Liquefaction Case Study of Volcanic Gravel Layer during 1993 Hokkaido-Nansei-Oki Earthquake

T. Kokusho

Y. Tanaka

K. Kudo

T. Kawai

Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics,
April 2–7, 1995, Volume I, St. Louis, Missouri

Liquefaction Case Study of Volcanic Gravel Layer during 1993 Hokkaido-Nansei-Oki Earthquake

Paper No. 3.20

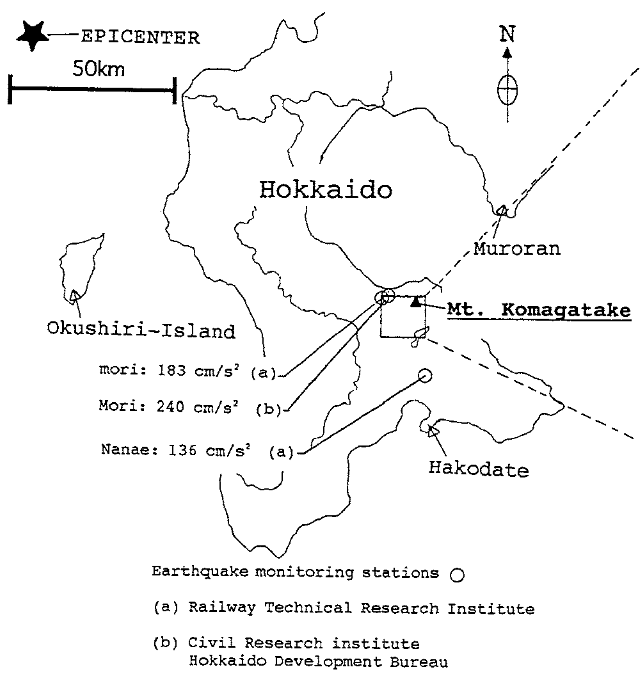
T. Kokusho, Y. Tanaka, K. Kudo and T. Kawai

SYNOPSYS During the Hokkaido-Nansei-Oki Earthquake, a high-land area around the foot of the volcano, Mt. Komagatake, in Hokkaido in Northern Japan experienced liquefaction phenomena inflicting damages of differential settlement in some tens of houses in the area. Several kinds of geotechnical site investigations were carried out to study the cause of the damage in a typical liquefaction site. Trenching and geological survey disclosed that Holocene debris avalanche deposit containing a plenty of large rocks actually liquefied. The undrained cyclic triaxial test was carried out on large intact samples taken out by the in-situ freezing sampling. The stress ratio measured in the test was in accordance with the seismic induced stress ratio estimated from the maximum acceleration in the area.

INTRODUCTION

During the Hokkaido-Nansei-Oki Earthquake(M=7.8) of July 12, 1993, the south-western part of the high-land area of Mt. Komagatake (Fig. 1) suffered liquefaction damage causing differential settlements in 44 wooden houses in Akaigawa-area in Mori Town as reported by the local municipal office. Similar damages were also found in the neighboring Nanae Town, too. Geology of the liquefied area was first investigated by Nirei et al. [1993] immediately after the quake and the liquefaction in gravelly layer called the Kurumizaka Rock Debris Avalanche Deposit was pointed out.

Based on these informations the authors started a systematic site investigations in one of the liquefied sites to examine the cause of the liquefaction including geological survey, geophysical explorations, boring, PS-wave logging, in-situ freezing soil sampling and laboratory test. This paper describes the results on the quantitative evaluation of the liquefaction susceptibility of this local gravelly soil.



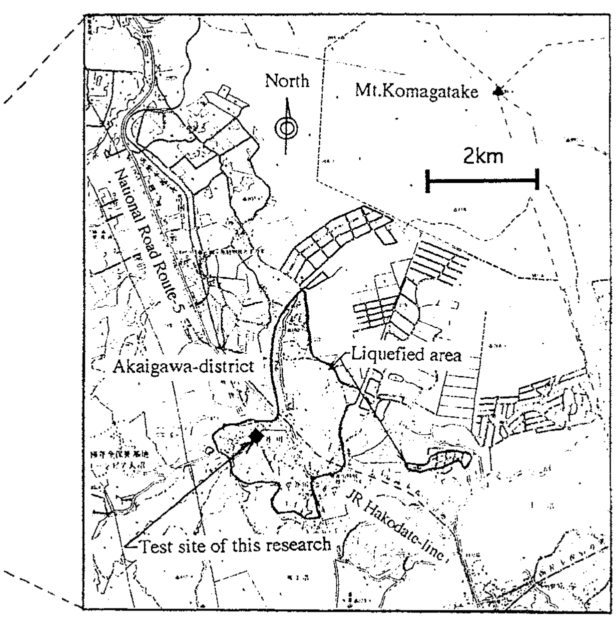


Fig.1 Investigated Site in Liquefied Area near Mt. Komagatake in Southern Hokkaido

GENERAL SITE SURVEY

The test site was selected in the liquefied area as plotted in Fig. 1 where a two-story big wooden pension house suffered differential settlement of 32 cm and needed considerable repair [Matsusaka 1993]. Fig. 2 indicates the land property belonging to the house where the detailed geotechnical investigations was carried out. After the earthquake, sand eruptions due to liquefaction were observed in many parts of the site, among which those near the pension were remarkable with visible ground surface fissures and settlement.

At first two bore-holes Bh-1 and Bh-2 were drilled in two separate points with and without conspicuous liquefaction-related phenomena at the ground surface as indicated in Fig.2. The soil profiles is shown in Fig.3 with the water table as high as 0.9 m from the surface. The site is covered with volcanic ash of about one-meter thick underlain by gravel layer with varying thickness and depth. This gravel layer identified as Kurumizaka Rock Avalanche Layer is said to have been formed by volcanic eruption of Mt.Komagatake in 1640.

Fig. 4 shows particle size distribution curves for the soil samples taken out from this gravel layer in one of the trenches as illustrated in Fig.2, indicating that the gravel is very well-graded with the uniformity coefficient, U_c, of more than one hundred, the sand content about 20% and the fines content about 3%. In the same figure representative grain size curves for the erupted sands and silts sampled from the ground surface are indicated. Obviously there exist big differences in particle size between the in-situ gravel and the erupted soil, as if denying the involvement of the gravel layer for the liquefaction.

Fig.5 shows a side wall of a trench excavated near the damaged house, in which the sand eruption path filled with sand and silt is clearly observed, demonstrating the path is initiated from the gravel layer about one-meter below going up through the upper volcanic ash all the way to the surface. It may well be concluded from this observation that liquefaction actually took place in the gravel layer during the earthquake and the sandy or silty matrix was selectively pushed up by the excess pore-pressure.

In order to clarify the age of the gravel layer, organic soils found at three locations in the boring core samples sandwiching the gravel as indicated in Fig. 3 were analysed with the carbon-14 dating technique. Though the results have some standard deviations, they are all concentrated at a little larger than 2000 years. Therefore this gravel layer believed to be rock avalanche may have been formed about 2,000 years ago, a little older than normally believed but still a young sediment in the Holocene Epoch.

GEOTECHNICAL INVESTIGATION IN BORE-HOLES

In the two bore-holes, Bh-1 and Bh-2, large penetration tests (LPT) were conducted down to the depth of 12m and 10 m respectively to investigate properties of the rock avalanche gravel layer. LPT, the specification of which is available in Table 2 in comparison with the standard penetration test (SPT), is an upgrading version of SPT often used in Japan for gravel layers. LPT blow-counts, N_L, is normally correlated to SPT blow-counts, N, as; $N = 2\ N_L$ for gravel and $N = 1.5\ N_L$ for sand (Yoshida et al. (1988)).

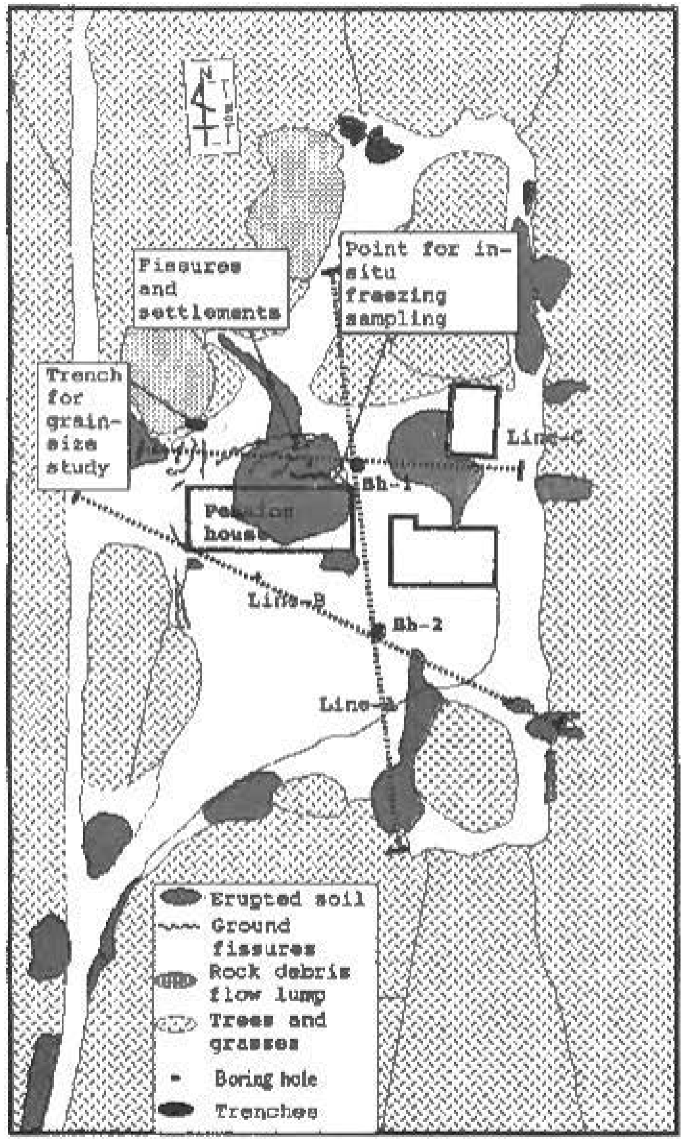
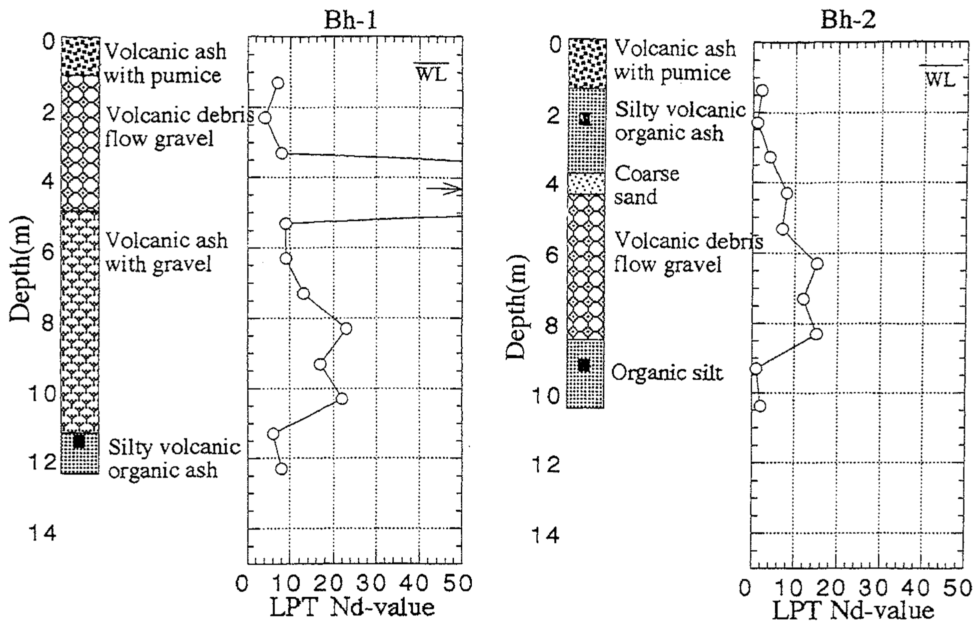


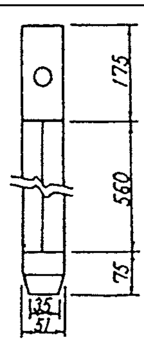
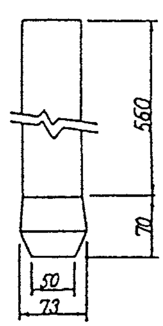
Fig.2 Land Properties of Investigated Site with Liquefaction-Related Phenomena and Points for Geotechnical Surveys



■ Samples for C-14 dating:
Bh-1 2070±250years B.P.
Bh-2 2160±240years B.P.(Upper sample)
2230±230years B.P.(Lower sample)

Fig.3 Soil Profiles in Two Boring Holes along with LPT Blow-Counts and C-14 Dating Results

Table 1 Specifications of Standard Penetration Test (SPT) and Large Penetration Test (LPT)

Test name	SPT: Standard Penetration test	LPT: Large Penetration Test
Weight(N)	622.3	980
Drop height (cm)	75	150
Drive length (cm)	30	30
Details of Penetration Probe		

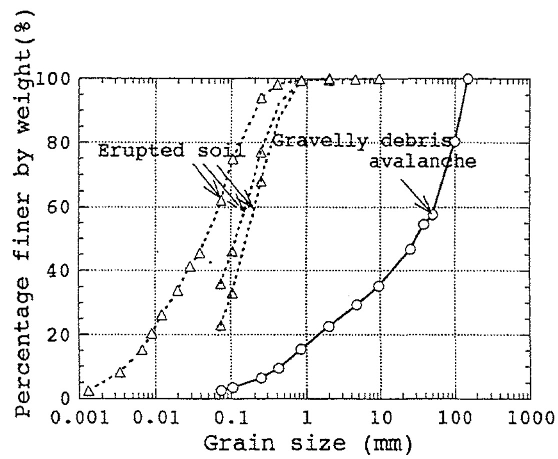


Fig.4 Grain Size Curves for Rock Debris Avalanche Layer and Erupted Soils

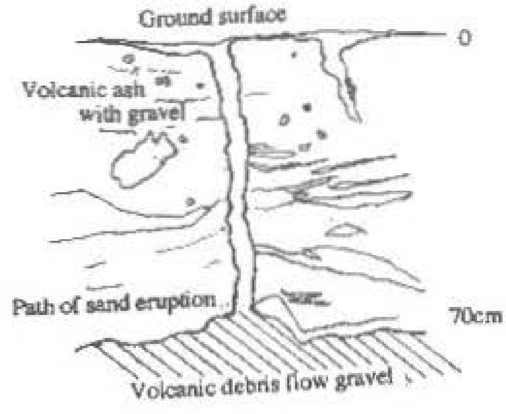


Fig.5 Photograph and Sketch of Sand Eruption Path Starting from Volcanic Debris Avalanche Deposit through Overlying Volcanic Ash Layer

Fig.3 shows the N_L vs depth relationships for the two bore-holes along with soil profiles. At one depth in Bh-1, the LPT probe could not penetrate due to a big rock, which was identified by the subsequent drilling. The upper part of the gravel layer probably liquefied during the earthquake exhibit N_L-values of 4 to 8, equivalent to SPT N-value of 8 to 16, exceptionally low as gravel soil with the much lower void ratio than sand as will be described later.

After drilling and casing the bore-holes with VCL tubes, the P and S-wave logging was carried out by means of the down-hole method. In fig.6, the travel times for P and S-waves are plotted against the depth for the two bore holes. Fig.7 is the velocity distributions interpreted from the original data in Fig.6. The thick solid lines correspond to the conventional data interpretation. In order to have finer velocity variations other data interpretations were tried; three different lines with different marks in Fig.7 for the S-wave velocity are obtained by the moving average method proposed by Okamoto et al. (1989), in which several neighboring data points in the original down-hole data as shown in Fig.6 are used to calculate the average local wave speed corresponding to the middle point of those neighboring data points. It is evidently seen with larger number of neighboring data points the more smoothed the curves are. The dotted curve also drawn in the same graph corresponds to the wave velocity measured by paired wave receivers connected together. The difference in travel time between the two receivers one-meter apart gives local wave speed in this method. Comparison of these three kinds of wave velocity distribution indicates that, while the conventional method can give a mean value in each interval, the moving average method and the paired receivers method provides finer velocity variations. These finer velocity changes are consistent with each other and also very similar to the change in the LPT blow-counts shown in the same figure.

It may be judged from these extensive data sets that, though there exist considerable velocity changes in the gravel layer probably due to the existence of large rocks in the layer, the upper part of the rock avalanche layer has the S-wave velocity as low as 80 m/s or less.

According to the authors' experiences, gravel layers in Pleistocene Epoch normally have the S-wave velocity larger than 300 m/s [Kokusho et al. 1994] . Considering that the S-wave velocity less than 100 m/s normally corresponds to soft cohesive soils with the void ratio larger than one while the void ratio of this gravel layer is about 0.3 as explained later, it may well be said that this gravel layer is offering a quite new challenging problem geotechnical engineers should look into. More detailed properties of this gravel layer is available in other literature [Tanaka et al.(1994), Kokusho et al. (1994)].

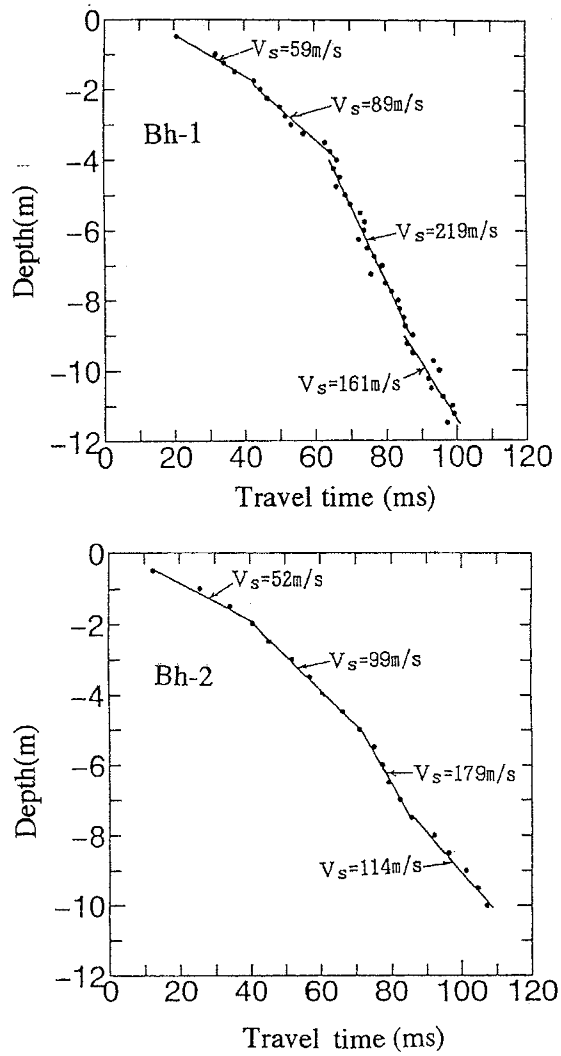


Fig.6 Travel Time vs Depth Relationship for Shear Wave in Two Bore-Holes

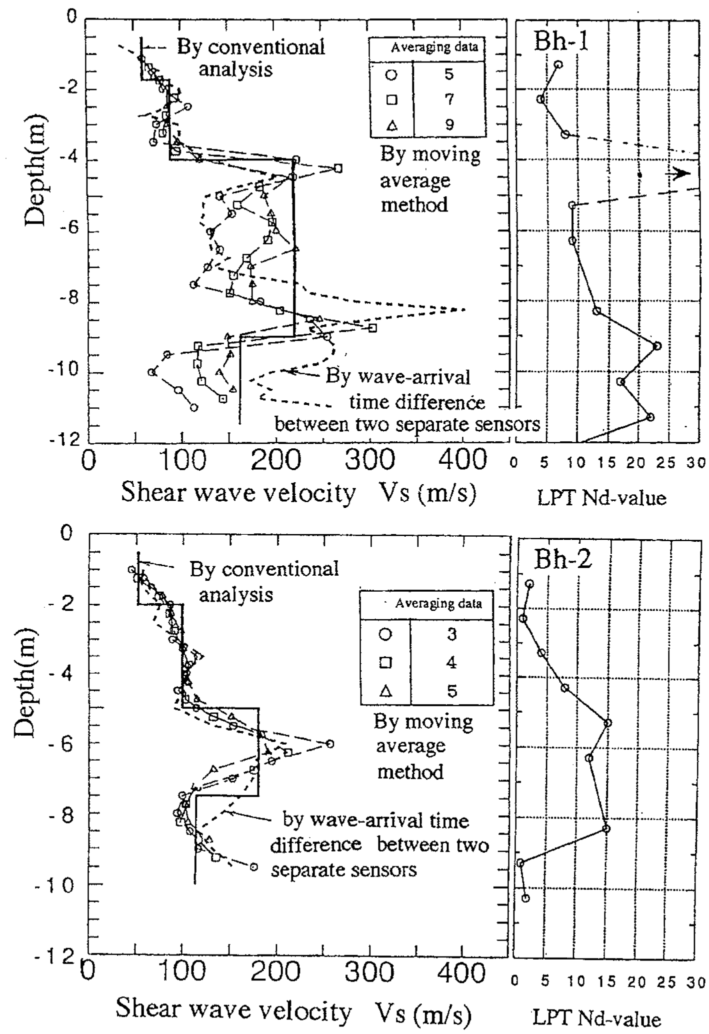


Fig.7 Shear Wave Velocity Distributions derived from Different Data Interpretation Methods compared with LPT Blow-Counts

IN-SITU FREEZING SAMPLING

Since there can be found few quantitative case studies on gravels liquefied in past earthquakes, it is of great importance to directly measure the undrained cyclic strength of this new type gravel by laboratory tests on intact samples. It is not an easy task , however, to recover an intact sample from this kind of gravel, and the in-situ freezing sampling is the only possibility to have samples as intact as possible. The sampling was carried out at the point as shown in Fig.2 where the liquefaction-related phenomena were most remarkably seen. There, the top layer of the volcanic ash was excavated by about one meter to reach the gravel layer from which the sand eruption paths as shown in Fig.5 started obviously.

Only the top part of the gravel layer was frozen to make a frozen bowl shape soil by means of the following procedures as illustrated in Fig.8;

1. A main freezing double-tube pipe of 1 m in length was gently inserted into the gravel by using a boring machine and then a top plate of 1.3 m in diameter with attached winding tubes was placed on the surface.
2. The liquid nitrogen (LN_2) was injected to the main freezing pipe and then guided to the winding tubes beneath the top plate. The temperature change in the frozen soil was monitored by buried thermocouples. The side of the frozen soil was step by step excavated and covered with theorem-insulators to eventually make the frozen soil of the maximum diameter 1.5 m and the height 1.35 m.
3. The frozen soil mass was then lifted (Photo.1) by a crane, from which core samples for laboratory tests, 30 cm in diameter and 60 cm in height, were drilled out.

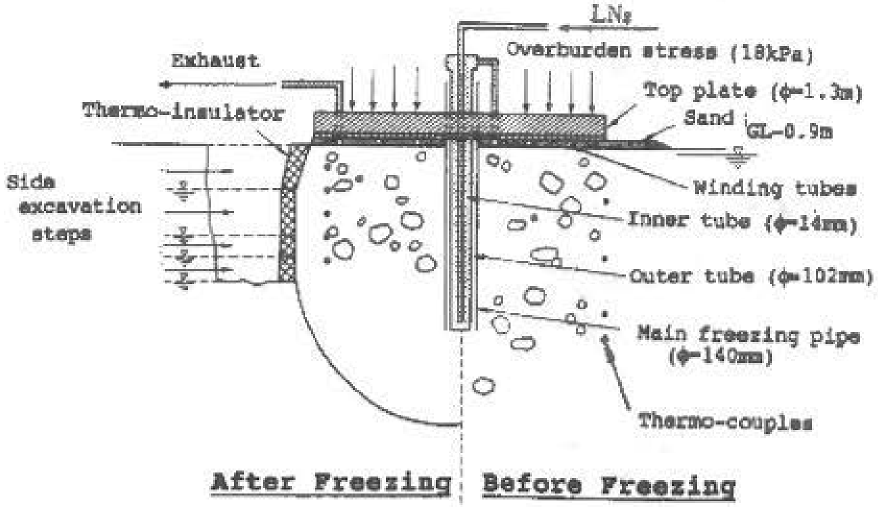


Fig.8 Cross-Sectional View on In-Situ Freezing sampling Method applied to Rock Debris Avalanche Deposit

Photo.1 Frozen Soil Sample Lifted by Crane (Max. Diameter=1.5m)

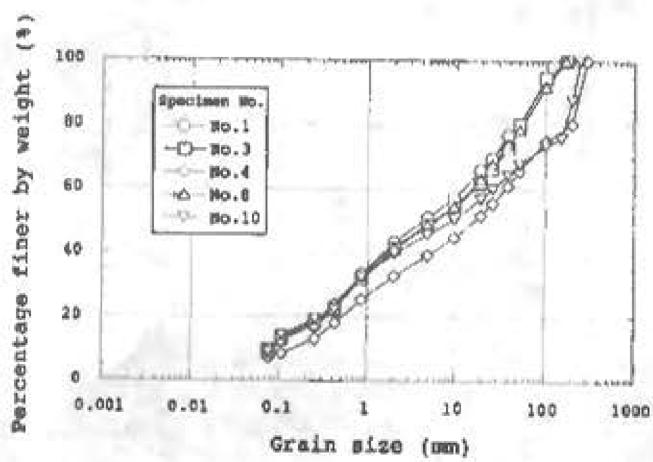


Fig.9 Grain Size Distributions for Tested Five Specimens

Table 2 Test Parameters for Five Specimens for Undrained Cyclic Triaxial Tests

Specimen No.	No.1	No.3	No.4	No.8	No.10
Dry density[g/cm^3]	2.0311	2.0378	2.1178	2.0725	2.1127
Wet density [g/cm^3]	2.2837	2.2879	2.3385	2.3098	2.3352
Void ratio	0.3379	0.3335	0.2832	0.3112	0.2863
Skempton's B-value	0.951	0.995	0.953	0.976	0.993
Compliance ratio: C_R	0.528	0.686	0.805	0.939	0.813
Stress ratio: $\sigma_d/2\sigma_c'$	0.221	0.193	0.168	0.181	0.169
No. of loading cycles (1%)	10.4	19.0	51.3	33.3	24.9
No. of loading cycles (2%)	13.7	22.8	57.2	38.7	28.1

UNDRAINED CYCLIC TRIAXIAL TEST

Undrained cyclic triaxial tests were conducted for the intact specimens of 30 cm in diameter and 60 cm in height taken by the in-situ freezing technique. The specimen set in the triaxial cell was thawed under the confining stress of 30 kPa and fully saturated with the aid of the carbon-dioxide gas, satisfying the Skempton's B-value above 0.95. It was then isotropically consolidated up to 50 kPa and cyclically loaded under undrained condition. The specimens were tested with sinusoidal wave of 0.1 Hz.

Among eleven specimens cored from the bowl-shaped big frozen soil, five of them with relatively smaller content of big particles were used for the test. The grain size curves and other test parameters are available in Fig.9 and Table 2 respectively. The maximum grain size was more than 10 cm in all specimens and in two specimens the longest grain axis was more than 30 cm. The void ratios of these samples were 0.34 to 0.28, remarkably lower than sands because the particles are very well-graded.

Representative time histories of the axial stress, the excess pore-pressure and the axial strain as well as the effective stress path are shown in Fig.10. It is seen that the axial movement can not catch up with the drastic increase in axial strain at the moment of complete liquefaction, thus causing the gradual decrease in the axial stress. This kind of behavior is very similar to what is normally observed for loose sand liquefaction. In order to evaluate the sress ratio for the axial strain double amplitude of 5 %, the strain record was linearly extrapolated as indicated in Fig.10.

Fig.11 shows the cyclic stress ratio vs number of loading relationship obtained for five specimens. The results are very consistent though those two specimens with particles bigger than 30 cm are included here. Considering that all specimens contain many gravels larger than 10 cm, it is really surprising that the test results are so stable.

Taking account the membrane penetration effect, the relationship in Fig.10 was then modified by the method proposed by Tokimatsu et al.(1987) in conjunction with the compliance ratio, Cr=0.8, as measured by the method by Tanaka et al.(1991), eventually giving the two thick lines shown in Fig.11 . Thus the undrained cyclic strength ($\sigma_d/2\sigma_c'$) for the number of cycles of 20 is finally evaluated as 0.18 and o.19 for the double strain amplitude is 2% and 5% respectively.

In the next step the undrained cyclic strength obtained in the laboratory test was compared with the earthquake induced stress estimated from neaby earthquake records. The maximum accelerations during the Hokkaido Nansei-Oki earthquake were summarized by Tokimatsu(1993) and some of the values near Mt. Komagatake are written in Fig.1. Although no records were available in the very vicinity of the investigated site, the maximum horizontal acceleration around this area may be assumed about 0.18 G as an average of these values.

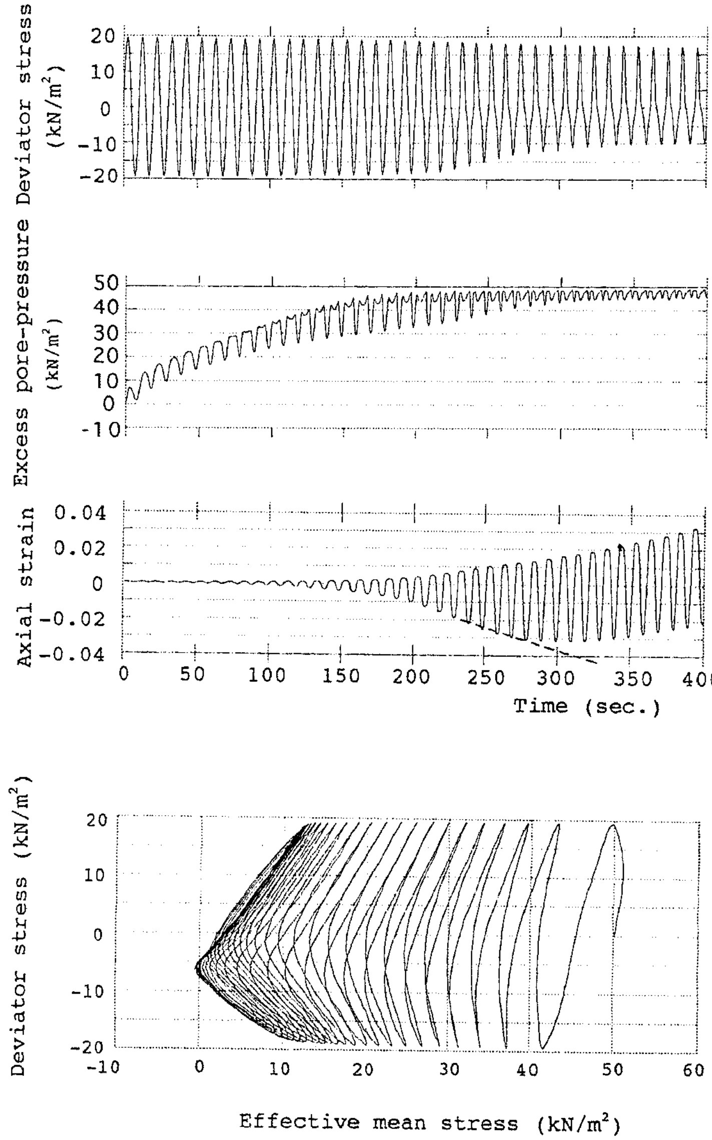


Fig.10 Time-Histories of Undrained Cyclic Triaxial Tests (Deviator stress, Pore-Pressure and Axial Strain) and Effective Stress Path

Based on the earthquake Magnitude of 7.8, the number of equivalent cycles of earthquake motion was assumed about 20. The water-table was assumed 0.9 m below surface though it may have been a little higher than this during the earthquake. The seismically induced maximum shear stress, τ_{maxL}, was calculated from the full overburden, σ_v, above the level of the center of the tested sample (1.3 m below the surface) multiplied by 0.18; thus, $\tau_{maxL}=0.18\sigma_v$. On the other hand the max. strength, τ_{maxR}, is calculated from the effective overburden,

σ_v', as $\tau_{maxR}=0.19\sigma_v'$ assuming that the stress amplitude ratio between the equivalent sinusoidal wave of 20 cycles and irregular seismic wave is compensated by the mean stress ratio, $(1+2K_0)/3$, where K_0 is the horizontal stress coefficient. The safety factor for liquefaction, F_1, is then given as $F_1=\tau_{maxR}/\tau_{maxL}=0.83$, implying that the gravel layer possibly liquefied during the earthquake.

CONCLUSIONS

Based on the intensive site investigations for the gravel layer in the liquefied site, the following principal findings have been drawn.

1. Boring and trenching demonstrated that the very coarse and well-graded gravel layer formed by rock debris avalanche during past volcanic eruption actually liquefied and erupted fine matrix of the layer to the ground surface.
2. Intact gravel samples of 30 cm in diameter have been recovered from the upper part of the layer by means of in-situ freezing sampling technique.
3. Small undrained cyclic strength of 0.18 and 0.19 for ε_{DA} = 2 % and 5 % respectively for N_1=20 has been obtained. This strength has been found smaller than seismically induced stress in the upper gravel layer estimated from the measured maximum surface accelerations, supporting the occurrence of liquefaction during the earthquake.

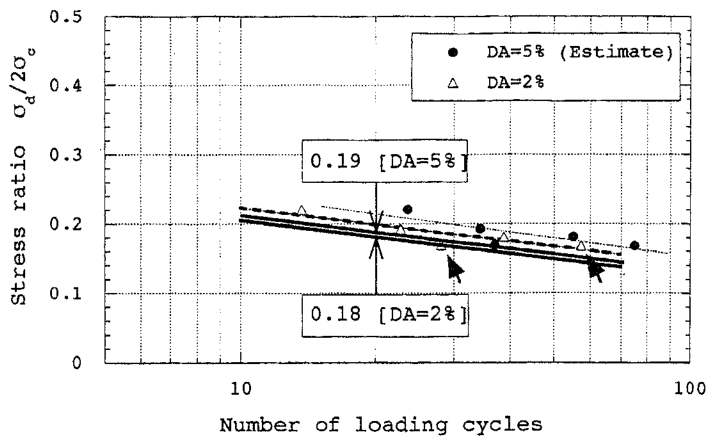


Fig.11 Stress Ratio vs Number of Loading Cycles obtained from Undrained Cyclic Triaxial Tests (Original Plots and Two Solid Lines after System Compliace Correction)

ACKNOWLEDGMENTS

Mr. Yukihiko Natsusaka who is the owner of the pension house is gratefully acknowledged for his great cooperation in the site investigation. Mr. Kiyoshi Nakanishi of Mori Municipal Office is also acknowledged for his help in this research. The authors gratefully appreciate the sampling work successfully carried out by the Kiso-Jiban Consultants Inc. They are also grateful to Mr. Shinji Tohda of CRIEPI who conducted geological research.

REFERENCES

Kokusho, T., Okamoto, T., Tanaka, Y., Kudo, K. and Kawai, T., "Liquefaction case study of gravelly debris avalanche layer during Hokkaido-Nansei-Oki Earthquake"(in Japanese) Proc. 9th Japan Earthquake Engineering Symposium, 1994

Kokusho, T. and Tanaka,Y. "Dynamic properties of gravel layers investigated by in-situ freezing sampling" Proc. "Ground Failure under Seismic Condition" ASCE Convention in Atlanta, 1994

Natsusaka,Y., Personal memorandum on damage and repair of pension house(in Japanese), 1993

Nirei,H., Natsusaka,Y., Nakanishi,K., Morisaki,M., Kimura,T. and Kusuda,T., "The liquefaction and fluidization phenomena of debris avalanche deposits from Mt. Komagatake volcano by the 1993 Hokkaido Nansei-oki earthquake" (in Japanese) The Journal of the Geological Society of Japan, 99, pp.xxi-xxii, 1993

Tanaka, Y.,Kokusho, T., Yoshida, Y., and Kudo, K., "A method to evaluating membrane compliance and system compliance in undrained cyclic shear tests," Soils and Foundations, Vol.31, No.3, pp30-42, 1991

Tanaka,Y., Kokusho, T., Okamoto,T., Kusunoki,K., Kawai,T., Suzuki,K., Kudo,K., Kataoka,T., Tohda,S., Abe,S. and Honsho,S., "Investigation on causes of liquefaction of Gravel Debris Avalanche Deposit Occured during Hokkaido-Nansei-Oki Earthquake of July 12, 1993(Part 1) - Geotechnical investigation and liquefaction assessment- (in Japanese) CRIEPI Report, 1994

Tokimatu, K. and Nakamura, K., "A simple correction for membrane compliance in liquefaction tests," Soils and Foundations, Vol.27, No.4, pp111-122, 1987

Tokimatsu,K., Suzuki,Y. and Tanuma,S.,"Preliminary Report on the Geotechnical Aspects of the Hokkaido-Nansei-Oki earthquake of July 12,1993" Special Volume "Performance of Ground and Soil Structures during Earthquakes" for 13 ICSMFE, 1993

Yoshida,Y. and Kokusho,T., "Empirical formulas of SPT blow-counts for gravelly soils" Proc. ISOPT-1 for Penetration Testing, pp.381-387, 1988