Mechanical properties of old embankment soil

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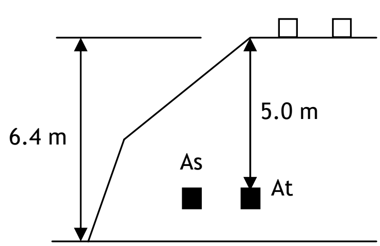
ABSTRACT

Civil engineering structures such as embankments should be stable for fifty to hundred years or more. To evaluate the long-term stability of embankment, any change in the mechanical properties of embankment soil should be well understood. The time-dependent behaviour of soils has been investigated extensively under one-dimensional and triaxial test conditions. Most of the observations in the literature have focused on the determination of the time-dependent behavior of natural soils, and the reported experimental studies of embankment soils are few. An embankment soil is remoulded and reconstituted. Although natural soils are exposed to very long periods of time, embankment soils are exposed to periods of typically up to 100 years. The purpose of this paper is to study the effects of time on the deformation and strength characteristics of embankment soil. Undisturbed sampling of and embankment soil was undertaken and consolidation, consolidated undrained (CU) triaxial tests with pore water pressure measurement, small cyclic loading, and bender element tests were performed to determine the consolidation parameters, strength parameters, and the small strain Young's modulus (E) and shear modulus (G). These were compared with the mechanical properties of freshly remoulded and compacted (reconstituted) soil.

1 SAMPLE PROPERTIES

Undisturbed samples were taken from about a 50-year old railway embankment near Hatanodai Station located south-west of Tokyo. Two block samples were taken from each of the locations shown in Figure 1, i. e. in the slope (As) and below the track (At).



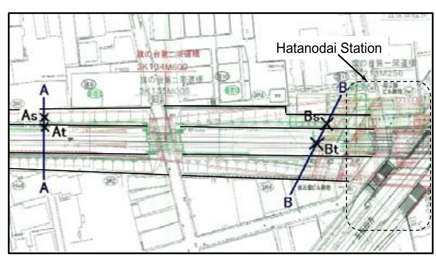


Figure 1: Sampling Location

The physical properties of the soil used are summarised in Table 1.

Table 1: Physical properties of soil

Sample ID.	Natural water content, w_n (%)	Soil density ρ_s (gm/cm^3)	Liquid limit w_L (%)	Plastic limit w_P (%)	Plasticity index I_P
As1	113.5	2.838	133.8	91.2	42.6
As2	109.5	2.819	148.8	101.4	47.4
At1	106.8	2.796	130.0	101.0	29.0
At2	117.4	2.862	138.5	101.5	37.0
Kanto Loam	80-150	2.7-2.9	80-150	40-100	20-50

2 CONSTANT RATE OF STRAIN CONSOLIDATION TESTING

2.1 Test Conditions

The size of the specimen was 6 cm in diameter and 2 cm in height. The specimen was trimmed from the block sample and a reconstituted specimen was made from the remaining part of the block. The water content of both undisturbed and reconstituted specimens were the same, and the reconstituted specimen was compacted to have the same void ratio as the undisturbed specimen. A back pressure of 196 kPa was applied to achieve a high degree of saturation and a constant strain rate of 0.05%/min was applied to allow the dissipation of excess pore pressure, taking into account the plasticity index of the samples.

2.2 Test Results

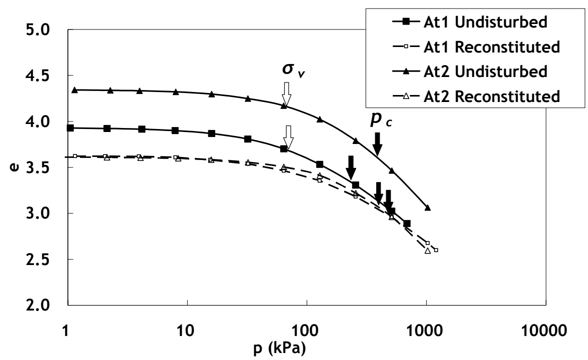


Figure 2: e versus log p relationships

Table 2: Coefficient of compression, C_c

Sample ID.	At1 Undisturbed	At1 Reconstituted	At2 Undisturbed	At2 Reconstituted
C_c	0.51	0.47	0.68	0.64

Figure 2 indicates e-logp relationships obtained for the undisturbed and reconstituted specimens. Here, p_c = consolidation yield stress and σ_v = estimated in situ overburden total stress.

2.3 Summary of the Results of Consolidation Test

The preconsolidation stress p_c is higher than the effective overburden pressure σ_v for both the undisturbed and reconstituted specimens. This may be due to the effects of compaction. The p_c value for the reconstituted specimens is slightly higher than that for the undisturbed specimens. This may be due to the initial void ratio of reconstituted specimen being slightly lower than that of undisturbed specimen. The overall consolidation behavior of the undisturbed and reconstituted specimen was almost the same, as shown in Figure 2. The coefficient of compression C_c was similar whether the specimen was undisturbed or reconstituted; for sample At1 the values were 0.51 and 0.47, and for sample At2 the values were 0.68 and 0.64 for the undisturbed and reconstituted specimens, respectively. There was no sudden drop in void ratio at stresses higher than the preconsolidation stress, as would be expected for cemented soil.

3 UNDRAINED TRIAXIAL COMPRESSION TESTING

3.1 Test Conditions

The diameter and height of the triaxial specimens were 7.5 cm and 15 cm, respectively. The thickness of the rubber membrane used was 0.03 cm and its diameter was 7.3 cm. For consolidation, the isotropic stress was increased at a rate of 0.5 kPa/min. After consolidation to 40, 60 and 294 kPa, undrained triaxial compression testing was carried out with an axial strain rate of 0.01%/min, with pore water pressure measurement.

3.2 Test Results

Figure 3 shows the deviator stress, q versus axial strain, ε relationship obtained from the undrained triaxial compression tests. Figure 3 shows a clear peak in q for the undisturbed specimens followed by a decrease in q. This suggests overconsolidated behaviour at low confining stresses and may be due to the destruction of the structure of the undisturbed specimen at high confining stresses. The peak strengths of the undisturbed specimens consolidated at 40 and 60 kPa are almost equal. The peak strengths of the reconstituted specimens consolidated at 60 kPa were relatively high compared with the strengths obtained at other confining stresses. This is probably because the initial void ratio of the specimen was lower.

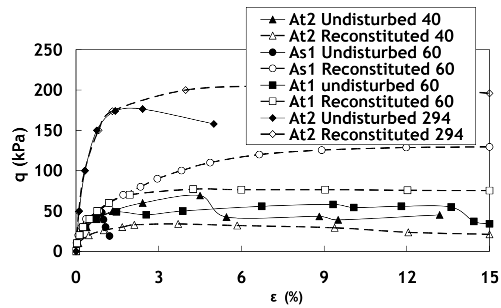


Figure 3: Stress versus strain relationships

Figure 4 shows the effective stress paths for the undisturbed and reconstituted specimens. Positive excess pore water pressure was generated in the undisturbed specimens. The effective stress path of the reconstituted specimen consolidated at 60 kPa is indicative of an over consolidated soil. The failure envelope shown in Figure 4 is a reasonable lower bound for both the undisturbed and reconstituted specimens, represented by the drained strength parameters c′= o kPa and φ′= 31.90°.

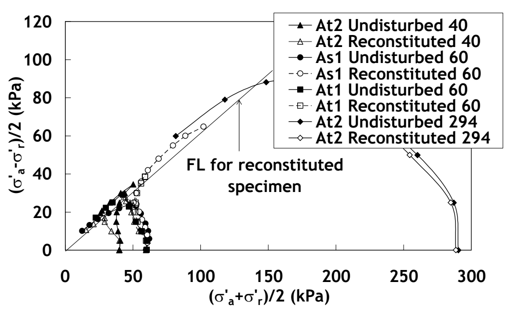


Figure 4: Effective stress paths

4 SMALL STRAIN STIFFNESS

4.1 Small Strain Cyclic Loading Testing

Small amplitude cyclic loading tests were carried out to measure the small-strain stiffness of specimens in the axial direction. A deviator stress q was applied to achieve strain amplitude of 0.005% to 0.01%. Cycles of unloading and reloading were repeated up to 5 times.

4.2 Bender Element Testing

A bender element is a piezo-ceramic plate, which can convert bending deformation and voltage, thereby transmitting and receiving shear waves, the velocities of which are measured. Equation (1) gives a relationship between shear wave velocity, *V* and shear modulus, *G*.

$$G = \rho V^2 \tag{1}$$

In which, ρ is the density of the material.

4.3 Test Condition

A bender element test was carried out at the end of consolidation and during shearing for selected undisturbed and reconstituted specimens. The shear wave velocity was measured at 10 kPa intervals of q for the specimen consolidated at 40 kPa and 60 kPa and at every 50 kPa interval of q for the specimen consolidated at 294 kPa. The measurement of shear wave velocity was taken when the creep deformation was almost finished. In the case of the specimen consolidated at 294 kPa, the shear wave velocity was also measured at confining stress of 60, 98 and 196 kPa.

4.4 Test Results

Figure 5 shows the vertical Young's modulus E_v and shear modulus G_{vh} values versus vertical effective stress σ'_v. Under isotropic consolidation, the amount of increase of both E_v and G_{vh} for the undisturbed specimens is slightly less than those for the reconstituted specimens. Small strain moduli *E* and G are affected by stress level and void ratio. If there are some differences in void ratio between the undisturbed and reconstituted specimens, such as in this study, the effect of these differences in void ratio should be considered. Under isotropic stress conditions, *E* and *G* are generally expressed by the following equation.

$$E = CF(e)(\sigma'_V)^n \qquad (2)$$

Here, *C* and *n* are constants determined by experiment. For the function *F*(e), the following equation was proposed by Shibuya et al. (1997) for cohesive soils.

$$F(e) = (1+e)^{-2.4} \qquad (3)$$

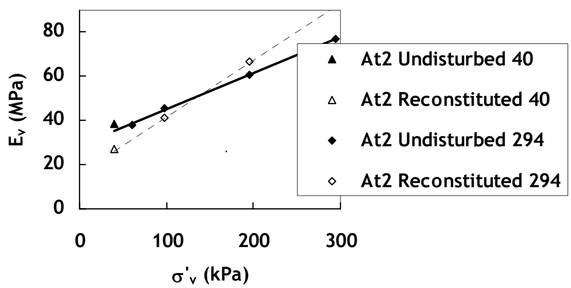


Figure 5(a): E_v versus σ'_v relationships

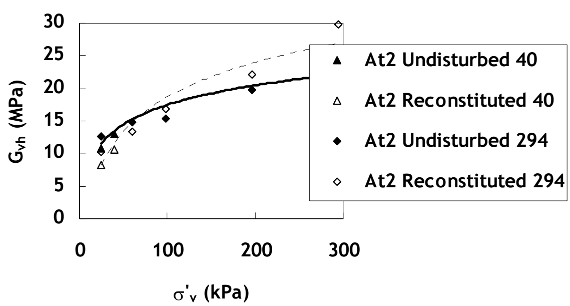


Figure 5(b): G_{vh} versus σ'_v relationships

Considering Equation (2) $\dfrac{E_v \;\; or \;\; G_{vh}}{F(e)}$ are plotted against σ'_V to log-log scales as shown in Figure 6. For both undisturbed and reconstituted specimens, linear relationships were obtained and therefore Equations (2) and (3) can be applied to the soil used in this study. From Figure 6, the modulus normalised by *F*(e) is higher for the undisturbed specimens. With increase in σ'_v the difference between the results for the undisturbed and reconstituted specimens becomes smaller because the initial structure of the undisturbed specimens changes with the stress level and becomes similar to the structure of reconstituted specimens at high stress level.

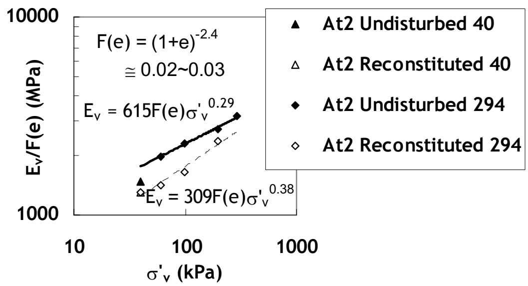


Figure 6(a): $\log \dfrac{E_v}{F(e)}$ versus $\log \sigma'_v$ relationships

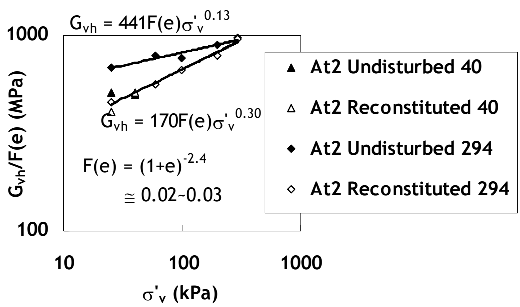


Figure 6(b): $\log \dfrac{G_{vh}}{F(e)}$ versus $\log \sigma'_v$ relationships

5 CONCLUSIONS

The following conclusions were drawn from this study from the tests on 50-year old undisturbed specimens and newly compacted specimens.

1. The strength of the undisturbed and reconstituted specimens was almost the same although the void ratio of the undisturbed specimens was slightly higher than that of the reconstituted specimens.
2. The modulus of the undisturbed specimens was higher than that of the reconstituted specimens, but the difference becomes smaller with increasing σ'_v.

REFERENCES

1. The Japanese Geotechnical Society (2000). *Laboratory Soil Testing Methodology and Commentary*, 2nd Edition, pp. 746-761 (In Japanese).
2. Yang, J. and Moroto, N. (1999). *Shear strength of undisturbed volcanic cohesive soils undergoing cementation*, Proceedings of JSCE, No. 617/III-46, pp. 175-189 (In Japanese).
3. Shibuya, S., Hwang, S. C. and Mitachi, T. (1997). *Elastic shear modulus of soft clays from shear wave velocity measurement*, Geotechnique, 47, 3, 593-601.