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Geotechnical properties of liquefied volcanic soil ground by 2003 Tokachi-Oki Earthquake

Propriétés géotechniques du sol volcanique liquéfié lors du tremblement de terre à Tokachi-Oki en 2003

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ABSTRACT
An earthquake with a magnitude of 8.0 occurred in Hokkaido, Japan, on September 26, 2003. Large-scale liquefaction failure occurred on embankments filled with volcanic ash. Liquefied volcanic ash soil of about 10,000 m^3 flowed from the liquefied area. Tests (SWS, CPT, DPT, SASW) were performed on soil in situ, and on undisturbed samples in the laboratory.

RÉSUMÉ
Le 26 septembre 2003, Hokkaido (Japon) a connu un tremblement de terre d'une magnitude de 8,0. Une liquéfaction de grande échelle s'est produite sur des remblais remplis de tufs d'origine volcanique. Sol volcanique de environ 10 000 m^3 s'est écoulé de la zone en fusion. Des essais (SWS, CPT, DPT, SASW) ont été réalisés sur site, ainsi que sur des échantillons de sol non remué testés en laboratoire.

1 INTRODUCTION

On September 26, 2003, an earthquake (2003 Tokachi-Oki Earthquake) with a magnitude of 8.0 occurred in Hokkaido, Japan. Large-scale liquefaction failure occurred on embankments filled with volcanic ash soil in the Kyowa area of Tanno Town close to Kitami City, situated about 230 km away from the epicenter. Kitami City recorded with a maximum acceleration of 123.6 gal (EW). In this site, to investigate the geotechnical characteristic of the liquefied ground, several kinds of in-situ tests were performed. Moreover, to examine the liquefaction characteristics of the volcanic ash soil ground, undrained cyclic triaxial and bender elements tests were performed on undisturbed samples retrieved by the block sampling method. In addition, these laboratory test results were compared with the in-situ test results. Finally, the cause of large-scale liquefaction failure is estimated from the geotechnical properties of a volcanic ash soil in Kitami area and the landform of this site.

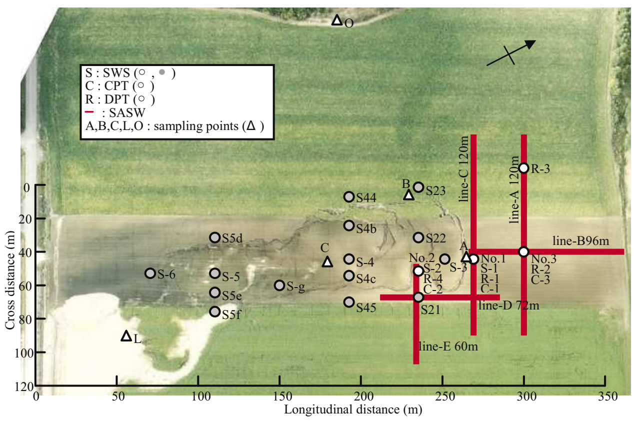

Photo 1. Site investigation points.

2 OUTLINE OF GROUND FAILURE

The ground failure caused by the liquefaction has been confirmed at six places in the Kyowa area, Tanno Town. The site investigated in this study is the largest liquefied place. The length and width were about 200 m and 60 m, respectively. The maximum settlement was 3.4 m. Liquefied volcanic ash soil of about 10,000 m^3 spouted out from the exhaust nozzles of the slope sides, as shown in photo 1. The liquefied soil blasted over the farmland, open ditches and roads for an area of about 100 m wide and 100 m long. Moreover, as the soil flowed into a river, the channel was filled for about 900 m downstream and 100 m upstream.

The old landform of this liquefied area was a narrow waste-filled valley, and then the bottom of the valley was exploited to be paddy fields. About twenty years ago, the valley was reclaimed by volcanic ash soil (Kutcharo pyroclastic flow deposits IV) cutting the hillsides, and the region was converted into gently-sloping farmland.

3 IN-SITU AND LABORATORY TESTS

The various in-situ tests were carried out three weeks after the earthquake, as shown in photo 1. The in-situ tests performed are the Swedish weight sounding test, SWS (19 points; S letters in photo 1.), mini diameter cone penetration test, CPT (3 points; area of cone base is 2 cm^2; C letters), mini dynamic penetration test, DPT (4 points; fall height is 35 cm; hammer weight is 300 N; R letters), and spectral analysis of surface waves test, SASW (5 lines; line-A to -E).

Physical properties tests were also performed on samples retrieved from 5 points (A, B, C, L, O). Furthermore, cyclic triaxial (CTX) and bender elements (BE) tests were performed on undisturbed samples of A and O points retrieved by the block sampling method.

Table 1 and Figure 1 show the physical properties and the grain size distribution curves of samples. In these table and figure, Fill A and B were retrieved from a depth of 0.5 to 1.0 m in slip down cliffs of north side, Fill C from the center of the liquefied area (depth is 0.3 m), and Runoff Soil L from a spouted point, as shown in photo 1. On the other hand, Cut O was retrieved from a cut sloop, situated about 100 m away from the liquefied area. It would seem that this O soil is a part of the recliamed materials.

From Fig.1, it is found that the grain size distributions of Fill A, B and Runoff Soil L are similar, and the sorting of soil particles had not occurred in Runoff Soil L. It is also found that the fine content of Fill C is slightly larger than those of other fill soils. This reason may be the effect of particle breakage caused by the cultivation etc., because the sampling depth is shallower than other samples. On the other hand, the particle size of Cut O is larger than those of other soils. This reason would seem because there is not the influence of particle breakage at the reclamation.

Table 1: Physical properties of volcanic ash soils.

sample	ρ_s (g/cm^3)	D_{50} (mm)	U_c	F_c (%)	ρ_{dmax} (g/cm^3)	ρ_{dmin} (g/cm^3)	$\rho_{d\ in\text{-}situ}$ (g/cm^3)	D_r (%)
Fill A	2.493	0.20	28	30.3	1.190	0.811	0.984	55
Fill B	2.485	0.20	34	30.3				
Fill C	2.510	0.13	36	37.4				
Runoff L	2.497	0.19	26	30.0				
Cut O	2.507	0.38	19	19.4	1.217	0.872	1.102	74

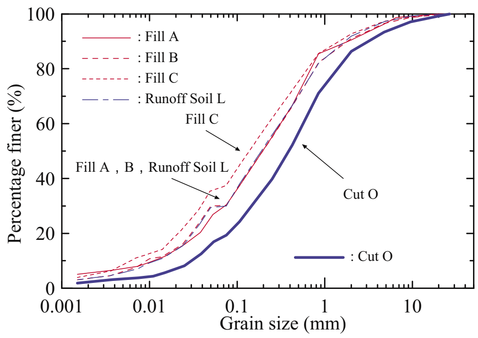

Figure 1. Grain size distribution curves of volcanic ash soils.

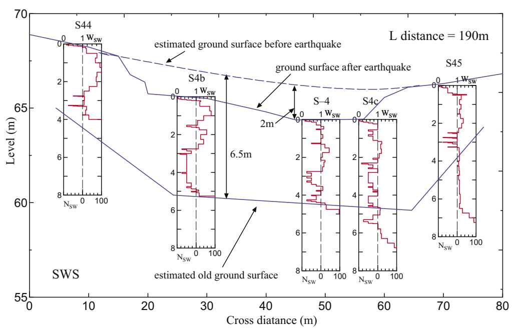

Figure 2. Swedish weight sounding test results on a cross direction.

3.1 *Results of in-situ tests*

Figure 2 shows an example of SWS test results on a cross direction (a longitudinal distance of 190 m in photo 1). In this figure, test results of both sides (S44, S45) are in the outside of the settlement area and other three test results (S4b, S-4, S4c) are in the inside of it.

The penetration resistance is relatively high from the ground surface to about 2 m deep, and then it becomes extremely low in the lower layer.The old ground surface before the reclamation, which is estimated from N_{SW} above about zero (W_{SW} > 1 kN), slopes in the both sides, and is relatively flat in the settlement area. From these test results, it is found that the bottom of the valley had been used as paddy fields. Furthermore, it would seem that the maximum depth from the ground surface before the earthquake estimated by the both sides to the old ground surface is about 7 m.

Figure 3 shows a comparison of different types of penetration tests results performed on No. 1 point in photo 1. Note that, the settlement had not occurred in this point. The penetration resistance profiles with depth by three types of penetration tests are similar. The penetration resistance is relatively high from the ground surface to about 1 m deep, and then it decreases with the increase of depth. Afterward, it decreases remarkably to about 4 m below the ground water level (about 2.5 m), which is determined from the value of pore water pressure on CPT, and then it increase at a depth of about 4.5 m because the tip may reached to the old ground surface.

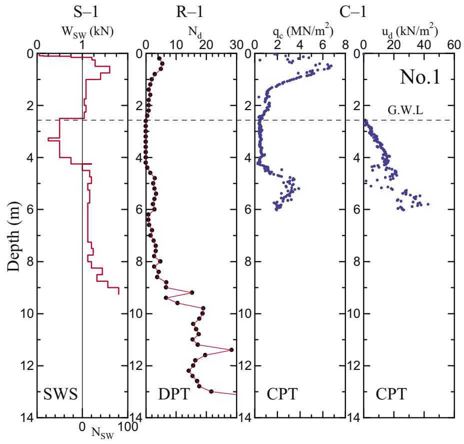

Figure 3. Comparison of defferent penetration test results (No.1 point).

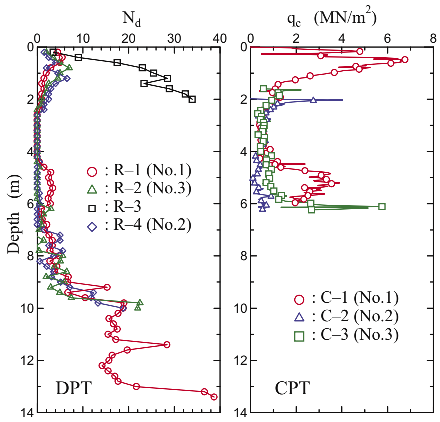

Figure 4. Comparison of penetration test results in different points.

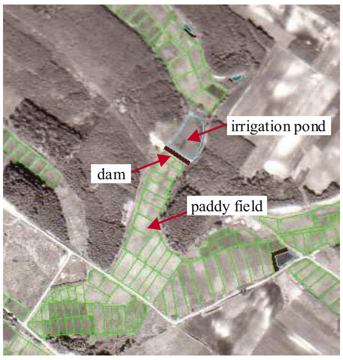

Photo 2. Aerial photograph taken in 1977.

Figure 4 shows comparisons of results of DPT and CPT on different points. In the case of DPT, the profile of N_d-value (= SPT N-value) with depth is similar except for the result on R-3, where the old ground surface is shallow in comparison with other points. However, the N_d on R-1 is slightly higher than other results on a depth of 4.5 to 6 m. The similar tendencies are recognized on the CPT (C-1) and SWS (S-1 in Fig. 3) test results performed on the same point. In this depth, there exists a buried dam, as shown in photo 2 taken before the reclamation. So that, it would seem that the penetration resistance became higher than that in the same depth of other points.

Figure 5 shows an example of SASW results (line-C in photo 1). It is found that the shear wave velocity V_S is slightly fast (V_S is over 100 m/s) from the ground surface to about 2 m deep, and then it becomes low (V_S is about 80 m/s) in the lower layer. Thereafter, the V_S is increase with the increase of depth. These tendencies are similar to the penetration test results. It is also found that the V_S is fast in both sides (V_S is over 150 m/s) in spite of the shallow depth. These layers seem the old ground judging from above a penetration test result (R-3 in Fig.4) and photo 2. Thus, the two-dimensional general ground profile can be estimated from the SAWS test without performing many penetration tests.

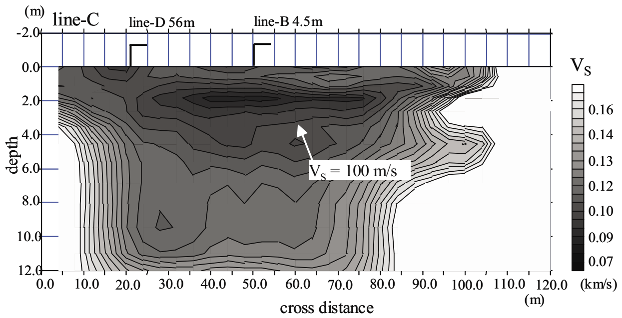

Figure 5. Spectral analysis of surface waves test result (line-C).

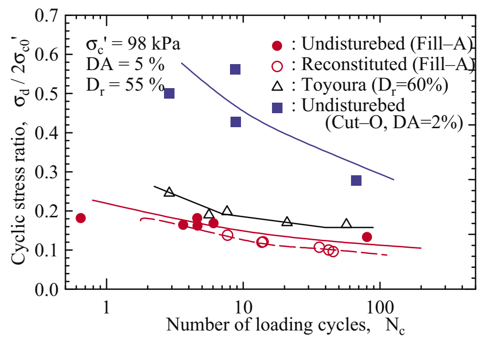

Figure 6. Comparioson of liquefaction strength.

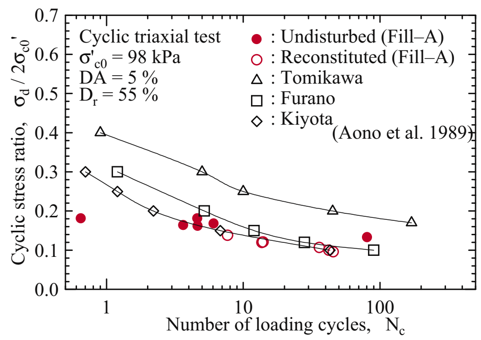

Figure 7. Comparison with other volcanic ash soils in Hokkaido.

3.2 *Results of laboratory tests*

The undrained CTX test (liquefaction test) and the BE test were performed on samples retrieved from point A (fill) and C (cut). Figure 6 shows the test results on undisturebed and reconsitituted specimens togather with a result for Toyoura sand of Dr =60 %. Test results were shown in the relation of the cyclic stress ratio versus the number of loading cycles where a 5% double-amplitude axial strain DA was observed. Note that, the test result on undisturbed O samples is for DA = 2 % because the necking phenomenon occurred before the DA reaches 5 %.

It is found that the liquefaction strength of undisturbed Fill A samples is very low in comparison with Cut O samples, and is also lower than Toyoura sand. Therefore, it is said that the reclaimed volcanic soil is the low resistance materials for the liquefaction. Furthermore, although the strength of undisturbed fill samples is higher than that of reconstituted fill samples, the difference is relatively small. Therefore, it is said that for this loose volcanic ash soil deposit only about twenty years old, the aging and structural effects were not significant with respect to the liquefaction strength.

As mentioned above, it was found that the liquefaction strength of the Kutcharo volcanic ash soil deposited in the liquefied area was very low. Then, to clear whether this low strength is a unique characteristic of the Kutcharo volcanic ash soil, the liquefaction strength was compared to those of other volcanic soils in Hokkaido.

Figure 7 is the comparison of liquefaction strength of the Kutcharo samples and reconstituted samples retrieved from the Tomikawa (Monbetsu Town), Furano (Kami-Furano Town) and Kiyota (Sapporo City) (Aono et al., 1989). The liquefaction strength of the Kutcharo fill soil is lower than those of other volcanic ash soils in Hokkaido. Therefore, it would seem that this low liquefaction resistance caused a lot of ground failure by the liquefaction in Kitami area in spite of the low maximum acceleration of about 120 gal and the long distance of about 230 km from the epicenter. It is also found that the liquefaction strength of the Kiyota volcanic ash soil is low as well as the Kutcharo volcanic ash soil. The volcanic ground failure by the liquefaction in the Kiyota area of Sapporo City is also reported (JGS, 2004). One of the reasons may be the lowness of liquefaction resistance of the Kiyota volcanic soil.

Figure 8 shows the relationships of the shear modulus versus the isotropic confining pressure obtained from BE and CTX tests on undisturbed and reconstituted specimens of Fill A samples, where the shear modulus G calculated from the V_S ($G = \rho_t V_S^2$) in BE tests and the G from the undrained Young's modulus E ($G = E/3$) at single amplitude axial strain $(\varepsilon_a)_{SA}$ of 10^{-5} in CTX tests. The difference of the shear moduli between undisturebed and reconstituted samples is relatively small as well as the liquefaction strength. Therefore, these results also imply that the aging effects are not significant for the small strain behavior of this loose volcanic soil deposit only about twenty years old.

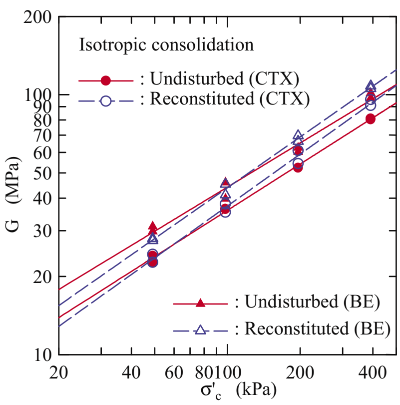

Figure 8. Comparion of G on undisturebed and reconstituted samples.

3.3 *Comparisons of in-situ tests and laboratory tests*

Figure 9 shows the shear wave velocity profile with depth on No. 1 point of line-C in SASW test together with the V_S from BE and CTX [$V_S = (E/3\rho_t)^{0.5}$] tests. In this figure, the laboratory test results of the depths different from the sampling depth (1 to 2 m) were estimated from the test results in the sampling depth.

In this estimation, the total density ρ_t in the upper layer of ground water level (2.5 m) was assumed as 1.203 g/cm^3 based on the value of undisturbed sample, and the ρ_t in the saturated lower layer was assumed as 1.589 g/cm^3. In other words, the dry density was assumed as the same irrespective of depth.

It is found that the V_S estimated from laboratory tests is slightly higher than that from SASW test except for the surface layer. As mentioned above, the V_S profile of laboratory tests is based on a result on shallow depth. The penetration resistance is relatively high from the ground surface to about 1 m deep, and then it decreases with the depth, as shown in Fig. 3. So that, it would seem that the V_S from laboratory tests became higher than that from SASW test. However, as the difference is relatively small, it is said that the similar result was obtained from the laboratory and in-situ tests.

Next, the liquefaction strength estimated from the in-situ test is compared to that from laboratory tests. The many prediction methods of liquefaction strength have been proposed on the basis of in-situ test results such as the *N*-value, corn penetration resistance and shear wave velocity. Then, the liquefaction strength estimated from the tip resistance of CPT and the V_S of SASW are compared to the laboratory test results in the following. The prediction of liquefaction strength by CPT was used the method proposed by Ishihara (1985). On the other hand, it by SASW was used the method proposed by Robertson et al. (1992).

Figure 10 shows the liquefaction strength τ/σ_c' in No.1 to 3 points estimated using the methods of Ishihara and Robertson et al. The laboratory liquefaction strength on undisturbed and reconstituted samples and the shear stress ratio on an acceleration of 120 gal estimated using the method proposed by Tokimatsu and Yoshimi (1983) are also plotted in this figure. It is found that the liquefaction strength by two different methods almost agrees. The liquefaction strength by laboratory tests also agrees, considering that it is a result on samples of a slightly hard surface layer. It is also said that the both prediction methods can apply to also the volcanic ash soil ground. Furthermore, the predicted liquefaction strength is lower than the estimated shear stress ratio on an acceleration of 120 gal below a depth of about 3 or 4 m, where is liquefied zone.

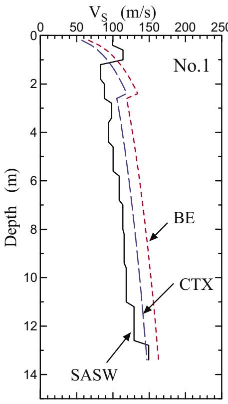

Figure 9. Comparison of SASW and laboratory tests.

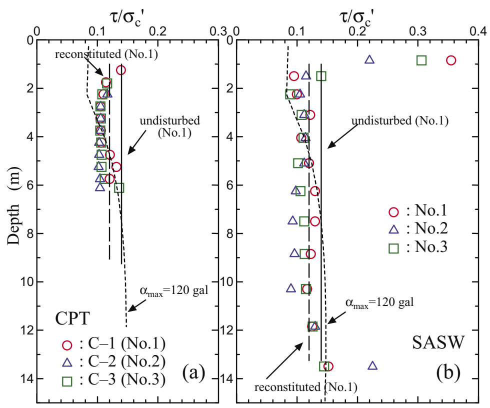

Figure 10. Pridiction of liquefaction strength; (a) method of Ishihara (1985), (b) method of Robertoson et al (1992).

4 CONCLUSIONS

It was estimated from in-situ tests that the maximum thickness of very loose volcanic ash soil is about 7 m from the old ground surface. The liquefaction resistance of the volcanic ash soil was very low in comparison with other volcanic ash soils in Hokkaido area. The aging effects were not significant for the liquefaction strength and small strain behavior of this volcanic soil deposit only about twenty years old. It would seem that the geotechnical characteristics of the reclaimed soil had not be changing from twenty years ago. The shear wave velocities obtained from laboratory and SASW tests almost agreed, and also the estimated liquefaction strength by CPT and SASW tests almost agreed with the laboratory test results.

Although a few vertical sand boils occurred in the collapsed area, a distinctive feature was a phenomenon that is the lateral sand boil of much volume from edges of the slope. One of the reasons for the lateral sand boils is that the water content of the volcanic ash soil is high because the void ratio is large and the volcanic soil is porous. Another is that the dissipation of excess pore water pressure was late and the duration of liquefied state was long because the permeability is low due to the fine content of about 30 %. Furthermore, the large-scale runoff of liquefied soil may have been caused by a phenomenon like the sloshing by a ground shock of relatively long cycle, because the old landform of the liquefied area was a narrow valley and the sides were restrained with a buried dam and a road fill, as shown in Fig. 11.

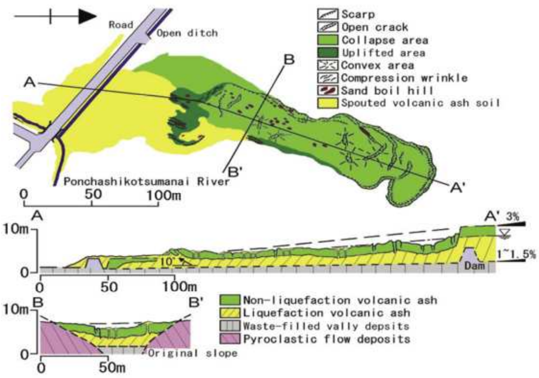

Figure 11. Schematic figure of liquefied area.

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