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STUDY ON LIQUEFACTION CHARACTERISTICS OF VOLCANIC ASH SOIL - REPORT ON FIELD INVESTIGATIONS IN HOKKAIDO -

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ABSTRACT: In the conventional design method, there is no particular method to judge the liquefaction phenomenon of the unusual soil such as volcanic ash soil. This study, in order to clarify and evaluate the liquefaction characteristics of volcanic ash soil, field investigations and a series of soil laboratory tests of volcanic ash ground were carried out. The field investigations and sampling were conducted in the areas where ground liquefaction occurred due to the 1993 Hokkaido-nansei-oki and the 2013 Tokachi-oki earthquakes. According to the results, the liquefaction strength ratio (R_L) was underestimated when using the formulation of the conventional design method. Moreover, it was found that the estimated formulation R_L need to be modified for evaluating the liquefaction phenomenon in volcanic ash soil.

Key Words: Volcanic ash soil, Liquefaction, Assessment of soil liquefaction

1. INTRODUCTION

Extensive and substantial soil liquefaction damage caused by the 2011 Off the Pacific Coast of Tohoku Earthquake and its aftershocks have left profound social impacts in Japan. In the light of this particular seismic liquefaction damage, the Ministry of Land, Infrastructure, Transport and Tourism (MLIT) launched the Committee to Study Countermeasures against Soil Liquefaction to comprehend actual situations of liquefaction damage and to review the conventional design method (assessment of soil liquefaction)[1)]. As a result, the conventional design method proved not to have missed any occurrence of liquefaction in this disaster. However, a number of places were judged to carry liquefaction potentials even though no signs including sand boil were detected. Thus, it was concluded that continuous efforts would be required by collecting and analyzing more data in order to improve assessment of soil liquefaction[2)]. To mitigate the liquefaction damage of future major earthquakes, a highly accurate assessment of soil liquefaction in response to various soil qualities and geological structures of Japan must be formulated.

In Japan, as a volcanic country, various kinds and natures of volcanic products have accumulated broadly as a result of active volcanism from the Quaternary period onward[3)-5)]. "Laboratory Testing Standards of Geomaterials" issued in 2009 by the Japanese Geotechnical Society classified volcanic ash soil as "unusual soil" that possesses geotechnical engineering properties clearly dissimilar to those of usual soil[6)]. Volcanic coarse-grained soil in particular is known to bear physical and mechanical characteristics different from those of sandy soil, exhibiting particle breakage due to its porous and vulnerable constituent particles and slightly taking on consolidation due to welding in the process of sedimentation[7)-12)].

An increasing number of slope failure and liquefaction damage in volcanic ash soil caused by past earthquakes have been reported, and most of them took place in the grounds of coarse volcanic ash soil[13)]. Disasters in volcanic ash soil induced by past earthquakes are mapped in Fig. 1. Among them, the locations of liquefaction damage are listed in Table 1. Prompted by those disasters, researchers have energetically conducted in-depth surveys on dynamic mechanical characteristics of volcanic ash soil[14)-19)], but only a limited number of studies directly evaluated the applicability of the conventional design method to volcanic ash soil. Kazaoka et al.[17)] clarified that the liquefaction strength of usual sand increased along with compaction whereas pumiceous volcanic ash (Hachinohe pumice) increased only a little, and that the liquefaction strength of sand reduced to 1 / 2 when the sand layer contained pumice merely at 10 %. In this case, the liquefaction strength of volcanic ash is possibly lower than that of sand with a same N value, and the liquefaction strength may be overestimated on the basis of the N value, which would make a judgment as potential hazard. In the meantime, Takada et al.[16)] demonstrated a contrary result. They proposed that when the dynamic shear strength ratio (liquefaction strength ratio) of secondary Shirasu was estimated from an N value, the result corresponded well to the dynamic strength test result if the measured N value was doubled. Kokusho et al.[20)] pointed out that liquefaction inflicted by an earthquake on a gently inclined farmland which was refilled with volcanic sandy soil could not be evaluated properly from the conventional F_L method.

Under these circumstances, with the aims of comprehending and evaluating liquefaction characteristics of volcanic ash soil which is unusual soil but not handled distinctively by the conventional design method, the authors conducted field investigations and laboratory soil tests for several sites damaged by the past earthquakes and their neighboring areas in Hokkaido, and survey results are discussed in this paper.

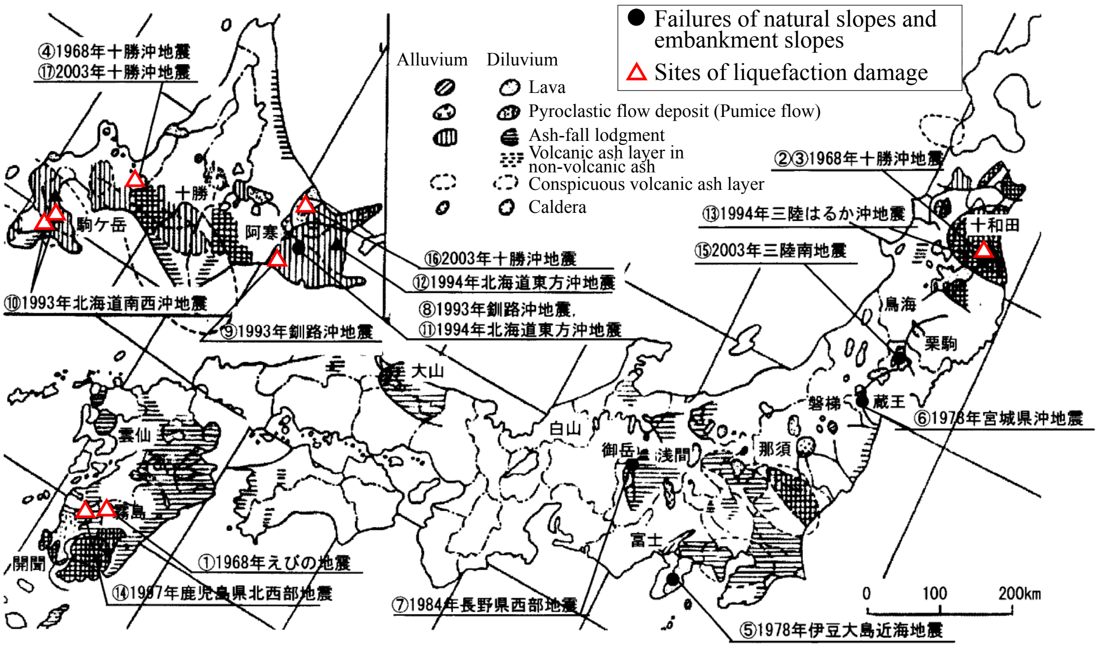


Fig. 1 Sites of disasters in volcanic ash soil induced by past earthquakes
(Fig.6-1 of Reference #13 was retouched)

Table 1 Sites of liquefaction in volcanic ash soil induced by past earthquakes

	Earthquakes	Liquefaction sites (the numbers correspond to those in Fig. 1)
1	1968 Ebino Earthquake	(1) Ebino Town, Miyazaki Pref.
2	1968 Tokachi-oki Earthquake	(2) Sannohe Town, Aomori Pref. (3) Gonohe Town, Aomori Pref. (4) Kiyota-ku, Sapporo City
3	1993 Kushiro-oki Earthquake	(9) Kushiro City, Hokkaido
4	1993 Hokkaido Nansei-oki Earthquake	(10) Mori Town, Hokkaido
5	1997 Kagoshima-ken Hokuseibu Earthquake	(14) Iriki Town, Kagoshima Pref.
6	2003 Tokachi-oki Earthquake	(16) Tanno-cho, Kitami City (17) Kiyota-ku, Sapporo City

2. OUTLINE OF FIELD INVESTIGATIONS AND LABORATORY TESTS ON LIQUEFACTION STRENGTH CHARACTERISTICS OF VOLCANIC ASH SOIL

Field investigations were carried out in several sites within Hokkaido, where liquefaction of volcanic ash soil was induced by past earthquakes: Mori Town[21] ((10) in Fig. 1), Utsukushigaoka, Kiyota-ku, Sapporo[22] (17), and volcanic ash ground in Bihoro Town near Tanno-cho, Kitami City[22] (16). In addition, laboratory soil tests were executed on samples collected from each site. Investigation points in each site are shown in Figs. 2 to 4.

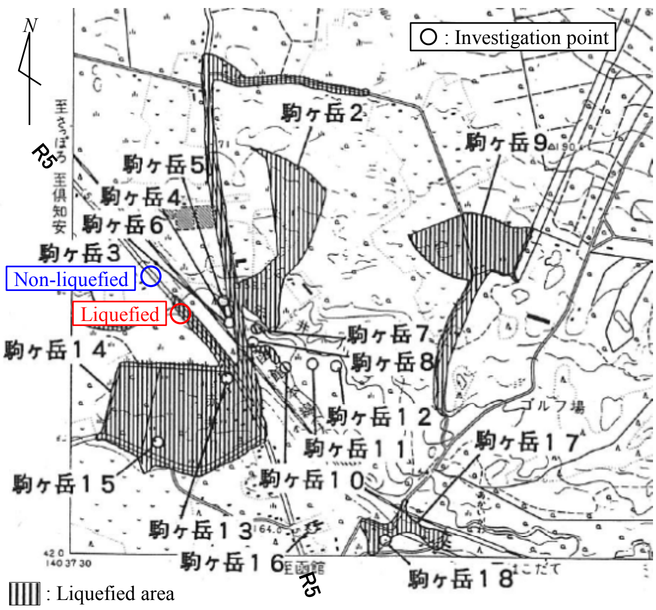


Fig. 2 The site of liquefaction caused by the 1993 Hokkaido Nansei-oki Earthquake and the investigation points in Mori Town
(Part excerpted from Fig. 39 of Reference #23 and modified)

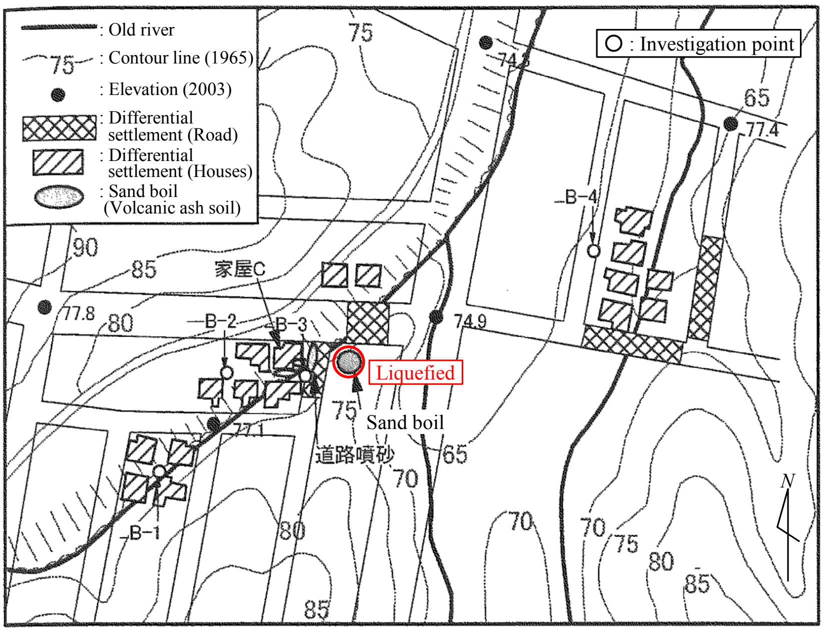


Fig. 3 The site of liquefaction caused by the 2003 Tokachi-oki Earthquake and the investigation point in Utsukushigaoka, Kiyota-ku, Sapporo City (Fig.4.3.2 of Reference #22 was retouched)

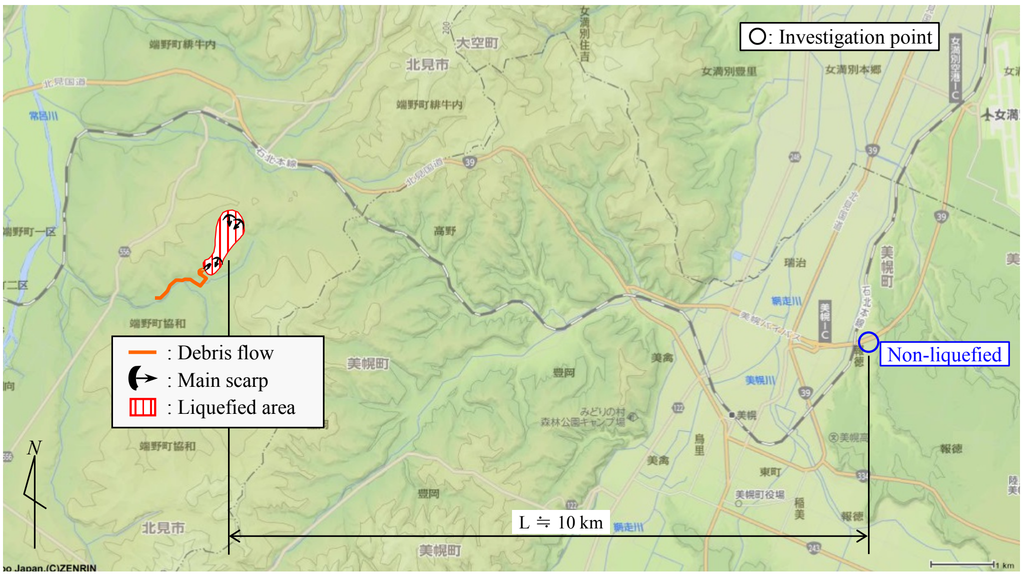


Fig. 4 The site of liquefaction caused by the 2003 Tokachi-oki Earthquake in Tanno-cho, Kitami City, and the investigation point in Bihoro Town

In Mori Town, the investigation was conducted in two places: one where liquefaction was observed and the other nearby spot of non-liquefaction. At the investigation point in Bihoro Town, no remains or signs of liquefaction caused by the past earthquake were detected but the conventional design method[24)] affirmed the possible occurrence of liquefaction. Items investigated and tested in each site are listed in Table 2.

Table 2 Items investigated and tested

Items investigated	Specifications
Mechanical boring	φ86 for SPT and PS logging φ116 for sampling
Standard penetration test (SPT)	Specifications of SPT used to determine *N*-value for design
Electric cone penetration test (CPT)	Measurements of three components (end resistance, skin resistance and pore water pressure)
PS logging	Suspension type Downhaul type (groundwater level and shallower)
Collection of undisturbed samples	Triple sampling*　GP sampling *One sample from Utsukushigaoka was through thin-wall sampling
Liquefaction test	Cyclic undrained triaxial test
Physical test	Grain size, soil particle density, water content, liquid limit and plastic limit

The standard penetration test (SPT) was undertaken at 1 m-deep intervals under a semi-automatic falling method. All the SPT samples collected were examined in the physical test. To grasp any minor change of faces with depth, the cone penetration test (CPT) was performed at the investigation points. However, CPT was not carried out in Mori Town, because cobbles were scattered about over the relevant ground, where CPT intrusion and accurate measurement were considered impossible.

To collect undisturbed samples for the liquefaction test, instead of frost soil sampling, which is known for the highest sampling accuracy, tube sampling was performed in a hole close to the φ86 bore hole. Because it was feared that the former would subject porous volcanic ash soil with many intra-particle voids (Fig. 5) to frost heave of pore water, possibly leading to particle breakage and change of soil structure. Methods adopted for the tube sampling were the conventional triple sampling (one sample from Utsukushigaoka was through thin-wall sampling) and GP sampling[25] which is recognized for higher accuracy among all tube samplings. The sampling accuracy was compared by collecting samples from holes close to each other at the same depth. The samples collected were not frozen with dry ice or in other ways on the spot to avoid an unfavorable influence from frost heave, but due care was exercised to maintain the quality of the samples while transported.

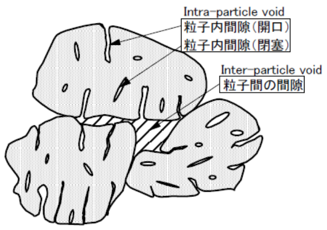


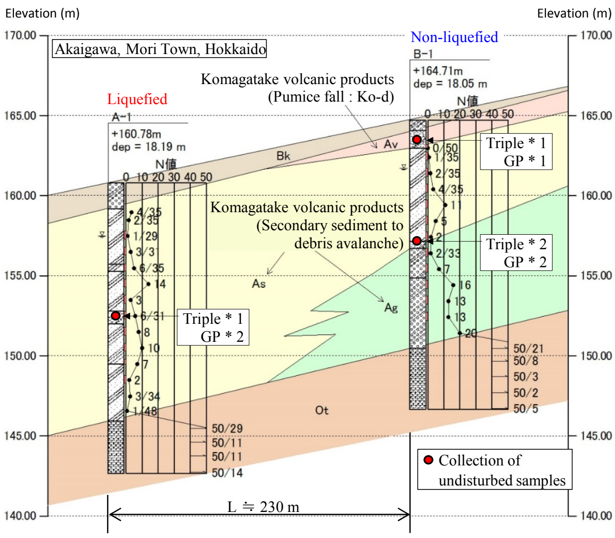
Fig. 5 Typical image of intra-particle voids of volcanic ash soil[5]

In these field investigations and soil tests, PS logging was utilized to evaluate the quality of undisturbed samples, and the initial in-situ shear modulus, G_0, was calculated from an S-wave velocity measured. As for the PS logging, the suspension type was used generally together with the downhaul type at a groundwater level and shallower, and measurements were taken at 0.5 m-deep intervals (measurement section in length of 1 m) so that the measurement section closest to the section of undisturbed samples could be selected later as a reference section.

Before the liquefaction test, weak S waves were applied to each specimen and the initial shear modulus, G_0, was calculated for each from the S-wave velocity. A cyclic undrained triaxial test was conducted as a liquefaction test, where the rate of loading was set at 0.1 Hz. The liquefaction strength ratio, R_L, of each specimen was made equivalent to the cyclic stress amplitude ratio, σ_d / $2\sigma'_0$, in response to DA = 5 % of double amplitude axial strain and the number of cycles at N_c = 20. After the liquefaction test, all the specimens were examined in the physical test.

3. EVALUATION OF LIQUEFACTION CHARACTERISTICS BESED ON PROPERTIES OF SUBSURFACE STRUCTURE

The results of the field investigation and soil test for Mori Town are discussed in this section. The geological profile of the investigation points in Mori Town is shown in Fig. 6.



Period	Geology	Layer		Thickness (m)	N value	
Quaternary	Bank	Bank	Bk	0.65～1.60	-	Sandy gravel of fine fraction nature
	Komagatake volcanic products (Pumice fall : Ko-d)	Volcanic ash	Av	1.10	0	Volcanic ash with pumice
	Komagatake volcanic products (Secondary sediment to debris avalanche)	Sandy soil	As	6.25～13.25	1～14	Gravelly sand of fine fraction nature
		Gravelly soil	Ag	6.25	2～20	Sandy gravel of fine fraction nature
Neogene	Oshironai layer	Rock mass	Ot	3.29～3.80	＞50	Tuff breccia

Fig. 6 The geological profile of the investigation points in Mori Town

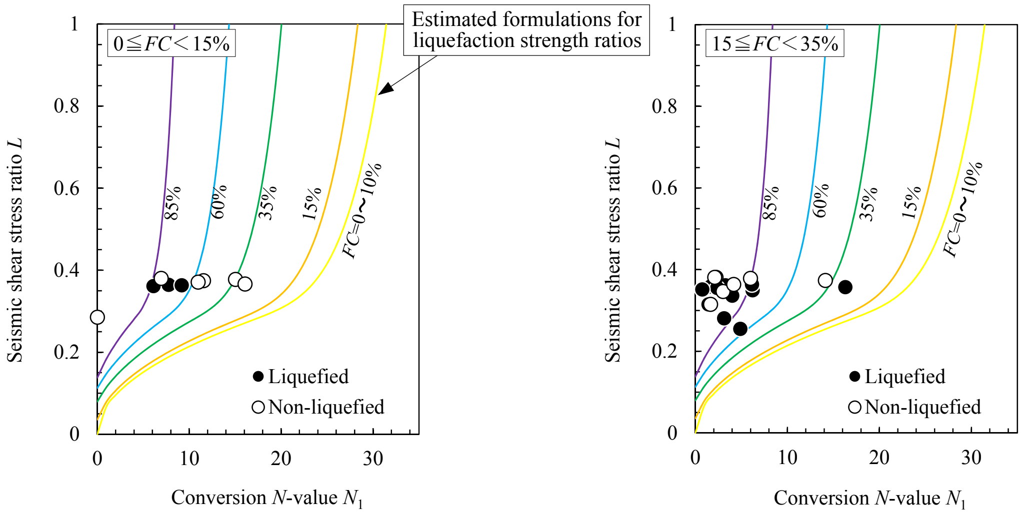
This area is located at the northeast foot of the Hokkaido-Komagatake volcano, where volcanic products (As and Ag) from Mt. Komagatake in the Quaternary period are deposited thick on the Oshironai layer (Ot) which is considered to be an engineering bedrock. This layer consists of volcanic products such as lava, lapilli, pumice and volcanic ash from Mt. Komagatake which has been active since the late Pleistocene epoch to date. These products are regarded as the secondary sediment to debris avalanche, containing a small amount of gravels.

The 1993 Hokkaido Nansei-oki Earthquake liquefied the As layer which had been accumulated widely in this area, inflicting immense damage on houses and roads[26]. Distributed on top of this layer are pumice fall (Av) which is alluvial deposit of Mt. Komagatake and sand gravel (Bk).

To explore the relation between liquefaction and ground strength, the As and Ag layers accumulating below the groundwater level were investigated through SPT at 1 m-deep intervals. Based on the resultant N values and results from the following physical test, converted N values, N_1, and seismic shear stress ratios, L, were obtained for the conventional design method[24] in accordance with the Specifications for Highway Bridges. In Fig. 7, these data are compared between the liquefied place and the non-liquefied place. In finding the seismic shear stress ratio, L, the design horizontal seismic coefficient, k_{hgL}, of the ground was calculated by reducing in depth profiling, where the peak ground acceleration (241 gal) was divided by the gravitational acceleration (980 gal). The data was among horizontal two-way acceleration records registered on the occasion of the 1993 Hokkaido Nansei-oki Earthquake by a strong-motion seismograph installed on the nearby bridge ground[27]. In the same diagram, estimated formulations for liquefaction strength ratios to be used in the assessment of soil liquefaction were drawn in curves according to the fine fraction content, *FC*. The correction factor, C_W, in response to seismic motion was given 1.0. The area above the curve is judged to liquefy whereas the area below the curve is judged not to liquefy.

Fig. 7 was completed according to the ranges of *FC* values obtained at every 1 m in depth. Each spot was plotted in the area above its corresponding curve, which means any non-liquefaction point was judged to have carried liquefaction potential in the conventional design method. But, this quantity of data wouldn't clearly distinguish geological tendencies by the ranges of *FC* values. Yet, no traces of liquefaction were observed at the non-liquefaction points. Concerning this matter, the Report of the Verification Result of Evaluation Method for Liquefaction[2] reasoned that it was a typical tendency of non-liquefaction sites to carry "larger F_L values" or "thinner layers with a factor of $F_L \leq 1$" in comparison with liquefaction sites under the conventional design method, so that the former was less susceptible to liquefaction although the both were judged to have liquefied.

If the Ag layer deposited below the As layer at this particular non-liquefaction place possessed the soil quality proof against liquefaction, the thickness of liquefaction layer (As) was considered thinner than that of the liquefaction place and therefore the ground was relatively less subject to liquefaction. In this context, the occurrence of liquefaction in this particular site cannot be judged correctly by the conventional design method, suggesting it is necessary to properly evaluate properties of ground, geological structure and seismic response of the ground.



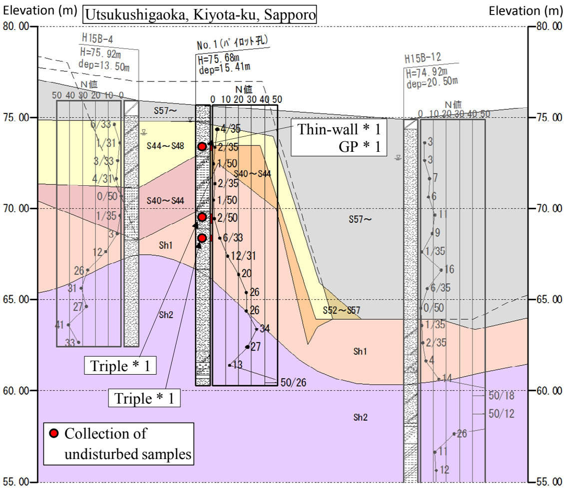
(a) $0 \leq FC < 15$ % (b) $15 \leq FC < 35$ %

Fig. 7 Relation between conversion *N*-values, N_1, and seismic shear stress ratios, *L*, according *FC* for liquefaction and non-liquefaction sites in Mori Town

4. EVALUATION OF LIQUEFACTION STRENGTH RATIO, R_L

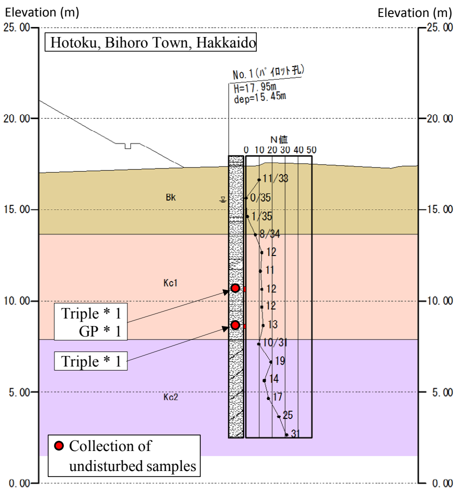
To comprehend properties of liquefaction strength in the volcanic ash ground liquefied by the past earthquakes and their nearby non-liquefaction areas, a liquefaction test (cyclic undrained triaxial test) was performed on samples collected at each point. The places where the test samples were collected in Mori Town are included in Fig. 6. The sampling places in Utsukushigaoka and Bihoro Town are also plotted on the geological profiles in Figs. 8 and 9, respectively.

The investigation site in Utsukushigaoka was land reclaimed by filling a morass and an old river channel with volcanic ash soil, where the layer Bk3 is known to have liquefied on the occasion of the 2003 Tokachi-oki Earthquake, inflicting substantial damage on houses and roads[22]. The investigation site in Bihoro Town was dominated by Kussharo pyroclastic flow deposits (Kc1 and Kc2) which were the secondary deposits to new Kussharo volcanic ash. In this place, no occurrence or traces of liquefaction by the 2003 Tokachi-oki Earthquake and other earthquakes were detected, but the entire area was judged to carry liquefaction potential under the conventional design method[24].



Period	Geology	Layer		Thickness (m)	N value	
Quaternary	Bank	Bank-1	Bk1	–	–	Volcanic ash
		Bank-2	Bk2	–	–	Volcanic ash
		Bank-3	Bk3	2.80	2～3	Volcanic ash
		Bank-4	Bk4	–	–	Volcanic ash
		Bank-5	Bk5	3.10	1～2	Organic silt
	Shikotsu volcanic products	Volcanic ash-1	Sh1	3.10	1～12	Volcanic ash Silty Volcanic ash
		Volcanic ash-2	Sh2	6.41	20～>50	Volcanic ash with pumice

Fig. 8 The geological profile of the investigation site in Utsukushigaoka



Period	Geology	Layer		Thickness (m)	N value	
Quaternary	Bank	Bank	Bk	3.35	0～10	Volcanic ash Organic soil
	Kussharo pyroclastic flow deposits	Volcanic ash-1	Kc1	6.75	7～13	Volcanic ash Volcanic sand
		Volcanic ash-2	Kc2	5.35	10～31	Volcanic ash with sand

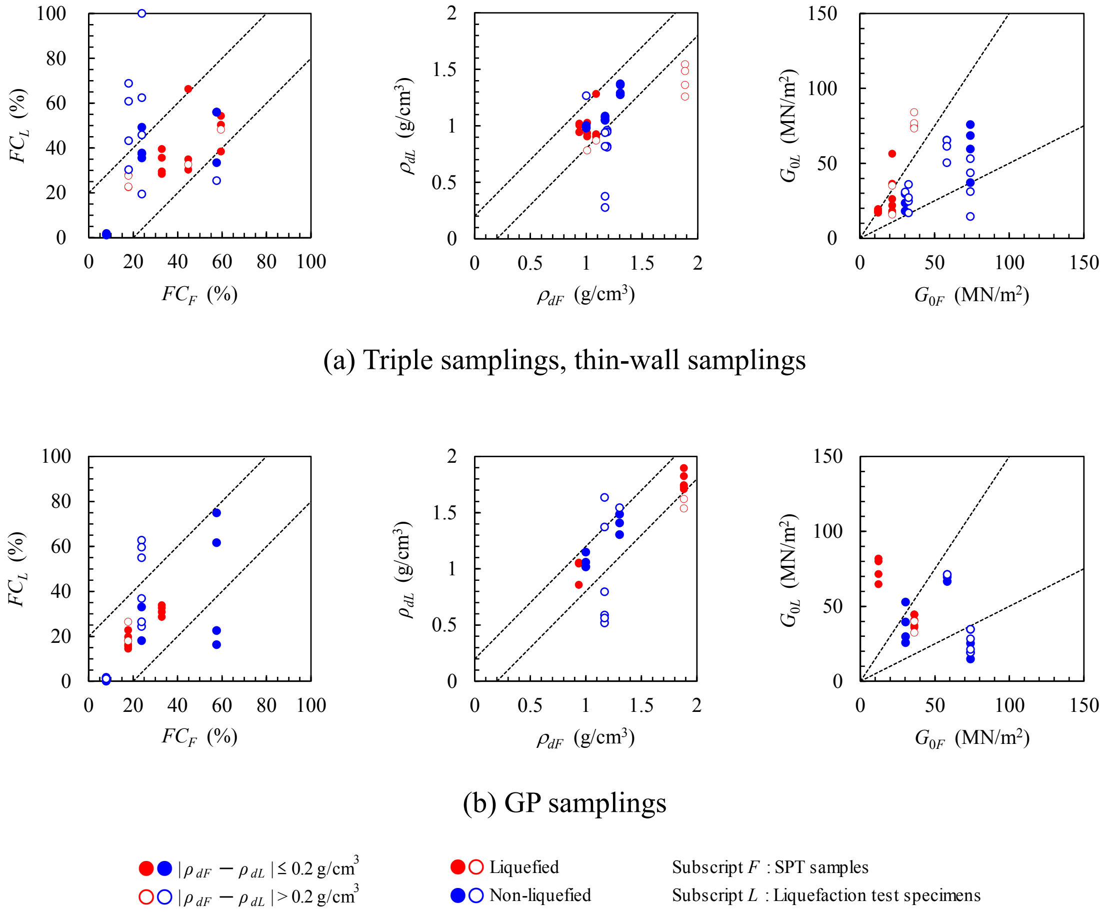
Fig. 9 The geological profile of the investigation site in Bihoro Town

In examining the correlation between liquefaction strengths, N values and grain sizes of the data collected in this research, the samples must be good in quality. To obtain the accurate correlation, their grain sizes need to be equivalent in the both field and laboratory, and the density and microscopic structure of soil in the field must be maintained till the laboratory test.

Disparities of properties between filed samples and laboratory specimens and also qualities of collected samples were evaluated with the fine fraction content, FC, used as a representative index of the grain size; the dry density, ρ_d, as the density; and the initial shear modulus, G_0, as the microscopic structure. The reason why FC was assigned as the index of the grain size was because a large quantity of volcanic ash soil in Japan possesses the nature of particle breakage. Impacts of different sampling methods were also compared. The field dry density, ρ_{dF}, was calculated in *Eq.* (1)[28] with the data of the SPT samples (the water density, ρ_w, was set at 1.0): the soil particle density, ρ_s, the natural water content, w_n, and the degree of saturation, S_r, (hypothesized as 100 %).

$$\rho_{dF} = \frac{\rho_w}{\rho_w / \rho_s + w_n / S_r} \tag{1}$$

The laboratory dry density, ρ_{dL}, was a value obtained after the specimen of the liquefaction test was consolidated. The initial shear modulus of the field, G_{0F}, was figured out on the basis of PS logging whereas that in the laboratory, G_{0L}, was derived from the S-wave velocity calculated before the liquefaction test. Fig. 10 demonstrates the relation between FC, ρ_d, and G_0 for the field samples and test specimens according to sampling methods. The subscript F stands for field and L for laboratory.

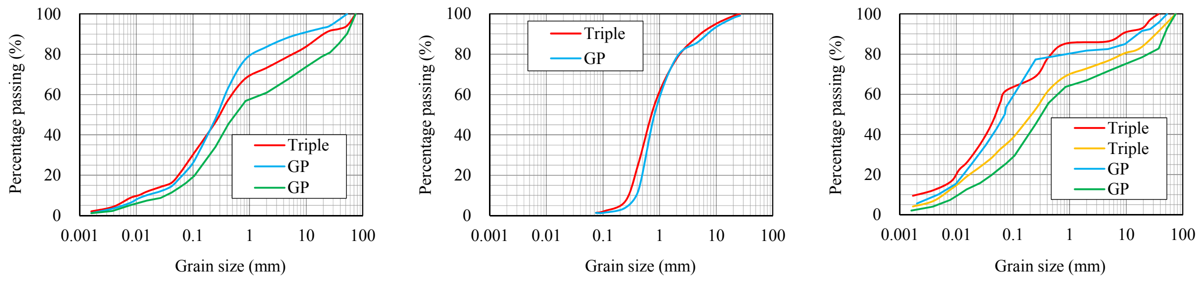


(a) Triple samplings, thin-wall samplings

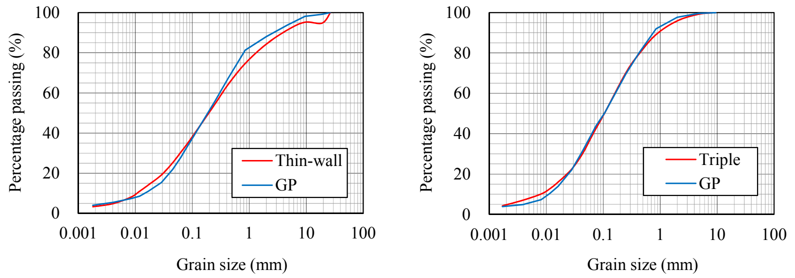
(b) GP samplings

Fig. 10 Disparities and qualities of the test specimens compared with the field samples

According to Fig. 10, each index of liquefaction test specimens dispersed from that of filed samples in any sampling method. It is unusual to carry out the grain size analysis on all of one sample and four specimens used for the liquefaction test. Nevertheless, the Japanese Geotechnical Society has norms that suggest ρ_d be assessed after consolidation as a result of the cyclic undrained triaxial test[29],

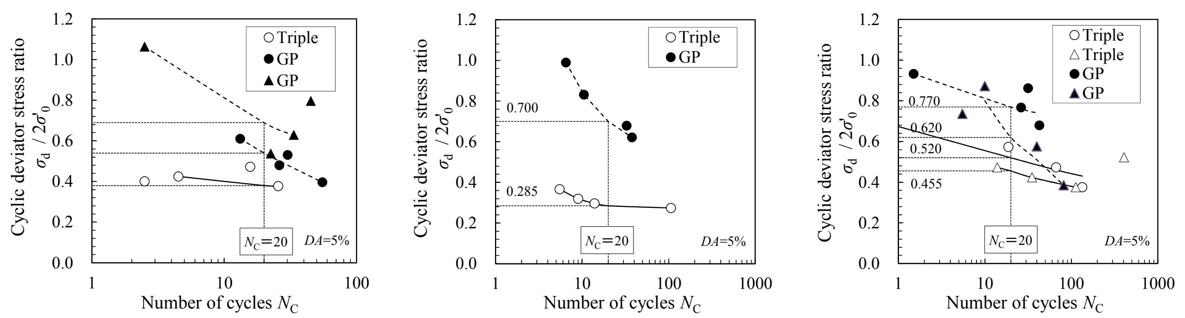


(a) Liquefied As layer, Mori (b) Non-liquefied Av layer, Mori (c) Non-liquefied As layer, Mori

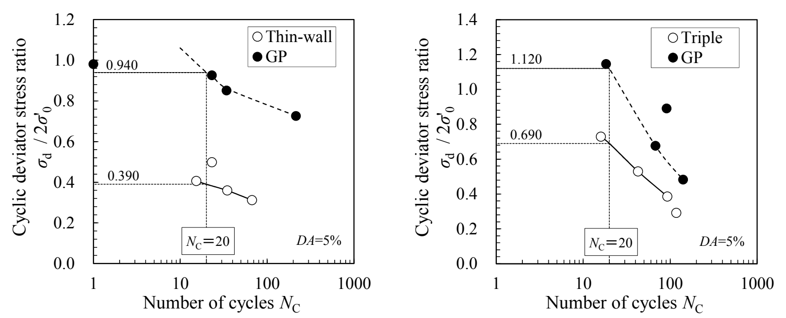


(d) Liquefied Bk3 layer, Utsukushigaoka (e) Non-liquefied Kc1 layer, Bihoro

Fig. 11 Average grain size distribution curves of the one sample and four specimens used for the liquefaction test



(a) Liquefied As layer, Mori (b) Non-liquefied Av layer, Mori (c) Non-liquefied As layer, Mori



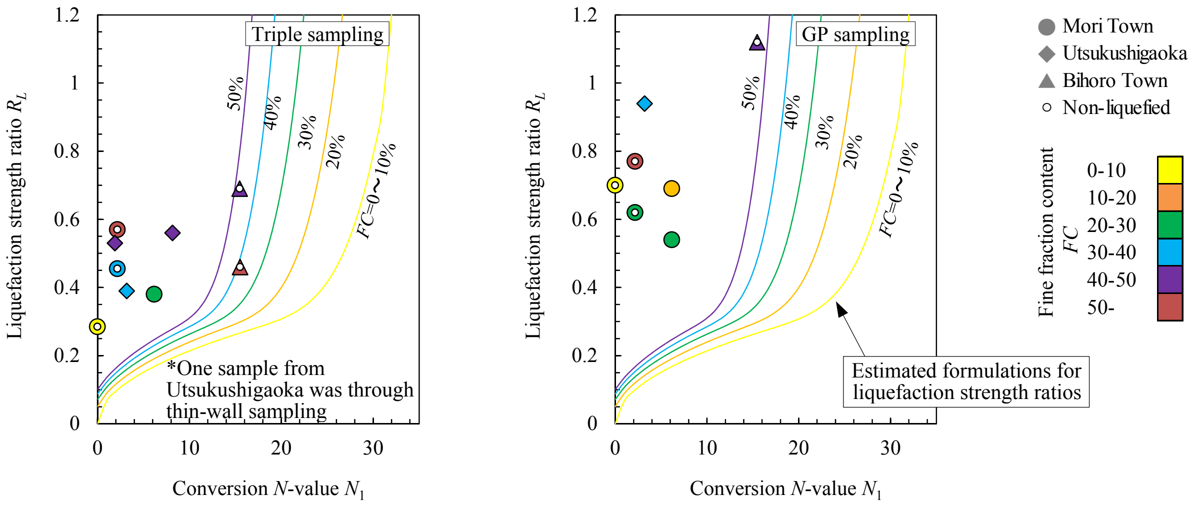
(d) Liquefied Bk3 layer, Utsukushigaoka (e) Non-liquefied Kc1 layer, Bihoro

Fig. 12 Liquefaction strength curves of the samples so that a relative difference in ρ_d is often used as an index to evaluate the quality of samples (or eliminate outliers). In the diagram, evaluation was attempted by separating specimens of $|\rho_{dF} - \rho_{dL}| \leq$ 0.2 g/cm^3 from the others. The samples which had ρ_d within the range of $|\rho_{dF} - \rho_{dL}| \leq 0.2$ g/cm^3 seem to keep other indexes dispersed within a limited range. But this tendency is not necessarily applied to all data. In this regard, ρ_d can be used as one of indexes but it does not fully present differences in grain size and microscopic structure by itself. The sites of liquefaction and non-liquefaction are separately shown in the diagram but their clear correlations to each index are not observed.

Through the quality evaluation of the samples, disparities of each specimen for the liquefaction test from the field samples were confirmed. Yet, the average of the one sample and four specimens was treated as a representative physical property at each depth in each site. The average grain size distribution curves of the one sample and four specimens used for the liquefaction test are shown in Fig. 11 and the liquefaction strength curves of each sample in Fig. 12, for the layers where triple sampling, thin-wall sampling and GP sampling were carried out at the same depth. As for the As layer of Mori Town, more than one sampling was conducted at the same depth in different holes, which were indicated with different marks.

No big difference in grain size distribution between sampling methods is seen in Fig. 11, whereas Fig. 12 reveals that the liquefaction strength ratio of GP samplings was larger than that of the other samplings.

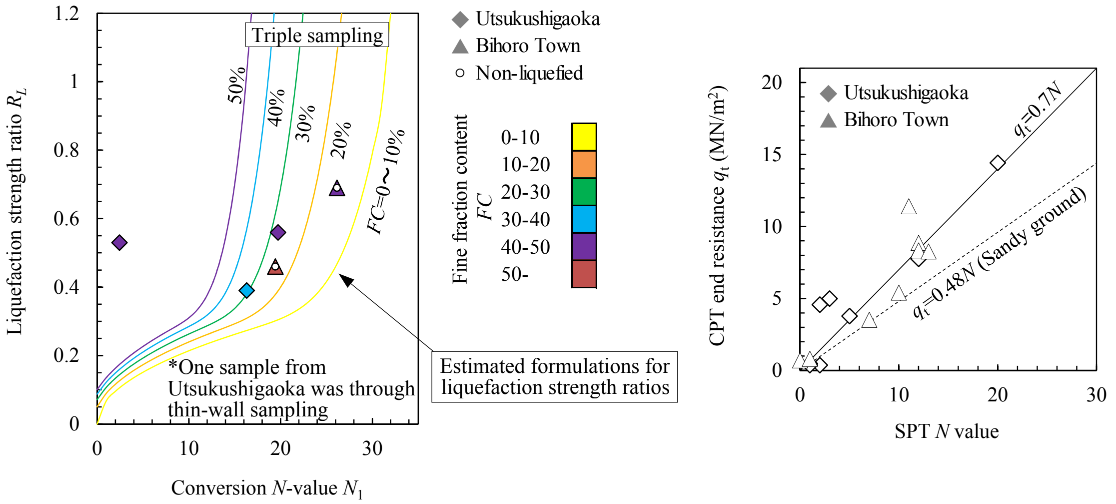
Fig. 13 illustrates the relation between liquefaction strength ratios, R_L, obtained by grouping all the samples in units of *FC* 10 %, and converted *N* values, N_1, according to sampling methods. The latter were calculated by converting SPT *N* values used for the conventional R_L-estimated formulation[24] under the Specifications for Highway Bridges, at the effective overburden stress equivalent to σ_v' = 100 kN/m^2. The conventional R_L-estimated formulation are drawn in curves according to *FC*. Because the liquefaction strength of ground is possibly affected not only by the amount of fines (*FC*) but by the quality of ground soil composition, the classification by plasticity index, *IP*, was also attempted but the result was all NP.



(a) Triple samplings

(b) GP samplings

Fig. 13 Relation between converted *N* values, N_1, and liquefaction strength ratios, R_L, by *FC* and sampling methods



(a) N_1 calculated at $N = q_t / 0.48$ (b) Relation between SPT N values and CPT end resistances, q_t

Fig. 14 Relation of converted N values, N_1, derived from CPT end resistances, q_t, to liquefaction strength ratios, R_L, and static/dynamic penetration resistances

According to Fig. 13, the triple samplings tend to increase R_L along with FC where the values were plotted in the area above their own corresponding FC curves, exceeding the conventional R_L-estimated formulation, except the data from Bihoro. Although the conventional R_L-estimated formulation is characterized by rapid decrease of R_L along with the reduction of N_1 near to 0, the diagram shows R_L remains high even when N_1 is small or close to 0. The triple samplings from Mori Town judged liquefaction potential fairly well (liquefaction will occur at $F_L \leq 1.0$)[24], when evaluated with seismic shear stress ratios, L, ($F_L = R / L$, $R = R_L$ in this case) shown in Fig. 7. The GP samplings proved to have R_L values larger than the triple samplings, indicating no clear relation with FC. This may be because GP samplings are used directly in the liquefaction test without trimming, sample strength can be affected by some factors: for examples, a difference of the maximum grain size ratio from ordinary liquefaction test samples, and seepage and adherence of special sampling gel (high density polymer solution) applied to sample surface. Thus, these effects need to be examined further.

In sum, the R_L value of volcanic ash soil from Mori, Hokkaido, is large and the occurrence of liquefaction can be fairly expressed by R_L of triple samplings. The conventional R_L-estimated formulation, however, underestimates the value. Therefore, it is necessary to add a proper coefficient or a correction value to the conventional method, or to newly develop an R_L-estimated formulation exclusive to volcanic ash soil.

Then, CPT data obtained through the filed investigation in Utsukushigaoka and Bihoro were extracted from Fig. 13(a) and the converted N values, N_1, calculated from the CPT end resistance, q_t, were plotted in the same way (Fig. 14(a)). In addition, the relations between SPT N values and q_t at the same points are shown in Fig. 14(b). The static/dynamic penetration resistance ratio, q_t / N, of Hokkaido's crushable volcanic ash ground is already known to be 0.7, which is larger than the average sandy ground of $q_t / N = 0.48$[18]. As Fig. 14(b) indicates, the investigation in this study exhibits the similar relation. In comparison with the sandy ground which holds an equivalent q_t level, the N value seems to be underestimated due to particle breakage induced by dynamic intrusion of SPT. In this case, static penetration resistance, q_t, which may involve no particle breakage, is considered to be inherent strength in volcanic ash ground, and the converted N value, N_1, was evaluated by calculating the N value ($N = q_t / 0.48$) from the average static/dynamic penetration resistance ratio of sandy ground. Consequently, although the result in Fig. 14(a) disagreed with the conventional R_L-estimated formulation for sandy soil, the converted N value, N_1, was overestimated and the correlation between N and R_L was expressed appropriately, suggesting that R_L of crushable volcanic ash soil can be evaluated with the static penetration resistance.

5. SUMMARY

In developing a highly accurate method of assessing soil liquefaction in response to Japan's diverse soil qualities and geological structures, the authors conducted field investigations and laboratory soil tests for several liquefaction sites damaged by the past earthquakes and their neighboring non-liquefaction areas in Hokkaido, with the aims of comprehending and evaluating the liquefaction characteristics of volcanic ash soil which is unusual soil but not handled distinctively by the conventional design method. The following are the investigation results.

(1) In the liquefaction site of Mori Town, Hokkaido, and its neighboring non-liquefaction site, which suffered the 1993 Hokkaido Nansei-oki Earthquake, the occurrence of liquefaction was surveyed on the basis of N values and ground surface accelerations observed nearby. As a result, the conventional design method was found incompetent for expressing the situation correctly as the non-liquefaction site was also judged to have carried liquefaction potential. The non-liquefaction site assumably possessed a thin $F_L \leq 1$ layer, so that the ground could be relatively little prone to liquefaction, suggesting that ground properties, geological structures and seismic response characteristics need to be evaluated properly.

(2) As for volcanic ash soil, the grain size and density may vary between SPT samples and liquefaction test specimens, or between specimens including one sample for the liquefaction test. Under these circumstances, in analyzing the correlation between data of the liquefaction test and N values, such factors as not only ρ_d but FC and G_0 should be used as indicators to eliminate outliers and evaluate the qualities of test specimens neatly.

(3) In this study, disparities were found wide between the liquefaction test specimens and the field samples. As the average of one sample and four specimens was used as a representative physical property of each sample, R_L derived from triple samplings was found to be able to express the liquefaction potential fairly well. The conventional R_L-estimated formulation underestimates the R_L value. Therefore, it is necessary to add a proper coefficient or a correction value to the conventional method, or to newly develop an R_L-estimated formulation exclusive to volcanic ash soil.

(4) The converted N value, N_1, used for the R_L-estimated formulation, was evaluated based on the CPT end resistance, q_t. As a result, it was suggested that R_L of crushable volcanic ash soil could be evaluated with the static penetration resistance.

For the future, our efforts will be directed to accumulate more data through further investigations and tests while carrying out studies on seismic response characteristics, to contribute to the development of a highly accurate method for assessing soil liquefaction for volcanic ash soil.

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