CASE STUDIES OF LIQUEFACTION IN THE 1964 NIIGATA EARTHQUAKE

KENJI ISHIHARA* and YASUYUKI KOGA**

ABSTRACT

Detailed soil investigations were conducted by means of the standard penetration test and Dutch cone penetration test, together with sampling by means of the large diameter sampler and Osterberg type piston sampler at two sites in the city of Niigata, Japan; one where liquefaction type failure had occurred, and the other where liquefaction type failure had not occurred during the 1964 earthquake. Undisturbed loose samples of sand were tested in the laboratory using the cyclic triaxial test apparatus. The cyclic strength thus determined at each depth of the deposits at the two sites was incorporated into the simple liquefaction analysis which was carried out on the basis of the ground surface acceleration records obtained at the basement of a nearby apartment building during the 1964 earthquake. The results of the analysis indicated the behaviors of two sites that were in accord with the actual performances observed at the time of the earthquake.

Key words: earthquake damage, laboratory test, liquefaction, sampling, sand

IGC: D7/C8

INTRODUCTION

An attempt to clarify the mechanism of the liquefaction of the ground that occurred in Niigata during the 1964 earthquake was made by Seed and Idriss (1967). The destruction in the city of Niigata due to the liquefaction was also investigated by Iwasaki et al. (1978) on the basis of the cyclic resistance of the sand deposits estimated from the blow count values of the standard penetration tests.

In order to obtain a better understanding of the resistance of sand deposits to liquefaction during earthquakes, accurate evaluation of the cyclic strength of insitu sand will be a prerequisite. One of the approximate ways to secure this goal would be to obtain undisturbed sand samples from the sites where clear evidence of liquefaction has been detected during past earthquakes, and to test them in the laboratory to determine the cyclic strength. Some attempts to this end have been made at several sites in the suburb of the Niigata city where liquefaction is known to have occurred during the 1964 earthquake (Ishihara, 1977; Ishihara, Silver and Kitagawa, 1978, 1979; Mori and Ishihara, 1979). The results of these studies disclosed that the cyclic stress ratio causing liquefaction in the specimen in 20 cycles ranged between 0.15 and 0.22 for the samples from the deposits which have shown apparent signs of liquefaction during the 1964 earthquake.

* Professor of Civil Engineering, University of Tokyo, Bunkyo-ku, Tokyo.

** Chief Research Engineer, Soil Dynamics Section, Public Works Research Institute, Ministry of Construction, Tsukuba, Ibaraki.

Written discussions on this paper should be submitted before July 1, 1982.

Another aspect of interest helpful in gaining in-depth understanding of the insitu liquefaction phenomena would be to compare the soil profiles and the cyclic strengths of intact sands between deposits at two sites; one where liquefaction is known to have occurred, and the other where liquefaction did not occur during past earthquakes. A study in this regard was performed by Ishihara, Silver and Kitagawa (1978) at two sites in the suburb of the Niigata city. However, the detailed investigation at these sites was not necessarily satisfactory, because these sites were located somewhat distant from the place where the acceleration records were obtained during the 1964 earthquake, and the findings of the investigations were judged not to be very precise enough to incorporate the recorded accelerations directly into the analytical procedure to obtain some insight into the mechanism of liquefaction occurrence in the field. Consequently, it was necessary to conduct a detailed soil investigation at another pair of sites in closer proximity to the place where the acceleration records were obtained.

An opportunity for performing such a study presented itself at two sites matching the above requirements in the center of the Niigata city. Undisturbed sand samples were obtained at these two sites and tested in the laboratory in the cyclic triaxial loading conditions. The test data were incorporated into the liquefaction analysis to compare the ground performances at these two sites observed during the 1964 earthquake. The results of these studies will be described in the following pages.

TEST SITES IN NIIGATA

For the purpose of comparing the cyclic resistance of soils and the soil profiles between two locations where clear manifestation of liquefaction was noted and was not during the 1964 earthquake, two sites were selected in the city of Niigata for detailed soil investigation as shown in Fig. 1. One site, called Kawagishi-cho site, located approximately 500 m from the north bank of the Shinano river was selected as a test site representative of the area where the ground damage due to liquefaction was most severe in the 1964 earthquake. Another test site, called the south bank site, was taken up for the present study as a representative location where practically no damage was incurred. These two sites located within 2 km of each other lay in the same general area of severe damage, but exhibited sharply contrasting ground performances.

Kawaguchi-cho Site

Photo. 1 shows an old map of the city of Niigata in 1849. The inlet portion on the right side of the picture is the place where the Kawagishi-cho is now located. Kawagishi-cho area had been a natural inlet close to the mouth of the Shinano river, thus providing an ideal site as a harbor for marine transportation. A recent literature survey has shown that this area ceased to function as a harbor when new ports were constructed nearer to the river mouth early in the 20th century, and during an approximately 10-year period between 1945 and 1955, the Kawagishi-cho area was reclaimed gradually to provide a new land mainly for use of public facilities. Methods of filling can not precisely be known, but it is believed that soils were carried to waterfronts by means of hand-pushing carts, railway cars or trucks and then dumped into the river without employing any artificial compaction technique. The westerly part of this reclaimed area as it existed in 1964 at the time of the earthquake is shown in detail on the map of Fig. 2. Following the earthquake, considerable amount of water emerged from the interior of the ground, leaving numerous sand vents and sand volcanoes all over the ground surface as indicated in Fig. 2. A number of surface crackings developed in loose fills along the waterfronts, causing lateral movement of the ground toward the unsupported river side. Many struc-

Fig. 1. Location of the test sites in the city of Niigata

Fig. 2. Exact location of the Kawagishi-cho test site and damage feature during the 1964 earthquake

Photo. 1. A map of the Niigata city in 1849

tures settled more than 50 cm, often accompanied by severe tilting. Undoubtedly the most spectacular case of settling and tilting was that experienced by the apartment buildings. This apartment house complex consisting of 8 buildings was constructed on the reclaimed land by the office of Niigata prefecture. Its location is shown in Fig. 2. The No. 4 building in this complex suffered the most severe damage, tilting through an angle of 80° , probably due

Photo. 2. Destruction to the apartment house complex in the Kawagishi-cho

to the fact that it had the poorest foundation work among the other neighboring buildings. The overall appearance of the destruction to this apartment complex is shown in the picture of Photo. 2. It is of interest to note that all the buildings tilted towards the side of the water tanks placed on the roofs. This fact indicates that the tilting occurred toward the center of gravity, and the building had been tilted sideways in the transverse cross section due to the weight of the water supply tank on the roof.

Fig. 3. Exact location of the south bank test site and damage feature during the 1964 earthquake

The exact location where the detailed soil investigation was made is indicated on the map of Fig.2 and also in the picture of Photo. 2. This test site was about 20 meters distant across a small road from the nearest apartment building that had suffered the damage.

South Bank Site

This test site is located close to the right bank of the Shinano river about 100 meters upstream from the Showa bridge as shown in Fig. 1. Although not clearly identified on the old map in Photo. 1, this site is believed to have been a shallow outcrop of a dense sand

deposit of diluvial age. Virtually no sign of ground damage was observed during the 1964 earthquake in this general area as indicated on the map of Fig. 3. The exact location of the present soil investigation is also indicated in Fig. 3.

INSITU SOIL INVESTIGATIONS

Kawagishi-cho Site

Sampling and sounding tests were performed side by side within 2.0 m from each other as shown in Fig. 4. A standard penetration test was first performed to provide a preliminary indication of the subsurface soil conditions. The result of the penetration test is presented in Fig. 5, where it may be seen that the Kawagishi-cho test site consisted of clean sand deposits to a depth of 20 m. The blow count values increase from about 5 at the depth of 5 m to about 10 at the depth of 14 m except for the depth of 11 m. It should be noted that the unusually small blow count values were encountered farther down around the depth of 19 m. It may be speculated that the unusually loose deposit exists at this depth due to the slow movement of the ground water along the channel of the Shinano river. Shown in Fig. 6 is the plot of the Dutch cone penetration resistance versus depth which was obtained in the bored hole 2.0 meters apart. Approximately the same trends in the distribution of the cone resistance are also observed in this plot as the resistance of the standard penetration test. It is to be noted that a sudden drop is also observed in the cone resistance around the depth of 19 m, in consistence with the similar

Fig. 4. Exact locations of the insitu tests at the Kawagishi-cho site

Fig. 5. Soil profile at the Kawagishi-cho site

Fig. 6. Penetration resistance by Dutch cone test at the Kawagishi-cho site

decrease in the standard penetration resistance as noted above. Sampling by means of a large diameter sampler was conducted at 9 depths below the water table down to a depth of 15 m as shown in Fig. 5. The sampling procedures were exactly the same as those employed in the previous investigation in Niigata (Ishihara and Silver, 1977; Ishihara et al., 1978). Sampling using a Osterberg type piston sampler was carried out side by side at 7 depths as shown in Fig. 5. The attempt was made wherever possible to obtain the Osterberg samples from the same depth as that of the large diameter sampling.

South Bank Site

Detailed locations of sampling and sounding tests at the south bank site are shown in Fig. 7. To establish a preliminary picture of the soil profile at this site the standard penetration test was first conducted. The soil profile and blow count values obtained are shown in Fig. 8, where it is seen that a soft surface deposit consisting of several thin layers of different materials exists to a depth of approximately 4 m. This surface deposit is deemed as the most recent alluvial or reclaimed fills laid over the old deposit of diluvial age. The diluvial sand deposit below the depth of 5.0 m is dense as represented by large values of the blow count on the order of 30 to 40. The resistance of the Dutch cone test is

shown in Fig. 9, where it may be seen that the resistance of the cone has the same pattern of distribution through the depth as that of the standard penetration resistance. The depths of sampling at the south bank site are indicated in Fig. 8 both for the samplings using the large diameter sampler and the Osterberg sampler. The sand deposit below the depth of 5.0 m was so dense that it was impossible to penetrate the sampling tubes farther down at this site. Therefore, no sample was obtained from deeper deposits. It was found that it is generally difficult to secure good-quality undisturbed sand samples

Fig. 7. Exact locations of the insitu tests at the south bank site

Fig. 8. Soil profile at the south bank site

Fig. 9. Penetration resistance by Dutch cone test at the south bank site

from stiff deposits with a blow count value exceeding $N=20$ by means of either the large diameter sampler or the Osterberg piston sampler. Their use appears to be limited to loose to medium dense sand deposits having a blow count value less than 20.

INDEX PROPERTY TESTS

Following the cyclic triaxial tests, the specimens were carefully removed from the triaxial cell and washed into a large container so that no particles were lost. Index property tests to determine density, grain size, specific gravity and limiting void ratios were performed on all test specimens following the procedures described in detail in a paper by Ishihara et al. (1978).

To determine the maximum and minimum limiting void ratios for each test specimen, reference values were set in advance for the standard Japanese sand (Toyoura sand) as being $e_{\max}=0.96$ and $e_{\min}=0.61$. These values are the averages among many test results on the same sand obtained by several investigators. Then, a minimum void ratio test method by vibration and a spooning maximum void ratio test method were developed and calibrated carefully so that they yielded the reference void ratio values as above for the Japanese standard sand. Using these calibrated techniques, all sand specimens from various depths were tested to determine the maximum and minimum void ratio values. Further, with this procedure, a United States standard sand (Monterey No. 0 sand) yielded void ratios of $e_{\max}=0.79$ and $e_{\min}=0.54$ which appear reasonable for this sand.

CYCLIC TRIAXIAL TESTS

Small specimens 12 cm in length and 6 cm in diameter obtained from the large diameter samples had been stored in the laboratory freezer. When performing the cyclic loading test, the frozen specimen was enclosed in a rubber membrane and put in place in the triaxial test chamber. A small vacuum of 20 kN/m^2 was applied to maintain the shape of the specimen and the initial dimension of the specimen were measured. The specimen was allowed to thaw for about 2 hours under vacuum with the aid of heat supplied by an electric heater. The triaxial cell was then assembled around the specimen, water was introduced into the triaxial chamber, and the vacuum was gradually reduced to zero while simultaneously increasing the chamber pressure to a value of 20 kN/m^2 .

The Osterberg sample about 80 cm long and 7.62 cm in diameter enclosed in the sampling tube had been stored in the laboratory freezer. When carrying out the laboratory cyclic triaxial test, the samples were cut into 15 cm length using an electric hacksaw. Then, the tubes were cut lengthwise by means of a band saw to split the tube to help remove the specimens. After taking out the frozen specimens, they were placed in the triaxial chamber. The procedures for measuring the dimensions of the specimens and thawing them under vacuum were the same as those employed in handling the specimens from the large diameter sample.

The subsequent laboratory procedures for having the specimens saturated and consolidated were the same as those adopted in the previous studies in testing the Niigata specimens. After the specimen had been saturated and consolidated, it was subjected, in undrained condition, to a sine wave cyclic axial load with a 1 Hz frequency. The cyclic load application was continued until the specimen deformed to a peak-to-peak axial strain of 10%. During the cyclic loading, the chamber pressure was kept constant and the axial load, axial deformation, and change in pore water pressure were recorded on chart paper. The specimens from any given depth were tested at least under three different values of peak cyclic axial load.

TRIAXIAL TEST RESULTS

A commonly used way of expressing soil strengths in cyclic loading conditions has been in terms of the cyclic stress ratio causing some types of failure within the soil specimen. The cyclic stress ratio is defined as the ratio between the amplitude of the cyclic axial stress, σ_{dl} , and twice the initial effective confining pressure, $2\sigma'_0$. The failure of the specimen may well be defined as the state of the initial liquefaction where the pore water pressure becomes equal to the initial effective confining pressure, or as the state of deformation where double-amplitude strains of 5% and 10% are produced in the test specimen. In this investigation, the failure criteria of the initial liquefaction, and 5% and 10% double amplitude axial strains will be used.

Kawagishi-cho Site

The results of the cyclic triaxial tests on the undisturbed specimens from the Kawagishi-cho site are summarized in Tables 1 and 2. Table 1 presents the test results on the specimens taken by means of the large diameter sampler and Table 2 refers to the test results on the Osterberg samples. The number of cycles which each specimen was able to withstand under given values of cyclic stress until the initial liquefaction, and 5% and 10% double-amplitude axial strains are tabulated in Tables 1 and 2. Also presented in these

Table 1. Index property values and cyclic triaxial test values, large diameter samples from Kawagishi-cho, Niigata

Sample No.	Depth (m)	Gradation		Specific gravity	Void ratio		Limiting void ratio		Relative density		$\frac{\sigma_{dl}}{2\sigma'_0}$	No. of cycles		
		D_{50} (mm)	U_c		e_c^*	e^{**}	e_{max}	e_{min}	D_{rc}^* (%)	D_{rc}^{**} (%)		init.	5%	10%
S-1	2.45	0.32	3.3	2.645	0.830	0.848	1.080	0.638	56.6	52.5	0.161	23	22	26
					0.828	0.844			57.0	53.4	0.136	130	127	132
					0.839	0.856			54.5	50.7	0.196	17	14	18
S-2	5.45	0.42	2.0	2.636	0.840	0.855	1.082	0.653	56.4	52.9	0.165	38	38	39
					0.832	0.850			58.3	54.1	0.187	11	10	12
					0.884	0.902			46.2	42.0	0.224	5	4	5
S-3	6.95	0.35	2.8	2.654	0.851	0.873	1.068	0.654	52.4	47.1	0.195	12	11	13
					0.851	0.867			52.4	48.6	0.175	14	13	15
					0.875	0.896			46.6	41.5	0.152	24	24	25
S-4	8.45	0.24	2.1	2.645	0.797	0.814	1.162	0.634	69.1	65.9	0.158	103	101	214
					0.856	0.878			57.9	53.8	0.198	18	17	26
S-5	9.45	0.35	2.1	2.640	0.836	0.850	1.071	0.656	56.6	53.3	0.182	34	37	40
					0.826	0.841			59.0	55.4	0.200	18	20	23
					0.906	0.921			39.8	36.1	0.225	5	5	6
S-6	11.45	0.26	2.0	2.669	0.870	0.893	1.091	0.657	50.9	45.6	0.177	12	11	13
					0.848	0.870			56.0	51.0	0.146	102	101	104
					0.853	0.878			53.7	49.1	0.164	43	42	45
					0.824	0.845			61.5	56.7	0.183	23	21	27
S-8	13.45	0.25	2.2	2.657	0.887	0.910	1.129	0.667	52.4	47.4	0.156	102	100	106
					0.895	0.923			50.6	44.6	0.176	21	15	19
					0.935	0.962			42.0	36.0	0.209	8	5	7
					0.821	0.843			66.7	62.0	0.177	38	33	41
S-9	14.45	0.28	1.9	2.651	0.811	0.831	1.138	0.705	75.5	71.0	0.160	201	200	213
					0.796	0.813			79.0	75.0	0.235	17	16	16
					0.765	0.783			86.1	82.0	0.260	19	13	22
					0.812	0.831			75.3	71.0	0.205	19	20	28

* Void ratio and relative density after consolidation

** Void ratio and relative density of undisturbed specimens before consolidation

Table 2. Index property values and cyclic triaxial test values, Osterberg samples from Kawagishi-cho, Niigata

Sample No.	Depth (m)	Gradation		Specific gravity	Void ratio		Limiting void ratio		Relative density		$\frac{\sigma_{dl}}{2\sigma'_v}$	No. of cycles		
		D_{50} (mm)	U_c		e_c	e	e_{max}	e_{min}	D_{rc} (%)	D_r (%)		init.	5%	10%
O-1	2.4	0.39	3.3	2.642	0.863	0.886	1.053	0.620	43.9	38.6	0.198	11	10	16
					0.807	0.820			56.8	53.8	0.235	6	6	9
					0.836	0.851			50.1	46.7	0.160	32	35	40
O-2	5.4	0.42	2.1	2.678	0.816	0.834	1.051	0.638	56.9	52.5	0.202	3	2	3
					0.823	0.841			55.2	50.8	0.199	20	18	—
					0.852	0.869			48.2	43.8	0.185	48	48	54
O-3	6.4	0.47	2.1	2.683	0.703	0.720	1.022	0.624	80.2	75.8	0.176	148	—	—
					0.803	0.818			55.0	51.3	0.240	6	8	21
					0.741	0.756			70.6	66.8	0.223	20	18	27
O-4	8.4	0.28	4.1	2.667	0.912	0.927	1.107	0.593	37.9	35.0	0.195	28	27	61
					0.780	0.795			63.6	60.7	0.225	14	20	40
					0.856	0.876			48.8	44.9	0.204	15	16	36
O-5	10.4	0.52	2.1	2.628	0.788	0.800	1.049	0.649	65.3	62.3	0.205	12	12	14
					0.931	0.947			29.5	25.5	0.254	4	4	6
					0.844	0.861			51.3	47.0	0.135	89	89	90
					0.888	0.908			40.3	35.3	0.172	8	8	9
O-6	12.4	0.35	2.4	2.680	0.878	0.895	1.080	0.652	47.2	43.2	0.232	8	8	11
					0.864	0.873			50.5	48.4	0.208	30	29	31
					0.859	0.873			51.6	48.4	0.176	89	89	91
O-7	14.4	0.28	2.4	2.675	0.844	0.859	1.137	0.670	62.7	59.5	0.198	22	21	26
					0.903	0.919			50.1	46.7	0.162	46	44	47
					0.890	0.905			52.9	49.7	0.250	6	6	11

Tables are the values of the void ratio and relative density of each test specimen before and after consolidation. The mean particle diameter and the uniformity coefficient of the specimens from each depth is also shown in Tables 1 and 2.

The cyclic triaxial test results listed in Tables 1 and 2 are plotted in Figs. 10 and 11. Fig.10 shows the test results on the specimens from the large diameter sampler, and the results of tests on the Osterberg samples are plotted in Fig.11. These figures show the cyclic stress ratio versus the number of cycles to cause the initial liquefaction, 5% and 10% double-amplitude axial strain in the specimens taken from each depth. For example, Fig.10(a) shows the cyclic strength of soils from a depth of 2.45 m where three small specimens were obtained from the large diameter sample. The relative density of these specimens ranged between 55% and 57%; therefore, the average relative density for three specimens was 56% as shown on the figure. It is to be noted that these relative density values are for the specimens at the time of consolidation prior to the cyclic loading test, and thus represent the cyclic strength of specimens having a slightly higher density than that encountered in insitu soils. It may be seen in Fig.10(a) that the cyclic strength curve is rather flat and that at 20 cycles the cyclic stress ratio causing initial liquefaction or 5% double-amplitude strain was on the order of 0.15. Figs.10(b) through 10(h) show similar results for specimens at successively deeper depths. It may be seen from these figures that in general the cyclic stress ratio causing failure in 20 cycles was on the order of 0.15 to 0.19, except at a depth of 14.45 m where a significantly higher average relative density of 79% was measured. Fig.11(a) showing the cyclic strength curve for the Osterberg specimen indicates that a similar relationship also exists for the Osterberg specimen between the cyclic stress ratio and the number of cycles. It may be seen from Fig.11(a) that the cyclic stress ratio causing initial liquefaction or 5% double-amplitude strain was

Fig. 10. Cyclic stress ratio versus number of cycles to initial liquefaction, and 5% and 10% double amplitude strain for the large diameter samples (Kawagishi-cho site)

Fig. 11. Cyclic stress ratio versus number of cycles to initial liquefaction, and 5% and 10% double amplitude strain for the Osterberg samples (Kawagishi-cho site)

on the order of 0.18, as compared to a value of 0.15 of the large diameter sample from the comparative depth of 2.5 m. Figs. 11(b) to 11(g) show the similarly arranged cyclic strength data at successively deeper deposits. It may be seen in these figures that the cyclic stress ratio causing initial liquefaction or 5% double amplitude strain in 20 cycles ranged between 0.15 and 0.20.

South Bank Site

The results of the cyclic triaxial tests on the large diameter samples and on the Osterberg samples taken from the south bank site are presented in Tables 3 and 4, respectively. In this test site, the depth of sampling was limited to depths of 4.45 m and 4.9 m. Therefore, tests on the large diameter sample were performed only on three specimens from a depth of 4.45 m. The Osterberg samples were obtained from five bored holes close to each other as shown in Fig. 7, but only three of them were tested in the cyclic triaxial apparatus. The grain size characteristics of the soils at the south bank were shown not to be appreciably different from that of the sand at the Kawagishi-cho site.

The cyclic strength data presented in Tables 3 and 4 are plotted in Figs. 12 and 13 in terms of the cyclic stress ratio versus the number of cycles. It may be seen in Fig. 12 that the cyclic stress ratio producing initial liquefaction or 5% double-amplitude strain in 20 cycles was on the order of 0.21 for the large diameter sample from a depth of 4.45 m with an average relative density of 70%. In contrast to this, much larger values of the cyclic stress ratio on the order of 0.30 were obtained for the Osterberg samples from a depth of 4.9 m having an average relative density of 85%. The large difference in cyclic strength between the two sands from depths of 4.45 m and 4.9 m may be accounted for by the sharp change in the blow count values around this depth as indicated in Fig. 8.

Table 3. Index property values and cyclic triaxial test values, large diameter samples from south bank of Showa bridge, Niigata

Sample No.	Depth (m)	Gradation		Specific gravity	Void ratio		Limiting void ratio		Relative density		$\frac{\sigma_{dl}}{2\sigma_0'}$	No. of cycles		
		D_{50} (mm)	U_c		e_c	e	e_{max}	e_{min}	D_{rc} (%)	D_r (%)		init.	5%	10%
S-1	4.45	0.28	2.4	2.693	0.785	0.799	1.098	0.653	70.3	67.2	0.203	20	23	45
					0.787	0.800			70.0	65.5	0.244	10	10	29
					0.782	0.794			69.5	68.3	0.165	114	120	170

Table 4. Index property values and cyclic triaxial test values, Osterberg samples from south bank of Showa bridge, Niigata

Sample No.	Depth (m)	Gradation		Specific gravity	Void ratio		Limiting void ratio		Relative density		$\frac{\sigma_{dl}}{2\sigma_0'}$	No. of cycles		
		D_{50} (mm)	U_c		e_c	e	e_{max}	e_{min}	D_{rc} (%)	D_r (%)		init.	5%	10%
O-1	4.9	0.29	1.5	2.692	0.753	0.768	1.070	0.633	72.5	69.1	0.373	12	17	—
					0.679	0.702			89.5	84.2	0.262	31	44	—
					0.694	0.706			86.0	83.3	0.239	132	152	—
					0.697	0.706			85.4	83.3	0.288	28	48	—
O-2	4.9	0.29	1.4	2.691	0.711	0.723	1.081	0.628	81.7	79.0	0.218	33	37	194
					0.663	0.673			92.3	89.8	0.358	12	38	—
					0.710	0.721			81.9	79.5	0.287	23	37	—
O-3	4.9	0.29	1.4	2.681	0.733	0.745	1.086	0.661	83.0	80.3	0.192	600	597	—
					0.711	0.725			88.2	85.0	0.250	21	32	—
					0.699	0.709			91.1	88.7	0.280	14	28	—

Fig. 12. Cyclic stress ratio versus number of cycles to initial liquefaction, and 5% and 10% double amplitude strain for the large diameter samples (south bank site)

Fig. 13. Cyclic stress ratio versus number of cycles to initial liquefaction and 5% double amplitude strain for the Osterberg samples (south bank site)

COMPARISON OF THE TEST RESULTS BETWEEN THE LARGE DIAMETER SAMPLE AND THE OSTERBERG SAMPLE

Since the samples were obtained side by side at the same place both by means of the large diameter sampler and the Osterberg piston sampler, it is of interest to compare the results of the tests on the specimens obtained by these two sampling procedures. Fig. 14 shows the void ratio values of each test specimen from two kinds of samples at the Kawagishi-cho site plotted versus depth. It may be seen in the figure that the void ratio values range between 0.72 and 0.97. Although individual data points scatter, average void ratio values at each depth appear to agree well between the specimens from the two samplers except at depths of 6.5 m and 14.5 m. Values of relative density for each test specimen from the two samplers are plotted versus depth in Fig. 15. The relative density values through the depth of sampling scatter within a wide range between 25% and 82%, but most of the test data fall in a relatively narrow range between 40% and 60%. In consistence with the distribution of the void ratio, the relative densities of the test specimens from the two different samplers appear to agree satisfactorily except at depths of 6.5 m and 14.5 m.

Fig. 14. Comparison of void ratio between the large diameter sample and the Osterberg sample

Fig. 15. Comparison of relative density between the large diameter sample and the Osterberg sample

Fig. 16. Cyclic stress ratio causing 5% double amplitude strain in 20 cycles plotted as a function of depth (Kawagishi-cho site)

Fig. 17. Acceleration records obtained at the basement of the Kawagishi-cho apartment building No.2 at the time of the 1964 earthquake

The cyclic stress ratio causing initial liquefaction or 5% double-amplitude strain in 20 cycles of load application was read off from the cyclic strength curves shown in Figs.10 and 11. The cyclic stress ratio thus obtained for two kinds of samples at the Kawagishi-cho site is shown in Fig.16. It may be seen from this figure that the cyclic stress ratios for soils from both samplers were nearly the same although the test specimens from the Osterberg sampler tended to exhibit a slightly higher resistance than the test specimens from the large diameter sampler. The test specimen at the depth of 7.0 m from the large diameter sample showed a smaller resistance as compared to that from the Osterberg sample. This

appears due to the higher density of the Osterberg specimen as seen in Fig.15. The test specimens at the depth of 14.5 m from the Osterberg sampler showed a smaller cyclic stress ratio, but this is in consistence with the smaller relative density of the Osterberg test specimens measured at this depth as shown in Fig.15.

COMPARISON OF LIQUEFACTION ANALYSES BETWEEN TWO TEST SITES

At the time of the 1964 Niigata earthquake, the ground at the Kawagishi-cho site was heavily destroyed due to the development of liquefaction in the sand deposit, whereas no visible sign of liquefaction was observed over the ground in the vicinity of the south bank site. Therefore, it would be of interest to make some simple analysis of liquefaction for the deposits at these two test sites and to compare the results of the analysis in the light of the damage feature observed at each site during the 1964 event.

The analysis of liquefaction during earthquakes requires a knowledge about the intensity of shaking in terms of the time history of acceleration on the ground surface at a given site in question. During the 1964 earthquake, acceleration records were obtained at the basement of the 4 story reinforced concrete apartment building at Kawagishi-cho. This building was the No.2 unit in the apartment complex consisting of 8 similar units, and its location is shown in Photo. 2. As seen in the photo, the No.2 building was located about 50 meters away from the test site in this investigation. The time histories of the accelerations obtained are shown in Fig.17.

The simple analytical procedure developed by Ishihara (1977) is used to make a crude analysis. In this method, the maximum stress ratio, $\tau_{\max}/\sigma_v'$, at each depth that is induced by earthquake loading is computed by the following formula.

$$\frac{\tau_{\max}}{\sigma_v'} = \frac{a_{\max}}{g} \cdot r_d \cdot \sigma_v / \sigma_v' \quad (1)$$

$$r_d = 1 - 0.015Z \quad (2)$$

where σ_v' is the vertical effective stress and σ_v is the vertical total stress at the depth in question. Z represents the depth in meter. The empirical formula expressed by Eq. (2) was proposed by Iwasaki et al. (1978). In the above formula, $a_{\max}$ is the maximum horizontal ground acceleration and g is the gravitational acceleration.

The cyclic strength to be compared against the induced maximum stress ratio is determined by the following formula, based on the cyclic triaxial test data which are expressed in terms of the cyclic stress ratio, $\sigma_{dl}/(2\sigma_o')$, required to cause initial liquefaction or 5% double amplitude strain in 20 cycles,

$$\frac{\tau_{\max,l}}{\sigma_v'} = \frac{0.9}{0.55} \cdot \frac{1+2K_0}{3} \left(\frac{\sigma_{dl}}{2\sigma_o'} \right)_{N=20} \quad (3)$$

where $\tau_{\max,l}/\sigma_v'$ indicates the maximum stress ratio producing failure in the soil as represented by the development of 5% double amplitude strain under uniform loading condition. K_0 is the coefficient of earth pressure at rest. The value, 0.55, on the right side of Eq. (3) represents a factor to take into account the irregular nature of time history changes of shear stress during an earthquake. This value was determined by actually applying the irregular torsional shear stresses having the same time histories as the Niigata records shown in Fig.17 on the hollow cylindrical sand specimens in the laboratory torsional shear test apparatus (Ishihara and Yasuda, 1975). A factor, 0.9, was incorporated on the right side of Eq. (3) to allow for the effect of changes in the direction of shear stress application in the horizontal plane of the deposit (Ishihara and Yamazaki, 1980). The maximum stress ratio given by Eq. (1) was compared against the corresponding maximum stress ratio required to cause failure in soils as given by Eq. (3). The

result of the comparison was expressed in terms of factor of safety against failure F_l , which is defined as

$$F_l = \frac{\tau_{\max, l} / \sigma_v'}{\tau_{\max} / \sigma_v'} \quad (4)$$

The simple analysis procedure as above was applied for both the deposits at the Kawagishi-cho site and the south bank site. While the Kawagishi-cho site was close to the place where the acceleration records were obtained, the south bank site was about 2 km distant from that place. In addition to this, the soil conditions at these two sites are different, and, therefore, the acceleration at the south bank site would have been different from that recorded at the Kawagishi-cho site. Yet, it was assumed that the deposit at the south bank site had been subjected to the same shaking as that recorded at the Kawagishi-cho apartment building during the 1964 earthquake. Thus, by incorporating into Eq. (1) the maximum ground accelerations of 159 gal and 155 gal as read off from Fig. 17, the factor of safety defined by Eq. (4) was calculated for both the deposits at the Kawagishi-cho and the south bank site on the basis of the cyclic triaxial test data shown in Figs. 12, 13 and 16. The results of the analysis are presented in Fig. 18 through Fig. 20.

The plots of the factor of safety versus depth shown in Fig. 18 indicate that the cyclic triaxial test data from the large diameter samples yield factors of safety below unity from the depths of 3 m to 13 m. Similar plots in Fig. 19 obtained from the Osterberg samples also indicate that the factor of safety drops below unity through the depth between 3 m and 12 m. Therefore, it may be concluded that the sand deposit at the Kawagishi-cho site must have developed liquefaction from the depth of about 3 m down to 13 m during the 1964 earthquake. This is also in agreement with the widespread ground surface manifestation of the liquefaction such as spouting of sand and water and the resulting inundation over the ground surface that were observed in this area as shown in Fig. 2 during the 1964 event.

The plot of the factor of safety for the south bank site shown in Fig. 20 indicates that

Fig. 18. Computed factor of safety versus depth at the Kawagishi-cho site (Large diameter sample)

Fig. 19. Computed factor of safety versus depth at the Kawagishi-cho site (Osterberg sample)

Fig. 20. Computed factor of safety versus depth at the south bank site

the liquefaction type failure occurred at a limited depth interval around 4.5 m depth. The near-surface layer above this depth consisted apparently of soils with some cohesion which are immune to liquefaction. The sand below the depth of 5 m was apparently dense with a blow count value of more than 30, indicating that a liquefaction type failure did not take place through the deposit of this layer. With these considerations in mind, it may well be speculated that the liquefaction type failure might have taken place in a thin layer around the depth of 4.5 m. However, it would appear that the occurrence of liquefaction, if any, must have extended only slightly through the depth and was never extensive enough to bring about any sign of liquefaction over the ground surface in this area as shown in Fig.3.

CONCLUSIONS

During the June 16, 1964 Niigata earthquake, acceleration records were obtained by means of a strong motion accelerogram (SMAC-B type) mounted at the basement of the Kawagishi-cho apartment building. Signs of extensive liquefaction such as ejection of sand and water were observed in the vicinity of the building during the earthquake all over the ground surface. With an aim towards clarifying the mechanism of liquefaction occurring at this site, detailed soil investigations were carried out near the Kawagishi-cho apartment lot, including the standard penetration test, Dutch cone penetration test and sampling of undisturbed sand samples by means of the large diameter sampler and Osterberg sampler. The undisturbed samples were tested in the laboratory using the cyclic triaxial test apparatus to determine the cyclic strength or liquefaction potential of insitu intact sands. The results of the site investigation showed that deposits of clean sands existed down to the depth of 20 m. From the surface layer to the depth of 5 m, the blow count value was on the order of 5, but it increased to a value of about 10 in the subsequent sand layer to the depth of 10 m. In the underlying layer to a depth of 20 m, the blow count value ranged between 15 and 30 except at a depth of 19 m where a surprisingly loose layer was encountered having a blow count value of 2. The results of the cyclic triaxial tests showed that the cyclic stress ratio producing initial liquefaction in 20 cycles ranged between 0.15 and 0.20. These cyclic strength values were incorporated into the simple analysis procedure together with the maximum acceleration values of 0.162 g and 0.158 g obtained during the 1964 earthquake. The result of the liquefaction analysis showed that the cyclic strength of the insitu sands was smaller than the corresponding cyclic shear stress that must have been induced in the deposit at the Kawagishi-cho site during the 1964 event.

It was also considered of interest to study soil conditions at a site where no evidence of liquefaction was observed on the ground surface during the 1964 earthquake. Such a site was found near the right bank of the Shinano river in the city of Niigata about 3 km distant from the Kawagishi-cho site. The detailed soil investigations at this site revealed that there existed some layers of cohesive soils to a depth of 4 m followed underneath by a 1 m thick loose sand layer. The sand deposit of diluvial origin underlying below the depth of 5 m was found to be medium dense to dense with an average blow count value of 35. The analysis of liquefaction showed that the cyclic shear stress induced in the ground was smaller than the corresponding cyclic shear strength of the insitu sands throughout the depth of the deposit except in a thin loose sand layer at the depth of 4.0 to 5.0 m. This analysis result is in accord with the non-occurrence of any ground surface devastation during the 1964 event.

ACKNOWLEDGEMENTS

This investigation was partially supported by the grant-in-aid from the Ministry of Education. The field sampling was supervised by Mr. M. Nakajima of the Token Geotechnique Co.. The laboratory cyclic triaxial tests were performed by Mr. M. Kuwabara of the Fudo Construction Co.. The original draft of this paper was reviewed by Dr. K. Mori of the University of Tokyo. The authors wish to extend their gratitude to these persons and agencies.

REFERENCES

- 1) Ishihara, K. and Yasuda, S. (1975): "Sand liquefaction in hollow cylinder torsion under irregular excitation," *Soils and Foundations*, Vol.15, No.1, pp.45-59.
- 2) Ishihara, K. (1977): "Simple method of analysis for liquefaction of sand deposits during earthquakes," *Soils and Foundations*, Vol.17, No.3, pp.1-17.
- 3) Ishihara, K. and Silver, M.L. (1977): "Large diameter sand sampling to provide specimens for liquefaction testing," 9th International Conference on Soil Mechanics and Foundation Engineering, Proceedings of Specialty Session 2, Tokyo, pp.1-6.
- 4) Ishihara, K., Silver, M.L. and Kitagawa, H. (1978): "Cyclic strengths of undisturbed sands obtained by large diameter sampling," *Soils and Foundations*, Vol.18, No.4, pp.61-76.
- 5) Ishihara, K. and Yamazaki, F. (1980): "Cyclic simple shear tests on saturated sand in multi-directional loading," *Soils and Foundations*, Vol.20, No.1, pp.45-59.
- 6) Iwasaki, T., Tatsuoka, F., Tokida, K. and Yasuda, S. (1978): "A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan," Proc. 2nd International Conference on Microzonation for Safer Construction-Research and Application. Vol.II, pp.885-896.
- 7) Mori, K. and Ishihara, K. (1979): "Undisturbed block sampling of Niigata sand," Proc., 6th Asian Regional Conference of Soil Mechanics and Foundation Engineering, Singapore, Vol.1, pp.39-42.
- 8) Seed, H.B. and Idriss, I.M. (1967): "Analysis of soil liquefaction : Niigata earthquake," Proc., ASCE SM 3, pp.83-108.

(Received August 18, 1980)