0 has to be evaluated from in-situ investigation or undisturbed sample
4. The K_0 values and strain dependency of G_{sec} indicate that the undisturbed samples have greater resistance to deformation than the reconstituted samples, as reflected by their lower K_0 values and higher G_{sec}. This behavior is likely attributed to cementation effects in the undisturbed samples, despite their relatively large void ratios (e).

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