

SOIL LIQUEFACTION STUDIES IN JAPAN STATE-OF-THE-ART

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**Earthquake Disaster Prevention Department
Public Works Research Institute
Ministry of Construction**

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**SOIL LIQUEFACTION STUDIES IN JAPAN
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by

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Synopsis:

This paper describes the state-of-the-art of studies on earthquake-induced soil liquefaction, with emphasis on experimental results and practical applications through investigations conducted at the Public Works Research Institute in Japan. Following a brief introductory review on previous studies of soil liquefaction, a case history of soil liquefaction during a number of past Japanese earthquakes is summarized.

Practical procedures for assessing occurrence of soil liquefaction of results of shaking table tests on the effects of soil liquefaction on pile foundations and earth embankments and provisions of soil liquefaction specified in current seismic design codes for various structures are described.

**Key Words: EARTHQUAKE, LIQUEFACTION, SANDY SOILS,
SOIL LIQUEFACTION**

SYNOPSIS

This paper describes the state-of-the-art of studies on earthquake-induced soil liquefaction, with emphasis on experimental results and practical applications obtained through investigations at the Public Works Research Institute in Japan. Following a brief introductory review on previous studies of soil liquefaction, a case history of soil liquefaction during a number of past Japanese earthquakes is summarized.

A practical procedure for assessing occurrence of soil liquefaction is proposed, and a procedure for accounting for degree of soil liquefaction is discussed. The procedures taking into account the effects of soil liquefaction can be employed in structural designing and microzoning. Some examples of the applications are shown.

Then, results of shaking table tests on the effects of soil liquefaction on pile foundations and earth embankments are described. Furthermore, several examples of countermeasures for reducing the soil liquefaction effects are introduced.

Finally, provisions of soil liquefaction specified in current seismic design codes for various structures are summarized. In the conclusions of this paper, future research subjects on soil liquefaction to be studied are pointed out.

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1. BRIEF REVIEW OF LIQUEFACTION STUDIES

1.1 Studies Prior to the Niigata Earthquake of 1964

The Niigata Earthquake of 1964 with a Richter magnitude of 7.5 brought about extensive damage to various engineering structures such as buildings, bridges, highways, railways, harbors, airports, river dikes, and lifeline facilities. Many of reinforced concrete buildings tilted considerably, and several spans of highway bridges fell down into river beds. It was disclosed that most of those damages were essentially caused by the effects of liquefaction of loose saturated sands distributed along Shinano River and Agano River in Niigata City.

Earthquake-induced liquefaction of saturated sand can be explained as the following process, by Iwamoto (1983):¹

1. Earthquake ground motion causes repeated shear stresses in saturated sands,
2. The shear stresses cause a tendency of negative dilatancy (or compression), yet the undrained condition does not permit compression,
3. Pore-water pressures generate and accumulate in the sands,
4. Effective confining pressures decrease,
5. Contact between sand particles loosen, and accordingly shear strength decreases, and
6. Sands become a liquefied state without any shear strength.

Prior to the epoch-making outbreak of the Niigata Earthquake, some preliminary studies relating to soil liquefaction had been performed in Japan. Mogami and Kubo (1953)² confirmed the existence of soil liquefaction phenomena by conducting laboratory tests. They measured shear strength of dry sands and unsaturated loams packed in a simple shear box capable of vibrating vertically up to a large acceleration of several times the gravity acceleration, and found that the shear strength decreases as vertical acceleration increases. They also found that the void ratio gradually decreases and approaches to a certain critical value as the acceleration gets great. Mogami and Kubo recognized a phenomenon of decrease in shear strength of sand specimens when subjected to a large vertical acceleration. They called this phenomenon "liquefaction" for the first time. The phenomenon observed by Mogami and Kubo,

however, is slightly different from liquefaction of saturated sands caused by earthquake motions.

Ishii, Tsuchida and Furube (1963)³ conducted laboratory experiments using a shaking table on which a sand box was mounted, and measured dynamic earth pressures and hydrodynamic pressures of saturated sands which are acting on the wall of the sand box during the horizontal shaking of the table. They found from the experiment that saturated sands in the box became liquid state during a large horizontal shaking, pore-water pressures go up, and void ratios extensively decrease during the course of the liquefaction. Although they did not disclose the mechanism of sand liquefaction during a horizontal shaking, they pointed out that outbreak of liquefaction of saturated sands in the box can be predicted from a relationship between the initial void ratio and the magnitude of acceleration of the shaking table.

Ohara (1963)⁴ also performed similar shaking table tests on liquefaction behavior of saturated sands. He measured considerable rises of pore-water pressures during shaking time. He also indicated that the excessive pore-water pressure is approximately equal to the product of the apparent density of the sand and the depth. The product can be expressed as the effective overburden pressure.

Although several investigators measured dynamic strength of dry sands and unsaturated soils with use of soil testing apparatus such as simple shear apparatus and dynamic tri-axial compression apparatus, it seems that no attempts were made before 1964 for directly measuring dynamic strength of saturated sands using laboratory apparatus.

1.2 Studies Following the Niigata Earthquake of 1964

After experiencing serious structural damage due to soil liquefaction during the Niigata Earthquake, extensive studies have been made on procedures for assessing liquefaction potential by a number of investigators. Typical procedures of liquefaction assessment could be classified into the following four categories:

1. Procedures based on in situ soil data such as N-values of the Standard Penetration Test (SPT),
2. Procedures based on comparison between dynamic shear strength (or liquefaction strength) of sand and earthquake-induced shear stress in sand,
3. Procedures based on excessive pore-water pressures in sands, and

4. Procedures based on case histories of liquefaction occurrence.

The following will give brief explanations of representative procedures for each of the above categories.

Procedures Based on In Situ Soil Data

Kishida (1966),⁵ Koizumi (1966)⁶, and Ohsaki (1966)⁷ proposed distribution curves of critical SPT N-values for assessing sand liquefaction potential, based on in situ soil data (SPT N-values) collected from building sites situated in both liquefied areas and non-liquefied areas in Niigata City. Fig. 1 illustrates distribution curves of critical N-values to cause soil liquefaction with respect to depth, proposed by the above three investigators. It should be noted that their proposals of critical N-values firstly provided with quantitative criteria for the assessment of liquefaction potential in designing building structures. The figure also shows several other proposals made thereafter. It is seen that critical N-values are considerably different among the various proposals.

Some investigators also discussed particle characteristics of sands which are vulnerable to liquefaction during earthquakes. As one of the examples, Fig. 2 indicates a relation between grain-size characteristics of soils and liquefaction potential proposed by Tsuchida (1971).⁸ He also provided a relationship between peak ground acceleration and critical N-values of the ground to cause liquefaction, which is shown in Fig. 3. Tsuchida's proposals are introduced in the design criteria of harbor structures, which will be described in 6.4 of this paper.

Procedures Based on Comparison Between Dynamic Shear Strength and Earthquake-Induced Shear Stress

Seed and Idriss (1967),⁹ (1971)¹⁰ first proposed the well-known practical method to assess liquefaction potential in which dynamic shear strength of sand is compared with earthquake-induced shear stress. Fig. 4 indicates a fundamental process for the assessment of liquefaction potential. In view of the results of dynamic tri-axial tests, they proposed a method in which dynamic shear strength of sand can be determined in terms of relative density and mean particle of the sand. Later, Seed (1979)¹¹ proposed a more simplified procedure using SPT N-values for estimating in situ dynamic strength of sands and liquefaction potential.

With reference to the proposals made by Seed and his colleagues, Iwasaki, Tatsuoka, Tokida, and Yasuda (1978)¹² proposed another practical simplified procedure in which the ratio of dynamic shear strength to earthquake-induced

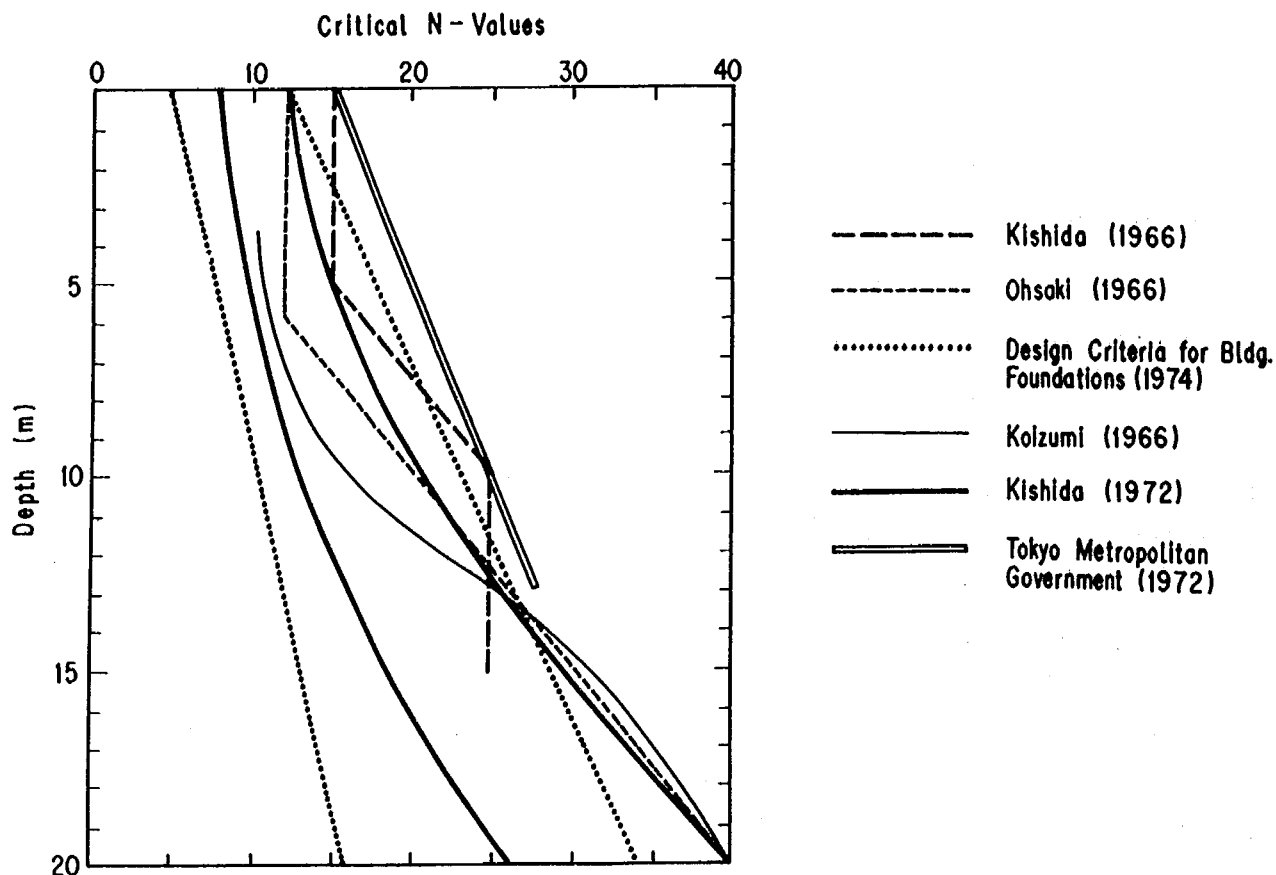


Fig. 1 Depth Versus Critical N-Values Cause Liquefaction

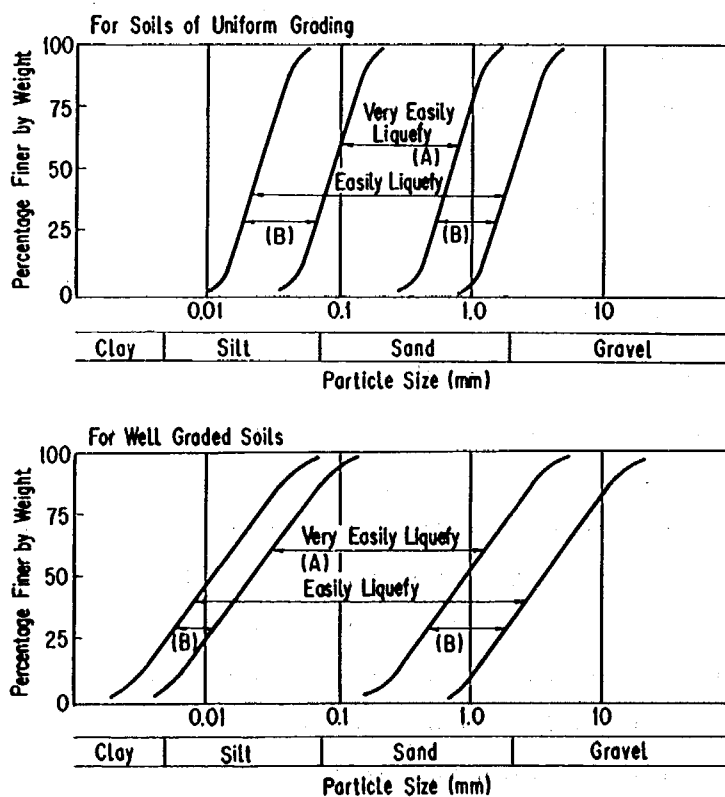


Fig. 2 Relation between Grain-Size (Characteristics of Soils) and Liquefaction Potential (After Tsuchida⁸)

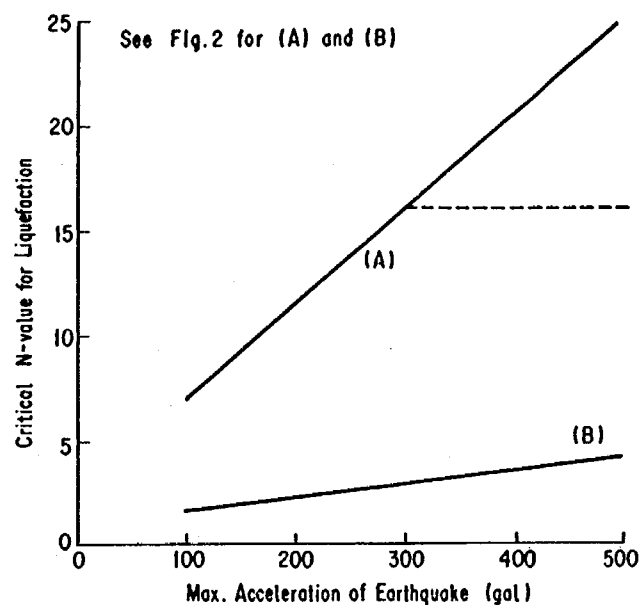


Fig. 3 Critical N-Values Causing Liquefaction in Terms of Maximum Earthquake Accelerations (After Tsuchida⁸)

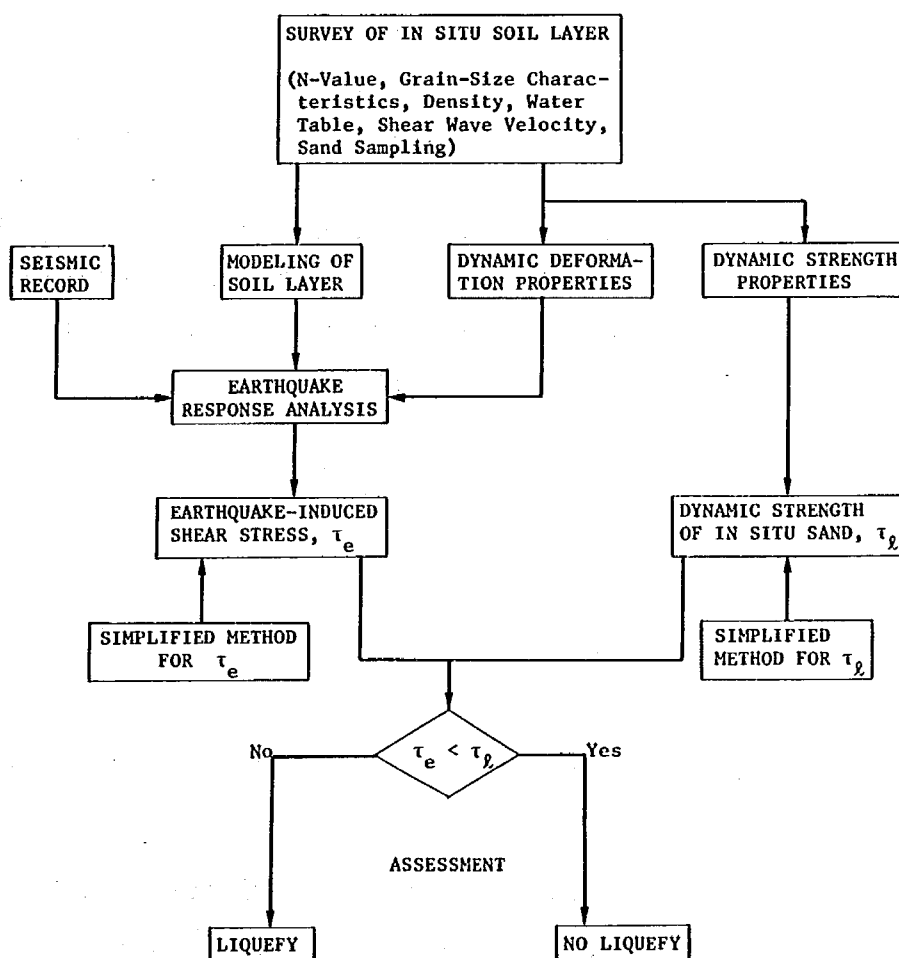


Fig. 4 Fundamental Flow for Soil Liquefaction Assessment (After Seed and Idriss¹⁰)

shear stress (defined as the liquefaction resistance factor $F_L = R/L$, R = shear strength and L = shear stress) is introduced as a key factor for the assessment of liquefaction potential. The details will be discussed in 3.2 of this paper.

Yoshimi and Tokimatsu (1980)¹³ examined the two procedures proposed by Seed (1979)¹¹ and Iwasaki and Tatsuoka (1978)¹², in view of data of various soils including medium to coarse sands, fine sands and silty sands. The result is shown in Fig. 5, in which $N_1 = 2N(\sigma'_v + 1)$, and R = cyclic stress ratio causing liquefaction = τ/σ'_v . In Fig. 5 the dotted line is 0.65 times the shear strength ratio obtained by Iwasaki-Tatsuoka procedure, which gives the maximum shear strength ratio, shown in 3.2 of this paper. They pointed out that Seed's procedure may underestimate the shear strength for the case of sands containing fine particles and having low N -values, and that Iwasaki-Tatsuoka procedure may underestimate the shear strength for sands with high N -values. Shear strength R (= cyclic stress causing liquefaction) for three types of sand (medium to coarse sand, fine sand, and silty sand) proposed by Yoshimi and Tokimatsu (1980) are illustrated by three solid lines in Fig. 5.

Shibata (1981)¹⁴ collected in situ data of SPT N -values of both liquefied sites and non-liquefied sites which sustained strong ground motions during the past earthquakes. By analyzing the collected data, he obtained empirical relations between SPT N -values and cyclic stress ratios ($R = \tau_L/\sigma'_v$) causing liquefaction, as shown in Fig. 6(a) and Fig. 6(b). In the figures, black and white circles indicate data points for liquefied and non-liquefied sites, respectively. The border lines can be expressed as

$$R = \frac{\tau_L}{\sigma'_v} = \frac{1}{80} \frac{N}{\sigma'_v} \quad , \quad \text{for } \sigma'_v \leq 0.75 \text{ kg} \cdot \text{f}/\text{cm}^2 \quad (1-a)$$

$$R = \frac{\tau_L}{\sigma'_v} = \frac{1}{60} N \quad , \quad \text{for } \sigma'_v > 0.75 \text{ kg} \cdot \text{f}/\text{cm}^2 \quad (1-b)$$

Those equations can be used for approximately estimating the maximum cyclic stress ratios causing liquefaction.

Procedures Based on Excessive Pore-Water Pressures in Sands

As described in 1.1, sand liquefaction can be explained to be caused by the effects of the rise of excessive pore-water pressures in sands. A number of investigations have been made regarding procedures for estimating the rise and fall of excessive pore-water pressures in sands during earthquakes.

Recent research done by Yamanouchi and Hyodo(1975),¹⁵ Tokimatsu and Yoshimi (1978),¹⁶ Ishihara and Towhata(1980),¹⁷ etc., is well recognized in Japan. In this connection investigations by Martin, Finn, and Seed (1975),¹⁸ Sherif and Ishibashi (1978),¹⁹ Martin and Seed (1979)²⁰ are also essential. In these procedures, time history of excessive pore-water pressures can be evaluated, and accordingly extent of liquefaction can be predicted as functions of elapsed time of earthquake ground motions.

Procedures Based on Case Histories of Liquefaction Occurrence

In order to correlate actual liquefaction phenomena and seismic and site conditions, Kuribayashi, Tatsuoka, and Yoshida (1974)²¹ conducted an extensive literature survey on liquefaction phenomena brought by forty-four past earthquakes which hit Japan since 1872. From the comprehensive study, they obtained a relationship between the earthquake magnitude and the maximum epicentral distance of liquefied sites, as shown in Fig. 7. From the figure one can define an area of zone in which liquefaction may take place, when an earthquake magnitude is given. They also disclosed that sites liquefied during the past earthquakes have high potential to re-liquefy by succeeding earthquakes. The details will be discussed in 2.1 of this paper. Sato, Iwasaki, and Tokida (1980)²² attempted to make microzonation maps which indicate (a) areas with high potential causing liquefaction, (b) areas with potential causing liquefaction, and (c) areas with low potential causing liquefaction. Maps cover major plains from northern Honshu to Kyushu. The definitions of the classifications are shown in Table 1. The definitions are derived from geological characteristics of liquefied sites during the past Japanese earthquakes.

Kotoda and Wakamatsu (1980)²³ also made similar microzonation maps for selected regions in northern Honshu.

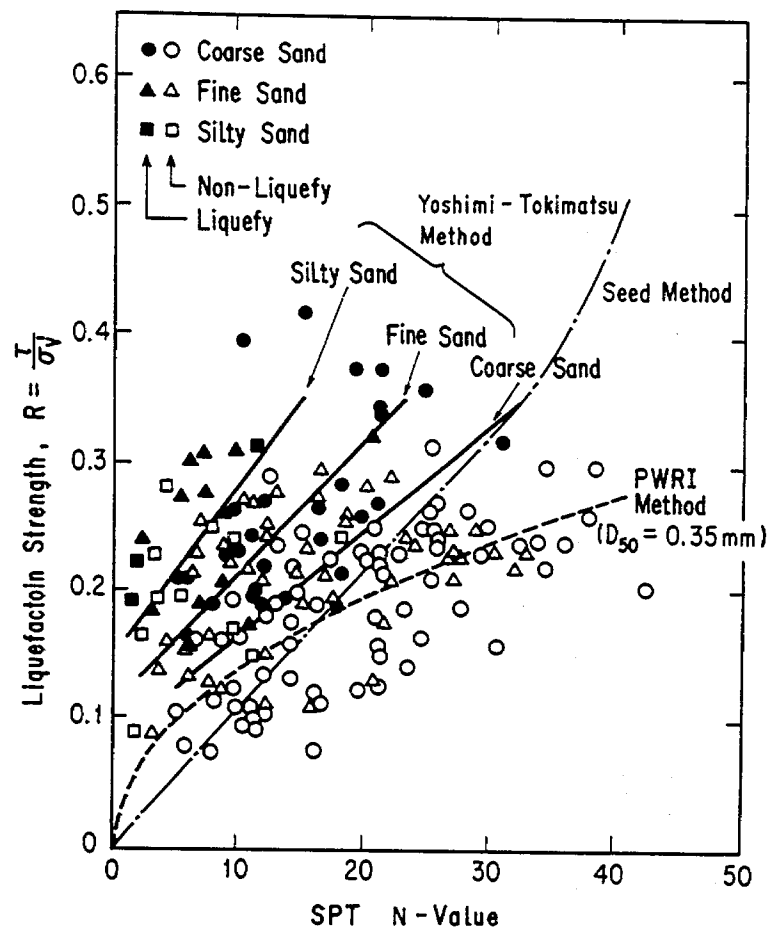


Fig. 5 Relations between N-Value and Liquefaction Strength of Sands (After Yoshimi and Tokimatsu¹³⁾)

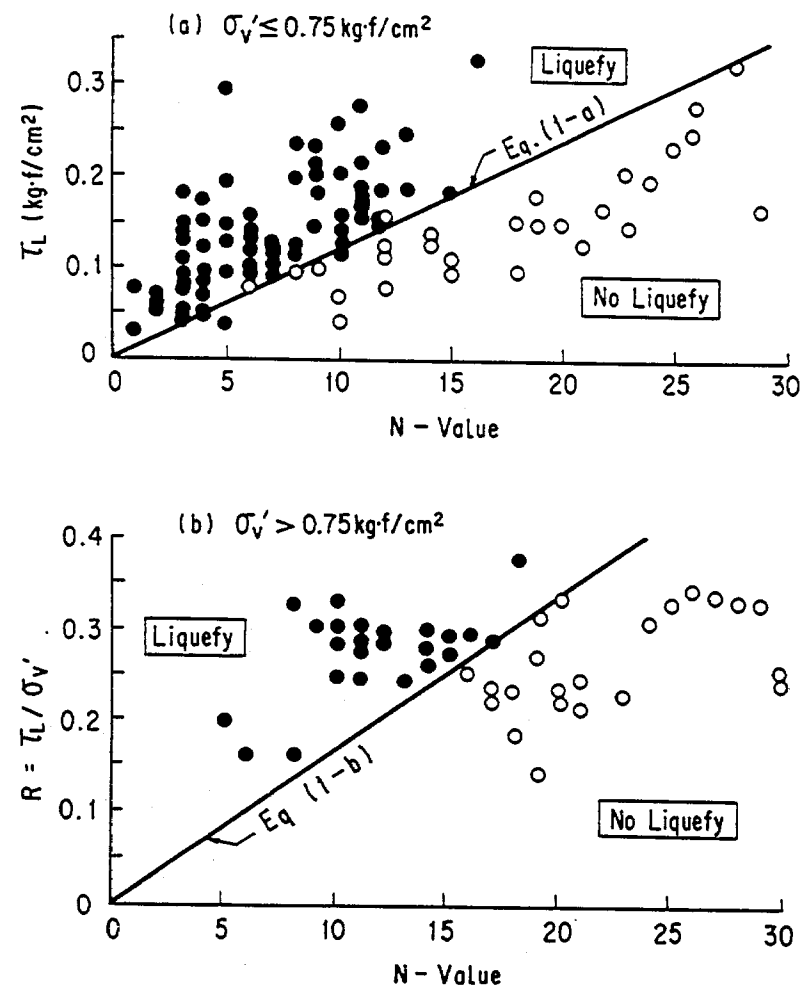


Fig. 6 Relations between N-Values and Liquefaction Strength and Strength Ratio (After Shibata¹⁴⁾)

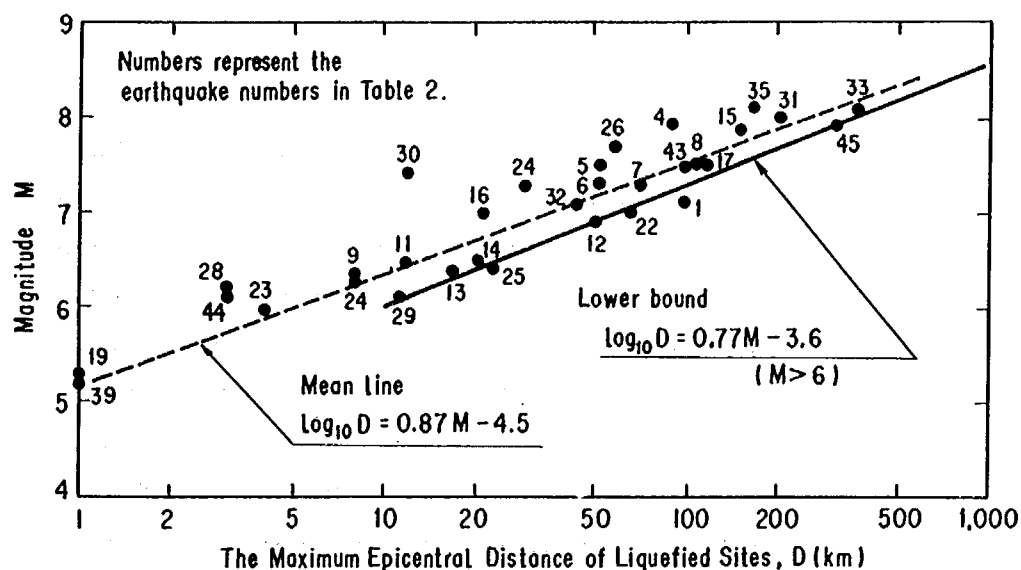


Fig. 7 Relationship between Maximum Epicentral Distance of Liquefied Sites (D) and Earthquake Magnitude (M) (After Kuribayashi, Tatsuoka, and Yoshida²¹)

Table 1 Classification of Areas in Terms of Liquefaction Vulnerability

CLASSIFICATION	DEFINITIONS
(A) Areas with high Potential to Cause Liquefaction	Older River Beds, Present River Beds, Low Lands between Sand Dikes (with High Water Tables), Filled Lands and Reclaimed Lands which formerly had Deep Water,
(B) Areas with Potential to Cause Liquefaction	Others than (A) or (C)
(c) Areas with Low Potential to Cause Liquefaction	Terraced Hill, Mountainous Land

2. HISTORY OF SOIL LIQUEFACTION DURING JAPANESE EARTHQUAKES

2.1 General

As briefly described in the previous chapter of this paper, Kuribayashi, Tatsuoka and Yoshida (1974)²¹ conducted an extensive literature survey concerning soil liquefaction phenomena brought by 44 past earthquakes in Japan since 1872. Table 2 shows a complete list of 50 earthquakes which induced soil liquefaction in these 112 years, including six recent earthquakes. The table indicates names of earthquakes in chronological order, dates of earthquake occurrences, Richter magnitudes determined by the Japan Meteorological Agency, focal depths, latitudes and longitudes of the epicenters, numbers of persons killed, and farthest distances of liquefied sites from the epicenters.

Evidences of liquefaction are pointed out from the facts, such as (1) water spoutings with sand from cracks on ground surfaces, (2) sand boils and sand volcanoes, (3) excessive settlements of heavy structures resting on sandy layers, and (4) floating up of old wooden piles (used as old bridge piers) out of rice paddy and floating up of caisson foundations from river beds.

Brief locations where liquefaction took place throughout Japan are illustrated in Fig. 8. As shown in this figure and Table 2, earthquakes having larger magnitudes induced soil liquefaction in wider areas with radii of several hundred kilometers.

Among these large earthquakes, the Nobi Earthquake of 1891, the Gono Earthquake of 1909, the Kanto Earthquake of 1923, the Nishi-Saitama Earthquake of 1931, the Fukui Earthquake of 1948, the Niigata Earthquake of 1964, and the Nihonkai-Chubu Earthquake of 1983 caused violent liquefaction in very wide areas. Furthermore, the Shonai Earthquake of 1894, the Rikuu Earthquake of 1896, the Tajima Earthquake of 1925, the Tonankai Earthquake of 1944, Tokachi-oki Earthquake of 1968, and the Miyagi-ken-oki Earthquake of 1978 brought about considerable spoutings of water and sands and made a number of large sand volcanoes.

Fig. 7 in 1.2 of this paper shows the relationship between the maximum epicentral distance of liquefaction areas (D in km) and the Richter magnitude of earthquake (M). It is found from this figure that the average line of the whole data points is expressed by

$$D = 10^{0.87M-4.5} \quad (2-a)$$

Table 2 List of Earthquakes Causing Soil Liquefaction (1872-1983)

No.	Earthquake	Date	Magnitude (M)	Focal depth (km)	Epicenter	Persons killed	Farthest Distance Liquefied (D) (km)
1.	Hamada	1872.3.14	7.1		34.8°N, 132.0°E	552	97
2.	Koshigun	1887.7.22	6.1		37.7°N, 139.0°E	0	
3.	Kumamoto	1889.7.28	6.3		32.8°N, 130.7°E	20	
4.	Nobi (Mino-Owari)	1891.10.28	8.4 (7.9)		35.6°N, 136.6°E	7,273	89
5.	Tokyo	1894.6.20	7.5		35.7°N, 139.9°E	31	52
6.	Shonai	1894.10.22	7.3		39.2°N, 139.5°E	726	52
7.	Tone-karyu	1895.1.18	7.3		35.9°N, 140.4°E	9	71
8.	Rikuu	1896.8.31	7.5		39.5°N, 140.7°E	209	108
9.	Kamitakai	1897.1.17, 4.30	6.3, 6.3		36.6°N, 138.2°E	0	8
10.	Minami Uonumagun	1898.5.26	6.7		36.9°N, 138.9°E	0	
11.	Fukuoka	1898.8.10, 8.12	6.5, 6.5		33.5°N, 130.2°E	0	12
12.	Gono (Anegawa)	1909.8.14	6.9		35.4°N, 136.3°E	41	51
13.	Ugosen	1914.3.15	6.4		39.5°N, 140.4°E	94	17
14.	Shimabara	1922.12.8	6.5, 5.9		32.7°N, 130.1°E	30	21
15.	Kanto	1923.9.1	7.9		35.2°N, 139.3°E	99,331	150
16.	Tajima	1925.5.23	7.0		35.7°N, 134.8°E	428	22
17.	Kitatango	1927.3.7	7.5	10	35.6°N, 135.1°E	2,925	117
18.	Ishinomaki	1927.8.6					
19.	Sekihara	1927.10.27	5.3	0-10	37.5°N, 138.8°E	0	1
20.	Kaga-nanseibu	1930.10.17					
21.	Kita-izu	1930.11.26	7.0	0-5	35.1°N, 139.0°E	272	
22.	Nishi-Saitama	1931.9.21	7.0	10-20	36.1°N, 139.2°E	16	65
23.	Noto	1933.9.21	6.0	15	37.1°N, 137.0°E	3	4
24.	Shizuoka	1935.7.11	6.3	10	35.0°N, 138.4°E	9	29
25.	Kawachiyamato	1936.2.21	6.4	20	34.5°N, 135.7°E	9	23
26.	Kinkazan-oki	1936.11.3	7.7		38.2°N, 142.2°E	0	58
27.	Oga	1939.5.1	7.0	0	39.95°N, 139.8°E	27	
28.	Nagano	1941.7.15	6.2	5-20	36.7°N, 138.3°E	5	3
29.	Tottoriken-oki	1943.3.4, 3.5	6.1, 6.1	20, 20	35.6°N, 134.2°E	0	11
30.	Tottori	1943.9.10	7.4	10	35.5°N, 134.2°E	1,083	12
31.	Tonankai	1944.12.7	8.0	0	33.7°N, 136.2°E	998	206
32.	Mikawa	1945.1.13	7.1	0	34.7°N, 137.0°E	1,961	44
33.	Nankai	1946.12.21	8.1	30	33.0°N, 135.6°E	1,330	364
34.	Fukui	1948.6.28	7.3	20	36.1°N, 136.2°E	3,895	
35.	Tokachi-oki	1952.3.4	8.1	4.5	42.15°N, 143.85°E	28	168
36.	Daishoji-oki	1952.3.7	6.8	20	36.45°N, 136.20°E	7	
37.	Tokushima-nanbu	1955.7.27			33.80°N, 134.30°E	1	
38.	Futatsui	1955.10.19	5.7	0-10	40.3°N, 140.2°E	0	
39.	Nagaoka	1961.2.2	5.2	20	37°27'N, 138°50'E	5	1
40.	Hyuganada	1961.2.27	7.0	40	31°36'N, 131°51'E	2	
41.	Miyagiken-hokubu	1962.4.30	6.5	0	38°44'N, 141°08'E	3	
42.	Oga Hanto	1964.5.7	6.9		40.3°N, 139.0°E	0	
43.	Niigata	1964.6.16	7.5	40	38°21'N, 139°11'E	26	99
44.	Ebino	1968.2.21/2.22/ 3.25	5.7, 6.1/ 5.6/5.7, 5.4	0/0, 10	38°01'N, 130°43'E	3	3
45.	Tokachi-oki	1968.5.16	7.9	0	40°44'N, 143°35'E	49	313
46.	Izu-Oshima Kinkai	1978.1.14	7.0	0	34.8°N, 139.3°E	1	20
47.	Northern Miyagi-oki	1978.2.20	6.0	60	38.7°N, 142.2°E	0	
48.	Miyagi-ken-oki	1978.6.12	7.4	30	38°09'N, 142°13'E	2	110
49.	Urakawa-oki	1982.3.21	7.2	10	42.1°N, 142.6°E	0	30
50.	Nihonkai Chubu	1983.5.26	7.7	5	40°24'N, 138°54'E	104	200

Table 3 Brief Information of 30 Liquefied Sites
(See Fig. 25 for Locations)

No.	River or Place	Site	Epic. Dist. Δ (km)	Evidence of Liquefaction	Damaged Structures	Typical N-values				
						2	5	Depth 10	15	20m
1	Abukuma	Abukuma Br.	114	Sand Boils	Bridge Piers	14	17	16	56	21
2	"	Niihama Weir	111	"	Rivetments	7	2(S)	30	32	-
3	"	River Mouth	110	"	Dyke	2	12	16	18	17
4	Arahama	Kokumin Ctr	110	Sand Boils, Cracks	Tank, Pool	5	12	36	37	45
5	Torinoumi	Shimayazaki	112	Sand Boils	Dyke, Rivetments	—				
6	Natori	Natori Br.	112	Sand Boils, Cracks	None	—				
7	"	Nakamura	109	Sand Boils	Dyke	4	17	19	-	-
8	"	Yuriage-Kami	109	"	Dyke	4	2(S)	50	50	-
9	"	Yuriage Br.	108	Sand Boils, Settle	Bridge Piers	3	8	25	25	15
10	Sendai New Port	New Port	102	Sand Boils	Pavement	12	17	24	29	7
11	Yoshida	Yamazaki	106	"	Dyke	2	3	0(C)	6	16
12	Naruse	Kimazuka Br.	99	"	Bridge	—				
13	Eai	Wabuchi	95	"	Dyke	—				
14	Old Kitakami	Oiri (1)	92	"	Dyke	2	9	3(S)	45	4(C)
15	"	Oiri (2)	91	"	Dyke	—				
16	Kitakami	Kawamae	85	"	Dyke	—				
17	"	Hagaki	80	"	Dyke	3	9	6	3(C)	-
18	Ishinomaki Port	Ohmagari	87	"	Coastal Protection	—				
19	Kyu Kitakami	Uomachi Port	80	"	Rivetment, Apron	6	4	7(S)	1(S)	50
20	Ishinomaki C.	Hebita	86	"	None	2	5	28	6(S)	4(S)
21	Suzakihama	Suzakihama	93	"	None	—				
22	Takagi	Matsushima	100	"	Tank, Pump Stn.	0(S)	2(S)	0(S)	0(S)	2(S)
23	Matsushima	Tedaru	98	"	None	—				
24	Rifu	JNR	107	"	None	6	4	0(S)	4	8
25	Sendai New Port	Power Stn.	102	"	None	12	20	25	23	7
26	Old Hazama	Yoneyama	104	"	None	5	10	11	19	47
27	Hazama T.	Municipal Off.	104	"	None	—				
28	Yoshida	Fukuro	109	"	Dyke	3	30	4(S)	1(S)	18
29	Ishinomaki Port	Shiomi	84	"	Pier	4	12	13	5(S)	3(S)
30	Eai	Kaniyaji	105	"	Dyke	1	3	13	18	5(S)

(Note) In the column of N-values, (S) and (C) denote silt and clay, respectively, and no parenthesis indicates sand or gravel.

and the lower bound for $M > 6.0$ is expressed by

$$D = 10^{0.77M-3.6} \quad (2-b)$$

On the basis of this relation, one can estimate that liquefaction may not occur at any site more distant than the distance obtained by Eq. (2-b). For earthquakes with magnitudes of 6, 7, and 8, for example, liquefaction may not take place at more distant sites than 10, 60, and 300 km from the epicenters, respectively.

2.2 Selected Regions Vulnerable to Liquefaction

As seen from Fig. 8, there are some selected regions where liquefaction took place considerably. Among them, Kanto Region (Tokyo and surrounding areas), Nobi Region (Nagoya and surrounding areas), and Hokuriku Region (Niigata and Yamagata Prefectures) are shown in Figs. 9, 10, and 11, respectively. Those maps provide microscopic features of the regions with densely distributed liquefied locations. The following will give detailed descriptions for the three regions.

Kanto Region (Tokyo and Surrounding Areas)

Spoutings of water containing fine sands (dark blue or black in color) from cracks on the ground were observed at many places in Kanto region during the Tokyo Earthquake of 1894 (No.5 in Table 2). In Fig. 9 the sites of liquefaction are denoted by solid circles with numbers 1 to 18 which are distributed only in alluvial areas close to the epicenter. After an interval of about 30 years from this earthquake, Kanto region was hit by the Kanto Earthquake of 1923 (No. 15) which was one of the most disastrous earthquakes in the Japanese history. Sites of liquefaction during this earthquake are denoted by hollow triangles in Fig. 9 and are very widely spread in alluvial plains in Kanto region. It is noteworthy that many liquefied sites are concentrated on old river beds of the Tone River located northeast of Tokyo. About three hundred and fifty years ago, the estuary of the Tone River was artificially connected to the Pacific Ocean by human excavation. Before that time the original Tone River flowed into Tokyo Bay, and was very deep. Soft deposits with sands sedimented in the area of the old river beds for a long period. Because of the change in the river course, high potential for liquefaction has remained in these sites in comparison with other sites in Kanto plain. Both of the above earthquakes and also the Tone-Karyu Earthquake of 1895 (No. 7) and the Nishi-Saitama Earthquake of 1931 (No. 22) brought about liquefaction of subsoils

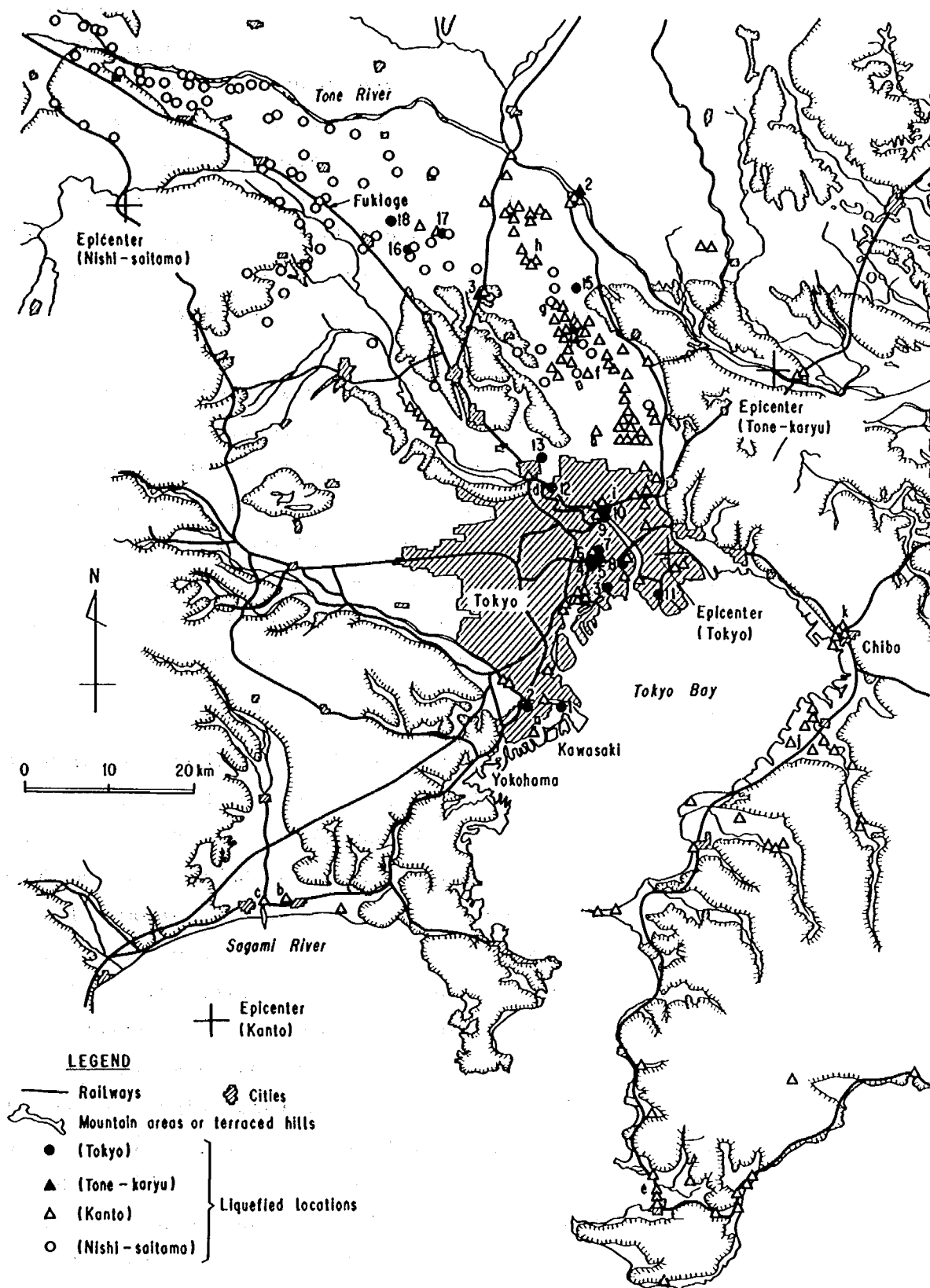


Fig. 9 Liquefaction Distribution Map in Kanto Region (Tokyo and Vicinity)
(After Kuribayashi, Tatsuoka, and Yoshida²¹)

in this area. Besides this area, sand boils, sand volcanoes and floating-up of wooden piles and caissons during the Kanto Earthquake of 1923 (No. 15) were reported at a number of sites shown in Fig. 9 where representative liquefied sites denoted by letters a, b, . . . , k. For example, the site a is the reclaimed land in Kawasaki City where water with dark blue and gray fine sand was ejected from ground surface cracks. At the site b near the epicenter, seven wooden piles floated up about 60 cm from a rice paddy during the quake (Figs. 12 and 13). These piles had been used as bridge piles about 800 years ago crossing over the original Sagami River. At the site c several bridge caisson foundations under construction floated up from the river bed of Sagami River (Fig. 14).

The Nishi-Saitama Earthquake of 1931 (No. 22) also induced liquefaction at a number of sites along the original and existing courses of the Tone River. The liquefied sites during this earthquake are denoted by hollow circles in Fig. 9. Violent water spouting was observed in Fukiage City located about 20 km east of the epicenter where a large quantity of water and fine sands ejected about 30 cm high and several hundred sand volcanoes were observed. Furthermore, the ground surface of liquefied sites settled about 15 cm and spouted water with a depth of about 15 cm covered the ground.

Nobi (Mino-Owari) Region (Nagoya and Surrounding Areas)

The Nobi region has been inflicted by four violent earthquakes during the past century which induced liquefaction at many sites, as shown in Fig. 10. Among them, the most violent one was the Nobi Earthquake of 1891 (No. 4) which brought about violent liquefaction in wide areas of the Nobi plain and Fukui plain which are denoted by solid triangles in Fig. 8 and by hollow triangles in Fig. 10. Liquefied sites are limited in alluvial soft deposits of three rivers of Kiso, Nagara and Ibi in Nobi plain and in the sites along river courses in Fukui plain which is located in the north of the epicenter. It should be noted that the Fukui Earthquake of 1948 (No. 34) also induced liquefaction in the entire Fukui plain as shown in Fig. 8 (also Figs. 15 and 16). The most violent water spouting during the Nobi Earthquake (No. 4) was observed at the site denoted by the letter a in Fig. 10 near the Shonai River where water with sand ejected high above 2 m from wells and sands deposited over the level of the house roof. And at the site b from the right bank of the Shonai River to the south of the site a, a thousand and several hundred sand boils were observed. The extent of liquefaction could be understood from the fact that rice paddy and field of about 40 km² in the prefectures of Aichi and Gifu became impossible to cultivate due to sand boils.

After an interval of eighteen years from this earthquake, the Gono Earthquake of 1909 (No. 12) brought liquefaction at many sites in the northwestern part of Nobi plain where liquefaction had been induced during the Nobi Earthquake (No. 4). Many sites liquefied during this earthquake which are denoted by hollow circles in Fig. 10. These are densely distributed in the area surrounded by three rivers of Ibi, Nagara and Kiso.

The Tonankai Earthquake of 1944 (No. 31) brought liquefaction in the coastal region with soft alluvial deposits between Cape Omaezaki and Lake Hamana, and in the south coastal part of Nagoya City which had been reclaimed during the past hundred and fifty years prior to this Earthquake. Such liquefied sites are denoted by hollow triangles in Fig. 7 and solid triangles in Fig. 10. It was disclosed that these liquefied sites were generally distributed inside zones where houses were damaged severely.

In the southern coastal region of Nagoya City, the damage (settlement and tilting) to wooden houses was mainly due to extensive ejections of sand and water. Furthermore, many sand boils and water spoutings were observed at the sites of severely damaged factories at Nishi-Inaei town which are denoted by the letter c in Fig. 10. It would be of interest that in the following Mikawa Earthquake of 1945 (No. 32) violent boiling of sand and water was observed again at the sites of these factories.

Hokuriku Region (Niigata and Yamagata Prefectures)

Fig. 11 shows a liquefaction history of prefectures of Niigata and Yamagata during the past century. The Shonai Earthquake of 1894 (No.6) brought about liquefaction at many sites which are denoted by hollow triangles in Fig. 11(b). In Sakata City located at the rivermouth of Mogami River, numerous sand boils were observed and the largest sand volcano reported was about 60 cm in height and about 3 m in diameter. The area where sands boiled most violently was covered by ejected sands and as a result this area became like a seashore. Similar violent sand boils were observed at site 2 in Fig. 11(b) where several tens of sand volcanoes were observed. The largest one was about 9 m in diameter (Figs. 17 and 18). It is interesting to note that liquefaction occurred again in this same site during the Niigata Earthquake of 1964 (No. 43) as shown in Fig. 11(b).

The Niigata Earthquake (No. 42) is well known for intensive liquefaction and extensive damage to modern civil engineering structures due to the liquefaction of saturated sandy deposits. Since the details are described in many other reports, only a summary is presented here. Liquefied zones during this earthquake are denoted by hatched areas with numerals 1 to 13 in Fig. 11(a)

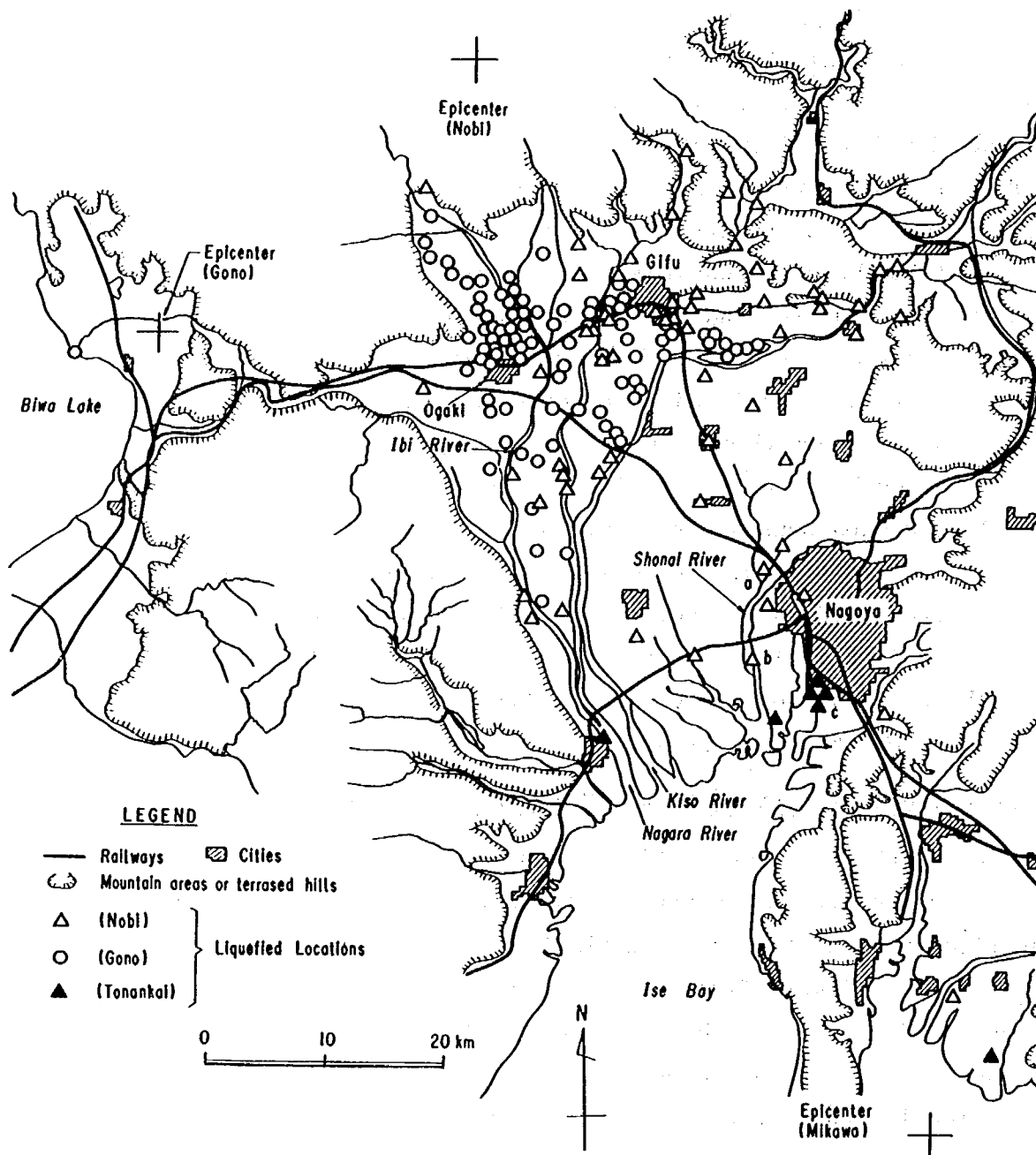


Fig. 10 Liquefaction Distribution Map in Nobi Region
(Nagoya and Vicinity)
(After Kuribayashi, Tatsuoka, and Yoshida²¹)

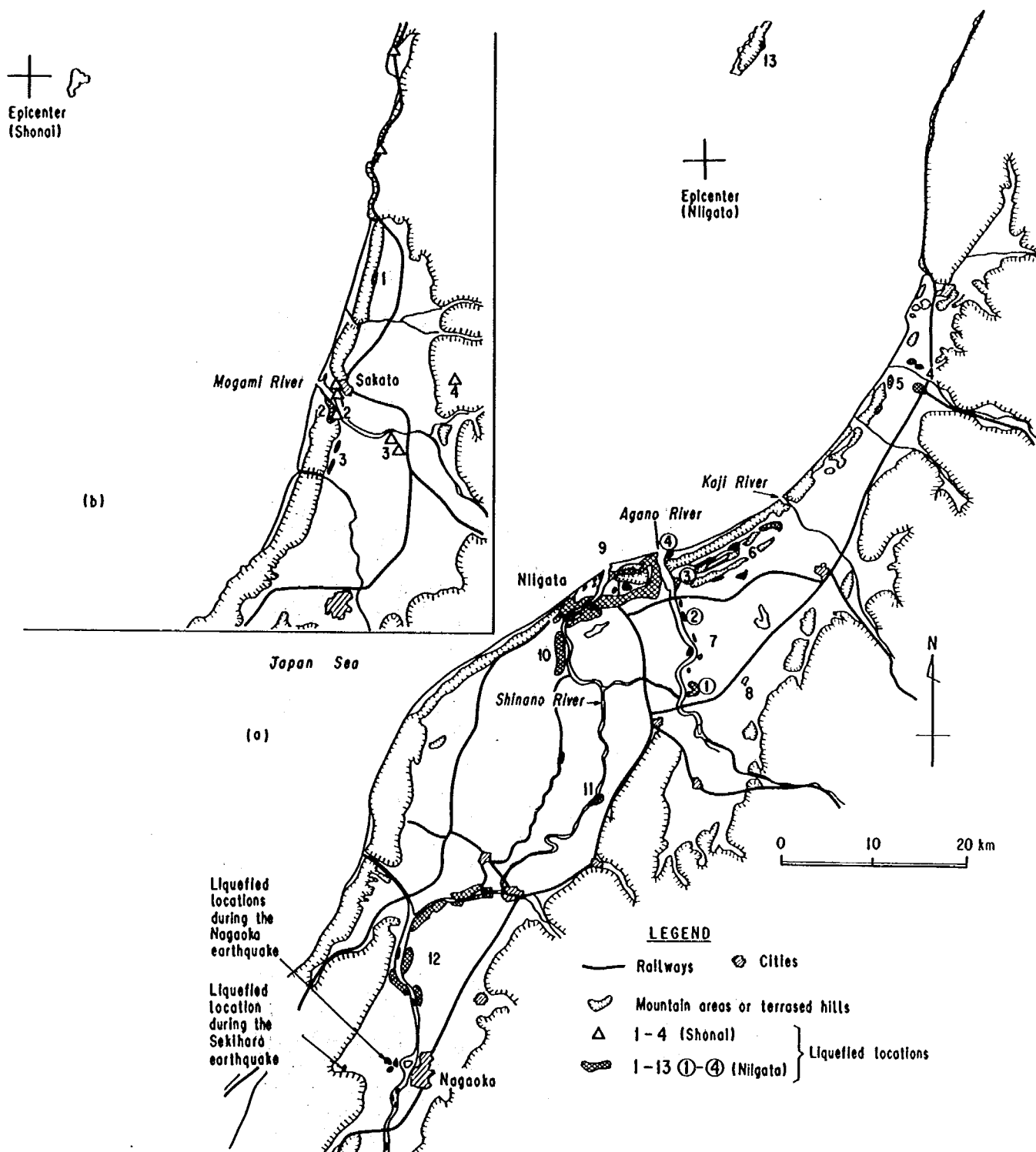


Fig. 11 Liquefaction Distribution Map in Hokuriku Region
(Niigata and Yamagata Prefectures)
(After Kuribayashi, Tatsuoka, and Yoshida²¹)



Fig. 12 Wooden Piles which Floated up from Rice Paddy at Chigasaki, Kanagawa Prefecture during the Kanto Earthquake of 1923



Fig. 13 A Closer View of a Wooden Pile which Floated up at Chigasaki during the Kanto Earthquake of 1923



Fig. 14 Caisson Foundations under Construction which Floated up from Riverbed of Sagami River, Kanagawa Prefecture during the Kanto Earthquake of 1923

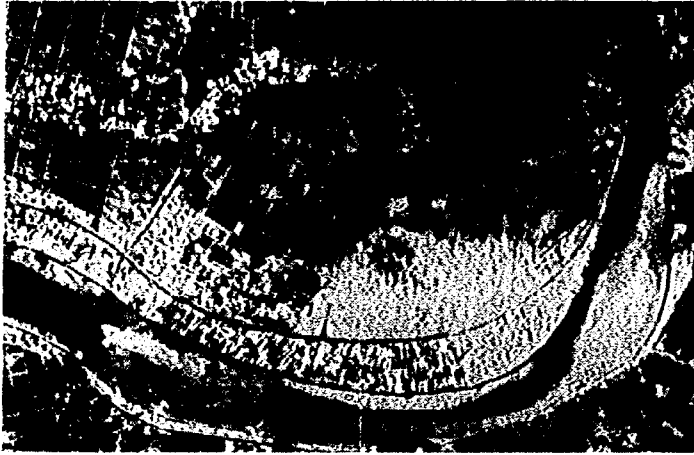


Fig. 15 Boiled Sands (White Color) in Rice Paddies along Kuzuryu River in Fukui Plain, Fukui City during the Fukui Earthquake of 1948



Fig. 16 Boiled Sands in Rice Paddy in the Suburb of Fukui City during the Fukui Earthquake of 1948

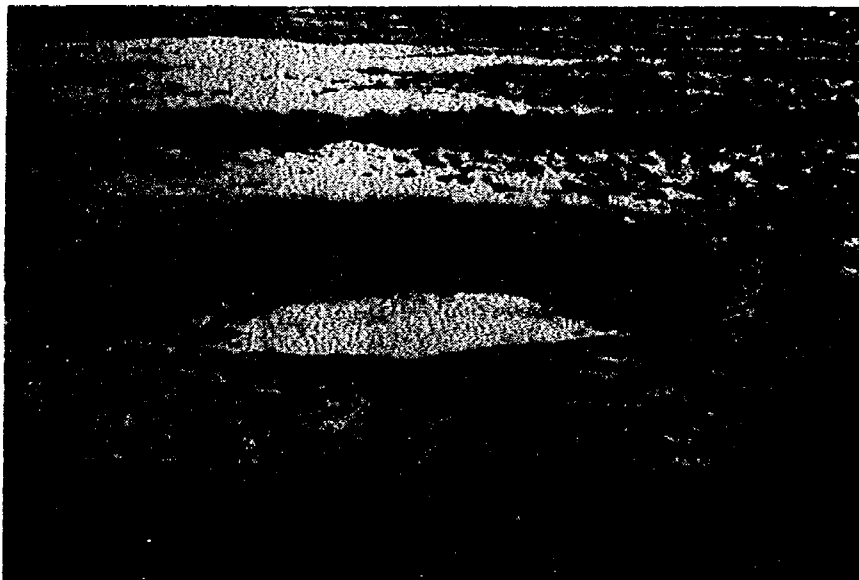


Fig. 17 Pond with Diameter of 2.4 m Made by Water Ejection near Mt. Iimori at the Rivermouth of Mogami River during the Shonai Earthquake of 1894

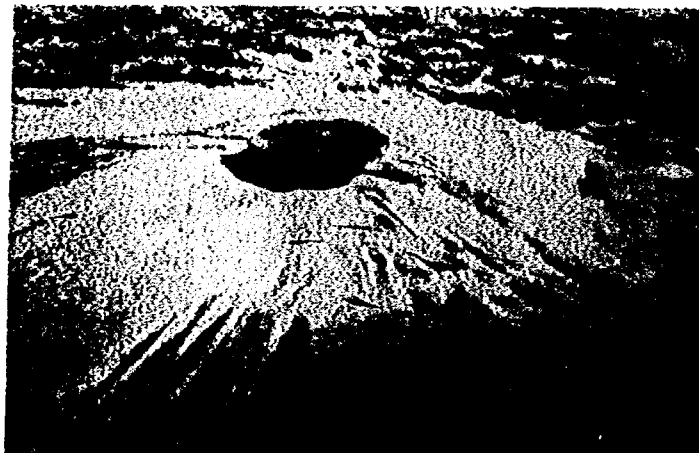


Fig. 18 Sand Volcano with Diameter of 3.6m and Height of 30cm near Mt. Iimori during the Shonai Earthquake of 1894

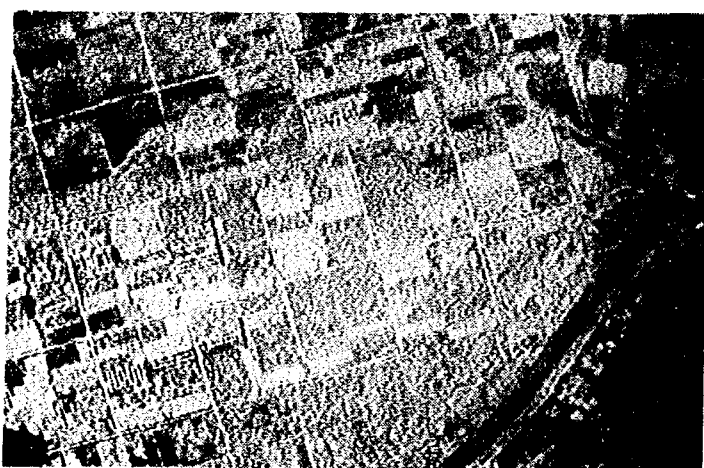


Fig. 19 Boiled Sands in Rice Paddies (Old Riverbeds of Agano River) near Niigata City during the Niigata Earthquake of 1964

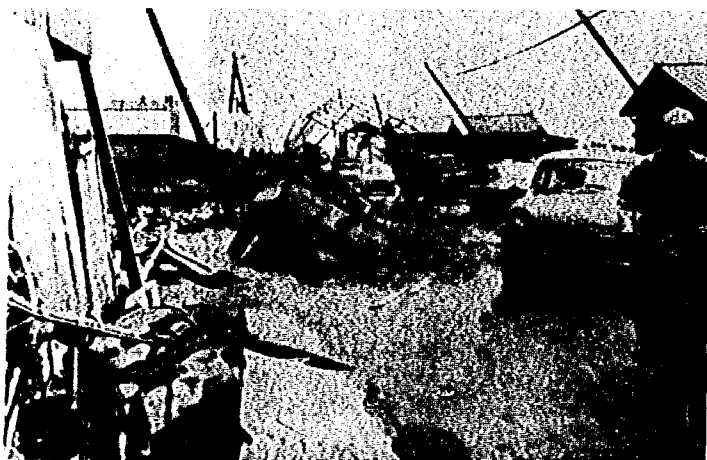


Fig. 20 Truck which Sank into Liquefied Low Bank of National Highway No.7 at Ebigase during the Niigata Earthquake of 1964



Fig. 21 Large Hole in Liquefied Ground at Area 10 in Fig.11. This Area Used to be Old Riverbed of Shinano River

and it may be seen that these zones are limited to old riverbeds of the rivers of Shinano, Agano, etc. For example, the area denoted by the letter a in Fig. 11(a) is the original riverbed of the Agano river which used to connect with Shinano River at the river mouth about three hundred years ago (Fig. 19). This area was reclaimed by sands about fifty years ago. National Highway No. 7 passes through this area and its 1 m high bank of sand moved down 1 m and laterally 4 m during this earthquake. This would be due to liquefaction of sandy bank and sandy ground because boiled sand covered the road surface and one truck on the bank sank into the ground (Fig. 20).

Furthermore, Fig. 21 shows large holes in the liquefied ground at the area No. 10 in Fig. 11(a) which used to be the riverbed of the Shinano River.

2.3 Soil Liquefaction Caused by Recent Earthquakes

Izu-Ohshima Kinkai Earthquake of 1978

A severe earthquake hit the eastern coast of Izu Peninsula on January 14, 1978, registering a magnitude of 7.0 on the Richter scale. The epicenter of the main shock is shown in Fig. 22(a) and (b), as well as JMA seismic intensities and epicenters of several foreshocks and numerous aftershocks.

Two tailing dams at Mochikoshi reservoir which stored mining wastes of fine silts (with diameter of about 0.02 mm) severely collapsed, as shown in Figs. 23 and 24. It is revealed that the main cause of the collapse was liquefaction of silts in the reservoir. The location of Mochikoshi tailing dams is shown in Fig. 22(b). The slimes which overflowed one of the dams caused extensive pollution of the Kano River and Suruga Bay. In view of the accident, the Ministry of International Trade and Industry made engineering provisions regarding countermeasures against liquefaction of soil materials stored in mining reservoirs. Contents of the provisions will be described in succeeding Section (6.5) of this paper.

Miyagi-Ken-Oki Earthquake of 1978

A strong earthquake hit Miyagi Prefecture on June 12, 1978, with a Richter magnitude of 7.4. The epicenter was located under the sea bottom off Miyagi, approximately 120 km east from Sendai City. The focal depth was reported about 30 km. The earthquake caused extensive damage to various engineering structures such as buildings, bridges, river dikes, industrial facilities, etc. Soil liquefaction phenomena were observed at 30 areas, among which engineering structures such as river dikes and bridges sustained damages in 22 areas. In the remaining liquefied areas, no structural damage was observed.

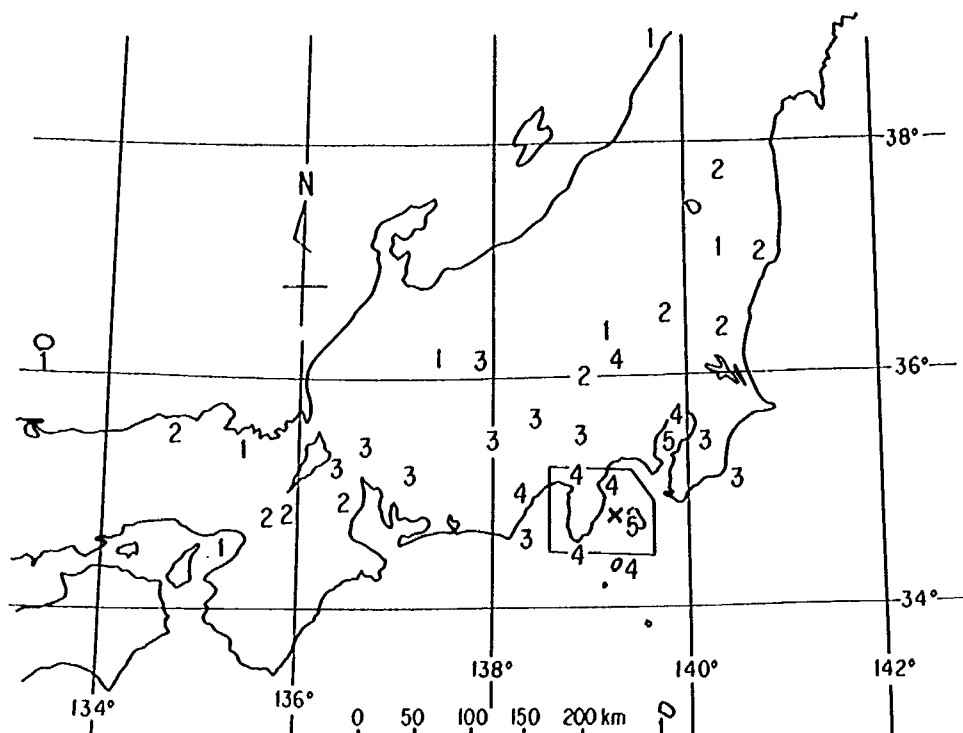


Fig. 22 (a) JMA Seismic Intensity and Epicenter of Mainshock of Izu-Ohshima Kinkai Earthquake of 1978

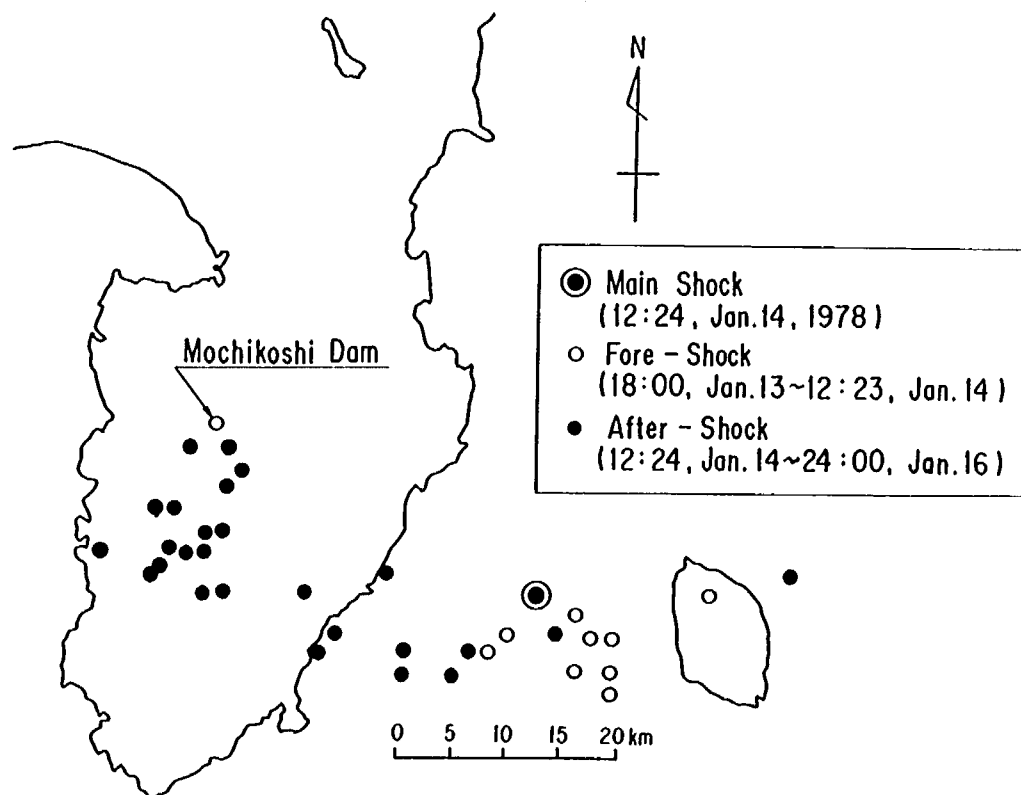


Fig. 22 (b) Distribution of Epicenters of Foreshocks, Mainshock, and Aftershocks, and Location of Mochikoshi Tailing Dam

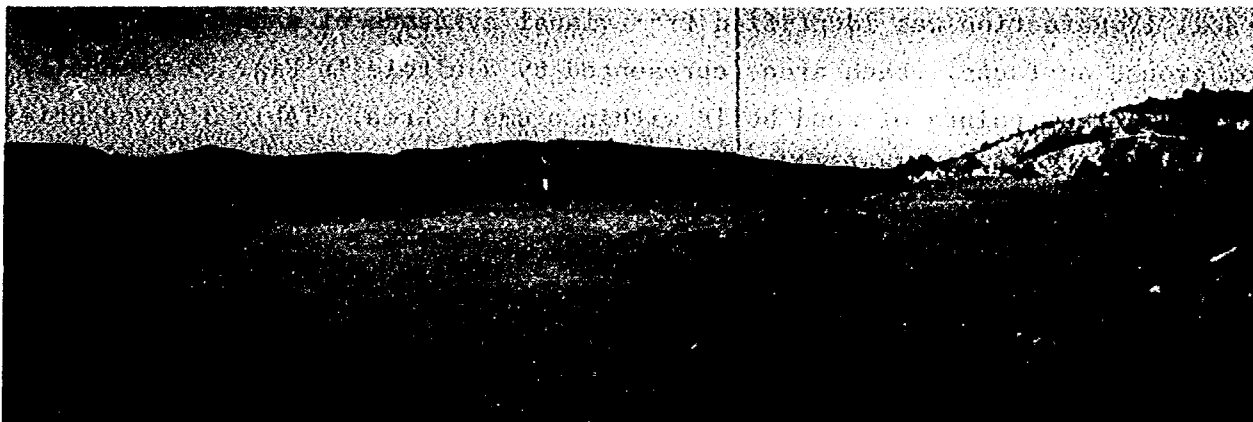


Fig. 23 Overall View of Mochikoshi Tailing Dam
for Storing Mining Waste



Fig. 24 Collapsed First Tailings Dam and Temporarily
Repaired Dam

Locations of the liquefied sites are indicated in Fig. 25. Open circles in the figure denote areas where soil liquefaction took place. An occurrence of soil liquefaction was identified from visual evidence of sand boils spouting from ground surfaces. Each area represented by a circle in Fig. 25 generally has a number of points of sand boils within a small area. Table 3 gives brief information about the liquefied sites. Numerals in the table coincide with those in open circles in Fig. 25.

Epicentral distances of liquefied areas are between 80 and 114 km. The farthest distance was 114 km, which coincides well with the empirical equation (2-b) described in 2.1 of this paper. Most of the major damage to civil engineering structures and soil liquefaction concentrated in areas of alluvial deposits along large rivers, such as Abukuma, Natori, Eai, Yoshida, and Kitakami.

As shown in Fig. 25, five areas close to the estuary of the Abukuma River were disclosed to liquefy from the evidence of sand boils at the ground surface. One of those is an area at Abukuma Bridge (on National Highway No. 6) which sustained heavy cracks to several pier columns, as shown in Fig. 26. A typical view of sand boils is shown in Fig. 27, which is a photograph taken at the site of Pier 6 of Abukuma Bridge. A boring log at that site is shown in Fig. 28. The standard penetration tests (SPT) were done in the Summer of 1978 (after the Earthquake), together with shear wave velocity measurement, sand sampling, cyclic triaxial tests for specimens taken from the site, and seismic response analysis of subsoils. Those studies were conducted to assess liquefaction potential and consider the effects of liquefaction in the design of New Abukuma Bridge which was completed in 1981 very close (15 m downstream) to the site of the damaged Abukuma Bridge. The results of liquefaction assessment are also shown in Fig. 27 in terms of liquefaction resistance factors (F_L) by the simplified method to be described in the following section (3.2) of this paper. In the assessment analysis, the peak ground acceleration was assumed to be 240 gals.

On the other hand, four areas close to the estuary of Natori River were also liquefied. One of them is an area at Yuriage Bridge, which was also damaged heavily. Fig. 29 shows a typical view of sand boils spouting from surface cracks near the site of Pier 8. From the careful observation of the Yuriage Bridge it was disclosed that eight of nine piers sustained cracks at the columns and inclined to the downstream direction. In order to assess liquefaction potential at this site, two procedures were employed. The results of liquefaction assessment using the simplified procedure are illustrated in Fig. 30, in which the liquefaction resistance factors ($F_L = R/L$) are plotted at two boring points: B-3 (sand boils observed) and B-5 (no sand boils). In

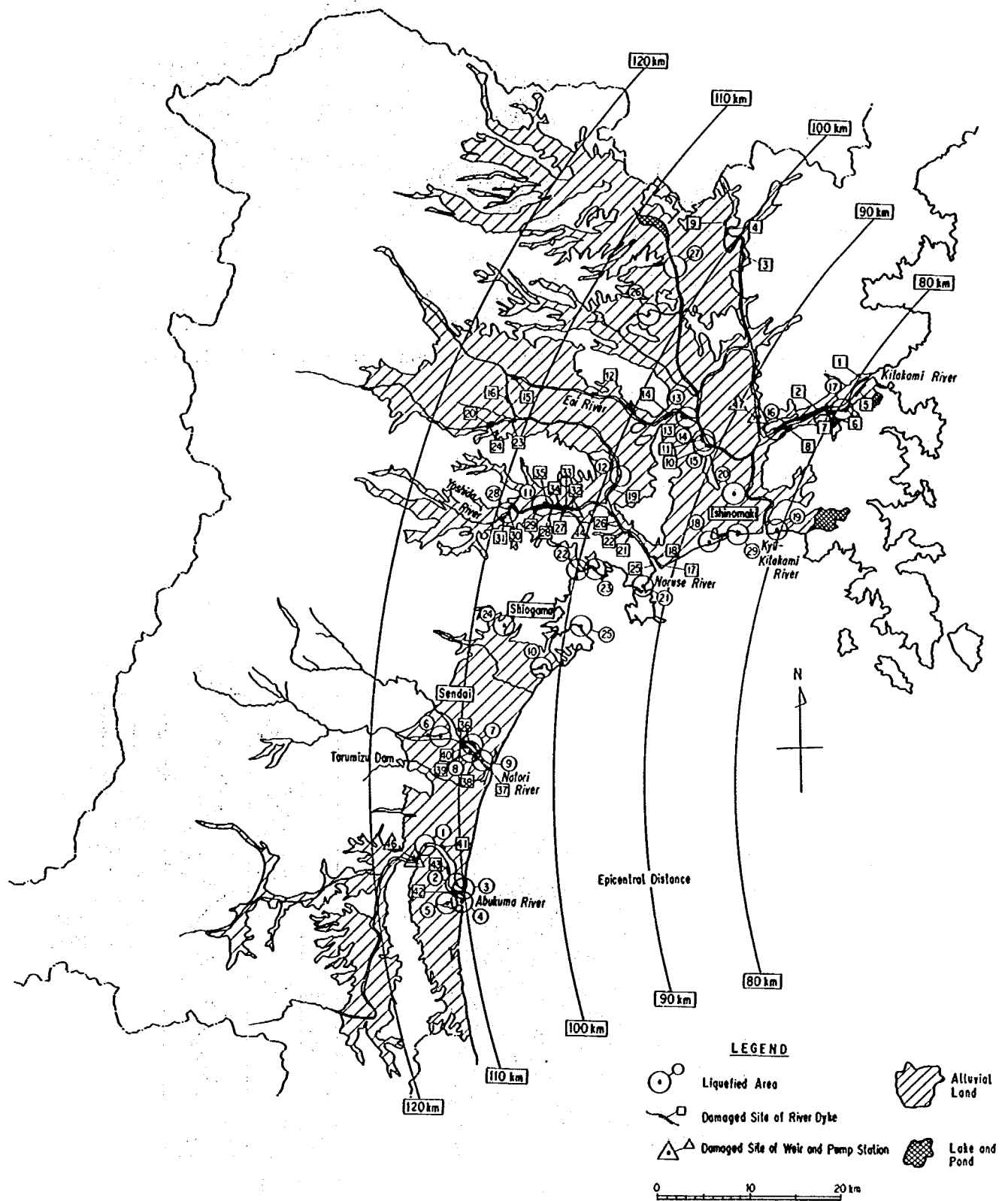


Fig. 25 Locations of Liquefied Sites during the Miyagi-ken-oki Earthquake of 1978

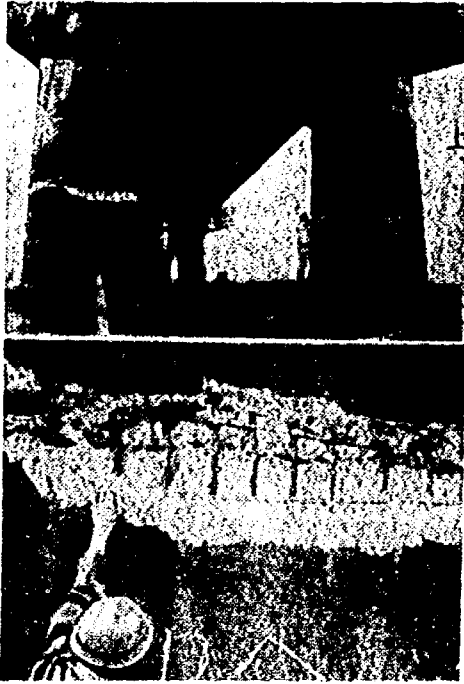


Fig. 26 Damage to a Pier Column of Abukuma Bridge

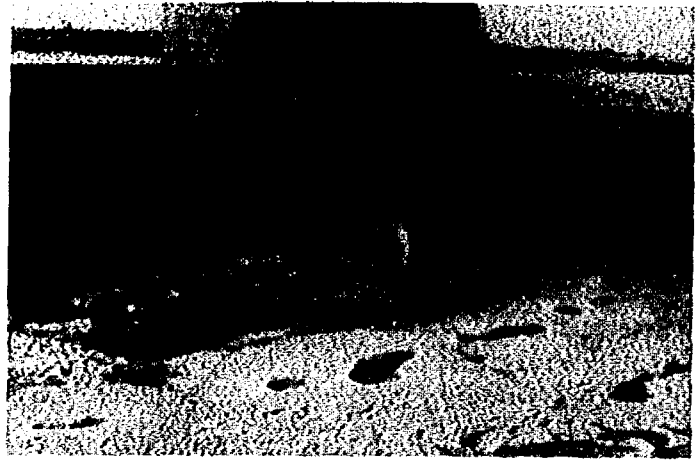


Fig. 27 Sand Boils around Pier-6, Abukuma Bridge

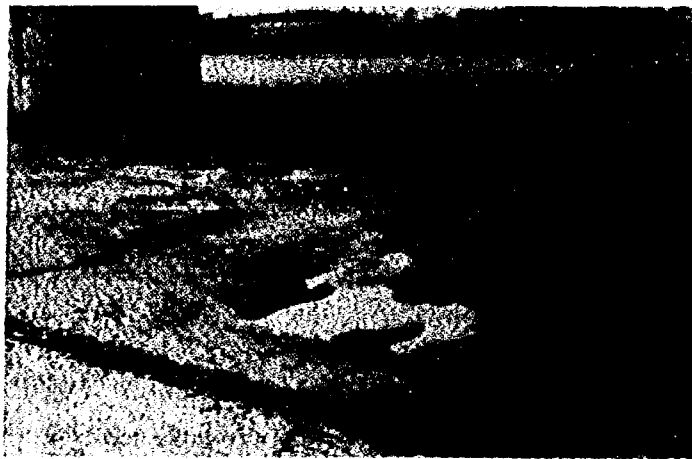


Fig. 29 Sand Boils from Ground Surface Cracks near Pier-8, Yuriage Bridge

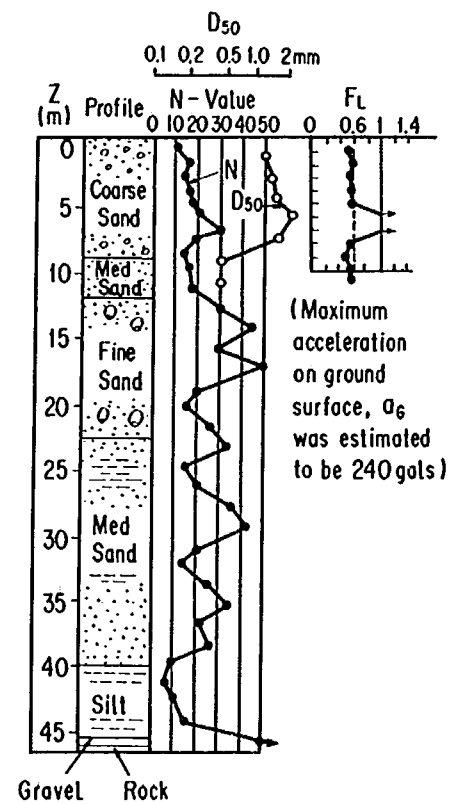


Fig. 28 Boring Log, SPT N-Values, and F_L -Values (Liquefaction Resistance Factor) at Pier-6, Abukuma Bridge

the analysis the peak ground acceleration was assumed to be 240 gals. The other is a rigorous procedure in which laboratory cyclic triaxial tests for undisturbed samples taken from the boring sites were conducted to obtain R (shear strength) and seismic response analysis was conducted (with use of a strong-motion record triggered at Tarumizu Dam site located 10 km west from the bridge site) to estimate L (shear stress). The results of the precise procedure are also plotted in Fig. 30 for the two boring points (B-3 and B-5).

Nihonkai-Chubu Earthquake of 1983

A large-size earthquake hit the coastal area of central Japan Sea, Akita and Aomori Prefectures, on May 26, 1983, registering a magnitude of 7.7 on the Richter scale. The focal depth was reported very shallow (about 5 km). Fig. 31 shows the location of the epicenter and the area of aftershocks and the locations of damages to civil engineering structures. It is reported that the peak accelerations were between 200 and 300 gals in the Japan Sea coastal areas of Akita and Aomori Prefectures. Although extent of damage to buildings and highway bridges was rather moderate, damage to soil structures such as river dikes and road embankments were very extensive, essentially due to the effects of liquefaction of supporting sandy deposits.

Fig. 32 shows excessive settlements of an approaching bank to Gomyoko Bridge in Hachirogata Lagoon, Akita Prefecture. It is observed that those settlements were caused by liquefaction of supporting sandy layers. Fig. 33 indicates a view of Gomyoko Bridge, which had very minor damages to reinforced concrete piles, despite the serious settlements of the approach road embankments.

At Showa Bypass (National Highway No. 7) which had been recently constructed on rice paddy areas consisting of very soft peat layers located in Showa Town (north suburb of Akita City), most of backfills approaching to highway bridge settled by several ten centimeters. Those portions where underground drainage ducts are crossing low banks and where peat materials had been replaced by sandy materials during the construction time, also settled, although adjacent portions on natural peat materials without drainage ducts did not settle distinctly. Fig. 34(a) illustrates soil profiles at one of longitudinal sections with drainage ducts. Fig. 34(b) indicates the end of the drainage duct, and Fig. 34(c) shows that the pavement surface of the low bank settled and cracked just above the duct buried in the replaced sands. It was disclosed that the replaced sands liquefied and came out around the end of the duct during the Earthquake.

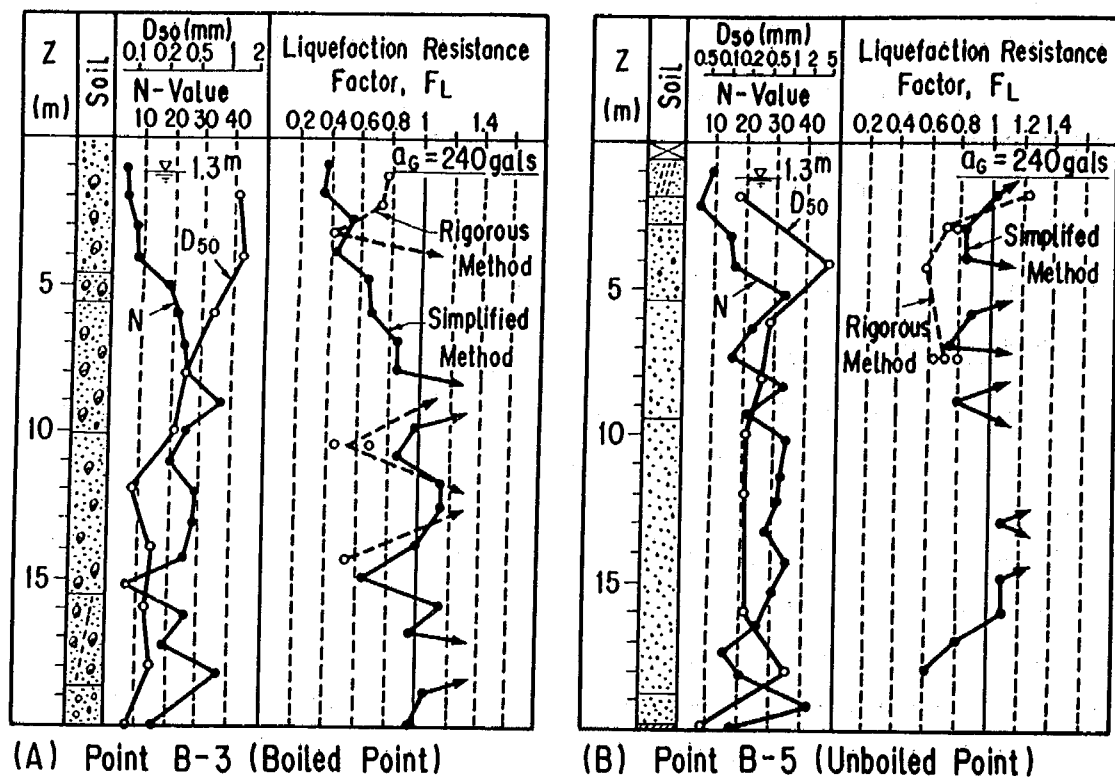


Fig. 30 Boring Logs, SPT N-Values, and F_L -Values (Liquefaction Resistance Factor) at Two Points, Yuriage Bridge

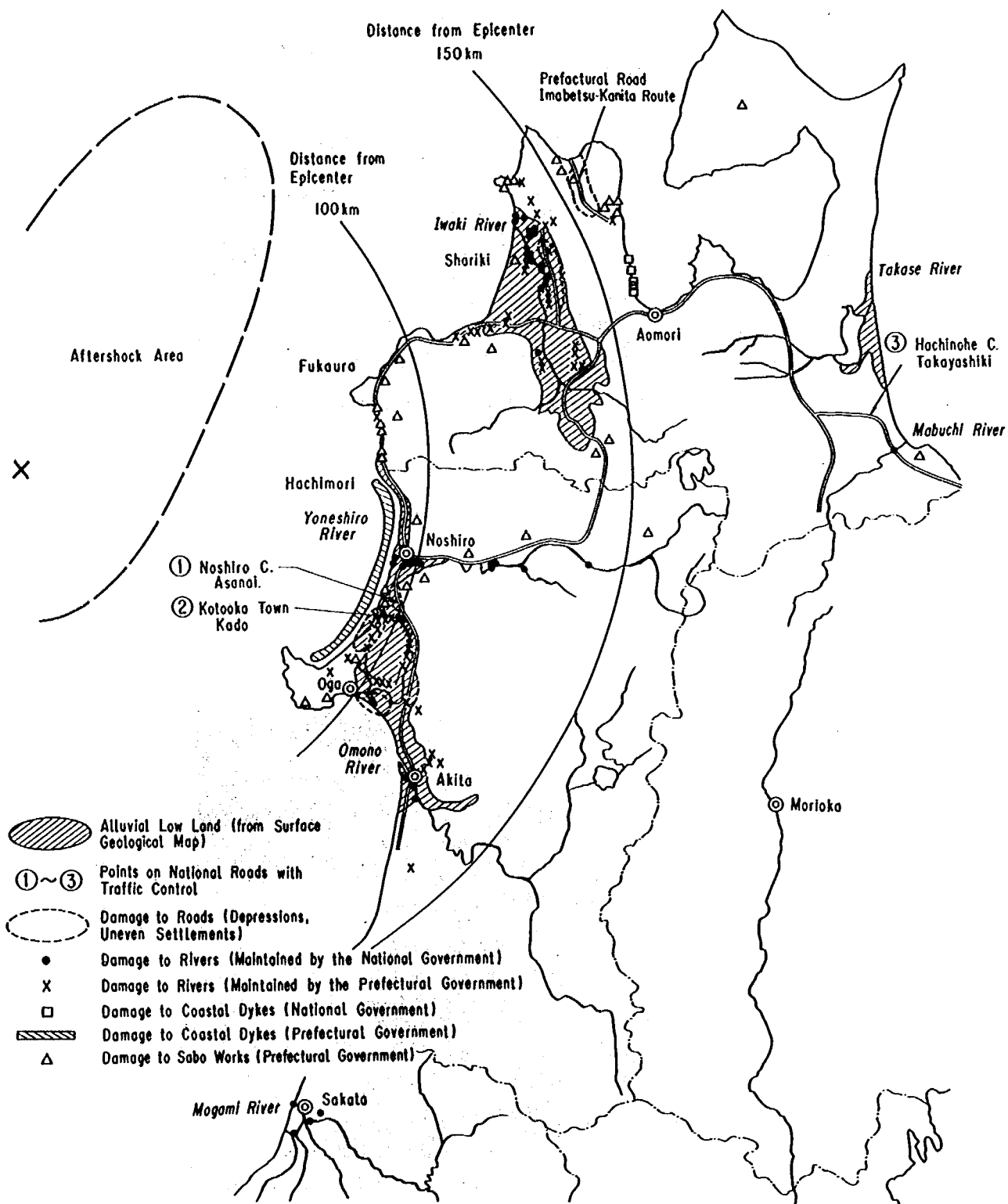


Fig. 31 Location of the Epicenter, Area of Aftershocks, and Locations of Highway Bridge Damages



Fig. 32 Excessive Settlement of Approach Bank to Gomyoko Bridge



Fig. 33 View of Gomyoko Bridge - No Major Damage to Bridge Structure

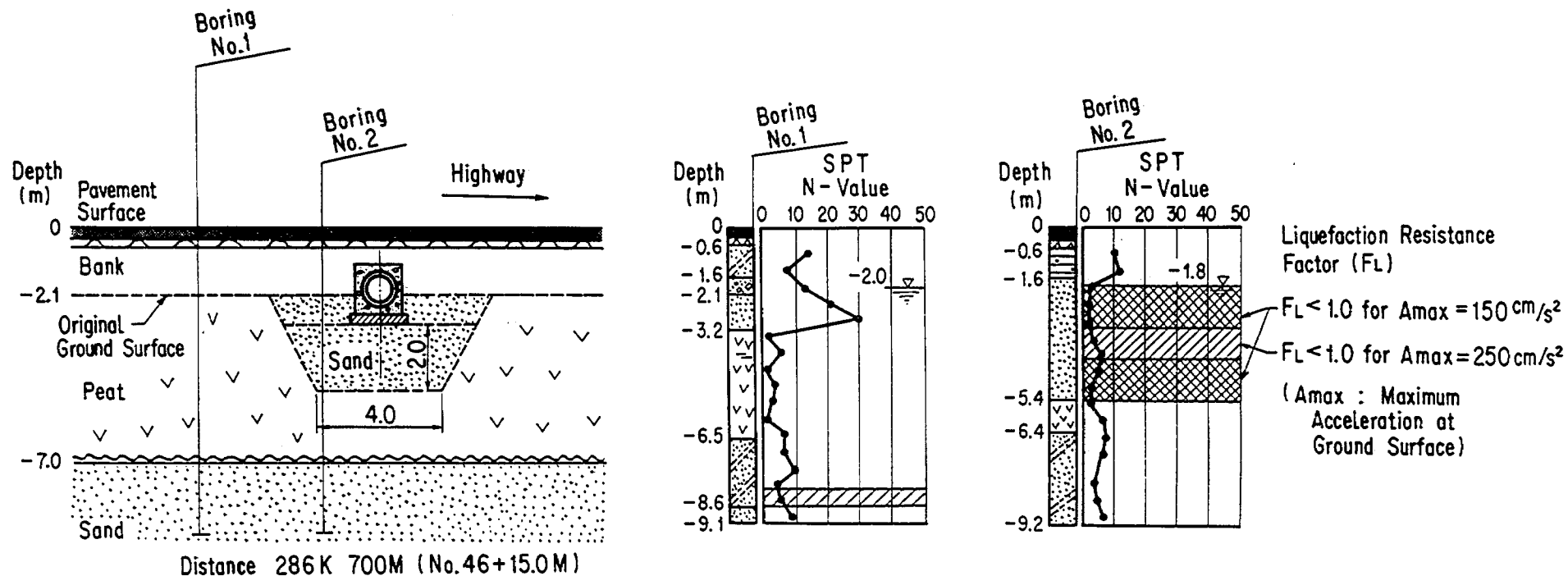


Fig. 34(a) Profile of Longitudinal Section of Highway and Soil Profiles with SPT N-Values and Liquefaction Resistance Factor (F_L)
(from Report of Akita Construction Office, Tohoku Regional Construction Bureau)

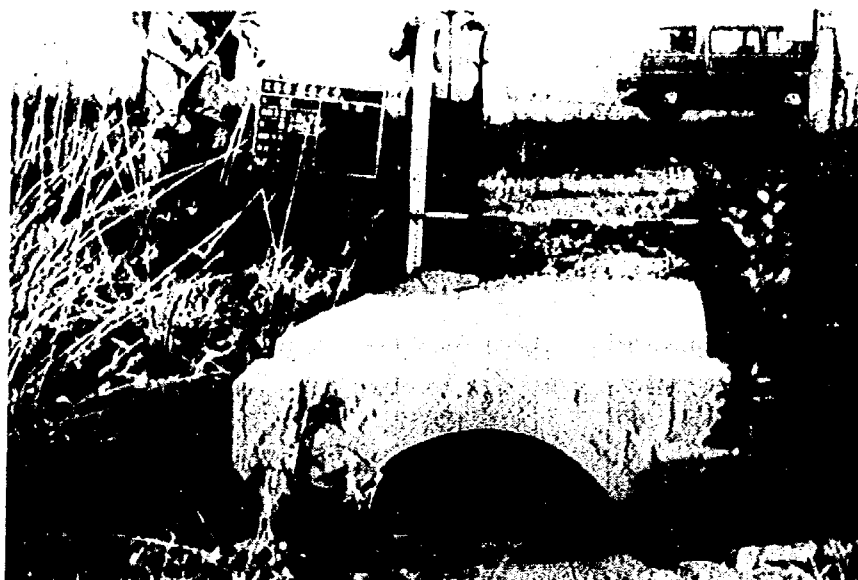


Fig. 34(b) End of Drainage Duct (Liquefied Sands Came out around the end of duct) (Courtesy of Dr. S. Yasuda, Kiso Jiban Co.)



Fig. 34(c) Settlement and Cracking at the Pavement Surface (Courtesy of Dr. S. Yasuda, Kiso Jiban Co.)

Fig. 35 shows a view of Jusanko Bridge in the northern part of Aomori Prefecture, where settlements (about 50 cm) were observed at neighboring ground surfaces due to sand liquefaction. Although serious settlements and cracks occurred at ground surfaces, the bridge did not receive serious structural damage, except a settlement of one pier (about 10 cm). Approach banks to both abutments settled considerably (about 1 m). Fig. 36 shows a large volcano with diameter of 7 m caused by soil liquefaction at Tomiyachi, Shariki Village, Aomori Prefecture.

River dikes along Hachirogata Lagoon settled seriously (1 to 2 meters). Along the total length of 50 km of the dikes surrounding Hachirogata Lagoon, about the length of 30 km sustained settlements. Figs. 37 and 38 are photographs indicating typical views of the dike damage.

2.4 Summary

From the extensive survey of soil liquefaction during the past Japanese earthquakes described in the previous sections, the following features can be summarized.

1. Re-liquefaction phenomena were observed at 9 areas as shown by the letters a to i in Fig. 8 and Table 4. This suggests that liquefaction potential at sites where liquefaction took place during past earthquakes is higher than other non-liquefied areas.
2. One of the important factors to cause liquefaction during earthquakes is the intensity of ground shaking. Seismic intensity at liquefied sites can be estimated by comparing both the liquefaction map and the seismic intensity map. From this, it is found that seismic intensity on JMA (Japan Meteorological Agency) scale at liquefied sites are 5 or higher, which is approximately corresponding to the maximum acceleration of 80 - 250 gals or higher.
3. The maximum distance of liquefaction caused by a given earthquake can be approximately estimated in terms of the magnitude of the earthquake. The lower bound of the longest distance of liquefaction (D in km) due to an earthquake with Richter magnitude of M can be expressed by

$$D = 10^{0.77M-3.6} \quad (2-b)$$



Fig. 35 View of Jusanko Bridge - Ground Settled about 50cm and One Pier Settled about 10cm

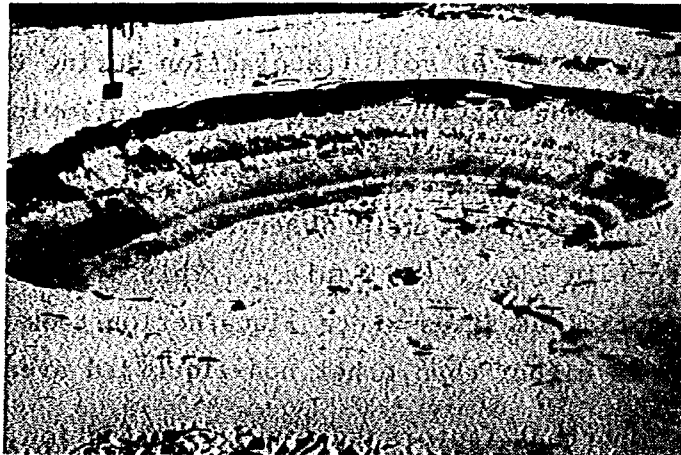


Fig. 36 View of Large Volcano Caused by Soil Liquefaction at Tomiyachi, Shariki Village, Aomori Prefecture

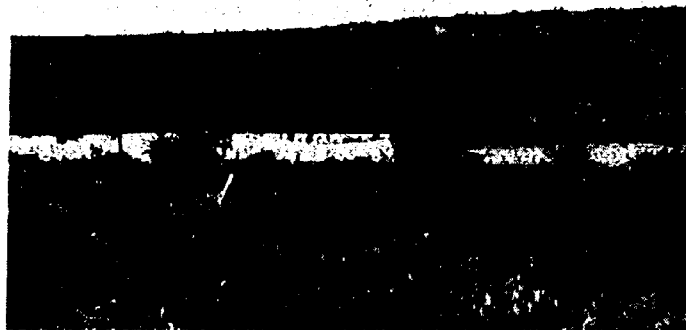


Fig. 37 A View of Damage to Dike of Hachirogata Lagoon

Table 4 Zones Where Re-liquefaction was Observed

(a, b, ..., g, h, i correspond to those)
in Fig. 10

Zones	Earthquakes
a: the narrow zone along the Omono River in the northwest of Oomagari city in Akita Prefecture	No. 8 (Rikuu) and No.13 (Ugosen)
b: on the left side at the river mouth of Mogami river	No. 6 (Shonai) and No.42 (Niigata)
c: in the original river course of Tone river on the northeast of Tokyo	No. 5 (Tokyo), No. 7 (Tonekaryu), No.15 (Kanto) and No.22 (Nishi-Saitama)
d: on the both sides of Chikuma river near Nagano city	No. 9 (Kamitakai) and No.28 (Nagano)
e: in the southern part of Fukui plain	No. 4 (Nobi) and No.24 (Fukui)
f: in the northwestern part of Nobi plain	No. 4 (Nobi) and No.12 (Gono)
g: in the southern part of Nobi plain	No.31 (Tonankai) and No.32 (Mikawa)
h: in the reclaimed land at Hachirogata Lake, Akita Pref.	No.42 No.43 No.45 No.50 (Nihonkai-Chubu)
i: on the left side (Nakamura Area, Tanetsugu Dike) of Natori River	No.47 No.48



Fig. 38 Another View of Damage to Dike of Hachirogata Lagoon

3. SIMPLIFIED PROCEDURES FOR ASSESSING LIQUEFACTION

3.1 Outline

As described briefly in Section 1.2 of this paper, several methods have been proposed to evaluate soil liquefaction potential. Iwasaki, Tatsuoka, Tokida, and Yasuda (1978)²⁴ proposed a simplified procedure using a liquefaction resistance factor (F_L) and a liquefaction potential index (P_L). In the simplified procedure the liquefaction potential of sandy soils can be evaluated from SPT N-values of the soils, unit weights (or densities) of the soils, mean particle diameters of the soils, and the maximum ground surface accelerations. In the following sections, the simplified procedure will be described, and the liquefaction resistance factors F_L and the liquefaction potential index P_L at 64 liquefied sites and 23 non-liquefied sites during a number of past Japanese earthquakes will be calculated in order to demonstrate the effectiveness of the procedure.

3.2 Liquefaction Resistance Factor, F_L

Outline of F_L -Evaluation

Fig. 39 illustrates schematically dynamic behavior of subsoils during earthquakes. Fig. 39(a) shows cyclic triaxial strength (R_ϕ) of soil specimens sampled from the field, and Fig. 39(b) indicates the dynamic shear stress (τ) of an arbitrary soil element when subjected to a bedrock acceleration with a peak acceleration of α_B .

To express the ability of a soil element to resist liquefaction, the liquefaction resistance factor F_L is defined as

$$F_L = \frac{R}{L} \quad (3)$$

where R is the in situ resistance (or cyclic shear strength) of the soil element to an earthquake loading, and L is the earthquake load in the soil element induced by a seismic motion. Evaluation methods of R and L will be described below.

Estimation of Cyclic Shear Strength (R)

An example of laboratory tests for an undisturbed soil specimen sampled from the underground of a site with sandy soils is shown in Fig. 40. This is the result of the undrained cyclic triaxial test for the specimen with the

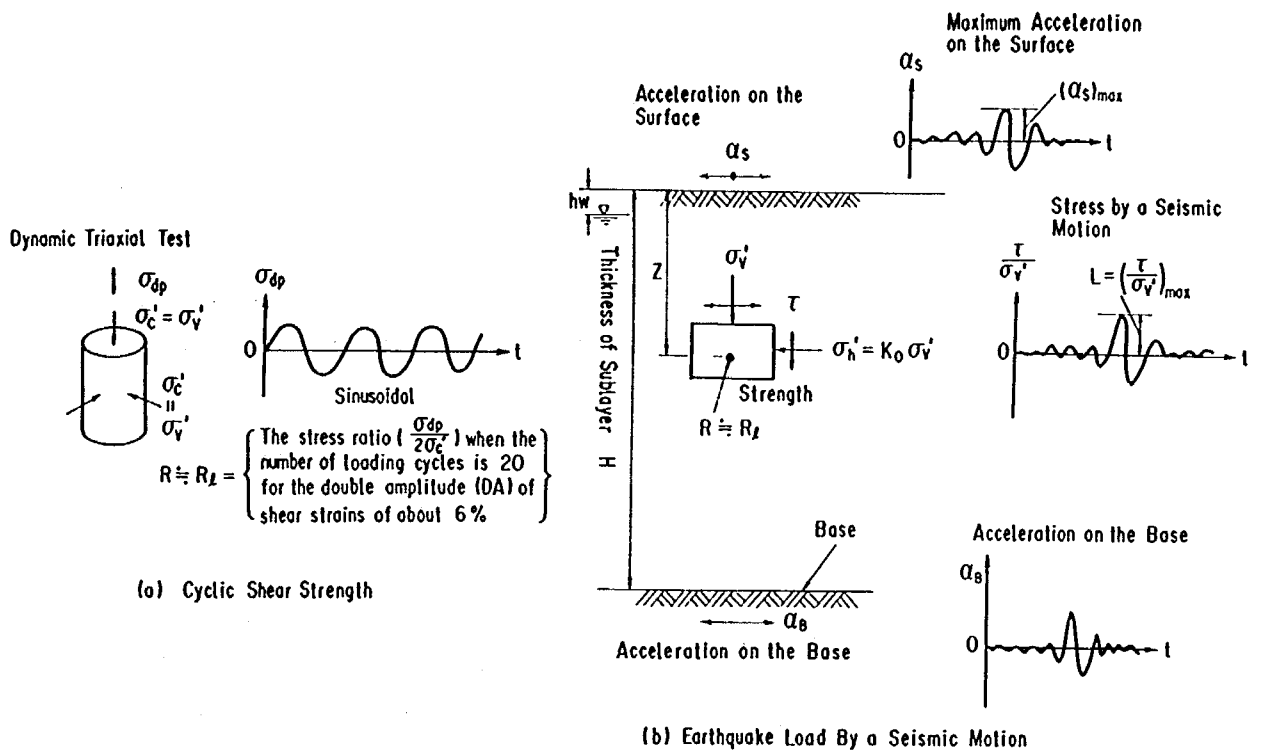


Fig. 39 Schematic Drawing on Dynamic Behavior of Subsoils during Earthquakes

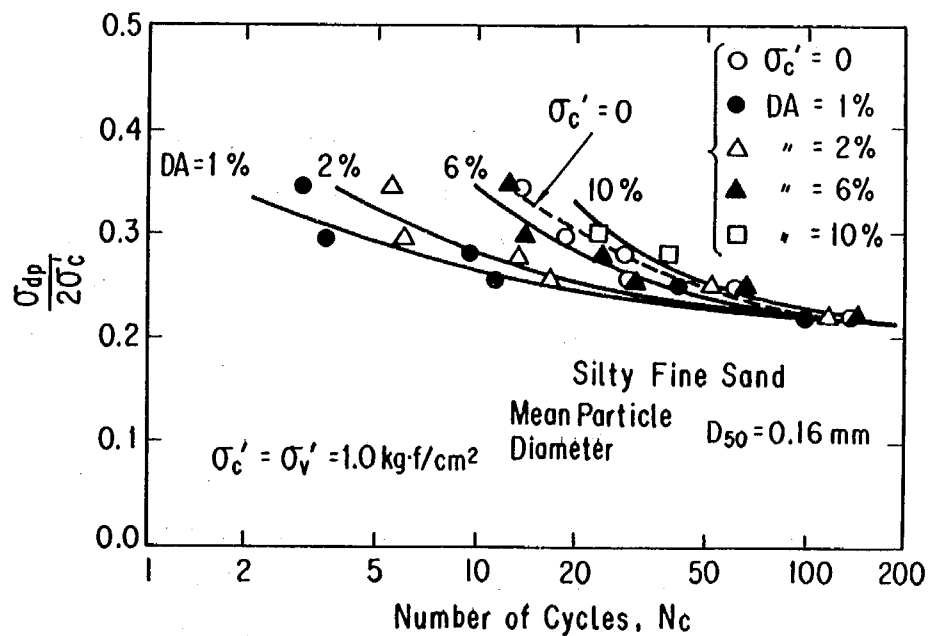


Fig. 40 Example of Dynamic Laboratory Test Results for Obtaining Dynamic Shear Strength of Undisturbed Soil Specimen

effective confining pressure (σ'_c) equal to the estimated in situ effective overburden pressure (σ'_v). The abscissa is the numbers of cyclic loadings (N_c), and the ordinate is one-half of the ratio of the dynamic axial load to the confining pressure ($\sigma_{dp}/2\sigma'_c$). In the figure, $N - \sigma_{dp}/2\sigma'_c$ relations for four cases when double amplitudes of axial shear strain become 1, 2, 6, and 10%, and also for the case when the pore-water pressure (Δu) becomes equal to the effective confining pressure (σ'_c) are shown. Cyclic shear strength of the soil specimen directly obtained from the cyclic triaxial tests (R_ℓ) is here defined as

$$R_\ell = \frac{\sigma_{dp}}{2\sigma'_c} \quad \text{for } N_c = 20 \text{ and } DA = 6\% \quad (4)$$

which is the stress ratio $\sigma_{dp}/2\sigma'_c$ when the number of loading cycles is 20 for the double amplitude (DA) of shear strains of about 6%, (see Fig. 40). The state of $DA = 6\%$ is close to that for the excessive pore-water pressure (Δu) is almost equal to the effective confining pressure (σ'_c). Accordingly the shear resistance at that state decreases entirely, and the soil element completely liquefies.

After knowing the triaxial cyclic shear strength (R_ℓ) defined above, in situ cyclic shear strength (R) can be obtained by

$$R = c_1 \cdot c_2 \cdot c_3 \cdot c_4 \cdot c_5 \cdot R_\ell \quad (5)$$

where c_1 to c_5 are correction factors accounting for several effects involved.

First of all, c_1 is the correction factor accounting for the difference between the in situ confining pressure and that for the triaxial test. Ishihara and Li (1972)²⁵ have proposed that $c_1 = (1 + 2K_0)/3$, where K_0 is the in situ stress ratio of lateral confining pressure to vertical confining pressure. For alluvial deposits and reclaimed land in which the value of K_0 can be considered about 0.5, the value of c_1 takes 2/3.

c_2 in Eq. (5) is the correction factor accounting for the difference between the in situ random loading pattern during actual earthquakes and the sinusoidal loading pattern normally used in the triaxial test. Ishihara and Yasuda (1975)²⁶ have proposed that the values of c_2 are 1/0.55 (for the case of smooth acceleration input) to 1/0.7 (for the case of shock-type acceleration input), and the average value of c_2 is 1.62.

c_3 in Eq. (5) is the correction factor for the effects of soil disturbances in the process of sampling and handling of specimens from the in situ to the laboratory. c_4 is the correction factor for the effects of densification in the process of sampling and handling. In the study by the authors, the product of c_3 and c_4 was assumed to be unity, because it was considered that the two effects may cancel out for relatively loose sands.

c_5 in Eq. (5) is the correction factor accounting for multi-direction shaking which can be anticipated in actual sites during earthquakes. Seed (1976)²⁷ estimated the factor of c_5 to be 0.9.

Using the values of c_1 through c_5 described above, Eq. (5) becomes

$$R = \frac{2}{3} \times 1.6 \times 1.0 \times 0.9 R_\ell \rightleftharpoons R_\ell \quad (6)$$

It is indicated from Eq. (6) that the in situ cyclic strength (R) can be nearly equal to the shear strength obtained by the undrained cyclic triaxial test (R_ℓ) for undisturbed specimens sampled from the field.

After Tatsuoka, et al. (1978)²⁸, Fig. 41 shows the relationship between the mean diameter (D_{50}) and the cyclic shear strength (R_ℓ), obtained from laboratory undrained triaxial tests for a number of undisturbed specimens taken from ten sites shown in Table 5. The ordinate in the figure is not R_ℓ but

$$DR_\ell^* = R_\ell - 0.0042 D_r^* \quad (7)$$

where

$$D_r^* = 21 \sqrt{\frac{N}{\sigma_v' + 0.7}} \quad (8)$$

is the empirical equation proposed by Gibbs and Holtz (1957)²⁹ for obtaining the relative density. In Eq. (8), N is the number of blows by the standard penetration test (SPT), and σ_v' is the effective overburden pressure (in kg.f/cm²). It is seen from Fig. 41 that DR_ℓ^* can be approximately expressed by the function of D_{50} by

$$DR_\ell^* = \begin{cases} 0.225 \log_{10} \frac{0.35}{D_{50}} & (\text{for } 0.02 \leq D_{50} \leq 0.6 \text{ mm}) \\ -0.05 & (\text{for } 0.6 \leq D_{50} \leq 2.0 \text{ mm}) \end{cases} \quad (9)$$

Table 5 List of Sand Samplings and Dynamic Triaxial Tests

(See NOTE)

Site	Sand Sampling	Void Ratio Measurement	e_{min} Measurement	e_{max} Measurement	Dynamic Triaxial Test Condition				References
					σ_v' (kg/cm ²)	Frequency (Hz)	DA for R ¹ (%)	Specimen (øcm, hcm)	
Site A (in Tokyo, along Tokyo Bay) Alluvial and Reclaimed	Modified Bishop-type sand sampler; the liner is 5.3 cmø and 65 cmø*	e_f ; estimated in situ void ratio by Eq. (3)	Tamping mold with air-dry sand** (0.64)	Lightly pouring air-dry sand into mold** (0.96)	$\sigma_v' = 0.2 \sim 1.15$	1.0	6	5ø, 10h	This Investigation *) Hanzawa and Matsuda (1977) **) Yoshimi and Tohno (1972)
Site B (in Shikoku, along Setonaikai Sea) Alluvial and Reclaimed	Twist sand sampler; the liner is 7 cmø and 80 cmø***)	e_f' ; average void ratio in the liner before freezing	Identical to Site A		$\sigma_v' = 0.45 \sim 1.75$	0.5	5	7ø, 14h	This Investigation ***) Ogura et al. (1978)
Site C (in Niigata City, along Shinano River) Alluvial and Reclaimed	Large diameter sand sampling; the liner is 20 cmø and 100 cmø	e_0 ; void ratio of thawed triaxial specimen at $\sigma_0' = 0.2$ kg/cm ²	Vibrating mold using air-dry sand on shaking table with 1 kg/cm ² being applied on the top (0.61)	Spooning air-dry sand into mold (0.96)	1.5 ($\sigma_v' = 0.3 \sim 1.15$)	1.0	5	5ø 10h	Ishihara (1976, 1977) Ishihara and Silver (1977)
Site D (in Tokyo) Alluvial	Identical to Site C				0.5 ($\sigma_0' = 0.265 \sim 0.610$)	1.0	5	5ø, 10h	Ishizawa, Nakagawa and Kurohara (1977)
Site E (in Yokohama, along Tokyo Bay) Reclaimed	Identical to Site A	—	—	—	$\sigma_v' = 0.37 \sim 0.77$	1.0	6	5ø, 10h	This Investigation
Site F (in Tokyo, along Tokyo Bay) Alluvial	Thin-wall sampling with enough cares	—	—	—	0.5, 1.0 ($\sigma_v' = 0.21 \sim 0.65$)	1.0	6	5ø, 10h	This Investigation
Site G (in Tokyo along Tokyo Bay) Alluvial and Reclaimed	Identical to Site A				$\sigma_v' = 0.8 \sim 1.14$	0.5	6	5ø, 10h	This Investigation
Site H (Kawagishicho, Niigata City) Alluvial and Reclaimed	Identical to Site A	e_f	Tamping mold using air-dry sand and applying pressure of 1 kg/cm ² on the top (0.60)	Average by two method: Kolbuszewski (1948) and Tanimoto (1975) (0.94)	—	—	—	—	JSSMFE (1976)
Site I (Oh-gi Shima) Reclaimed	Identical to Site A	e_f	Identical to Site H		—	—	—	—	Saito (1977)
Site J (in Yokohama, along Tokyo Bay) Reclaimed	Frozen Column Method; making frozen 5mø, 40 cmø large column in ground	Void ratio of frozen specimen	The Yoshimi-and-Tohno method (0.62) (0.98)		—	—	—	—	Yoshimi, Hatanaka and Oh-oka (1977) Hatanaka (1977)

(NOTE) The numbers in () represent the maximum and minimum void ratios of Toyoura Sand determined by each method.

After Tatsuoka, Iwasaki, Tokida, Yasuda, Hirose, Iwai, and Kon-no (1978)28)

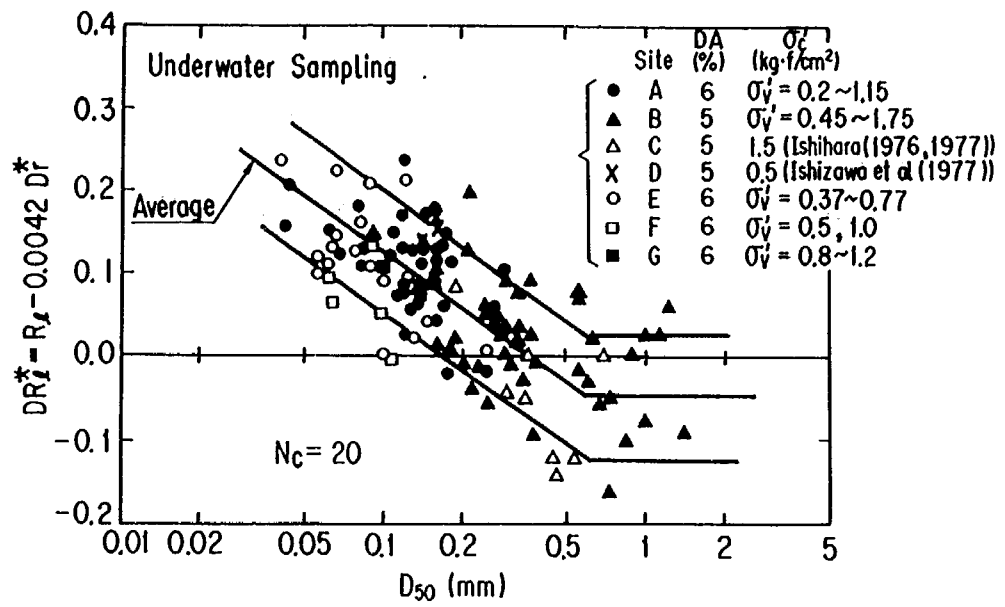


Fig. 41 Relationship between Mean Particle Diameter of Soils (D_{50}) and Cyclic Shear Strength (R_d)

From Eqs. (7), (8) and (9), cyclic triaxial shear strength (R) can be expressed by

$$R_{\ell} = \begin{cases} 0.0042 D_r^* + 0.225 \log_{10} \frac{0.35}{D_{50}} & (\text{for } 0.02 \leq D_{50} \leq 0.6 \text{ mm}) \\ 0.0042 D_r^* - 0.05 & (\text{for } 0.6 \leq D_{50} \leq 2.0 \text{ mm}) \end{cases} \quad (10)$$

or

$$R_{\ell} = \begin{cases} 0.0882 \sqrt{\frac{N}{\sigma_v' + 0.7}} + 0.225 \log_{10} \frac{0.35}{D_{50}} & (\text{for } 0.02 \leq D_{50} \leq 0.6 \text{ mm}) \\ 0.0882 \sqrt{\frac{N}{\sigma_v' + 0.7}} - 0.05 & (\text{for } 0.6 \leq D_{50} \leq 2.0 \text{ mm}) \end{cases} \quad (11)$$

The effective overburden pressure (σ_v') (in $\text{kg}\cdot\text{f}/\text{cm}^2$) at a depth of z can be obtained by (see Fig. 39(b))

$$\sigma_v' = \left[\int_0^{h_w} \gamma_t \cdot dz + \int_{h_w}^z \gamma' \cdot dz \right] / 10 \quad (12)$$

where

γ_t : unit weight of soil above the water table ($\text{t}\cdot\text{f}/\text{m}^3$),

γ' : unit weight of soil below the water table ($\text{t}\cdot\text{f}/\text{m}^3$),

$\gamma' = \gamma_t - \gamma_w$ (γ_w : unit weight of water = $1 \text{ t}\cdot\text{f}/\text{m}^3$),

h_w : depth of the water table from the ground surface (m),

z : depth of a soil element from the ground surface (m).

By using Eqs. (11) and (6), in situ cyclic shear strength (R) can be approximately obtained as functions of SPT N-values, effective overburden pressures (σ_v' in $\text{kg}\cdot\text{f}/\text{cm}^2$) and the mean particle diameter (D_{50} in mm).

Estimation of Dynamic Load, L

Dynamic load, L (or maximum shear stress) in a soil element caused by earthquake motions (see Fig. 39(b)) can be expressed by

$$L = \frac{\tau_{\max}}{\sigma_v'} \quad (13)$$

in which $\tau_{\max}$ denotes the maximum single amplitude of shear stress in the soil element, and σ_v' is the effective overburden pressure at the static condition.

After Seed and Idriss (1971)³⁰, the dynamic load can be estimated from the following equation

$$L = \frac{\alpha_{s \cdot \max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \quad (14)$$

in which $\alpha_{s \cdot \max}$ (in gals) denotes the maximum acceleration on the ground surface and g is the acceleration of gravity ($g = 980$ gals), σ_v is the total overburden pressure, σ_v' is the effective overburden pressure, and r_d denotes a reduction factor of dynamic shear stress in soils accounting for the elastic deformation of the ground. The value of $\alpha_{s \cdot \max}$ can be estimated in view of the input bedrock motion and response characteristics of the ground layer. According to Iwasaki, et al. (1978)³¹, the value of $\alpha_{s \cdot \max}$ at a site for an anticipated earthquake with magnitude of M can be obtained by the following equation

$$\alpha_{s \cdot \max} = 18.4 \times 10^{0.302M} \times D^{-0.8} \quad (15)$$

which is the average for the entire strong acceleration data (by SMAC-Type accelerographs) obtained at various ground surfaces in Japan. In Eq. (15), M is the earthquake magnitude on the Richter scale, and D is the epicentral distance in km.

Nextly, Figs. 42(a) and 42(b) illustrate typical results of earthquake response analysis for two alluvial deposits. The analysis was done with the use of the computer program "SHAKE" developed by Schnabel, Lysmer, and Seed (1972)³², by inputting several acceleration records triggered during actual earthquakes in Japan. It can be seen from the figures that the value of r_d is generally smaller for input motions with shorter predominant periods (T_e), and also for grounds with shorter natural periods (T_g). It is estimated that T_e is a function of earthquake magnitude (M), epicentral distance (D), and ground conditions, and that T_g is a function of thickness and rigidity of subsurface soils above the bedrock. From a number of earthquake response analyses for various ground layers, the following equation is proposed for simply estimating r_d

$$r_d = 1 - 0.015z$$

(16)

in which z is the depth (in m).

Case Study of Liquefaction Resistance Factor, F_L

Liquefaction resistance factor (F_L) defined above was examined for the case of the Niigata Earthquake of 1964. Fig. 43 shows a liquefaction occurrence map in Niigata City during the Niigata Earthquake of 1964. The maximum horizontal acceleration recorded at the ground surface in the city was about 170 gals. Table 6 lists 23 points (all in Niigata City) for which the liquefaction resistance factors (F_L) are computed. Of 23 points, 17 points were liquefied and 6 points (located close to liquefied sites) were not liquefied. In the table liquefied depths were approximately estimated by either damaged features of structural foundations, or by the comparison of particle characteristics of boiled sands and those of boring logs. Figs. 44 and 45 show the distributions of F_L with respect to depth at two sites (Points 23 and 4 in Table 6) where liquefaction did not take place. It can be seen from the figures that the values of F_L are larger than unity. On the other hand, Figs. 46 and 47 indicate the distributions of F_L at two other sites (Points 2 and 5 in Table 6) where severe liquefaction took place. It can be seen from the figures that the values of F_L are generally smaller than 1.0.

Fig. 48 summarizes the results of the analysis for the 23 points in Niigata City. Data points of small black circles correspond to soils where liquefaction took place. On the other hand, data points of open circles correspond to soils where liquefaction did not occur. It can be understood from Fig. 48 that most of data points with F_L -values less than unity are those for liquefied zones and that most of data points with F_L -values greater than unity are those for non-liquefied zones. This tendency is especially obvious for the depth shallower than 10 m. Furthermore, Fig. 49 illustrates the frequencies and the cumulative incidences of F_L -values calculated for both liquefied layers and non-liquefied layers at 87 points: (64 liquefied points and 23 non-liquefied points located close to liquefied points -- see Tables 7 and 8). It is found from the figure that the distribution characteristics of F_L at the liquefied layers are obviously different from those at non-liquefied layers. At the liquefied layer most (about 87 percent) of F_L -values distribute in the range less than 1.0, while at the non-liquefied layers most (about 85 percent) of F_L -values distribute in the range higher than 1.0. However, it should be also noticed that 13 percent of F_L -values exceed 1.0 at

Table 6 Characteristics of 23 Points Where Liquefaction Resistance Factors (F_L) are Computed, using The Simplified Method (All Points are Located in Niigata City)

POINT NO.	NAME OF SITE AND BORING POINT		LIQUEFIED OR NON-LIQUEFIED*	LIQUEFIED DEPTH ESTIMATED	
1	Showa Bridge	Boring 1	L	10.5 m	Heavy damage to Bridge Piers
2		Boring 2	L	10.5 m	
3		Boring 3	L	10.5 m	Light Damage
4		Boring 4	NL	0	
5	Niigata Railway	Boring 1	L	15 m	
6	Hospital	Boring 2	L	15 m	
7	Niigata Airport		L	10 m	
8	Sekiya		L	11.5 m	
9	Shinano R. Railway Bridge	Boring 1	L	6.5 m	
10	Bandai Bridge	Boring 6	L	9 m	
11	Higashi-Kosen Bridge	Boring 2	L	8 m	Heavy damage to Bridge Piers
12		Boring 4	L	5 m	
13		Boring 5	NL	0	No Damage
14	Yachiyo Bridge	Boring 1	L	12 m	
15		Boring 5	L	10 m	
16		Boring 7	L	10 m	
17	Shin-Matsuhama Bridge	Boring 1	NL	0	
18		Boring 2	L	10 m	
19	Taihei Bridge	Boring 1	L	10 m	Site (Surface to 2 m)
20		Boring 2	L	5 m	
21	Shindoji		NL	0	
22	Ohmiya		NL	0	
23	Nishi-Ohata-Cho		NL	0	

*L: Liquefied, NL: Non-Liquefied

Table 7 Characteristics of 64 Liquefied Points Where P_L -Values are Computed

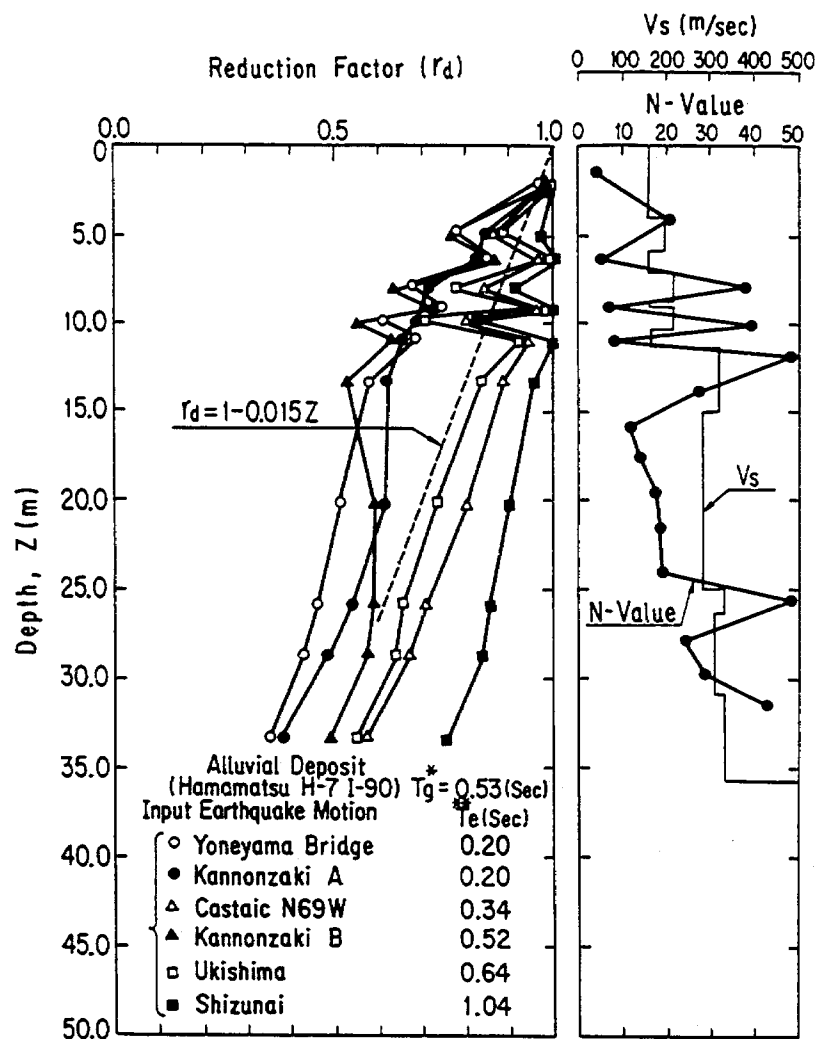
POINT NO.	LOCATION OF POINT		EARTHQUAKE	SURVEY TIME	WATER TABLE	D ₅₀	SOILS	A _{max}	P _L
L1	Shinano R.	Bo. 1	Niigata 1964 M = 7.5	A	2 m	M	FS-MS	170 gals	9.0
L2	Railway Bridge	Bo. 2		A	2.5	M	"		24.9
L3	Higashi-Kosen	Bo. 1		A	0.5	E.M	FS-CS		5.8
L4	Bridge	Bo. 2		B	2.5	M	"		1.3
L5		Bo. 4		B	2.5	M	"		0.6
L6	Bandai Bridge	Bo. 6		A	0	M	MS		20.2
L7		Bo. 1		B	0	E.M	FM-MS		18.3
L8	Yachiyo Bridge	Bo. 5		B	0	E.M	"		38.5
L9		Bo. 7		B	0	E.M	"		26.2
L10	Shin-Matsushima Br.	Bo. 2		A	1.56	M	MS-CS		24.0
L11	Taihei Bridge	Bo. 1		A	0	M	CS		32.9
L12		Bo. 2		A	0	M	"		19.6
L13		Bo. 1		A	0	E	"		28.2
L14		Bo. 2		A	0	M	MS-CS		16.5
L15	Showa Bridge	Bo. 3		A	0	M	"		14.3
L16		Bo. 2		B	0	E	"		22.7
L17		Bo. 3		B	0	E	"		23.2
L18	Niigata Airport			B	0.8	E	"		13.1
L19	Sekiya			A	0.6	M	CS		28.6
L20		Bo. 1	Tokachi- oki 1968 M = 7.9	B	1.4	M	FS	200 gals	20.1
L21	Niigata Railway	Bo. 2		B	1.3	M	FS-MS		5.8
L22	Hospital	Bo. 1		A	0.63	M	FS		13.5
L23		Bo. 2		A	1.18	M	"		10.5
L24		Bo. 1		A	1.25	M	CS		18.9
L25		Bo. 2		A	1.25	M	"		20.1
L26	Kawagishi-cho	Bo. 3		A	1.25	M	"		18.9
L27		Bo. 4		A	1.25	M	"		14.5
L28		BC21-2		B	1.2	E	MS		5.1
L29	Kawagishi-cho	BC21-3		B	1.2	E	"		9.7
L30		BC104		A	1.2	E	"		14.2
L31		BC14		A	1.35	E	FS-MS		4.4
L32	Nanaehama	Bo. 1	Hakodate City	A	0.95	M	FS	200 gals	0
L33	Reclaimed	Bo. 2		A	0.95	M	"		0
L34	Land	Bo. 3		A	1.1	M	"		5.8
L35	Hachinohe Paper Co.		Hachinohe	M = 7.9	A	0.57	E	"	235
L36	Gifu City		Nobi 1891 M = 8.0	A	0.9	M	MS	255	36.1
L37	Unuma	Kagamigahara C.		A	0.75	M	S/Gra	210	13.0
L38	Ogaseike			A	2.1	M	"		8.8
L39	Mangoku-cho, Ogaki C.		Tonankai 1944 M = 8.0	A	1.2	M	S-Gra	270	28.7
L40	Meikodori, Minato-ku			A	0.6	M	MS	200	19.5
L41	Komei-cho, Minato-ku			A	0.9	M	CS		26.3
L42	Inaei Shinden			A	0.23	M	SIS-FS		17.7
L43	Takaya-cho 45		Fukui 1948 M = 7.3	A	4.2	M	MS-CS	325	22.7
L44	Maruoka-cho 2			A	1.8	M	MS-Gra	315	24.9
L45	Takaya-cho 2-168			A	4.2	M	SSI-MS	325	11.4
L46	Abukuma Bridge	Bo. 4	Miyagi- ken-oki 1978 M = 7.4	A	0	M	CS	175	12.2
L47	Estuary of Abukuma R.			B	0	E	"	180	20.4
L48	Yuriage-kami,	Y-1		A	1.82	M	SIS-CS		12.3
L49	Natori R.	Y-2		A	0.85	M	FS-CS		9.4
L50		No. 1		A	1.7	M	FS-MS	185	7.0
L51	Yuriage Bridge	No. 2		A	1.3	M	MS-CS		0.5
L52		No. 3		A	0.26	M	"		21.8
L53	Yamazaki Dike, Yoshida R.			A	0.87	E	SSI-MS	190	15.5
L54	Oiri, Old	No. 1		A	4.3	M	ClS-MS	210	4.1
L55	Kitakami R.	No. 2		A	2.4	M	"		4.1
L56	Uomatchi,	B-1		B	0	E	MS-SIS	230	41.7
L57	Ishinomaki	B-2		B	0	E	"		37.9
L58	Rifu Marshaling Y.	No. 12		B	2.72	E	ClS-CS		22.2
L59		No. 1		B	0	E	FS-MS	225	14.6
L60	Shiomi Pier,	No. 2		B	0	E	FS		28.5
L61	Ishinomaki	No. 3		B	0	E	"		20.3
L62	Tanetsugu Dike,	N-4		A	0.5	M	SSI-MS	180	12.3
L63	Natori R.	N-5		A	1.3	M	FS-CS		5.6
L64	Wabuchi, Eai R.	W-2		A	2.45	M	SIS-FS	295	21.2

- Notes: 1) Survey Time means the time of insitu soil survey. A is after the earthquake, and B is before the earthquake.
2) Water Table denotes the depth of water table below ground surface.
3) D₅₀ is mean particle diameter of soil; M is measured, and E is estimated.
4) For Soils, FS=fine, MS=medium sand, CS=coarse sand, SIS=silly sand, SSI=sandy silt, Gra-gravel, ClS=clay, and ClS=clayey sand.
5) A_{max} denotes the maximum ground surface acceleration. The values are either measured or estimated by an empirical equation.

Table 8 Characteristics of 23 Non-Liquefied Points Where P_L -Values are Computed

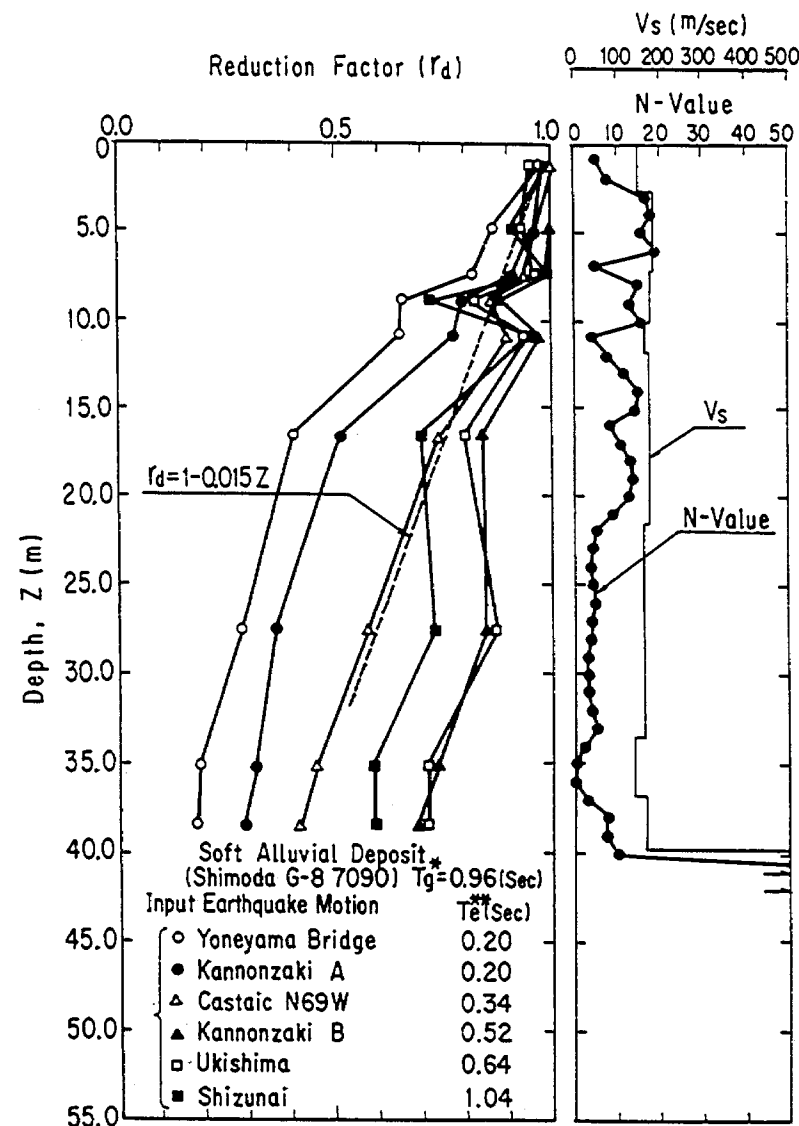
POINT NO.	LOCATION OF POINT			EARTHQUEKE	SURVEY TIME	WATER TABLE	D ₅₀	SOILS	A _{max}	P _L
N1	Shindoji		Niigata City	Niigata 1964 M=7.5	A	2.3 m	E	FS		0
N2	Kogane-cho				A	5.2	E	MS		0
N3	Higashi-Kosen Bridge, Bo. 5				B	2.5	M	FS-CS		0
N4	Shin-Matsuhama Bridge, Bo. 1				A	1.2	M	MS-CS	170 gals	18.6
N5	Ohmiya 266K712M				A	0	M	FS		8.2
N6	Showa Bridge, Bo. 4				A	0	E	MS		4.6
N7	Nishi-Ohata-cho				A	3.5	E	FS-MS		0.4
N8	Gotanda Bridge Bo. 1	Kamo Town	A	0.1	M	FS-SiS		5.3		
N9	Bo. 2		A	4.3	M	FS-Si		0		
N10	Farmers Union, Maruoka	Fukui P.	Fukui, 1948 M=7.3	A	1.8	M	Si-MS	295	18.7	
N11	Tantetsugu D'ke N-1	Miyagi Pref.	Miyagi-ken-oki 1978 M=7.4	A	0.85	M	FS-MS	180	0.3	
N12	Natori River N-2			A	0.90	M	SiS-CS		1.0	
N13	Yuriage-kami, Natori R. Y-3			A	21.5	M	"		0.9	
N14	Right Estuary, Kitakami, R. Bo. 10			A	1.55	E	FS	230	2.3	
M15	Right 3.2K, Natori R.			A	2.50	E	ClS-CS	180	0.8	
M16	Kin-noh Bridge P8			A	4.0	M	Si-SiS	195	0.6	
N17	Abukuma Bridge Bo. 1			A	4.3	M	MS-CS	175	0	
N18	Bo. 2			A	3.4	M	MS-CS		0.7	
N19	Eai Bridge No. 1			A	8.0	E	FS-Cl		0	
N20	Minami Sendai No. 2			B	0.8	E	SSi-Gra	180	0	
N21	Uomachi, A-1			B	0	E	MS-SiS	230	13.9	
N22	Ishinomaki A-2			B	0	E	MS-SiS		17.1	
N23	Wabuchi, Eai R. W-3			A	3.35	M	FS-MS	260	3.7	

Note: Refer to footnotes of Table 7.



* First Natural Period of Surface Deposit above Bed Rock
 ** Predominant Period of Earthquake Motion at the Surface of Bed Rock

Fig. 42(a) Earthquake Response Analysis of Alluvial Deposit at Hamamatsu City (Reduction Factor, r_d)



* First Natural Period of Surface Deposit above Bed Rock
 ** Predominant Period of Earthquake Motion at the Surface of Bed Rock

Fig. 42(b) Earthquake Response Analysis of Alluvial Deposit at Shimoda City (Reduction Factor, r_d)

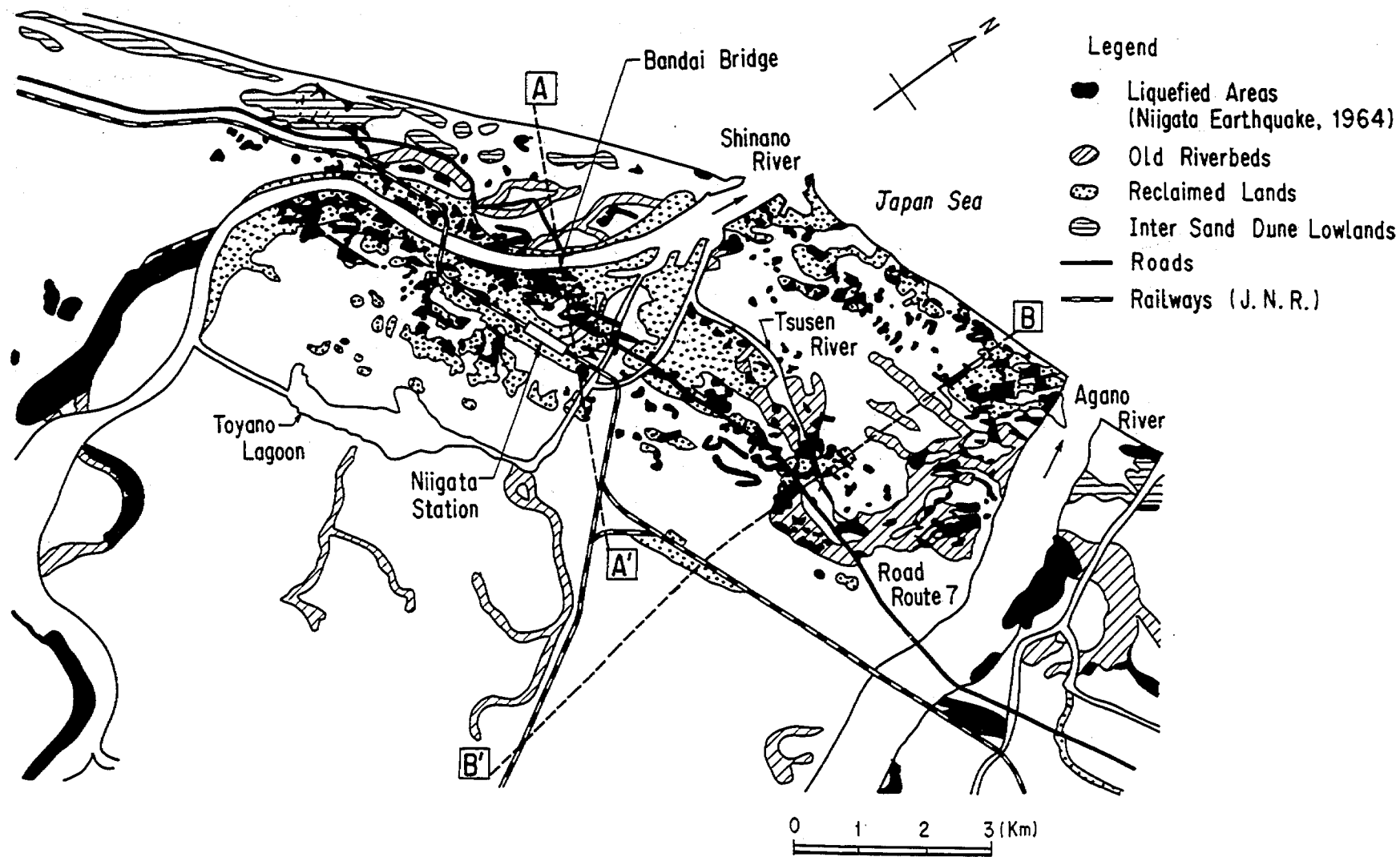


Fig. 43 Liquefied Areas in Niigata City during the Niigata Earthquake of 1964

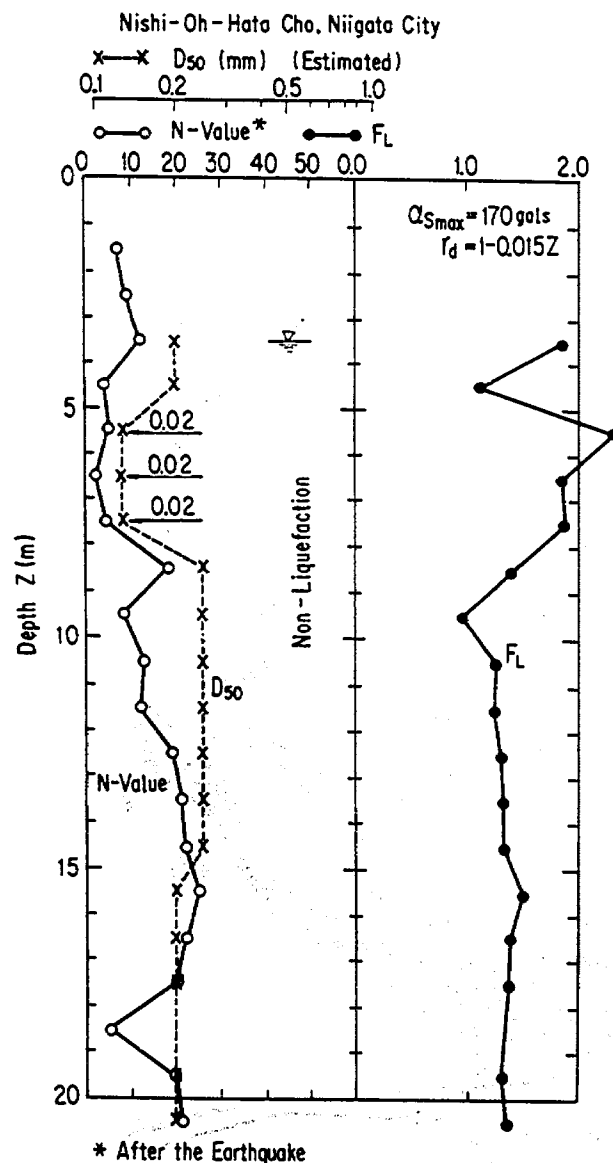


Fig. 44 Distribution of F_L -Values with Respect to Depth at Nishi-Ohata-Cho (Point 23 of Table 6-Non-Liquefied)

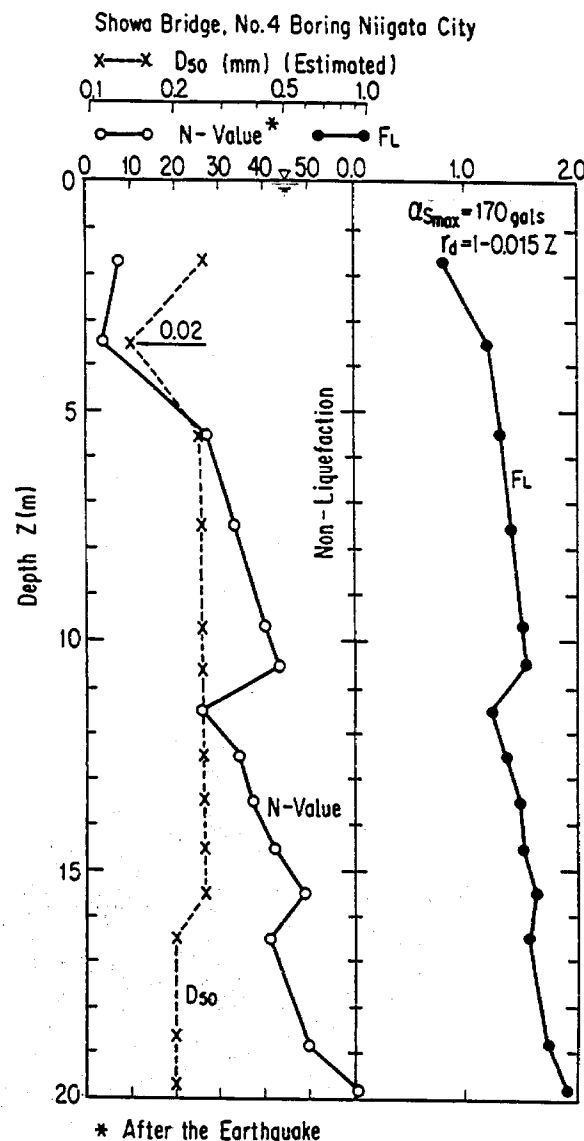


Fig. 45 Distribution of F_L -Values with Respect to Depth at Boring 4 of Showa Bridge (Point 4 of Table 6-Non-Liquefied)

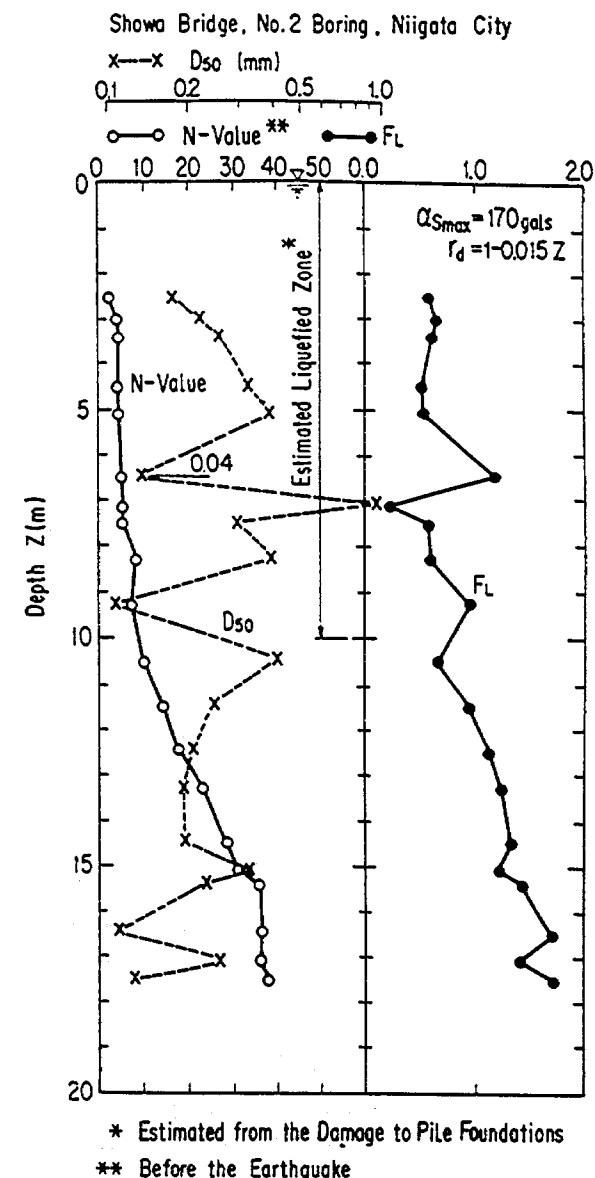


Fig. 46 Distribution of F_L -Values with Respect to Depth at Boring 2 of Showa Bridge (Point 2 of Table 6-Liquefied)

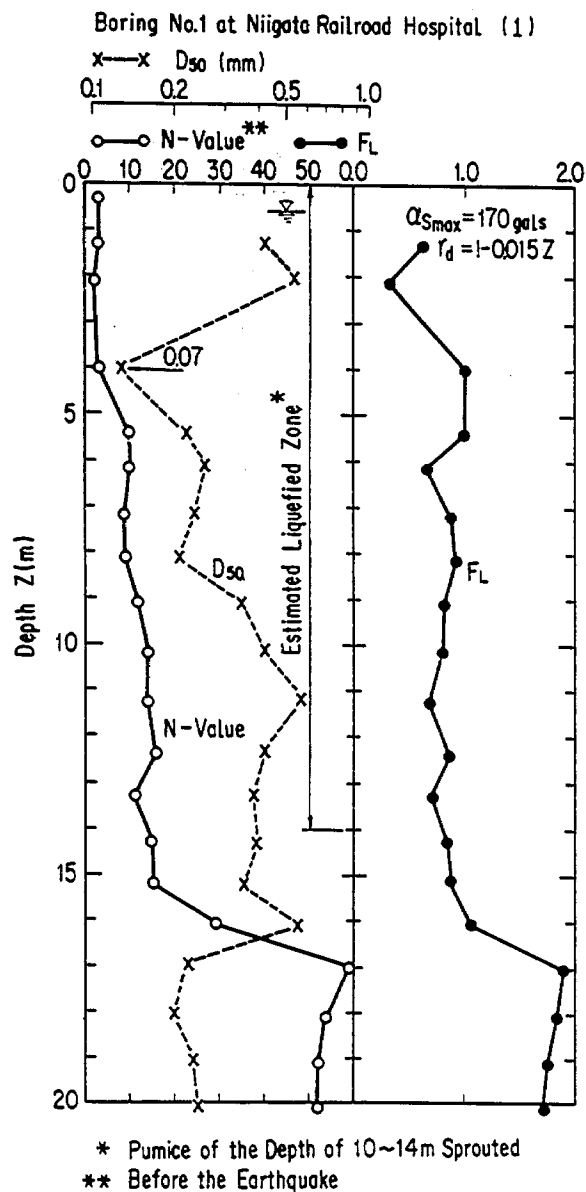


Fig. 47 Distribution of F_L -Values with Respect to Depth at Boring 2 of Niigata Railway Hospital (Point 6 of Table 6-Liquefied)

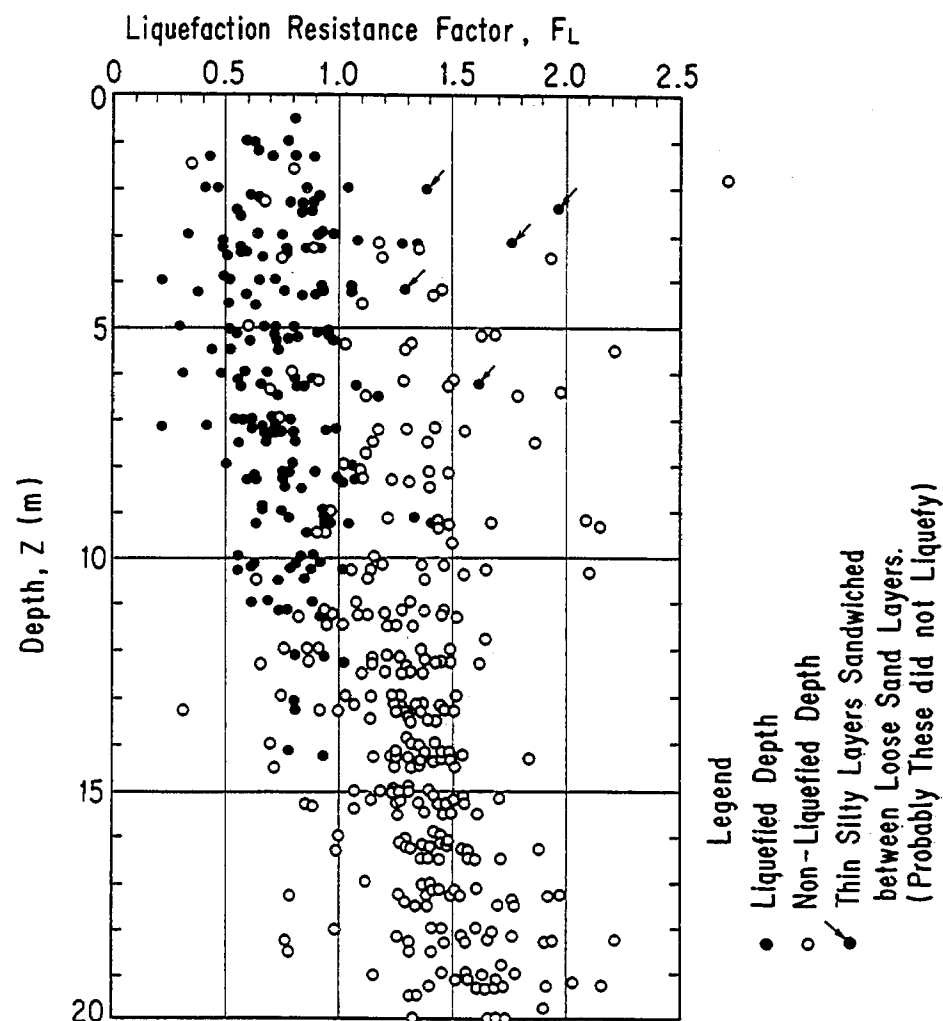


Fig. 48 Summary of Analysis of F_L -Values at 23 Points in Niigata City

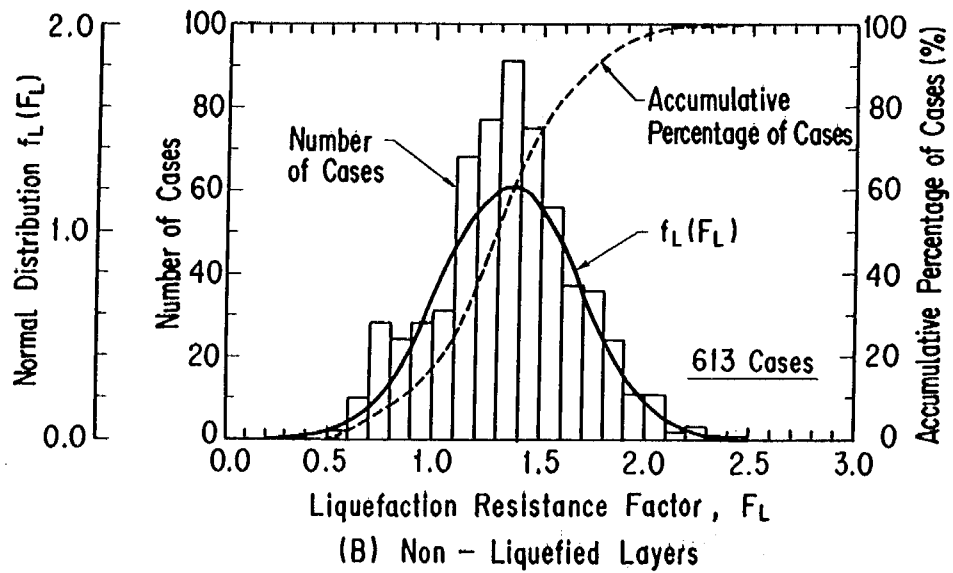
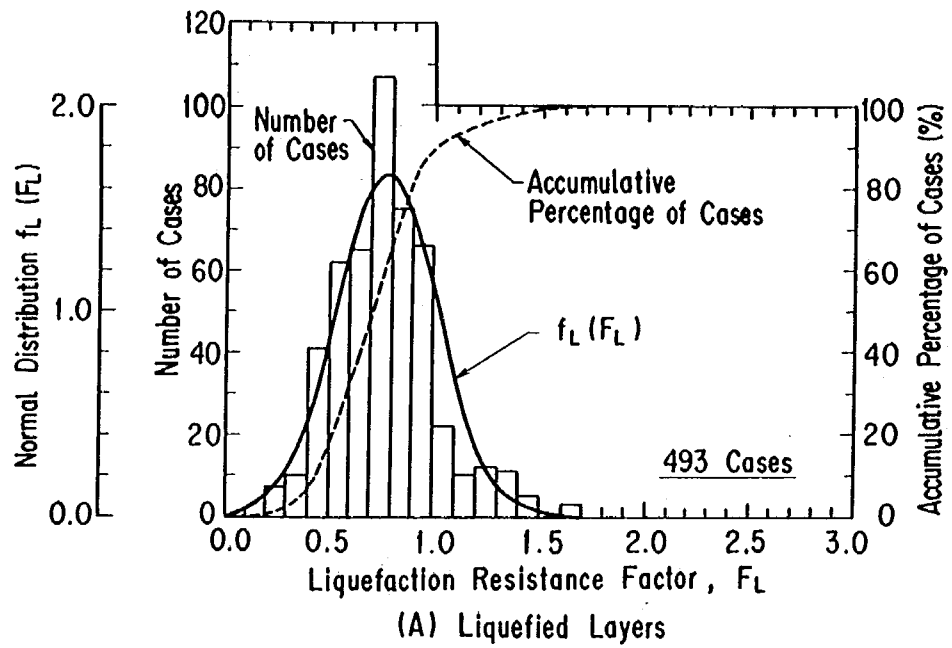


Fig. 49 Frequencies and Accumulative Incidences of F_L -Values Calculated for both 64 Liquefied Sites and 23 Non-Liquefied Sites

the liquefied layers and 15 percent of F_L -values become less than 1.0 at the non-liquefied layers. From the above study, it can be concluded that the simplified procedure using the liquefaction resistance factor (F_L) proposed above can be employed for approximately evaluating liquefaction potential with respect to depth.

3.3 Liquefaction Potential Index, P_L

Outline of P_L -Procedure

An ability to resist liquefaction at a given depth of a site may be evaluated using the liquefaction resistance factor described in the previous section. It should be noticed, however, that the damage to structures due to soil liquefaction is considerably affected by the severity of liquefaction extent. In view of this fact, Iwasaki, Tatsuoka, Tokida, and Yasuda (1978)²⁴ also proposed the liquefaction potential index (P_L) defined by Eq. (17) to estimate the severity of liquefaction extent at a given site for a seismic motion.

$$P_L = \int_0^{20} F \cdot w(z) dz \quad (17)$$

where $F = 1 - F_L$ for $F_L \leq 1.0$ and $F = 0$ for $F_L > 1.0$, and $w(z) = 10 - 0.5z$ (z : depth in m), as shown in Fig. 50. The function $w(z)$ is introduced to account for the extent of soil liquefaction according to the depth. The triangular shape of $w(z)$ and the depth of 20 meters are decided with the consideration of liquefaction phenomena during the past earthquakes. For the case of $F_L = 0$ for the entire depth, P_L becomes the maximum of 100; and for the case of $F_L \geq 1.0$ for the entire depth, P_L is the minimum of zero. For general cases P_L will take values between 0 and 100.

Case Study of Liquefaction Potential Index, P_L

The liquefaction potential index (P_L) described in the above is calculated for 64 liquefied points and 23 non-liquefied points where appropriate geotechnical information is available. In the analysis are considered six great earthquakes: the Nobi Earthquake of 1891 (Richter magnitude, $M = 8.0$), the Tonankai Earthquake of 1944 ($M = 8.0$), the Fukui Earthquake of 1948 ($M = 7.3$), the Niigata Earthquake of 1964 ($M = 7.5$), the Tokachi-oki Earthquake of 1968 ($M = 7.9$), and the Miyagi-ken-oki Earthquake of 1978 ($M = 7.4$). The character-

istics of the 64 liquefied points and 23 non-liquefied points (located close to the liquefied points) analyzed in this case study are summarized in Tables 7 and 8, respectively.

Fig. 51 summarizes the relation between P_L and the number of cases, and the relation between P_L and the accumulative percentages at all liquefied and non-liquefied points shown in Tables 7 and 8. It is found from this figure that P_L -values for liquefied points are higher than those for non-liquefied points. For non-liquefied points P_L -values are mostly less than 15, and the percentage of P_L that is less than 5 is about 70%. On the other hand, for liquefied points the percentage of P_L that is less than 5 is about 20%, and for about half of the liquefied points P_L -values are greater than 15. From these results, a simplified procedure for assessing soil liquefaction in terms of the liquefaction potential index P_L may be proposed as follows:

- $P_L = 0$: Liquefaction risk is very low,
- $0 < P_L \leq 5$: Liquefaction risk is low,
- $5 < P_L \leq 15$: Liquefaction risk is high, and
- $P_L > 15$: Liquefaction risk is very high.

As mentioned in the above, the liquefaction potential index defined by Eq. (17) may be employed in assessing the liquefaction potential for a given site.

For all the 64 liquefied sites and the 23 non-liquefied sites, the relationship between the total thickness of saturated sandy layers (H_s) and the underground water table from the surface (H_w) or the thickness of soil layers from the surface (H_0) which are not vulnerable to liquefaction is obtained in Fig. 52. In the figure, a liquefied site is shown by a black circle, and a non-liquefied site is shown by a white circle, and H_s is taken as 10 meters for cases of H_s of 10 meters or greater. It may be indicated from the figure that the liquefied sites can be approximately separated by a shaded zone from the non-liquefied sites. It is also seen that for most of the liquefied sites, H_s (thickness of saturated sandy layers) is larger than 2 meters and H_w (water table) or H_0 (thickness of soils not vulnerable to liquefaction) is smaller than 2 meters.

In the Appendix of this report, soil profiles, SPT N-values and other data for the 64 liquefied sites and the 23 non-liquefied sites are shown for the reference of the reader. On the bottom of each figure information on settlement of the ground surface caused by earthquakes is also provided.

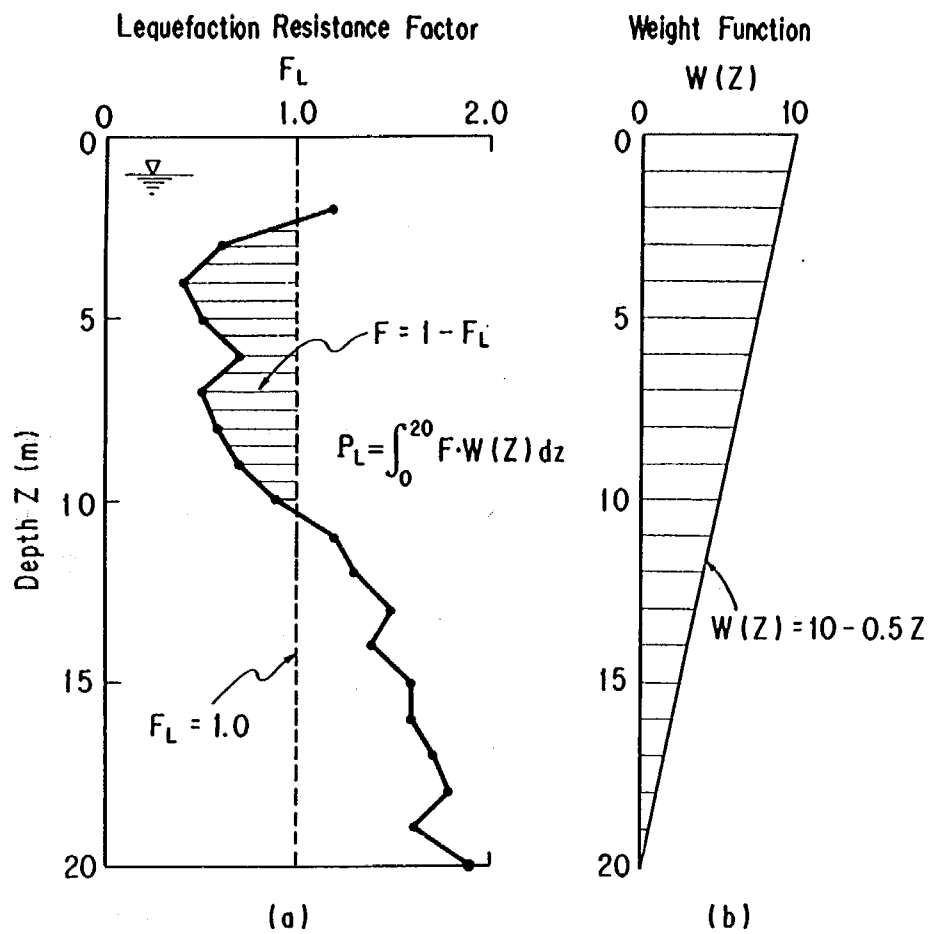


Fig. 50 Integration of $F \cdot W(Z)$

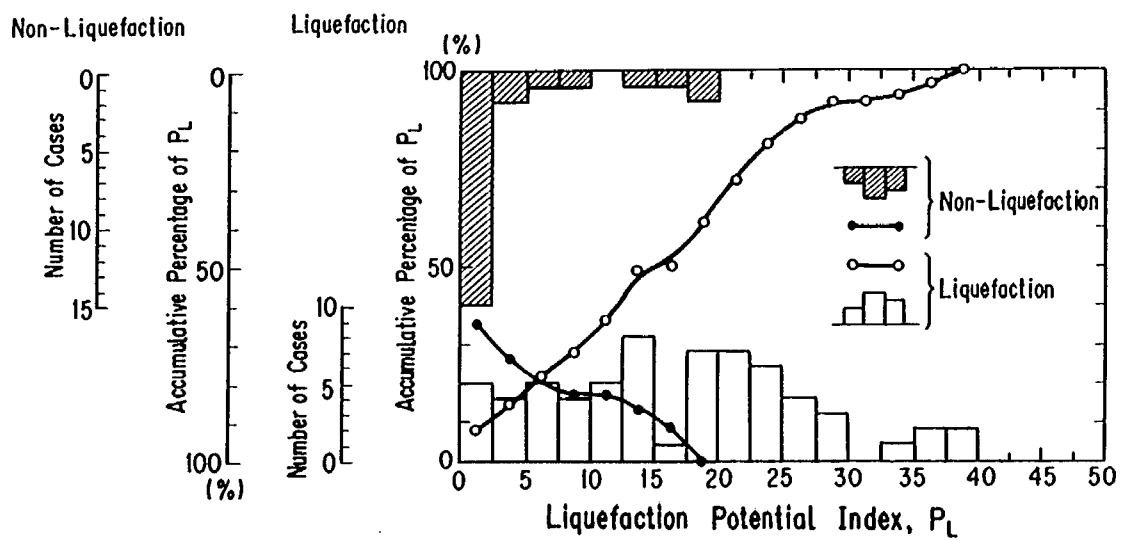


Fig. 51 Relation between P_L -Values and Number of Cases of Liquefied Sites and Non-Liquefied Sites

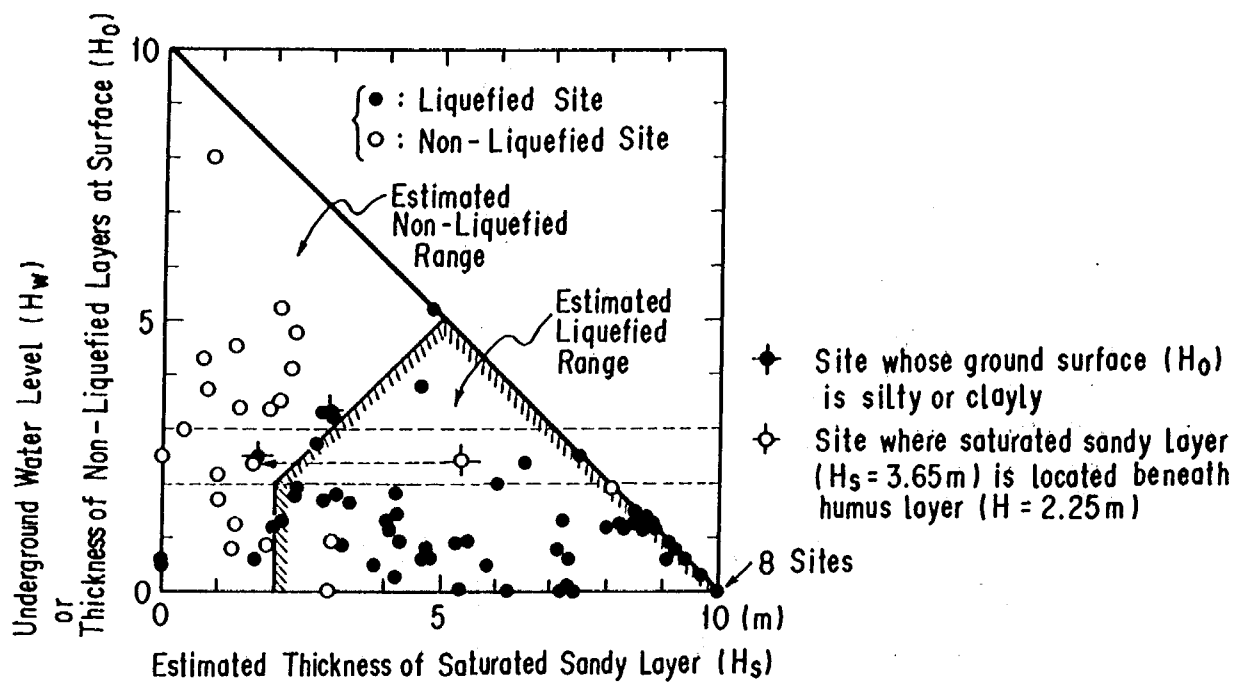


Fig. 52 Relationship between Estimated Thickness of Saturated Sandy Layers (H_s) and Underground Water Level (H_w) or Thickness of Non-Liquefied Layers at Surface (H_o)

4. SHAKING TABLE TESTS ON SOIL LIQUEFACTION EFFECTS

4.1 Outline

Shaking table tests have been carried out to clarify the properties of soil liquefaction, the effects of soil structures on structures, and the effectiveness of the proposed simplified procedure for assessing liquefaction potential.

In this chapter two kinds of shaking table tests conducted at the Public Works Research Institute will be described.

4.2 Behavior of Pile Foundations

A loose saturated sand layer 0.95 meters deep, 6 meters long, and 3 meters wide was prepared on a shaking table which was shaken in a sinusoidal wave. The principal characteristics of the shaking table are shown in Table 9. Four pile foundation models were set up in the sand layer. The frequency of input motion was 7 Hz, and the magnitude of acceleration ranged from 30 to 250 gals. Accelerations and pore-water pressures in the sand layer, accelerations and displacements of the pile foundations, and earth pressures acting on the pile foundations were measured at several points.

Fig. 53 illustrates a set up of the test arrangement. The waterproof sand container is 6 m long, 3 m wide, and 2 m deep and has two rotational hinges at the bottom of the longitudinal end walls in order to facilitate shear deformation of the model grounds. The container also has transparent glass in the two side faces so that the investigators can easily watch the behavior of the models. Table 10 shows input conditions for the shaking table tests. Fig. 54 is an example of test results. The figure shows the distribution of completely liquefied zones for several elapsed times for Test No. 2 of Table 10. In the tests the extent of liquefaction is defined by the ratios of the excess pore-water pressure (Δu) to the effective overburden pressure (σ_v') at a given depth. Zones with the ratio ($\Delta u / \sigma_v'$) of 1.0 can be judged to be completely liquefied. Fig. 55 indicates characteristics of the sand model during tests (Test No. 1-8), showing settlements, void ratios, relative densities, etc., for each stage of the tests.

Fig. 56 illustrates a relationship between displacements at the top of the Model-2 pile and extent of soil liquefaction. In the figure the abscissa denotes the ratios of the thickness (H_L) of the completely liquefied zone to the total soil layer depth ($H_0 = 95$ cm). It may be seen from the figure that

Table 9 Specification of Shaking Table at PWR1

Items	Performances
Table Size	6 m x 8 m (1 table)
Maximum Loading Capacity	100 ton
Maximum Tipping Moment	150 ton-m
Maximum Excitation	100 ton
Maximum Stroke (Horizontal)	±75 mm
Maximum Velocity (Horizontal)	±60 cm/sec
Maximum Acceleration (Horizontal)	±700 gal: Under Maximum Loading ±2,500 gal: Under No Loading
Direction	Two Directions (One Horizontal and Vertical) Simultaneously
Control of Input Motions	Analog Control and Digital Control
Excitation Motions	Sinusoidal Waves, Triangular Waves, Rectangular Waves, Earthquake Waves
Control Item	Acceleration Velocity Displacement
Frequency Range	DC ~ 30 Hz
Control System	Electro-Hydraulic Servo System

Table 10 Input Conditions of Shaking Table

Test No.	Input Motion	Input Acceleration (gal)
1	Sinusoidal 7 HZ	30
2		80
3		80
4		50
5		80
6		150
7		150
8		250

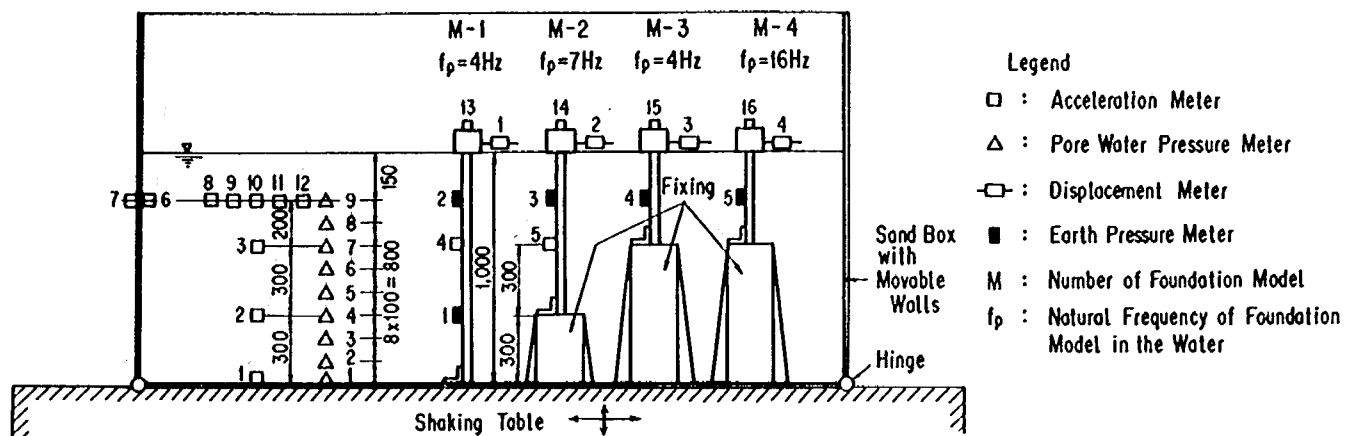


Fig. 53 Outline of Test Models and Measurement System

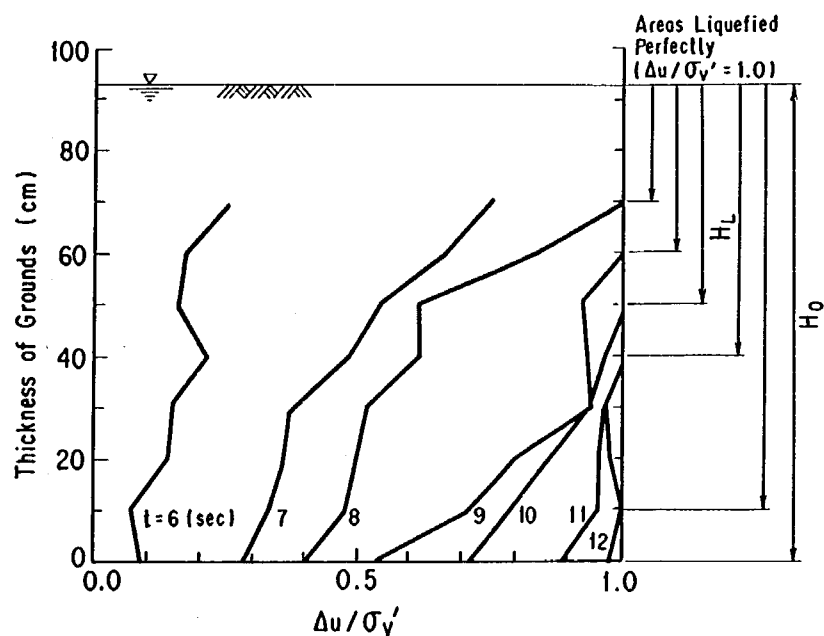


Fig. 54 Time History of Relation between Rate of Liquefaction ($\Delta u / \sigma_v'$) and Thickness

Test No.	Stage	Average Settlement Ratio (%)	Void Ratio e				Relative Density D_r (%)					Dry Unit Weight γ_d (g/cm ³)		Wet Unit Weight γ_t (g/cm ³)	
		2 4 6	0.6	0.7	0.8	0.9	40	50	60	70	80	1.4	1.5	1.9	2.0
0	Before Shaking														
1	After Shaking of 30gal														
2	After Shaking of 80gal														
3	After Shaking of 80gal														
4	After Shaking of 50gal														
5	After Shaking of 80gal														
6	After Shaking of 150gal														
7	After Shaking of 150gal														
8	After Shaking of 250gal														

* Average Settlement Ratio = (Surface Settlement) x 100 / (Total Thickness)

Fig. 55 Characteristics of Sand Model during Tests

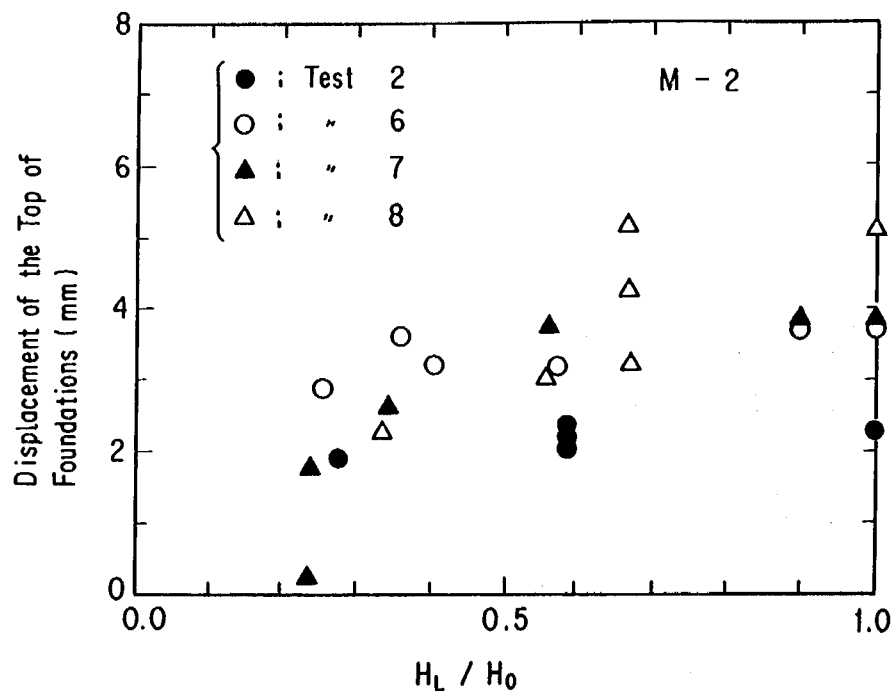


Fig. 56 Relation between Top Displacement of Foundation Model and Degree of Soil Liquefaction (H_L/H_0)

displacements at the top of piles tend to increase according to the increase in the depth of liquefied layers.

Fig. 57 indicates the relationship between ground surface accelerations (α_g in gals) and cone penetration resistance (q_c in $\text{kg}\cdot\text{f}/\text{cm}^2$) obtained by the static cone penetration tests conducted in the sand box immediately following each shaking test. Fig. 58 illustrates relations between ground surface accelerations (α_g) and cyclic shear strengths (R_d). Cyclic shear strengths (R_d) were obtained by the following two procedures: The first one is Eq. (11), for which N -values are estimated from q_c -values (in $\text{kg}\cdot\text{f}/\text{cm}^2$) by the empirical equation (18), namely

$$N = 4 \cdot q_c \quad (18)$$

The second one is the following equation proposed by Seed (1966)³³ for clean sands.

$$R_d = 0.0042 D_r \quad (19)$$

in which D_r denotes the relative density whose values are shown in Fig. 55. From these figures one can judge the level of ground accelerations (40 gals or greater) to cause complete liquefaction (defined by $\Delta u/\sigma'_v = 1.0$) in terms of either R_d -values or q_c -values.

Next, Figs. 59 and 60 show liquefaction resistance factors (F_L), excessive pore-water pressures (Δu) and ground surface accelerations (α_g) as functions of elapsed time for non-liquefied layers and liquefied layers, respectively. In the figures, F_L -values are estimated by equations (11) and (14), using measured values during the tests. It can be seen from the figures that for liquefied layers F_L -values decrease to less than 1.0 according to the increase in the pore-water pressure (see Fig. 60). On the other hand, for non-liquefied layers F_L -values are higher than 1.0 (see Fig. 59).

Finally, Fig. 61 summarizes the relation between F_L -values and the extent of soil liquefaction, $L_u = \Delta u/\sigma'_v$ (Δu = excessive pore-water pressure, and σ'_v = effective overburden pressure) for liquefied layers. It is seen that F_L -values decrease in accordance with the increase in L_u and F_L -values average less than 1.0 for L_u of 0.5 or higher. Furthermore, F_L -values decrease to less than 0.6 when $L_u = 1.0$ (or the sand layer liquefy completely).

From model shaking table tests mentioned in the above, it may be concluded that F_L -values are adequately employed in assessing soil liquefaction potential of sand layers.

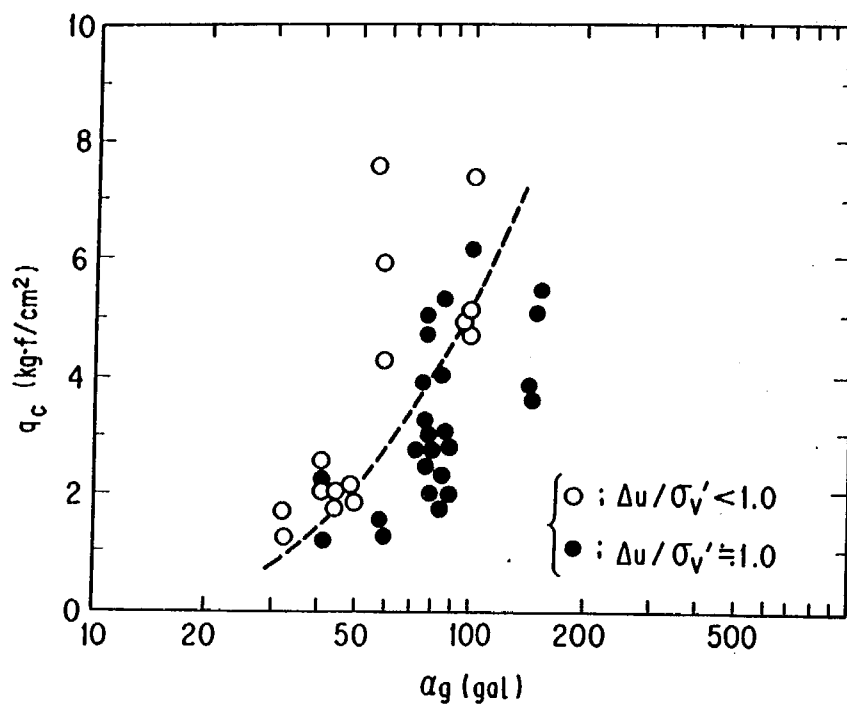


Fig. 57 Relation between Ground Surface Acceleration (a_g in gals) and Static Cone Penetration Resistance (q_c in kgf/cm^2)

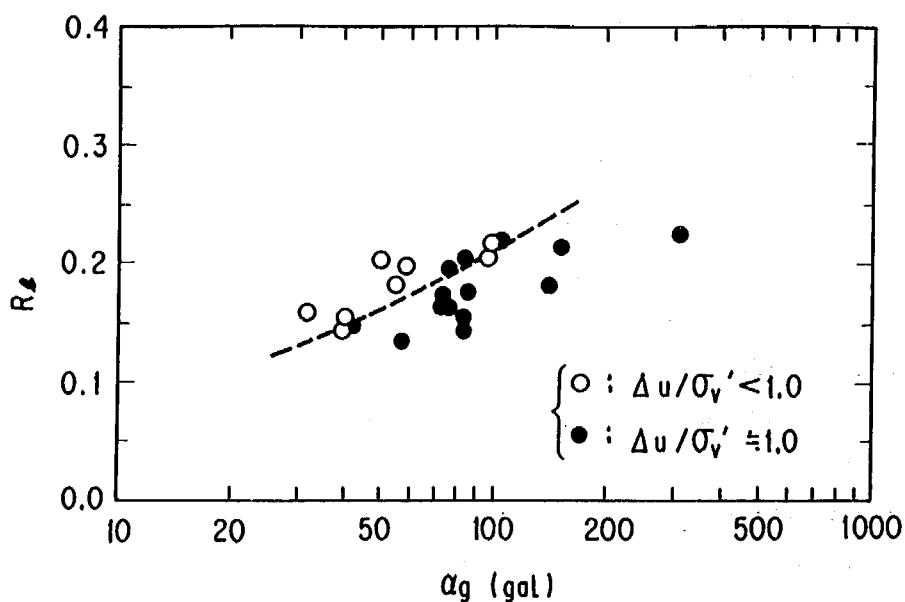


Fig. 58 Relation between Ground Surface Acceleration (a_g in gals) and Cyclic Shear Strength

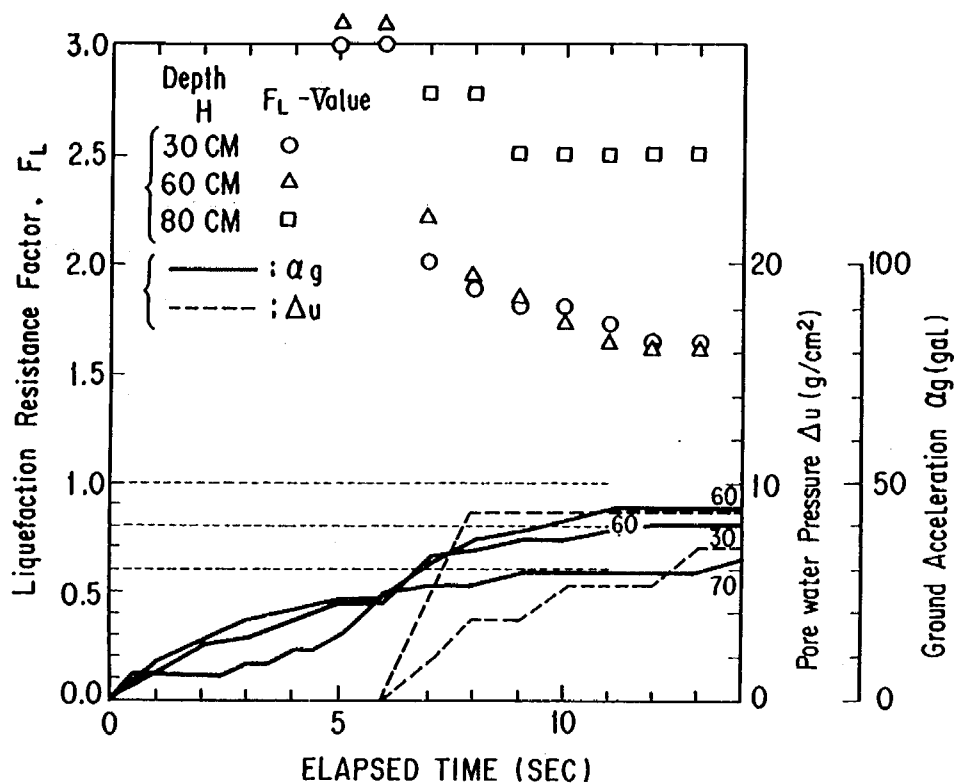


Fig. 59 Time Histories of Pore-Water Pressure (Δu), Acceleration (α_g) and Liquefaction Resistance Factor (F_L) of Non-Liquefied Sand Layers during Shaking Table Tests

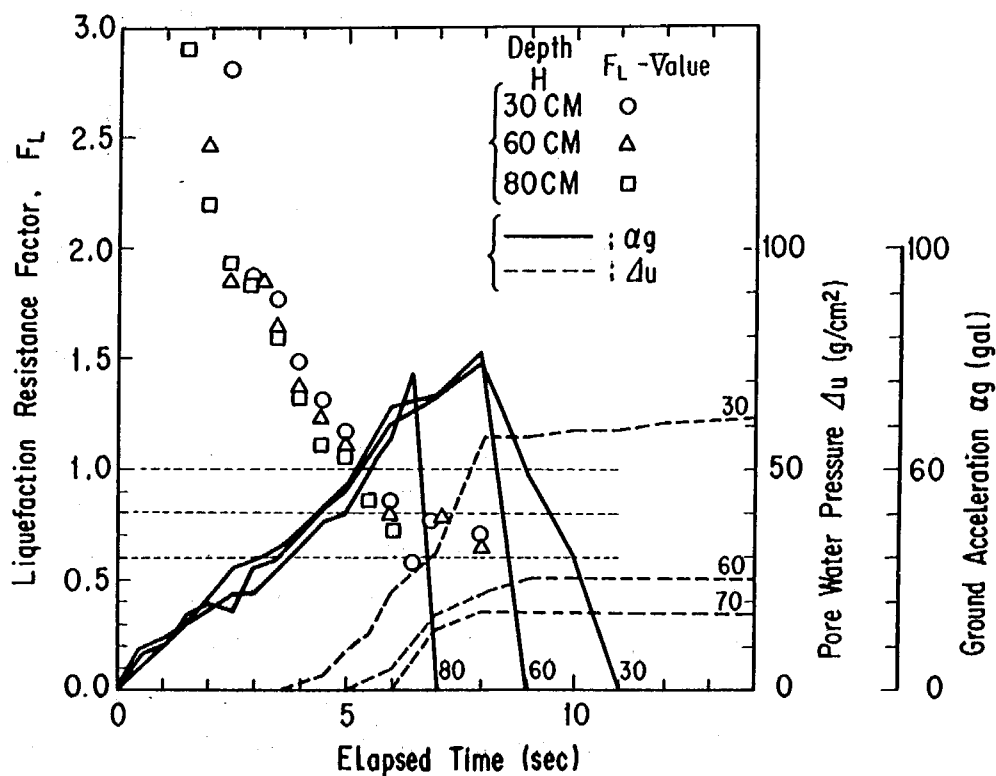


Fig. 60 Time Histories of Pore-Water Pressure (Δu), Acceleration (α_g) and Liquefaction Resistance Factor (F_L) of Liquefied Sand Layers during Shaking Table Tests

4.3 Behavior of Earth Embankments Supported by Sand Layers

Outline

In view of the past experiences of damage to earth embankments due to the effects of liquefaction of sandy layers underneath the embankments, Arakawa, Kimata, and Kondo (1983)³⁴ have recently conducted experiments of embankment models constructed on loose sandy layers. For the experiments they used a large sand box resting on a large-size shaking table, which is the same as the one used in the studies described in 4.2. The objectives of the tests are to clarify the effects of liquefaction of the underlying sand layers on the behavior of earth embankments, and to obtain their failure patterns.

Procedure of Experiments

Sandy embankment models were made on sandy grounds in a container 6 meters long, 3 meters wide, and 2 meters deep, which were firmly fixed on a large shaking table. The characteristics of the shaking table were shown in Table 9 in the previous section. Experiments of four sandy grounds with different relative densities were carried out.

The dimensions of the models are shown in Fig. 62. The thicknesses of the embankment and the underlying ground are 110 and 60 cm, respectively, the crown width of the embankment is 100 cm, and the inclination of both slopes is 1:1.5 (vertical to horizontal).

First wet sands were placed into the container and the ground and the embankments with the above dimensions were made. Then, water was poured through the cock at the bottom of the container up to the level of the ground surface. Table 11 shows four different cases of ground compaction tested, and compaction methods are shown in the table.

Physical properties of sands used in the experiments are indicated in Table 12. Characteristics of ground layers and embankments made for the experiments are shown in Table 13. The results of static cone penetration tests (q_c in $\text{kg}\cdot\text{f}/\text{cm}^2$) are illustrated in Fig. 63. As shown in Table 13, the ground for Case 1 was loose with a relative density of 36%. The ground for Case 2 was looser with a relative density of 19%. The ground for Case 3 was dense with a relative density of 62%. The ground of Case 4 was densest with a relative density of 88%. The relative densities of embankments were about 40% for Cases 1, 2, and 3, and 62% for Case 4. Degree of compaction ($\gamma/\gamma_{\text{max}} = (\text{dry density})/(\text{maximum dry density})$) of these embankments were 88 to 92%.

After each model was completed on the shaking table, resonance tests were carried out in order to clarify frequency characteristics. In the resonance tests, the shaking table was excited into sinusoidal motions with a constant small acceleration of 10 gals having the frequency range from 5 to 30 Hz. The duration time for each frequency was about 30 seconds.

After resonance tests were finished, failure tests were conducted to get failure patterns of embankments. In the failure tests, sinusoidal motions with a constant frequency of 5 Hz were inputted. Three levels of table accelerations, taking 200, 400, and 600 gals as targets, were inputted. In each test it took about 5 to 10 seconds for the table to reach the given acceleration. The duration time for each acceleration level was 50 seconds after the motion took the constant acceleration. For the cases where models completely failed at lower levels of accelerations, tests of higher levels were ignored. Accelerations measured during the tests are shown in Table 14. Observed accelerations were slightly different from target accelerations.

As shown in Fig. 62, 26 accelerometers and 18 pore-water pressure meters were installed in the ground and the embankment. In order to visually measure large permanent movements, vertical and horizontal grid lines of white color with an interval of 30 cm were made at a side surface (on glass) of the ground and embankment, and vertical lines were also made inside the ground.

Results of Experiments

Resonance curves at the top of the embankment and the ground surface for the Case No. 3 are shown in Fig. 64. The ordinate amplification of the figure indicates the ratios of accelerations at two points to that on the shaking table. It is seen from the figure that the acceleration amplification on the ground surface takes its maximum at the frequency of 24 Hz and that of the embankment crown takes its maximum at 21 Hz. Table 15 shows frequencies for which amplifications on the ground surface and embankment crown take their maximums for the four cases. In view of the resonant frequencies, the shaking frequency for the following failure tests was chosen to be 5 Hz, which was sufficiently lower than any resonant frequencies.

As understood from Table 15, both embankments of Case 1 (with loose supporting ground) and Case 2 (looser) failed at step 1 (target acceleration of 200 gals). The embankment of Case 3 (dense) did not deform remarkably at step 1, but failed at step 2 (target acceleration = 400 gals). The embankment of Case 4 (very dense) did not deform largely at step 1 or step 2, but surfaces of the embankment cracked heavily at step 3 (target acceleration = 600 gals). Figs. 65 to 68 are photographs indicating failure features for

Table 11 Methods of Compaction for Four Cases

Case No.	Grounds	Embankments
1	5 Times of Compaction with Foot	5 Times of Compaction with Foot
2	1 Time of Compaction with Foot	the same as above
3	5 Times of Compaction with Mallet	the same as above
4	5 Times of Compaction with Rammer	the same as above

Table 12 Physical Properties of Sands Used for Experiments

Items	Sign	Unit	Value
Specific Gravity of Soil Particle	G _s	-	2.704
Optimum Moisture Content	w _{opt}	%	15.5
Maximum Dry Density	γ _{d, max}	gf/cm ³	1.668
Maximum Void Ratio	e _{max}	-	0.972
Minimum Void Ratio	e _{min}	-	0.613
60% Grain Size	D ₆₀	mm	0.30
10% Grain Size	D ₁₀	mm	0.15
Mean Grain Size	D ₅₀	mm	0.26
Coefficient of Uniformity	U _c	-	2.0

Table 13 Characteristics of Models of Embankments and Grounds

Case No.		1	2	3	4
Items					
Wet Unit Weight γ _t (ton/m ³)	Embankment	1.61	1.63	1.59	1.79
	Ground	1.61	1.55	1.72	1.84
Dry Unit Weight γ _d (ton/m ³)	Embankment	1.47	1.49	1.49	1.55
	Ground	1.47	1.42	1.55	1.63
Water Content w (%)	Embankment	9.6	9.2	6.9	15.4
	Ground	9.9	9.2	11.4	12.7
Void Ratio e	Embankment	0.839	0.815	0.815	0.745
	Ground	0.842	0.904	0.749	0.655
Saturated Unit Weight γ _{sat} (ton/m ³)	Embankment	-	-	-	-
	Ground	1.93	1.89	1.98	2.03
Relative Density D _r (%)	Embankment	37.0	43.7	43.7	63.2
	Ground	36.2	18.9	62.1	88.3
Degree of Compaction D/D _{max} (D _{max} =1.668 t/m ³)	Embankment	0.881	0.893	0.893	0.929
	Ground	0.881	0.851	0.929	0.977

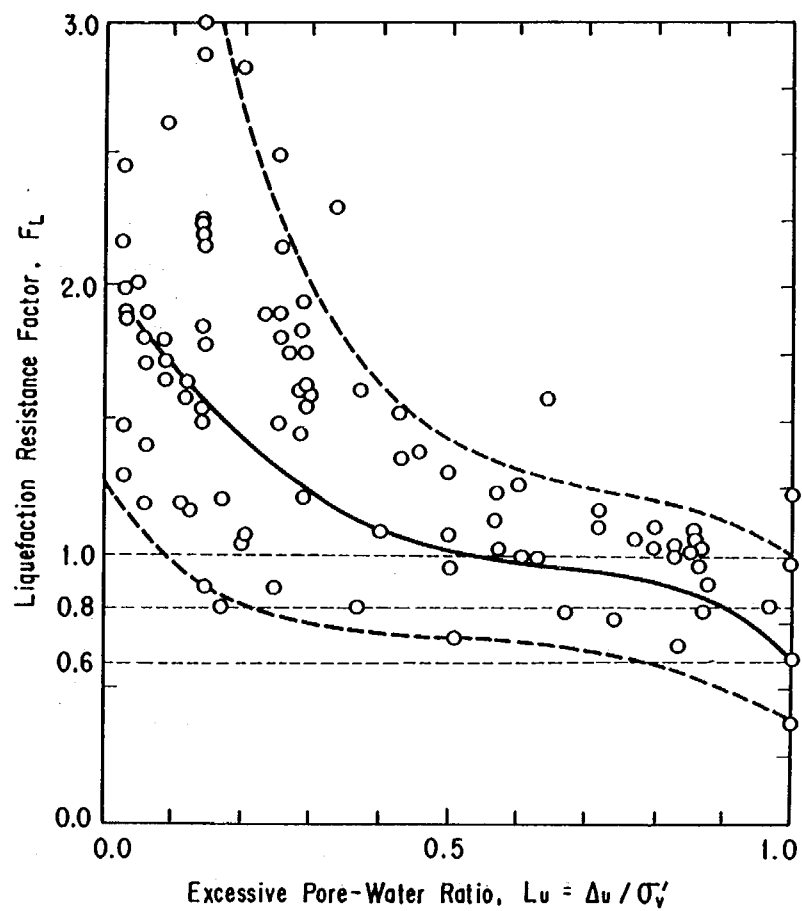


Fig. 61 Relationships between Liquefaction Resistance Factor (F_L) and Rate of Ground Liquefaction ($\Delta u / \sigma'_v$) during Shaking Table Tests

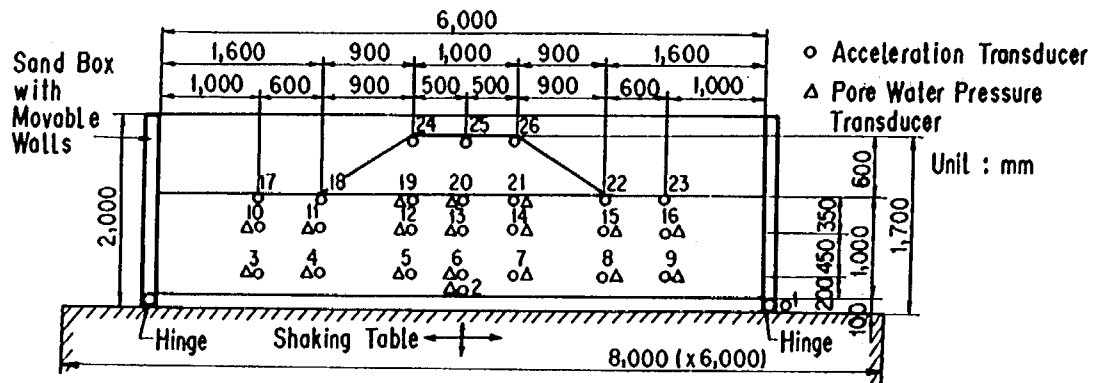


Fig. 62 Model of Ground and Embankment

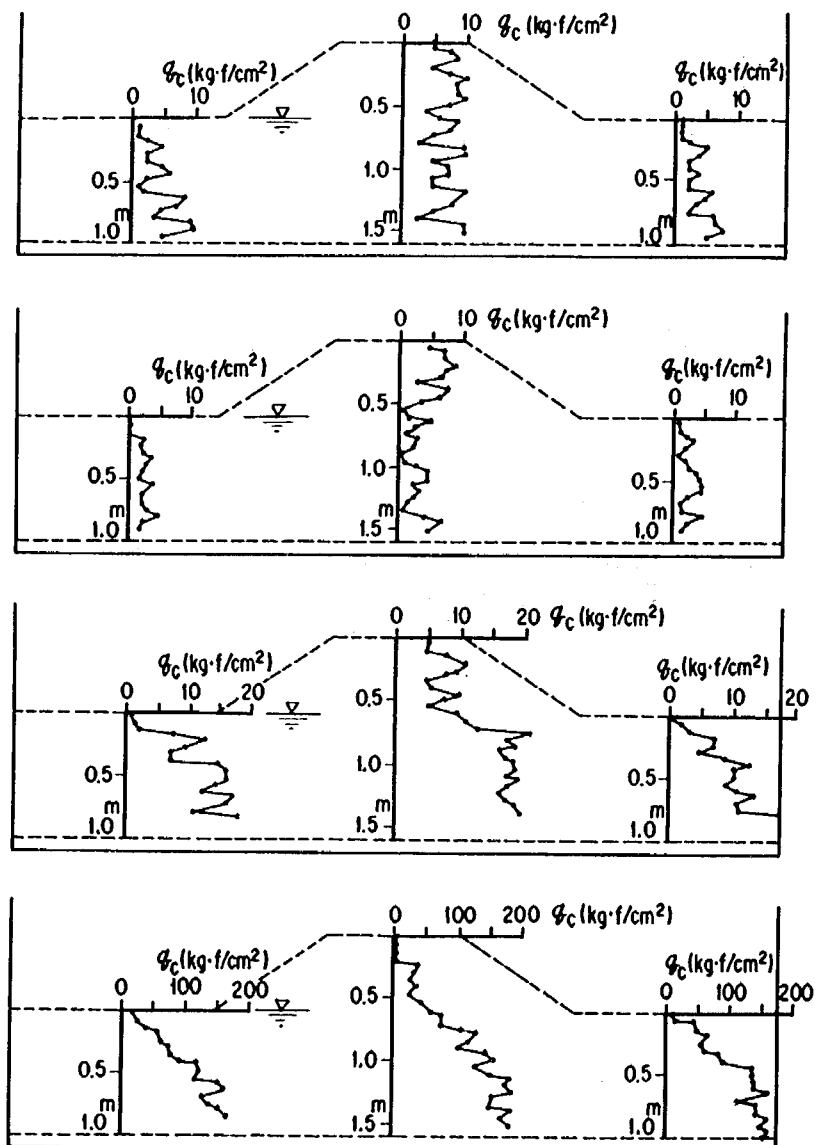


Fig. 63 Distribution of q_c -Value by Cone Penetration Tests

Table 14 Acceleration of Shaking Table

Case No.	Step No.	Target Acceleration (gal)	Observed Acceleration (gal)	Remarks
1	1	200	201	Failed
2	1	200	186	Failed
3	1	200	223	Not Damaged
	2	400	341	Failed
4	1	200	221	Not Damaged
	2	400	425	Not Damaged
	3	600	682	Failed

Table 15 Resonant Frequencies at Preliminary Shaking Table Tests

Case No.	Embankment		Ground	
	Resonant Frequency (Hz)	Amplification Ratio	Resonant Frequency (Hz)	Amplification Ratio
1	14	11.0	14	3.5
2	14	11.5	14	4.0
3	21	22.0	24	8.5
4	26	24.0	higher than 27	higher than 27

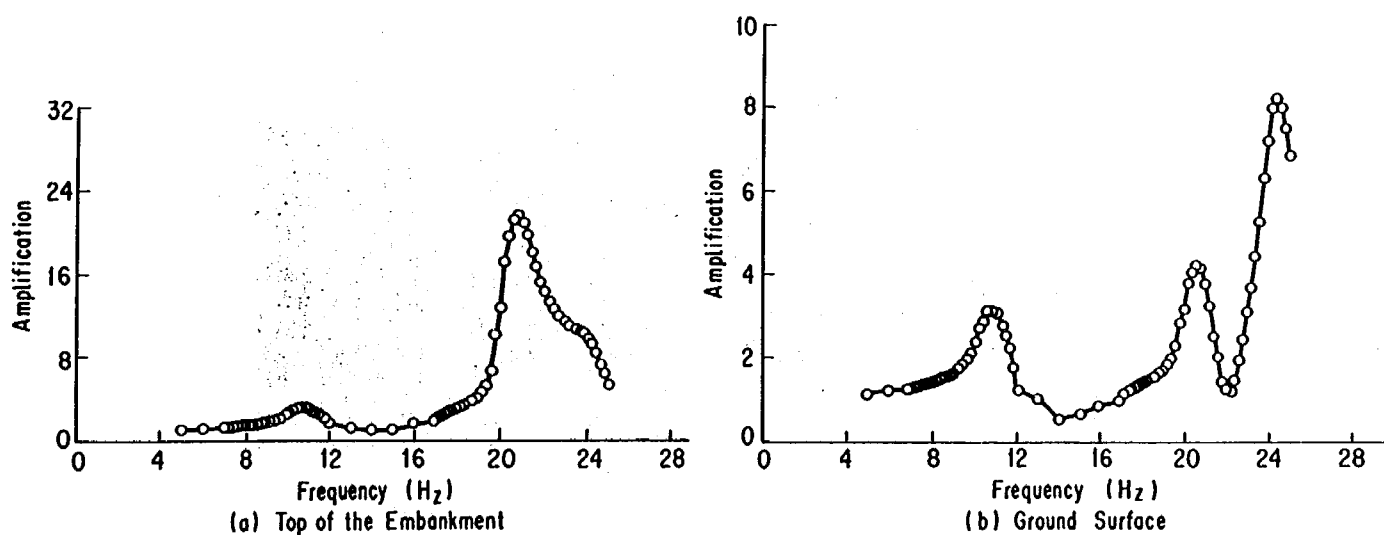
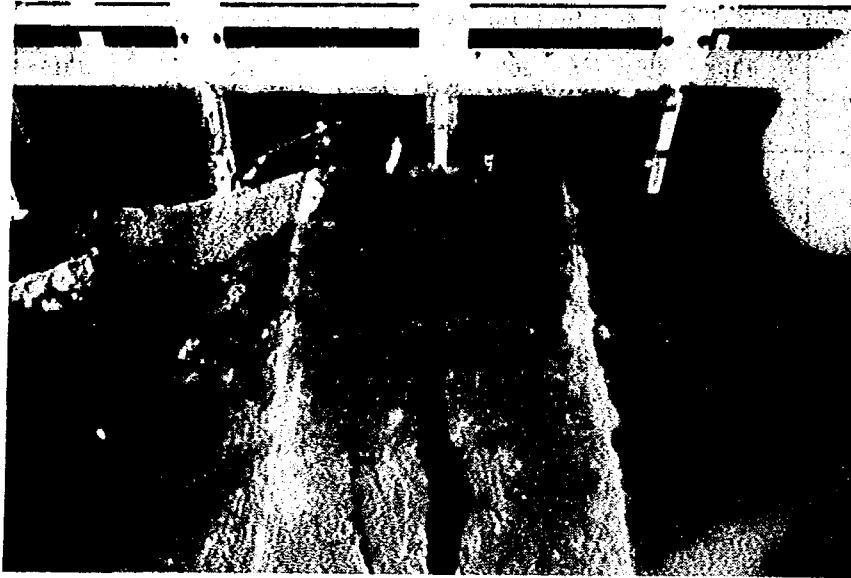
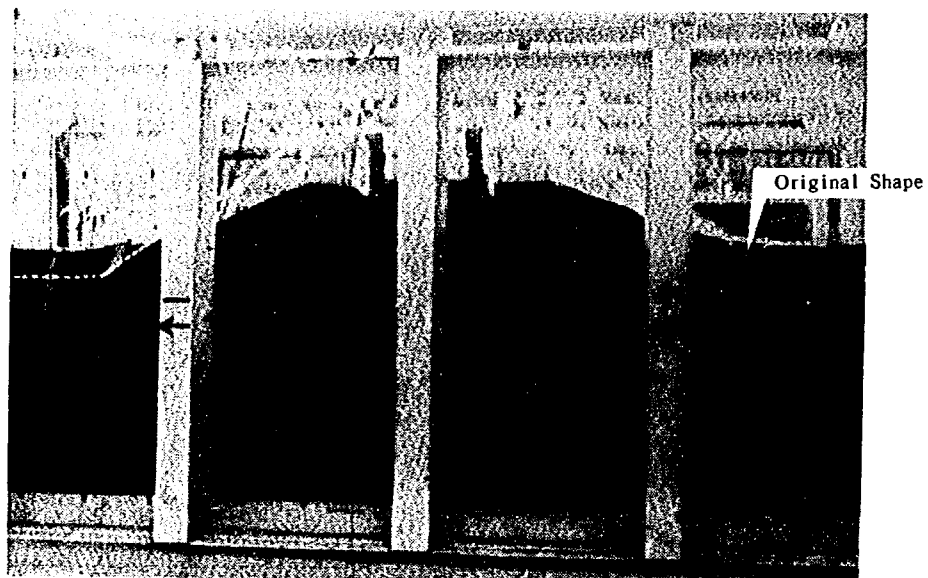


Fig. 64 Examples of Resonance Curves (Case 3)

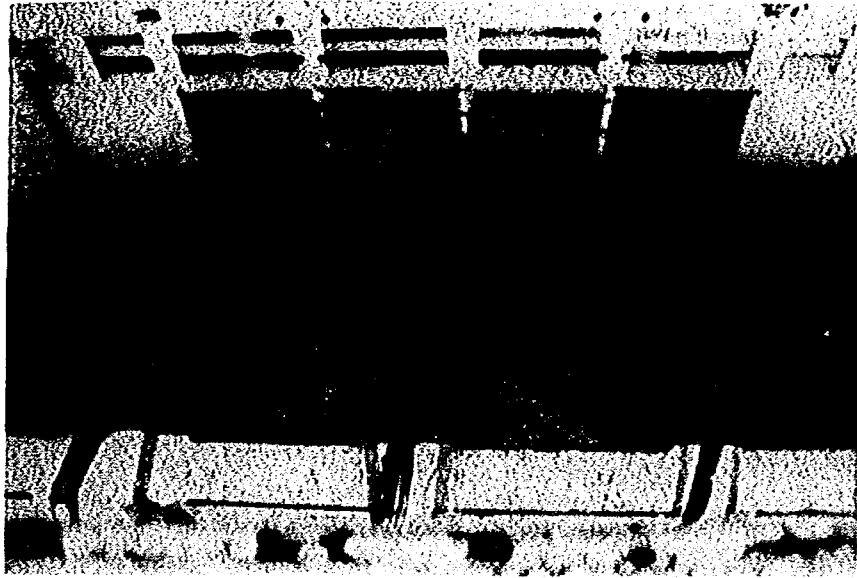


(A) Top View

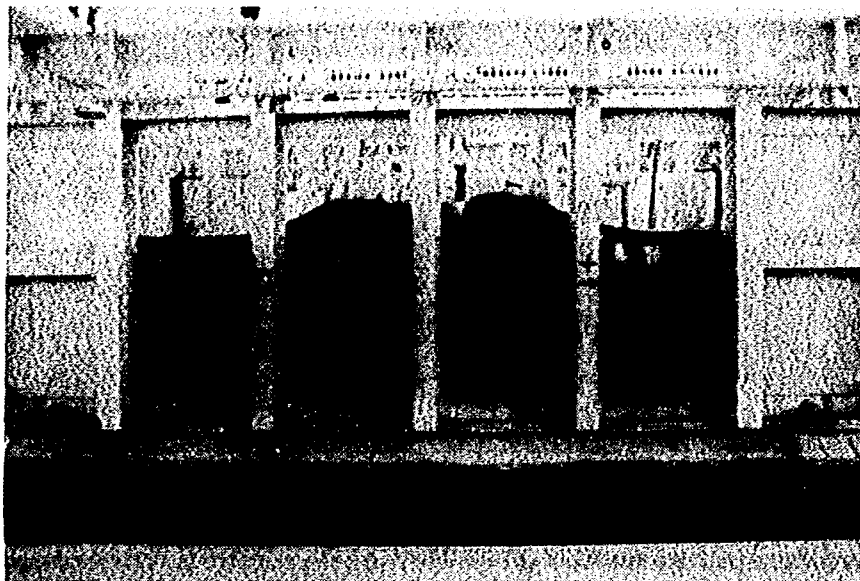


(B) Side View

Fig. 65 Failure Mode of Case 1

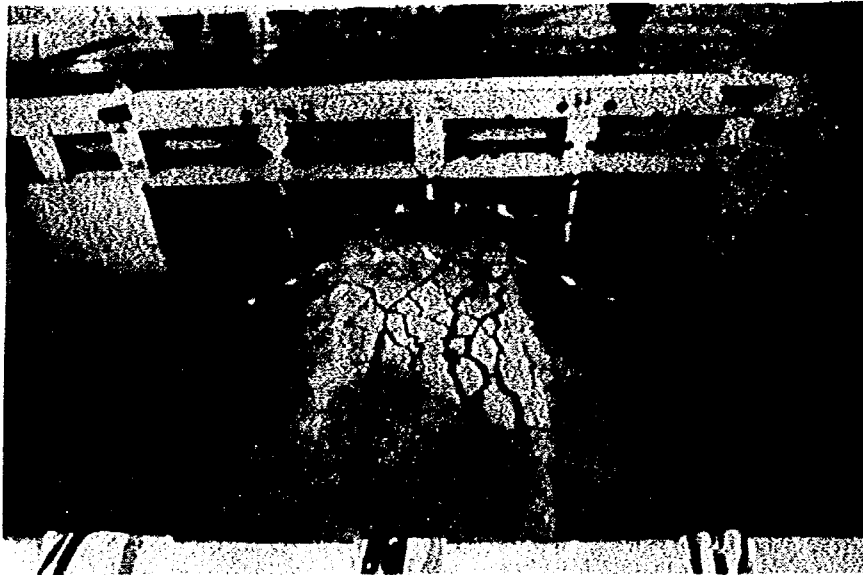


(A) Top View

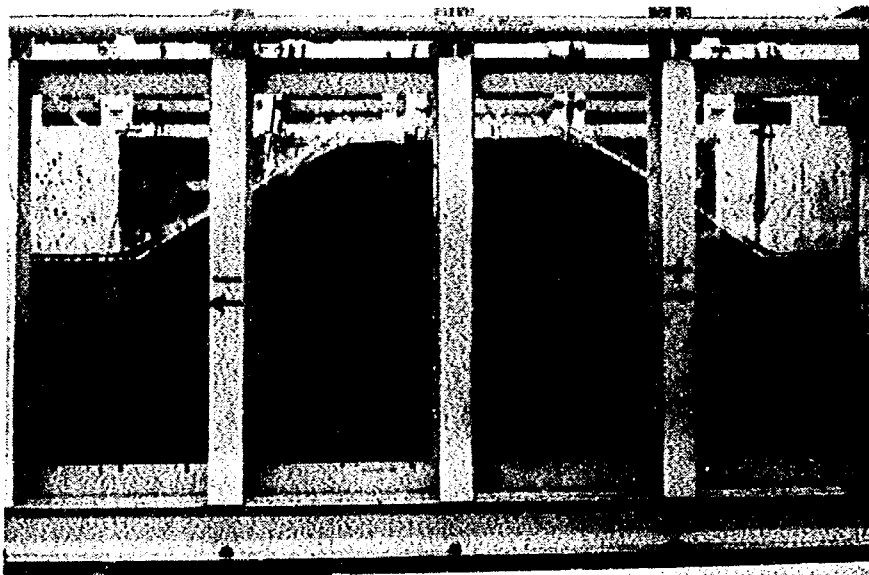


(B) Side View

Fig. 66 Failure Mode of Case 2



(A) Top View



(B) Side View

Fig. 67 Failure Mode of Case 3



(A) Top View



(B) Side View

Fig. 68 Failure Mode of Case 4

each case after the last steps. Fig. 69 shows a relation between the relative density and the maximum acceleration causing embankment failure. It is obvious that the acceleration at failure increases as relative density increases.

Next, the failure pattern of model embankments will be described. In Cases 1 and 2, heavy cracks broke off at the centers of the embankments due to tensile forces, as shown in Figs. 64 and 65. The values of acceleration when the cracks occurred were about 200 gals. After cracks started to open, the embankments began to settle, and accordingly the tension cracks widened. Eventually widths of the cracks became 14 cm in Case 1, and 25 cm in Case 2. Average settlements at the embankment crowns were 20 cm in Case 1, and 23 cm in Case 2. The lower parts of the embankments flowed toward the outer sides, and surrounding ground surface heaved up, as shown in Figs. 65 and 66 in which the white dotted lines denote the original shapes.

Prior to the occurrence of cracks and settlements, pore-water pressures began to rise in the ground near the embankment and slightly later in the ground just underneath the embankment. Examples of time histories of the accelerations (at the shaking table and the ground surface) and pore-water pressures are shown in Fig. 70. Sand boilings were observed at the ground surface near the walls of the sand box. This evidence indicates that liquefaction of the sandy ground took place and accordingly the embankment cracked, settled and even flowed toward the outside directions.

In Case 3, several cracks appeared on the embankment crown at the elapsed time of about 15 seconds of Step 2 (341 gals - see Table 15). Cracks, however, did not grow large when comparing with Cases 1 and 2. The average settlement at the crown was 6.5 cm. In this case the lower part of the embankment slightly flowed toward the outer sides, as shown in Fig. 67. In this case, several of the smaller cracks appeared at the crown of the embankment. Pore-water pressures also rose but not so remarkable when comparing with Cases 1 and 2.

In Case 4, the surfaces of the embankment slopes began to fail at about the elapsed time of 15 seconds of Step 3 (acceleration was 682 gals - see Table 15). A number of small cracks appeared at the embankment crown, as shown in Fig. 67. The peak acceleration measured at the crown was over 1,000 gals. The ground and the inner embankment, however, did not sustain any serious deformation. Pore-water pressures did not rise in Case 4.

Excessive pore-water pressures were also measured for Cases 1, 2, and 3 where soil liquefaction of sandy grounds took place. The excessive pore-water

pressure ratio ($L_u = \Delta u / \sigma'_v$) is defined as the ratio of excessive pore-water pressure (Δu) to effective overburden pressure (σ'_v). The values of Δu were obtained from time-history records measured during the tests and the values of σ'_v were calculated using the common method proposed by Osterberg (1959).³⁵

Fig. 71 illustrates an example of the distributions of excessive pore-water pressure ratios ($L_u = \Delta u / \sigma'_v$) at different elapsed times (after initiation of shaking) in Case 2. It is obviously seen that the value of L_u increases as time elapses. Values of L_u , however, are very different between ones underneath the embankment and ones under the side of the embankment. For example, the values of L_u are around 1.0 at points under the side of the embankment, but 0.3 to 0.5 at the sites just underneath the embankment. Fig. 72 shows a relation between L_u -values and settlements of the embankment crown in Case 2. It is clear from Figs. 71 and 72 that liquefaction conditions under the sides of the embankment are completely different from those just underneath the embankment. The tendency shown in Case 2 is the same for the other cases. The embankments did not settle instantaneously, but settled gradually in accordance with elapsed time. Velocities of settlements of the embankments are approximately 0.5 cm/s in Case 1, 0.7 cm/s in Case 2, and 0.2 cm/s in Case 3. The reason why the embankments settled gradually may be that the time of occurrence of excessive pore-water pressure is different from point to point, and that occurrences of complete liquefaction propagate from one point to another.

Fig. 73 illustrates a relationship between the average value of $L_u = \Delta u / \sigma'_v$ in the sand ground and the settlement ratio of the embankment crown, S_e / H (S_e = settlement of the embankment crown, and H = height of the embankment). It is obvious from the figure that the average settlement ratios are very low in the range of $L_u < 0.3$, and increase abruptly in the range of $L_u > 0.8$. This tendency is important in estimating settlement of embankments resting on sandy layers subjected to earthquake motions.

As a summary of the comprehensive experimental investigations of seismic failure patterns of embankments constructed on saturated sandy layers, the following may be concluded.

1. Embankments on loose sandy layers may fail due to the effects of liquefaction of the supporting sandy layers during earthquakes.
2. Failure patterns and degrees of failure of the embankments on loose sandy layers depend remarkably on relative densities of the supporting sandy layers.

3. In cases of loose sandy ground, the ground under the sides of the embankment first liquefies, the embankment tends to flow toward the outer directions, and the embankment considerably loses its stability.
4. Excessive pore-water ratios, $L_u = \Delta u / \sigma'_v$, are high at points under the sides of an embankment, and rather low just underneath the embankment.
5. Settlement ratios, S_e/H , are low for cases where the average excessive pore-water pressure is less than 0.3 and S_e/H increases extensively for cases where the ratio, $L_u = \Delta u / \sigma'_v$ is 0.8 or higher.

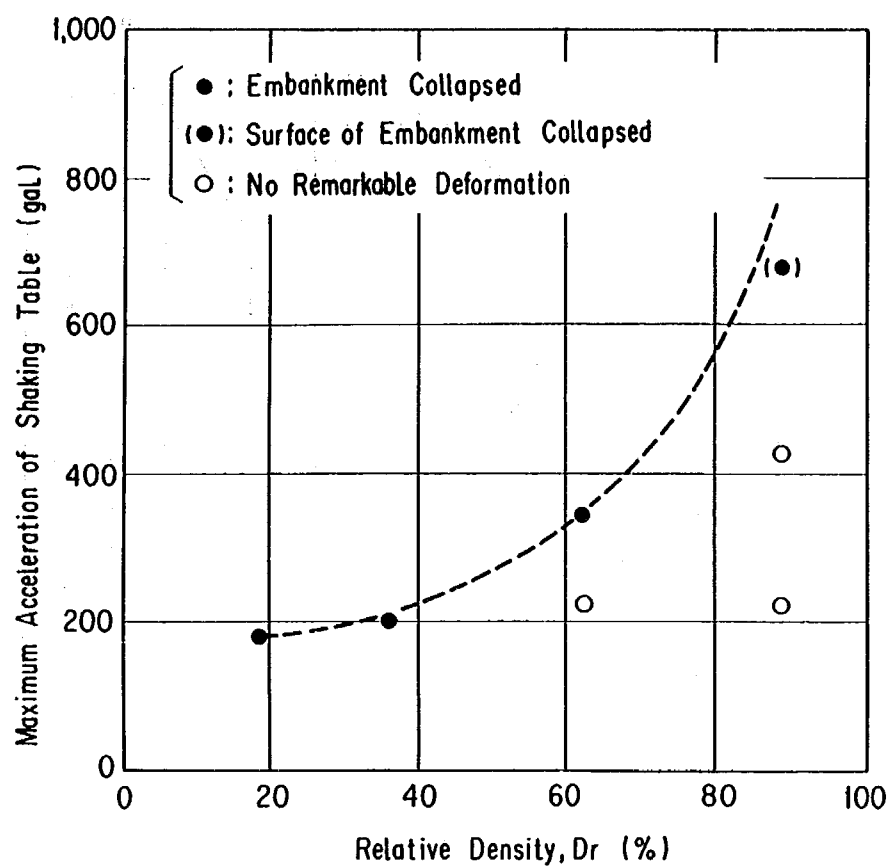
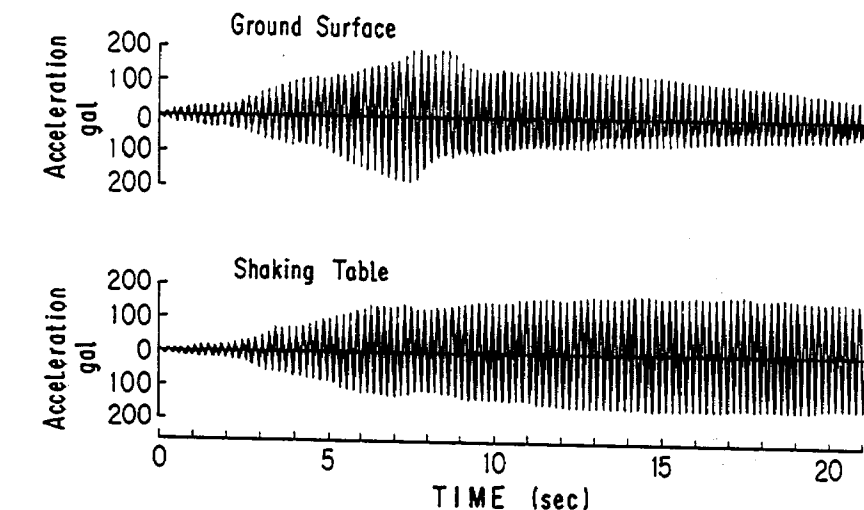
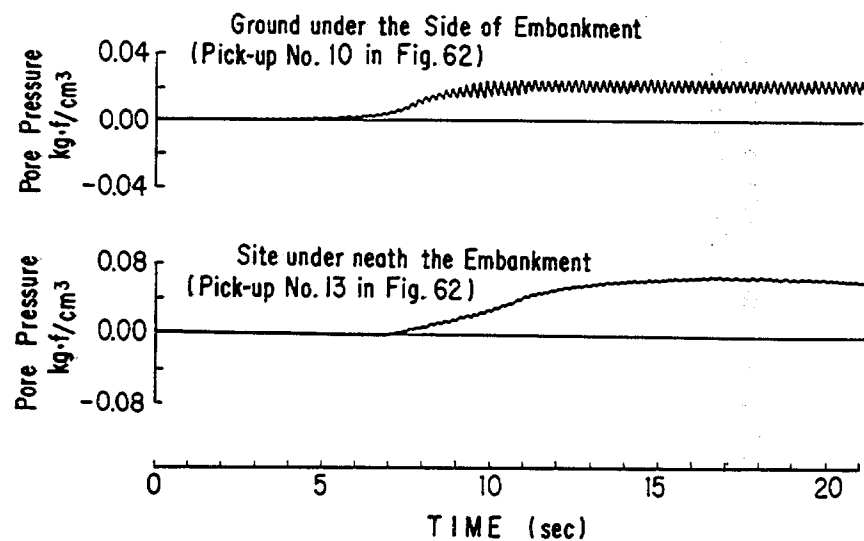


Fig. 69 Relation between Relative Density of Ground and Maximum Acceleration of Shaking Table



(a) Acceleration



(b) Pore Water Pressure

Fig. 70 Examples of Records of Accelerations and Pore-Water Pressure at Case 2

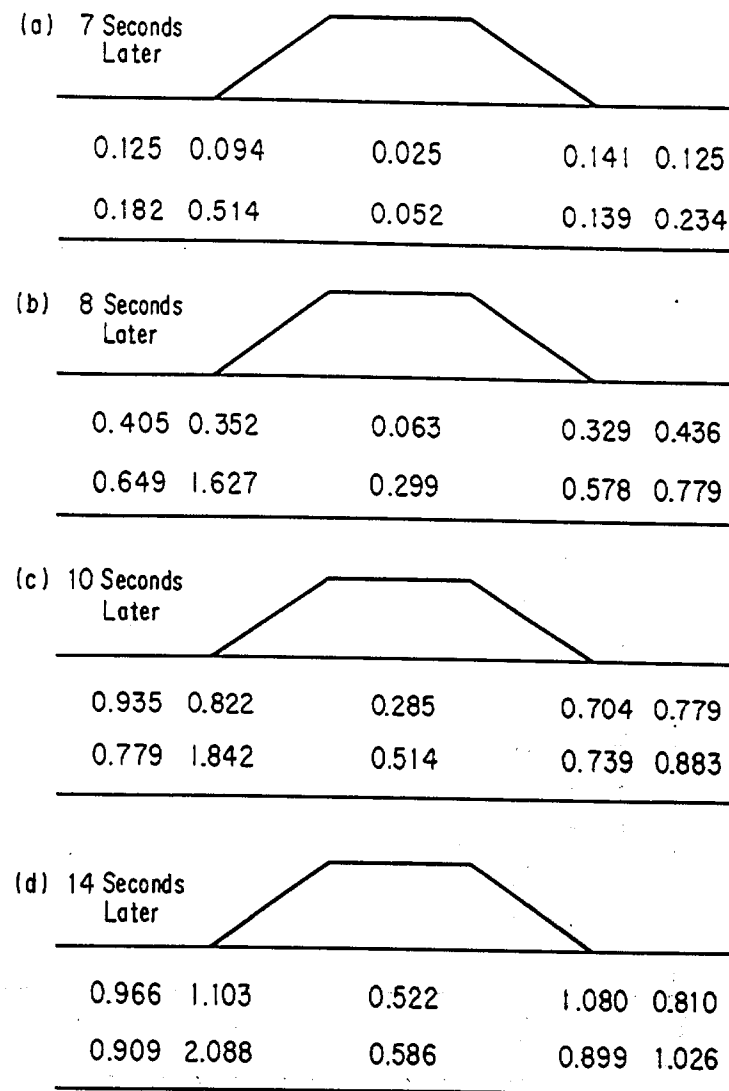


Fig. 71 Distribution of Excessive Pore-Water Pressure Ratio, $L_u = \Delta u / \sigma_v'$ (Case 2)

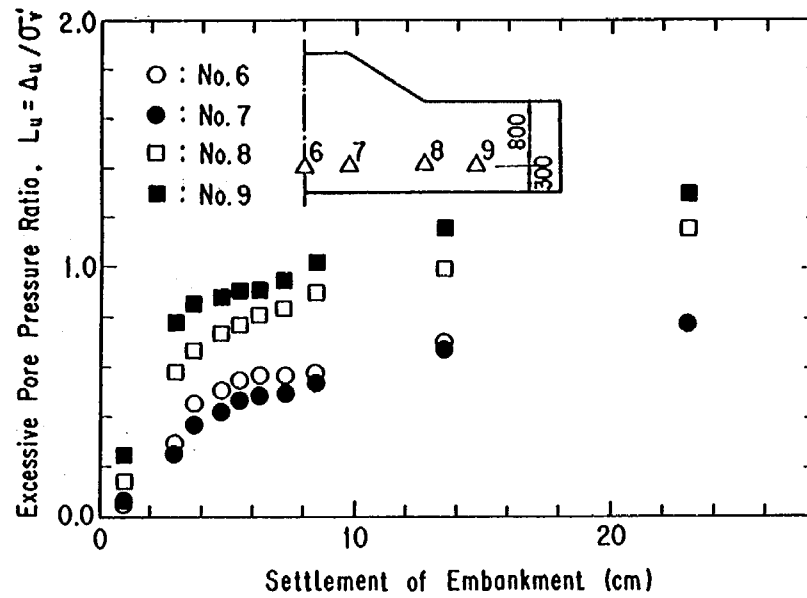


Fig. 72 Relation between $\Delta u / \sigma_v'$ and Settlement of Embankment (Case 2)

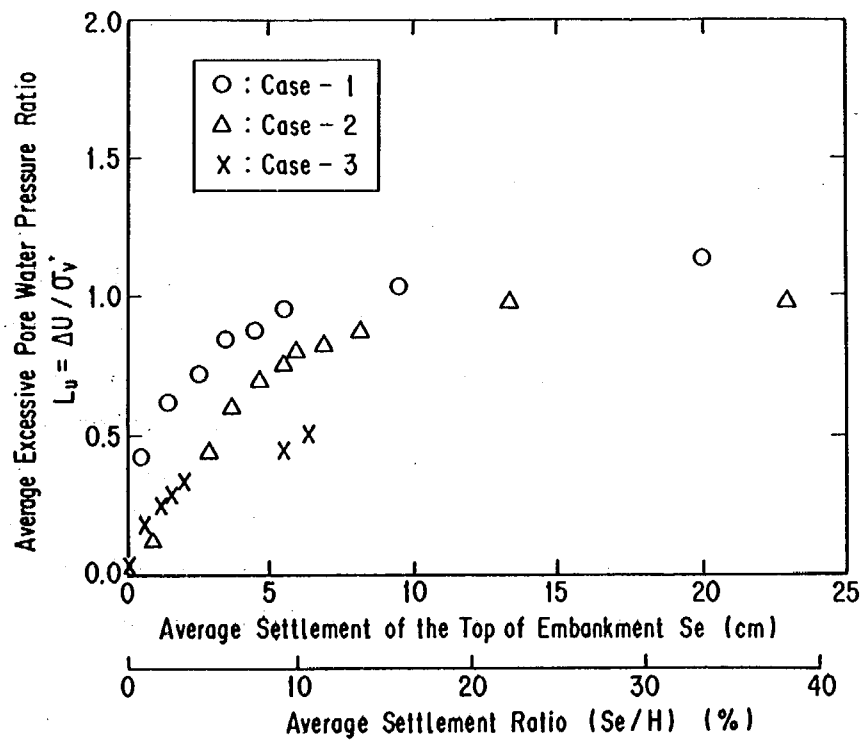


Fig. 73 Relation between Average Value of $L_u = \Delta u / \sigma_v'$ and Settlement of Embankment

5. COUNTERMEASURES AGAINST SOIL LIQUEFACTION

5.1 Outline

In designing an important civil engineering structure on soils vulnerable to liquefaction during earthquakes, it seems inevitable to consider the effects of soil liquefaction on the seismic stability of the structure. This chapter will briefly describe, according to a recent paper by Suematsu, Yoshimi, and Sasaki (1982),³⁶ the fundamental concept and examples of its application of the countermeasures for reducing the influences of soil liquefaction.

Countermeasures to reduce seismic damage caused by soil liquefaction can be classified into two categories:

- A. Soil Improvement Countermeasures - prevent positively the development of soil liquefaction by improving soil properties to make the soils stronger.
- B. Structural Improvement Countermeasures - prevent structural failures by making structures stronger (such as rigid foundations with pile driving) without directly improving soil properties.

5.2 Soil Improvement Countermeasures

The principal concept and practical application procedure for (A) Soil Improvement Countermeasures are illustrated in detail in Table 16. Next, Table 17 shows construction methods for countermeasures in increasing soil strength against liquefaction occurrence, and their characteristics. This table shows the number of cases employed by 25 major Japanese construction companies in 1975 through 1980. Table 18 indicates the number of cases applied to a variety of structures. The numbers of countermeasure cases are increasing every year. As shown in Table 18, it is seen that among several principles for increasing soil strength against soil liquefaction, densification is most frequently used. It is found from Table 17 that the methods of sand compaction piles is often employed.

When sand densification construction methods are applied at sites near residential areas in cities, vibration and noise problems should be carefully considered. In such cases, the method of injection, hardening of chemical materials, and deep mixing may be effectively used, although the number of the cases actually employed so far is rather small. The soil substitution method to aid particle improvement or hardening is effective for cases where liquefiable layers are shallow, and where excavation is inevitably needed.

The well point method and the deep well method to lower the degree of saturation and increase the effective stress need long-time operations and accordingly the maintenance of equipment is one of the troublesome factors to be considered. The banking method will increase effective stress by placing heavy material (like gravel) several meters deep on the surface of liquefiable sands, although it is not often used in Japan.

The gravel drain method can be applicable to sites where sand is not too loose and where densification is not possible. The method is not recommendable to sites where sand is very loose, because the effectiveness seems low.

Effectiveness of the vibrofloatation method is reported by Watanabe (1966)³⁷ for the Niigata Earthquake of 1964, and also by Ohsaki (1970)³⁸ for the Tokachi-oki Earthquake of 1968. Effectiveness of sand compaction pile method is described by Ishihara (1980)³⁹ for the Miyagi-ken-oki Earthquake of 1978, and also by several other investigators. Effectiveness of gravel drains is introduced by Tokimatsu and Yoshimi (1980)⁴⁰ and Sasaki and Taniguchi (1981)⁴¹ from laboratory tests, and by Ishihara, Saito, and Arima (1980)⁴² and Takehara, Tanaka, and Ohira (1980)⁴³ from field experiments. Since gravel drains are actually used only a little, the effectiveness is not yet verified during real earthquakes.

5.3 Structural Improvement Countermeasures

Earth Embankments

Materials of earth embankments such as river dikes, highway banks, railway banks and reclaimed lands are, in general, not completely saturated, so they are not vulnerable to complete liquefaction. Embankments constructed on loose liquefiable sandy grounds, however, are very much vulnerable. The typical examples can be seen in cases from the recent Nihonkai-Chubu Earthquake of 1983 as described in 2.3 of this paper. For those earth structures resting on loose liquefiable sands, some countermeasures for increasing liquefaction strength should be implemented. Among a number of countermeasures shown in Tables 17 and 18, the densification procedures utilizing compaction methods are most reliable and are most frequently used in Japan. Soil substitution procedure is rather expensive except for the cases when the volume of substituting soils is very small. Procedures for banking, lowering the water table, and constructing underground walls are not sufficiently effective for normal cases.

As for liquefaction countermeasures for existing earth banks, a method of increasing soil strength just underneath the earth structures is rather

Table 16 Principles and Procedures for Soil Improvement Countermeasures

PRINCIPLES AND OBJECTIVES		PROCEDURES	
Improvement of Soil Properties	Particle Improvements or Hardening	→	Soil Substitution
	Densification	→	Mixture Treatment
	Lowering Saturation Degree	→	Compaction
Improvement of Stress-Strain Conditions	Increase in Effective Stress	→	Banking or Lowering Water Table
	Dissipation of Pore-Water Pressure	→	Gravel Drains and Others
	Shut-off of Pore-Water Pressures	→	Gravel Drains and Others
	Restraint of Shear Deformation	→	Underground Walls

After Suematsu, Yoshimi, and Sasaki (1982)³⁶

Table 17 Characteristics of Methods of Countermeasures to Increase Soil Strength

Principles	Countermeasure Methods	Approximate Applicable Depth	Effectiveness	Pollution Problems	Number of cases employed in 1975 to 1980	Remarks
Densification	Vibro-Floatation	Up to -20m	Up to N-values of 15 to 20	Less troublesome because of horizontal vibration	16	Permeability may increase in some cases
	Sand-Compaction Piles	Up to -35m	Up to N-values of 25 to 30	Some troublesome because of vertical vibration	123	Permeability may increase in some cases
	Blasting	Up to -20m	Up to Dr of 70 to 80%	Large shock	0	Execution management is difficult
	Rod Compaction	Up to -20m	Up to N-values of 15 to 20	Some troublesome because of vertical vibration	9	Insitu sands are often used
	Dynamic Consolidation (Heavy Weight Dropping)	Up to -10m	Apt to execution practices	Large shock	4	Compaction for shallow layers
	Vibro-Tamper	Up to -3m	Apt to execution practices	Less troublesome	17	Compaction for shallow layers, other methods may be jointly used.
	Roller Compaction	Up to -0.2m to -0.3m, Shallower than water table	Up to Dr of 95%	Less troublesome	1	Waterproof measure is necessary for depth below water table
	Pile Driving	Up to -10m to -12m	Apt to execution practices	Vibration and noise from diesel pile hammer	1	
	Chemical Piles	Up to -20m	Apt to execution practices	Less troublesome, Fragments	1	
Particle Improvement or Hardening	Substitution	Generally up to -5m	Effective by using Gravels	Less troublesome	14	
	Injection Hardening	Up to the depth of boring	Apt to execution management	Influences on neighboring structures	9	Execution management is difficult in case of cement grouting
	Surface Mixing Treatment	Up to -5m	Apt to mixing contents	Less	3	
	Deep Mixing Treatment	Up to -30m	Apt to mixing contents	Less	8	
Decrease in Saturation Degree	Well Point	Lower water table by 5 to 6m	Uncertainty of Permeability	Lowering water table at neighboring site	9	Long-time operation is necessary
	Deep Point	Lower water table by 15 to 20m	Uncertainty of Permeability	Lowering water table at neighboring site	2	Long-time operation is necessary
Dissipation of Pore-Water Pressures	Gravel Drain	Up to -20m		Less	4	Retrofitting of existing structures
Restraint of Shear Deformation	Sheet Pile	Up to -10m	Difficult to assess the effects of restraint	Vibration from driving of sheet piles	1	Underground continuous walls may be used instead of sheet piles.

Table 18 Numbers of Cases Employed for Various Structures as Countermeasures to Increase Soil Strength Against Liquefaction (1975 to 1980)

STRUCTURES	PRINCIPLES				TOTAL
	DENSIFICATION	PARTICLE IMPROVE- MENT OR HARDENING	LOWERING SATURATION DEGREE	DISSIPATION PORE-WATER PRESSURE	
Civil Engineering Structures (Including Reclaimed Lands)	29	9	0	0	38
Port and Harbor Structures	38	6	1	0	45
Bridge Foundation	2	0	0	0	2
Tank Foundation	91	8	0	0	99
Building Foundation	37	3	2	2	44
Underground Structures, Buried Structures	18	6	5	1	30
Others (such as Plant Foundations)	17	2	3	2	24
TOTAL	232	34	11	5	282

Suematsu, Yoshimi, and Sasaki (1982)³⁶

difficult, so a method of increasing soil strength below the toes of earth banks can be implemented. Other than the above methods, the following three methods can be effectively implemented for strengthening existing embankments (see Fig. 74).

- a. Supplementary bank,
- b. Substitution by gravels,
- c. Sheet piles and tie-rods.

In the case of the sheet pile and tie-rod method, it seems that tie rods connecting two rows of sheet piles will considerably increase stability of earth banks against sliding, even if the surrounding sandy layers liquefy during earthquakes.

In the case of earth dams and reclaimed lands where completely saturated soil materials are likely to liquefy, measures to reduce the water table inside the soil may be necessitated to avoid liquefaction. In the case of tailing dams containing cohesionless loose materials with high-water content, it is very important to have effective drainage systems. It should be considered to have facilities to prevent rain water from entering the reservoir and to reduce the degree of saturation of reservoir materials by installing surface drainages and bottom channels.

Building Foundations

Four types of countermeasures are proposed for preventing damage to building foundations due to soil liquefaction. Fundamental concepts of the four methods of countermeasures, (a) to (d), are illustrated in Fig. 75.

Category (a) (Soil Improvement Countermeasures) is to improve soil properties underneath the building. Densification particle improvement, hardening, and dissipation of pore-water pressures can be employed as methods of this category. Those methods may be applicable to medium or low-rise buildings.

Category (b) (Structural Improvement Countermeasures) is to use deep pile foundations as well as densification and/or dissipation of pore-water pressures. This category may be applicable to high-rise buildings which will be constructed on reclaimed lands and on liquefiable loose sands having soft clays below the sands.

Category (c) (Removal of Liquefiable Soils) is to remove all of liquefiable soils underneath the buildings and to have deep underground foundations. This category may be applicable to super high-rise buildings and buildings resting on rather shallow loose sandy layers.

Category (d) (Enclosing Foundations) is to construct enclosing foundations to shut out propagation of excessive pore-water pressures toward the base of the building. It is common to have inside soils strengthened by any of the densification, dissipation of pore-water pressures, soil particle improvement, hardening and lowering water table methods. As for practical examples, two of those categories are jointly employed in order to make the countermeasures more effective. Fig. 76 shows some examples. Fig. 76(a) is a combination of underground continuous walls and compaction, and Fig. 75(b) is a combination of gravel matts, gravel drains, and sand compaction. Fig. 77 illustrates an example of the well point method to lower the water table. Further, Fig. 78 draws an example of a structural improvement countermeasure.^{44,45} As for existing buildings, either underground continuous wall underneath the outer fringes of the building bases, or gravel drains can be effectively used. In the case of the gravel drains, however, excessive settlements should be carefully checked in advance.

Harbor Structures

Three typical procedures of countermeasures for harbor structures are illustrated in Fig. 79(a) to (c). Fig. 79(a) shows the case of a cassion-type breakwater in which countermeasures using densification, soil particle improvement, hardening, and dissipation of excessive pore-water pressures can be used. Those countermeasures are similar to those for common earth structures on land. Fig. 79(b) illustrates an example of a sheet pile quay retaining wall in which countermeasures using densification, soil particle improvement, hardening, and dissipation of excessive pore-water pressures can be used, as well as structural countermeasures such as strengthening sheet piles and tie-rods by neglecting the lateral subgrade reactions of liquefiable sandy soils in the design. Finally, Fig. 79(c) is an example of a structural pier in which pile structures form the main part of the pier. Even in this case soil improvement can be jointly employed to effectively strengthen the stability.

In improving existing harbor structures resting on liquefiable soils, densification of foundation soils, dissipation of excessive pore-water pressures, and hardening can be effectively used as countermeasures to reduce seismic liquefaction effects.

Bridges

In constructing bridge substructures on liquefiable soils, either soil strengthening or structural strengthening is usually considered. In the case

of highway bridges, liquefaction potential shall be assessed with use of the liquefaction resistance factor, F_L , for saturated sandy soils with the mean particle diameter of 0.02 to 2.0 mm at the depth of 20 meters or shallower. The procedure for obtaining F_L -values was introduced in 3.2 of this paper. If F_L -values are less than 1.0, the coefficient of the lateral subgrade reaction shall be reduced or neglected by multiplying the normal coefficient by the D_E -value shown in Table 19. Driving piles, excavation piles, place-in-case piles, and well and caisson foundations are normally used as foundations of bridge substructures resting on liquefiable soils. Types of foundation will be selected according to the type of superstructure and the in situ construction conditions. Fig. 80 illustrated an example of countermeasures for a bridge abutment in which soil improvement and long pile foundations are jointly employed in order to upgrade the soil liquefaction vulnerability.

Underground Pipelines and Buried Structures

In the past earthquakes, a number of underground pipelines and buried structures such as water supply pipelines, sewage conduits, fuel pipelines, etc., sustained heavy damage due to large earth pressures and uplifts caused by the effects of soil liquefaction. Also those pipelines were completely broken off due to the concentrated stresses acting at soil discontinuity points, such as points between liquefied sites and non-liquefied sites and at structural discontinuity points such as those between pipes with foundations and pipes without foundations.

In order to reduce the effects of soil liquefaction on the stability of pipelines, several procedures can be proposed. Fig. 81 illustrates two examples: (a) place stabilized soils (such as soil cements) into the excavated spaces surrounding pipelines, and (b) construct zones of gravel drains to dissipate excessive porewater pressures. In both cases soils surrounding pipelines should be densely compacted. Fig. 82 indicates two additional examples for strengthening seismic stability of conduits for electric wires by using the effects of the restraint of sand bags and retaining piles together with sand bags.

In constructing underground sewage treatment facilities and semi-buried ditch highways, the effects of soil liquefaction (such as seismic earth pressures and uplifts acting on the structures) should be carefully considered. It is important to reduce the effects of soil liquefaction on the stability of these underground structures. Fig. 82 illustrates two examples of countermeasures for preventing damage due to soil liquefaction, by using gravel drains. Fig. 83(a) can apply to the case of shallow layers, and Fig. 83(b)

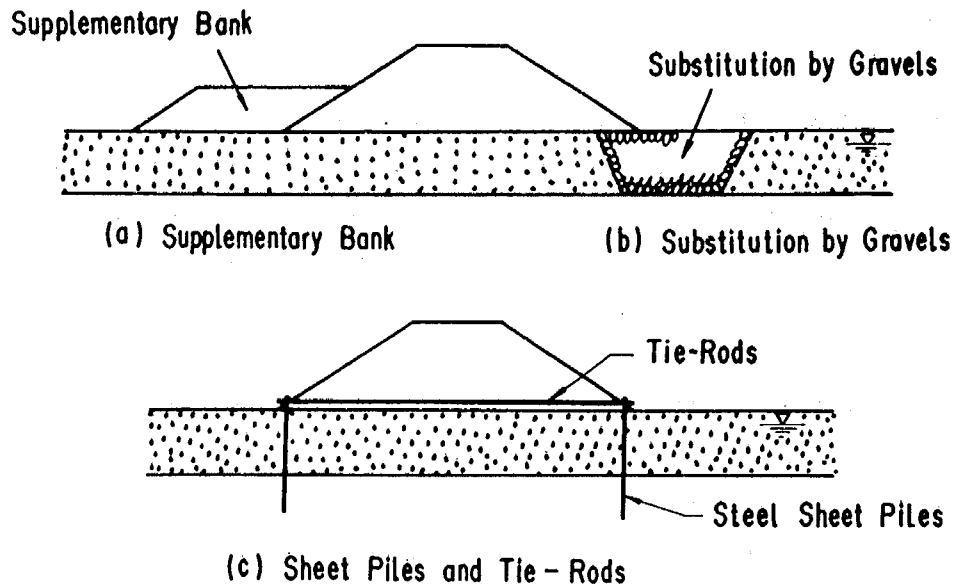


Fig. 74 Liquefaction Countermeasures for Strengthening Existing Embankments (After Suematsu, Yoshimi, and Sasaki (1982)³⁶)

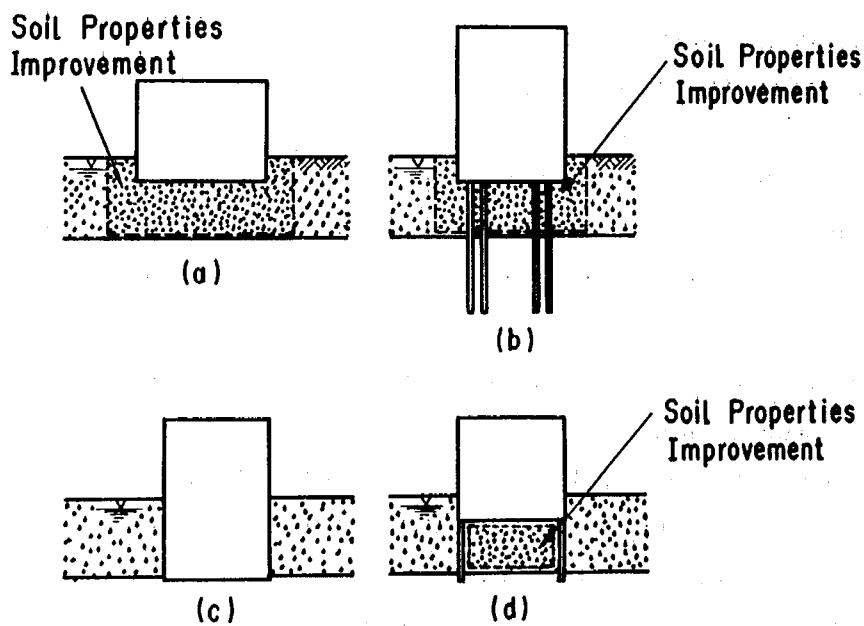


Fig. 75 Four Types of Countermeasures for Building Foundations (After Suematsu, Yoshimi, and Sasaki (1983)³⁶)

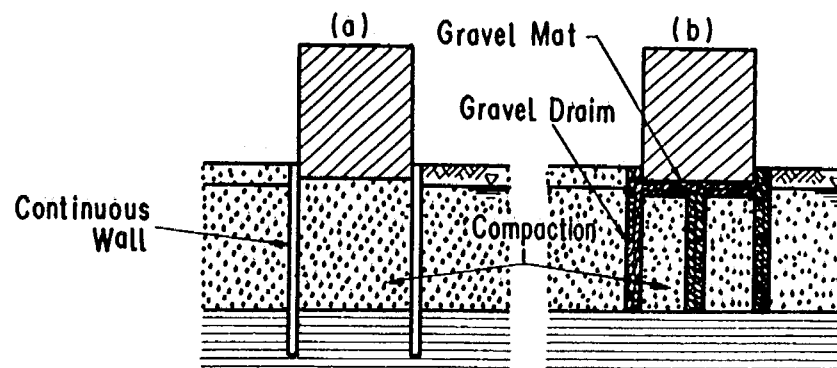


Fig. 76 Countermeasures in Which Two Procedures are Combined

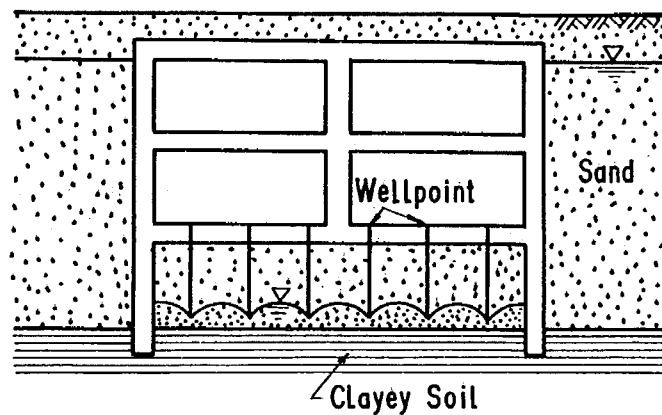


Fig. 77 Countermeasure to Lower Water Table

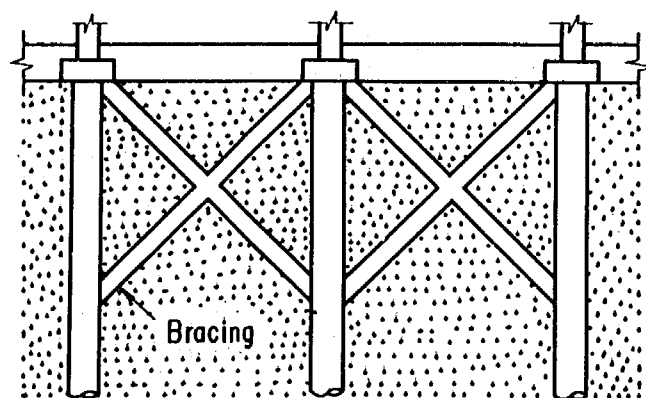


Fig. 78 Structural Improvement Countermeasures for Building^{44), 45)}

Table 19 Relation between Liquefaction Resistance Factor (F_L) and Reduction Factor (D_E)

F_L	Depth, $Z(m)$	Reduction Factor, D_E
$F_L \leq 0.6$	$Z \leq 10$	0
	$10 < Z \leq 20$	$1/3$
$0.6 < F_L \leq 0.8$	$Z \leq 10$	$1/3$
	$10 < Z \leq 20$	$2/3$
$0.8 < F_L \leq 1.0$	$Z \leq 10$	$2/3$
	$10 < Z \leq 20$	1
$1.0 < F_L$	Any	1

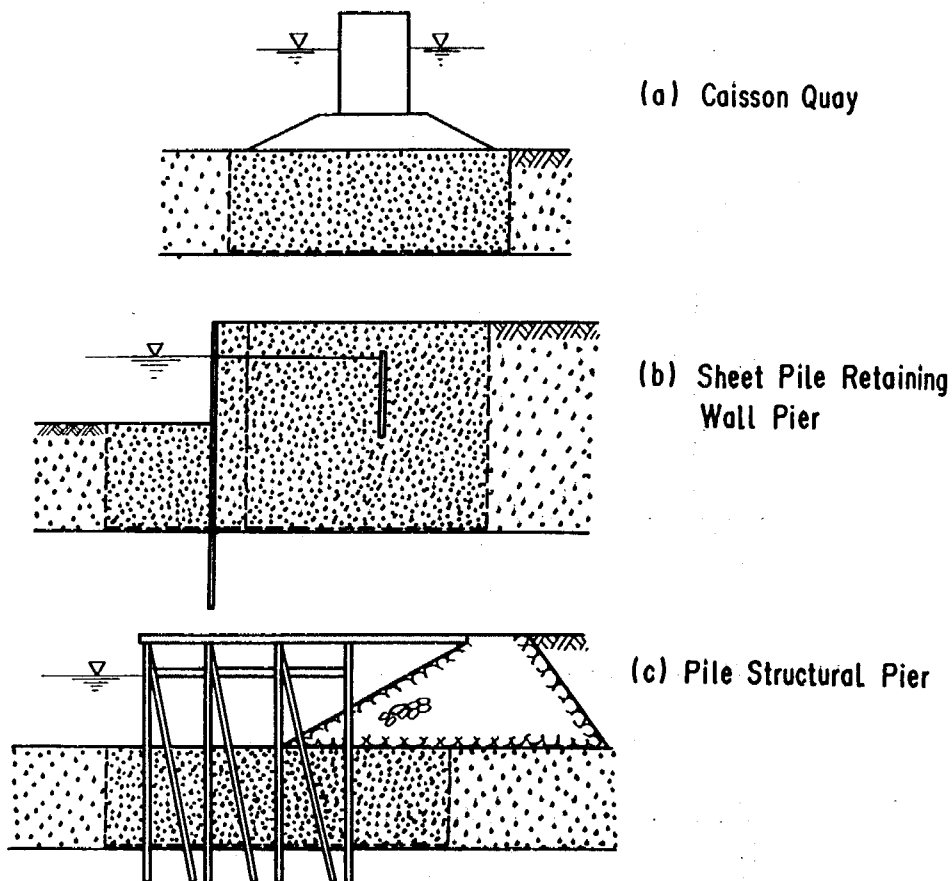


Fig. 79 Countermeasures for Harbor Structures

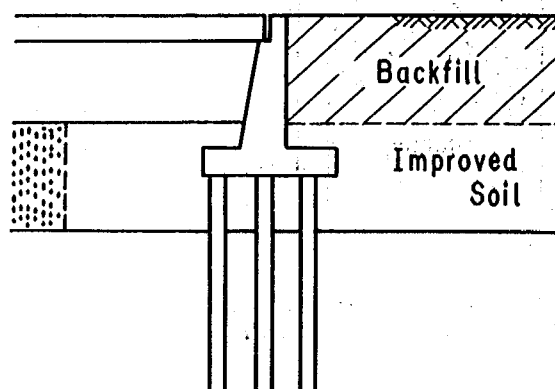


Fig. 80 Liquefaction Countermeasures for Abutment - Soil Improvement and Deep Piles are Jointly Employed

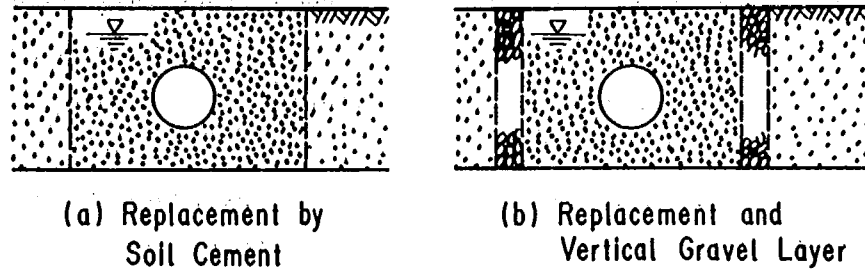


Fig. 81 Example of Countermeasures for Pipelines by Soil Replacement

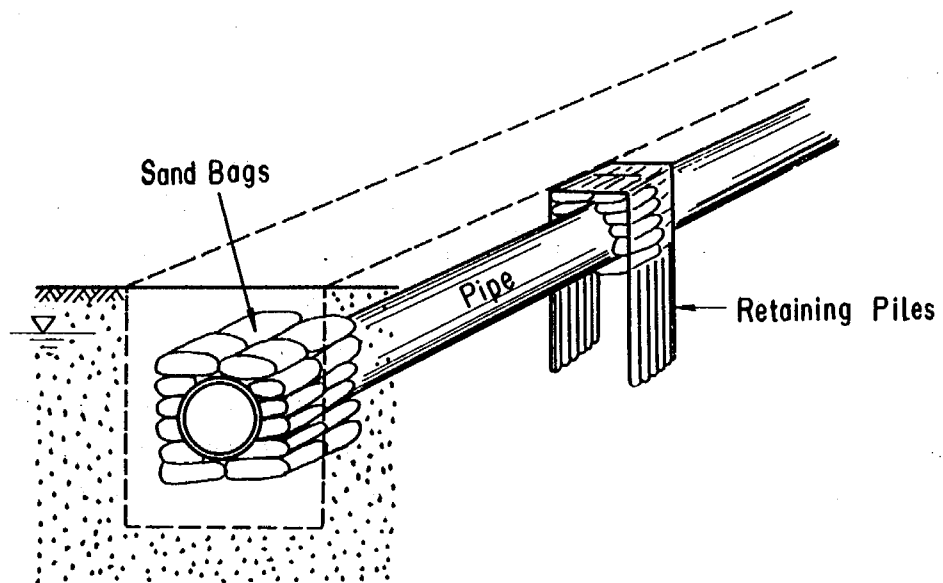


Fig. 82 Restraint of Pipelines by Sand Bags and Retaining Piles

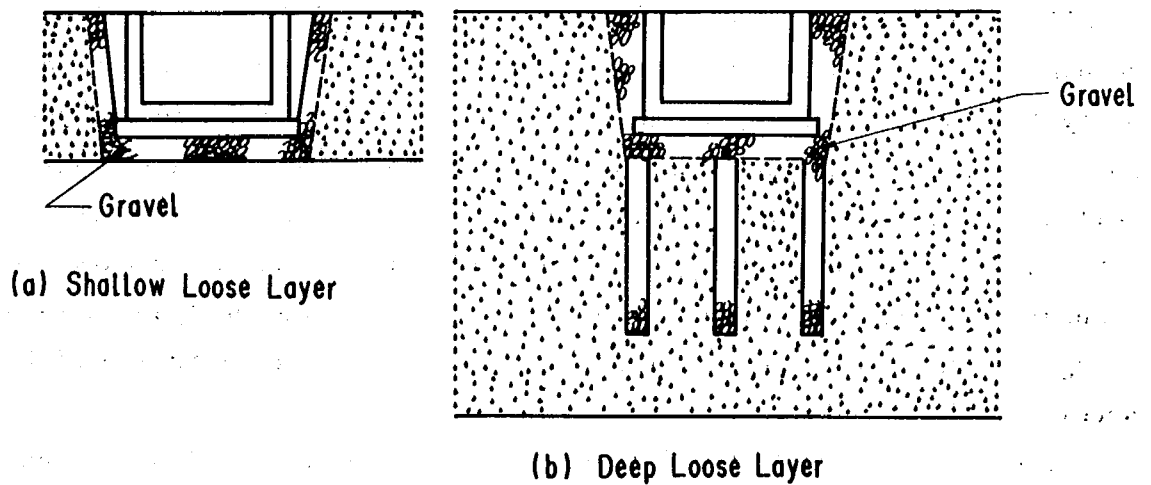


Fig. 83 Liquefaction Countermeasures for Underground Conduits

can apply to the case of deep layers. The gravel zones may have the effects of densification as well as the effects of shutting off the propagation of excessive pore-water pressures and of dissipating the pore-water pressures.

Fig. 84 indicates a result of a shaking table test to evaluate the effects of gravel drains on upward displacements of underground ditch models.⁴⁷ In the extreme left of the figure is the case without gravel drains, and the remaining five cases are those with five types of gravel drains shown in the bottom of the figure. Since the ditch model has the density of 1.05, it will move upward when the surrounding soils completely liquefy. It is seen from the figure that upward displacements for the cases with gravel drains are considerably smaller (one over 4 to 10) than that for the case without gravel drains.

Storage Tank Foundations

In constructing storage tank foundations on liquefiable soil layers, two procedures are normally employed to strengthen the foundations against seismic damage due to soil liquefaction. Fig. 85 illustrates the two procedures. Fig. 85(a) indicates the procedures of soil property improvement such as densification, dissipation of excessive pore-water pressures, soil particle improvements, and hardening. Furthermore, Fig. 85(b) shows the procedures of structural improvement by pile foundations, as well as soil property improvement.

Design Methods for Increasing Soil Strength

As for design methods for increasing soil strength associated with soil liquefaction, simplified procedures specified in the design regulations are usually employed. For very important structures, however, detailed dynamic response analyses are sometimes performed. Fig. 86 shows the comparison of cyclic shear stresses (white circles) obtained by dynamic analyses during an earthquake and cyclic shear strength for (a) before, and (b) after the soil property improvement. Soil improvement work was done for the surface soils up to 20 meters by the sand compaction method with the target of the standard penetration test value of 20. These dynamic analyses were done using the SHAKE program⁴⁷ by inputting the bedrock acceleration with the peak acceleration of 200 gals.

Fig. 87 illustrates fundamental concepts for zones of soil property improvement. From the experiences of the past earthquakes, (a) and (c) did not sustain any damage, although (b) had a serious settlement of about 50 cm and an inclination. Provisions for regulating hazardous facilities specified

by the Fire Defense Agency state that the length of improvement area (L) is two thirds of improvement depth (ℓ), as shown in (a) of Fig. 87. Also L is specified to take a value between 5 and 10 meters. Furthermore, Fig. 87(d) illustrates the zone of soil property improvement proposed by Tsuchida, Iai, and Kurata (1976)⁴⁸. In view of the laboratory experiments, they proposed the area inside the line of A to B shown by dotted marks to be improved. In the figure $\alpha_1 (=45^\circ + \phi/2)$ denotes the angle of passive failure and $\alpha_2 (=45^\circ - \phi/2)$ denotes the angle of active failure, where ϕ is the angle of friction of the soil. In cases where the area of improvement is not sufficient due to site conditions, other procedures such as underground walls, sheet piles, or gravel drains should be jointly implemented.

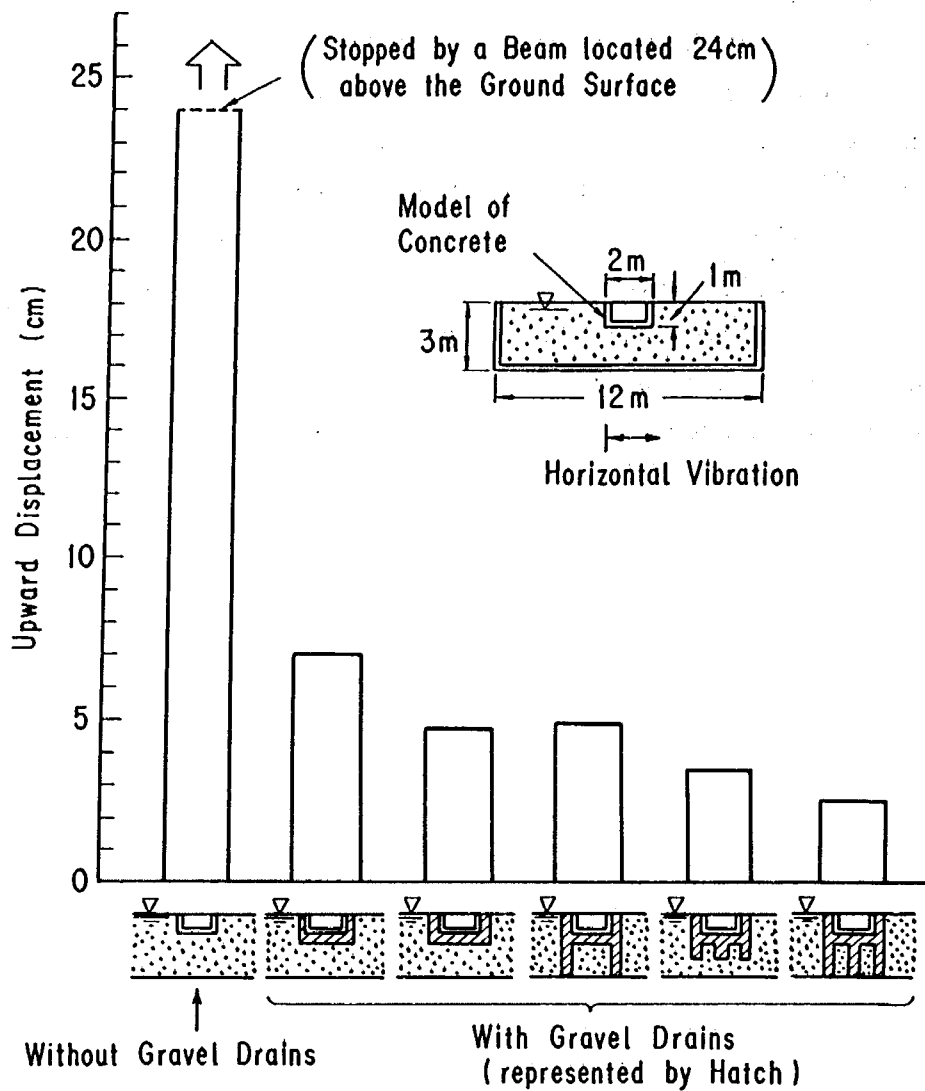


Fig. 84 Upward Displacements of Underground Conduits with and without Gravel Drains

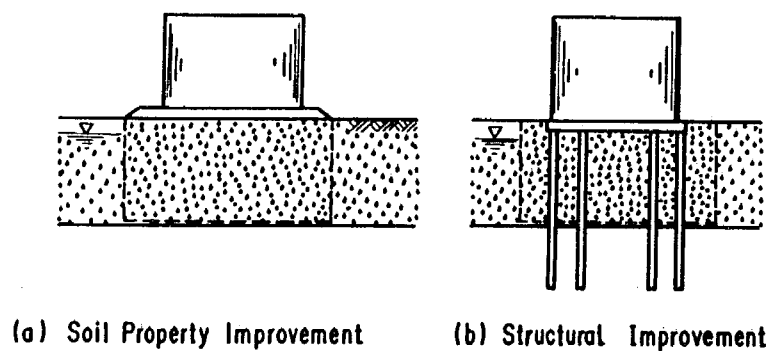


Fig. 85 Countermeasures for Storage Tank Foundations

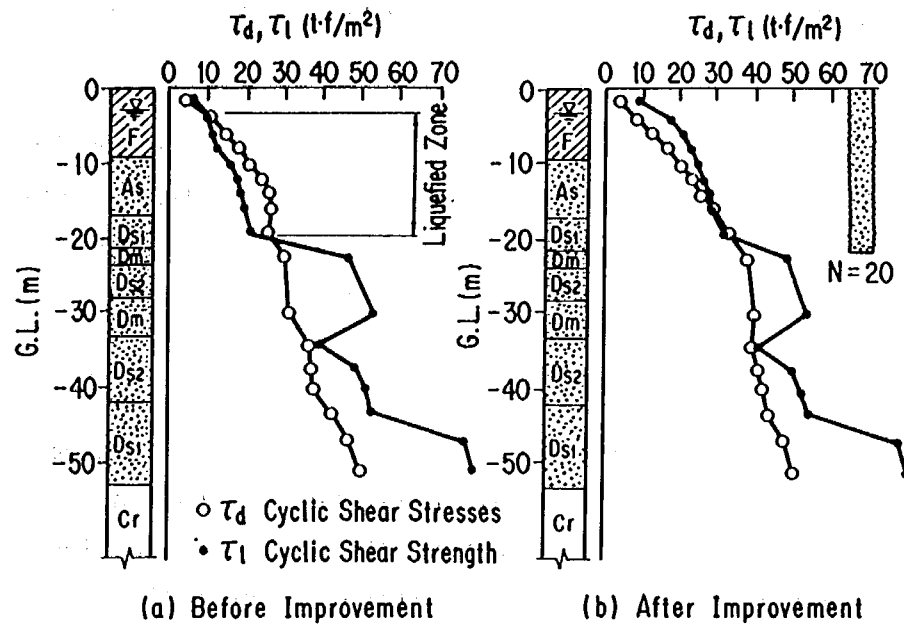


Fig. 86 Comparison of Dynamic Analyses between Before- and After-Soil Improvement

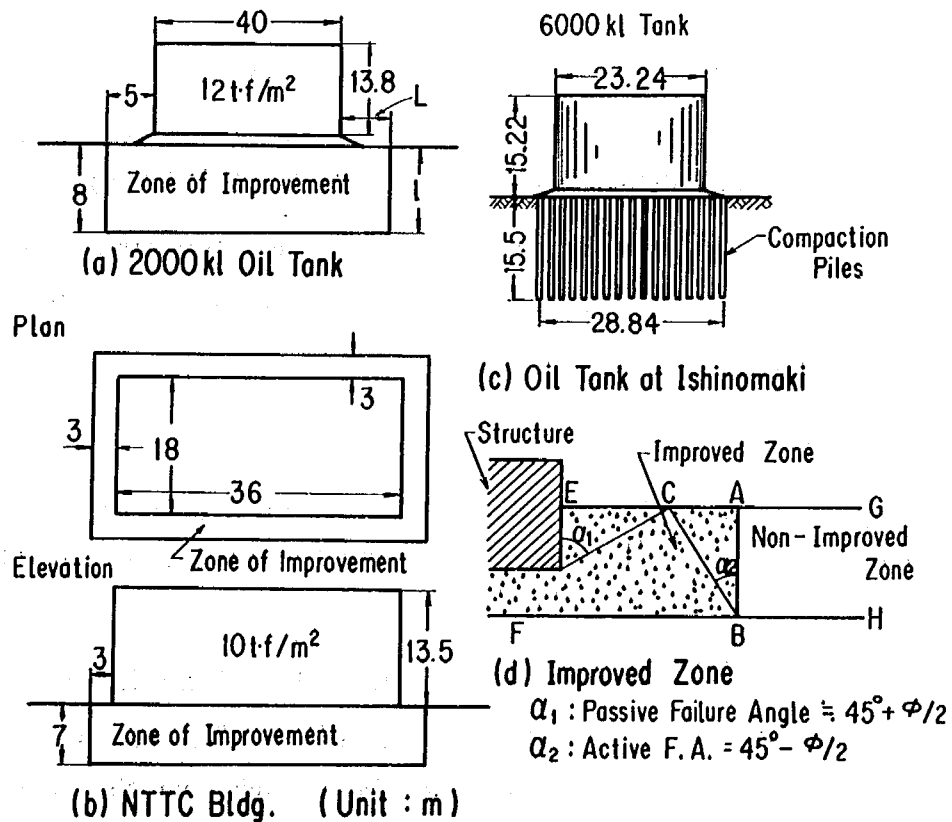


Fig. 87 Fundamental Concepts of Zones of Soil Property Improvement

6. PROVISIONS OF SOIL LIQUEFACTION SPECIFIED IN CURRENT DESIGN CODES

6.1 Outline

There are various provisions regarding soil liquefaction in Japan. In this chapter provisions of soil liquefaction for buildings, highway bridges, port and harbor structures, tailing dams, oil tanks, road earth works, and common utility ducts will be described briefly.

6.2 Buildings (AIJ)

In the Criteria for Design of Building Foundations⁴⁹ issued in 1974 by the Architectural Institute of Japan, it is provided that a specific investigation on the effects of soil liquefaction on building foundations shall be necessary for such sandy grounds as specified in the following:

- a. Soil layers up to the depth of 15-20 meters from the ground surface.
- b. Clean sandy layers of medium size with uniform gradings, namely, clay content less than 10%, mean particle $D_{50} = 0.07 \sim 2.0$ mm (especially, 0.15 - 1.0 mm), and a coefficient of uniformity (U_c) less than 10 (especially, less than 5),
- c. Saturated layers below the water table, and
- d. Loose layers (the definition of loose layers can be evaluated by SPT N-values and depth relationship given by Fig. 88).

Compaction of sands, substitution of sands, and lowering of the water table are recommended as effective countermeasures to prevent the occurrence of soil liquefaction.

6.3 Highway Bridges

In the Specifications for Earthquake Resistant Design of Highway Bridges⁵⁰ issued in 1980 by the Japan Road Association, the provisions for liquefaction are provided as follows:

a. Sandy Soil Layers Need to be Checked for Liquefaction.

Saturated alluvial sandy layers which have the high water table within 10 meters of the ground surface, and have the mean particle diameter (D_{50}) on the grain size accumulation curve between 0.02 and 2.0 mm, may be vulnerable to liquefaction for the depth between 0 and 20 meters, and liquefaction potential of these layers shall be judged according to Item b which follows.

b. Judgment of Liquefaction. For those layers which may be vulnerable to liquefaction according to Item a, liquefaction potential shall be checked, based on the liquefaction resistance factor, F_L , defined by the following equation.

$$F_L = \frac{R}{L} \quad (19)$$

where

F_L : liquefaction resistance factor,

R : the resistance of soil elements to dynamic loads, and

$$R = R_1 + R_2 \quad (20)$$

R_1 and R_2 can be determined in accordance with Figs. 89 and 90, respectively.

L : the dynamic loads of the soil elements induced by the earthquake motion to be considered in design, and

$$L = r_d \cdot k_s \cdot \frac{\sigma_v}{\sigma_v'} \quad (21)$$

$$r_d = 1.0 - 0.015z, \quad (22)$$

z : depth from the ground surface (m)

k_s : seismic coefficient for evaluation of liquefaction and can be determined by

$$k_s = v_1 \cdot v_2 \cdot v_3 \cdot k_{s0} \quad (23)$$

v_1 : seismic zone factor; $v_1 = 1.0, 0.85$ and 0.7 are given for A (active), B (moderate), and C (less active) zones, respectively, in terms of the past seismic activities.

v_2 : ground condition factor; $v_2 = 0.9, 1.0, 1.1$, and 1.2 are given for 1st (rock), 2nd (diluvial), 3rd (alluvial), and 4th (soft alluvial) ground conditions, respectively.

Sites where liquefaction occurrence is a concern generally have either 3rd or 4th ground conditions.

v_3 : importance factor of the bridge. $v_3 = 1.0$ is normally taken.

- k_{s0} : design seismic coefficient to be considered at the input bedrock level. $k_{s0} = 0.15$ is taken as the standard.
- σ_v : total overburden pressure ($\text{kg}\cdot\text{f}/\text{cm}^2$)
- σ'_v : effective overburden pressure at the static condition ($\text{kg}\cdot\text{f}/\text{cm}^2$)

Soil layers having the liquefaction resistance factor, F_L , smaller than 1.0 shall be judged to be likely to liquefy during the design earthquake. Figs. 89 and 90 are graphic illustrations of the first term R_1 and the second term R_2 represented by equation (24), which is proposed in view of the results of laboratory dynamic triaxial tests on soil specimens taken from several sites in Japan, as described in 3.2 of this paper.

$$R = \begin{cases} 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.19 & (0.02 \leq D_{50} \leq 0.05^{\text{mm}}) \\ 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.225 \log_{10} \left(\frac{0.35}{D_{50}} \right) & (0.05 \leq D_{50} \leq 0.6^{\text{mm}}) \\ 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} - 0.5 & (0.6 \leq D_{50} \leq 2.0^{\text{mm}}) \end{cases} \quad (24)$$

c. Treatment of Soil Layers Judged to Liquefy

For those soil layers which are judged to be likely to liquefy by Item b in the above, bearing capacities, coefficients of subgrade reactions, and other soil constants shall be either neglected or reduced intentionally in the seismic design, by multiplying the original static soil constants by the reduction factor, D_E . Values of the reduction factor, D_E , are given in Table 19, with reference to the laboratory experiments done by Iwasaki, Tokida, and Kimata.⁵¹

d. Example of Application

Judgment of liquefaction and an example of the application to bridge design will be briefly described for the case of Showa Bridge^{52,53} according to the above mentioned provisions. Showa Bridge sustained heavy damage during the Niigata Earthquake of 1964, as shown in Figs. 91 and 92. Five central spans with simply supported girders fell down into river beds. One of the causes of the damage is supposed to be considerable liquefaction of surrounding sandy soils.

An investigation on liquefaction potential and seismic displacement

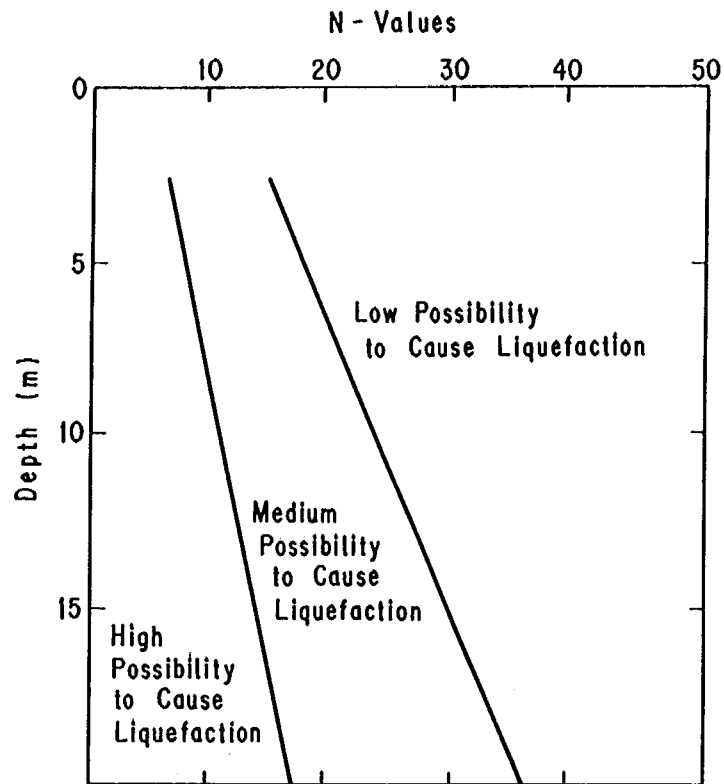


Fig. 88 Design Critical SPT N-Values to Cause Soil Liquefaction Specified in the Building Code in Japan

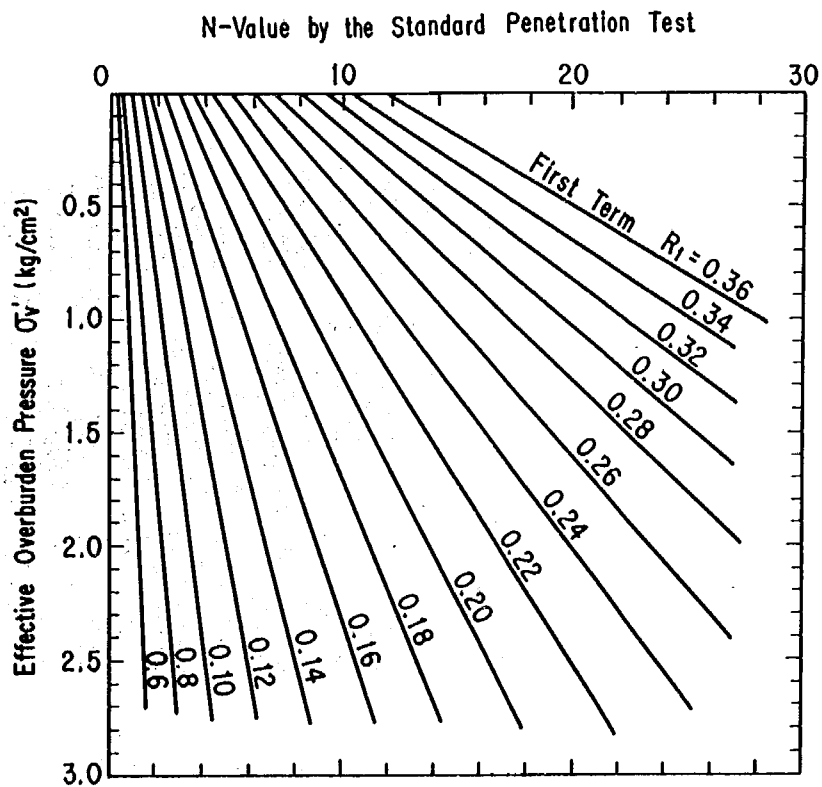


Fig. 89 R_1 -Value (First Term of Resistance R)

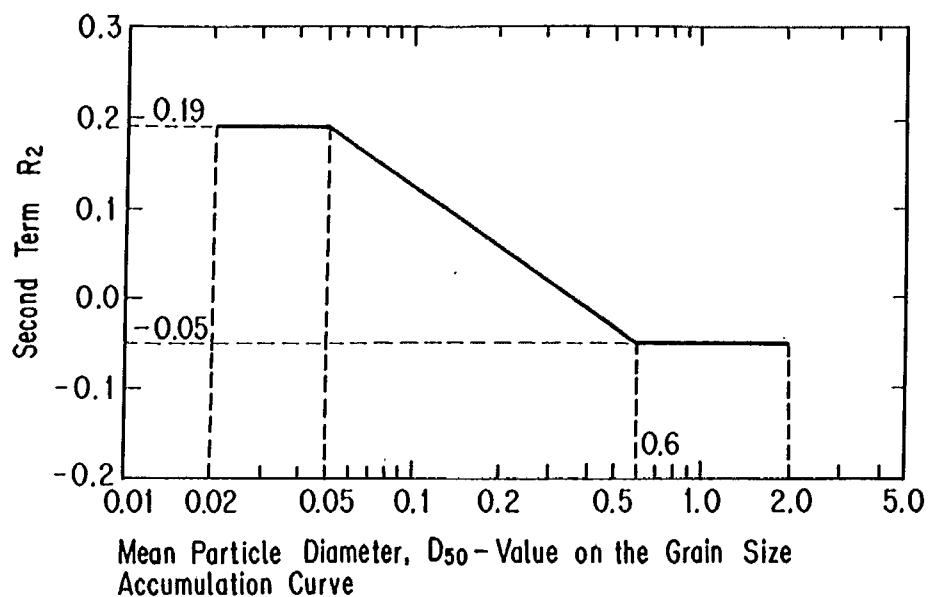


Fig. 90 R_2 -Value (Second Term of Resistance R)



Fig. 91 Photograph of Showa Bridge Damaged during the Niigata Earthquake of 1964

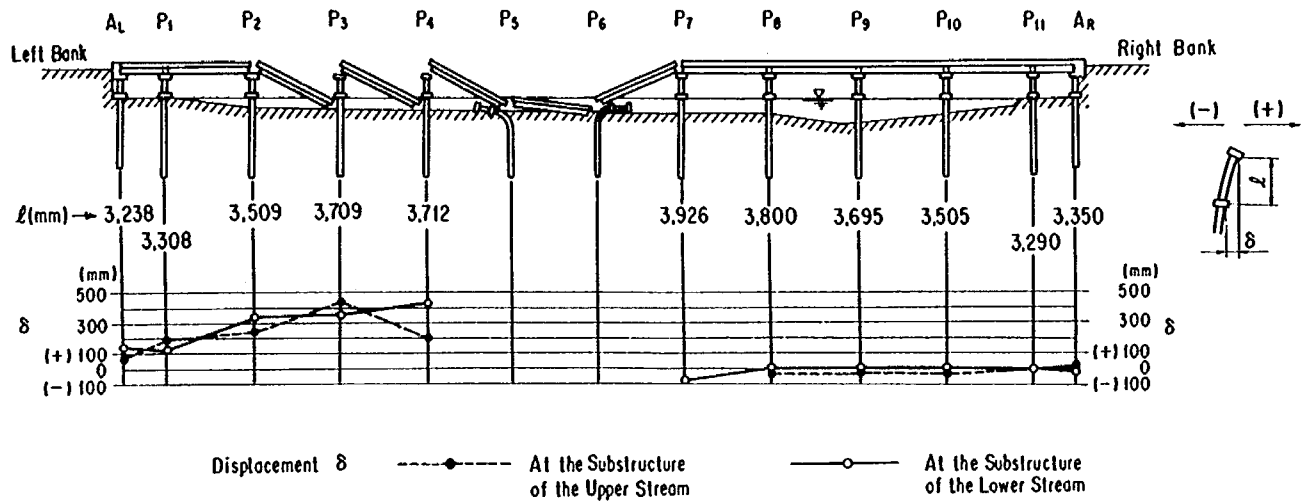


Fig. 92 Side View of Showa Bridge Damaged during the Niigata Earthquake of 1964

of the bridge is done for Pier 5 of the Bridge which failed completely. Fig. 93 illustrates an outline of the substructures for both before and after the Niigata Earthquake of 1964, and the results of the assessment of soil liquefaction using the liquefaction resistance factor, F_L . After the Niigata Earthquake all substructures were reinforced by doubling the number of piles. According to F_L -values computed with depth shown in Fig. 92, the ground supporting Pier 5 is estimated to be likely to liquefy up to the depth of 10 meters from the surface.

Seismic displacements of the crown of the Pier 5 under the conditions for both before and after the reinforcement are calculated by using the conventional seismic coefficient method specified in the Specifications for Highway Bridges, Part IV, Substructures.⁵⁴ In the analysis, three values of horizontal seismic coefficients, k_h , are taken as 0.15, 0.20 and 0.25, although the design coefficient is $k_h = v_1 \cdot v_2 \cdot v_3 \cdot k_0 = 0.85 \times 1.2 \times 1.0 \times 0.2 = 0.20$, according to the current specifications. In the analysis, the coefficients of lateral subgrade reaction, K (in $\text{kg} \cdot \text{f}/\text{cm}^3$) at the depth of $1/\beta$ of 400 cm ($\beta = \{KD/(4EI)\}^{1/4}$, D = diameter of pile (in cm), E = Young's modulus (in $\text{kg} \cdot \text{f}/\text{cm}^2$), I = moment of inertia of pile (in cm^4)) are changed according to the extent of soil liquefaction. The original K -value is estimated as $K_0 = 1.0 \text{ kg} \cdot \text{f}/\text{cm}^3$ from the static field survey. Four values of K -values of 0, 0.33, 0.67, and $1.0 \text{ kg} \cdot \text{f}/\text{cm}^3$ are taken in the analysis by assuming D_E -values of 0, $1/3$, $2/3$, and 1.0, from Table 19.

Fig. 94 summarizes a relationship between the maximum displacement (δ in cm) at the pier crown, the reduction factor (D_E) and the coefficient of lateral subgrade reaction (K in $\text{kg} \cdot \text{f}/\text{cm}^3$). It is found from this figure that the displacements at the pier crown increase according to the decrease in the D_E -value (or K -value), namely the increase of the extent of soil liquefaction. It is also noted that the displacements remarkably increase when the coefficient of subgrade reaction is zero. It is also seen that the displacements considerably decrease when the reinforcement work was implemented after the occurrence of the Niigata Earthquake of 1964. As shown in the above, it is confirmed that a consideration of the effects of soil liquefaction plays an important role in seismic design of bridges.

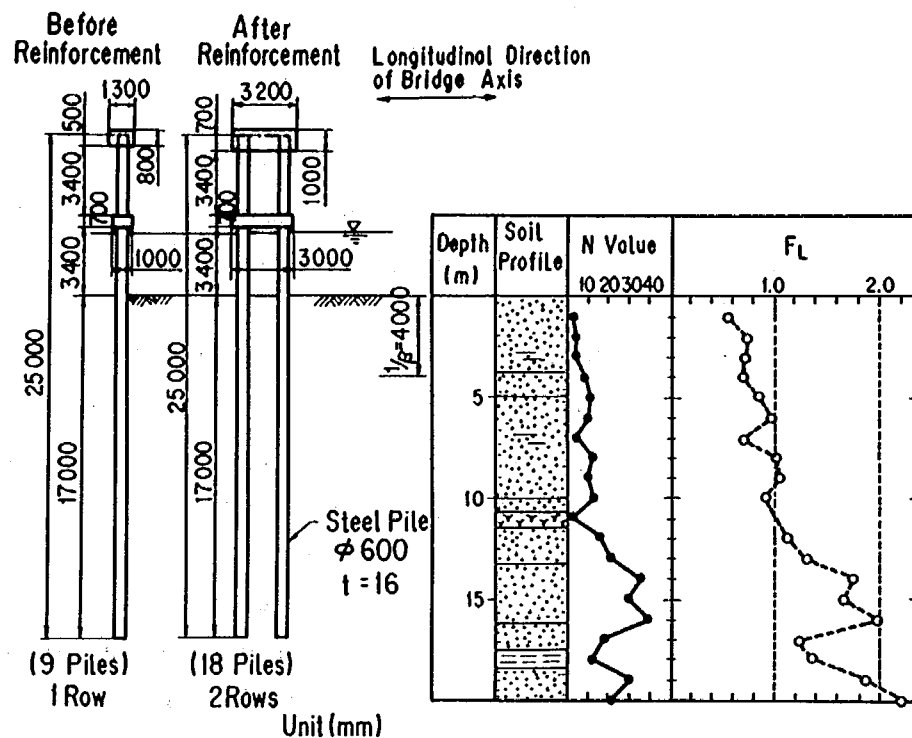


Fig. 93 Showa Bridge Pier 5 and Soil Liquefaction Assessment with Use of F_L -Values

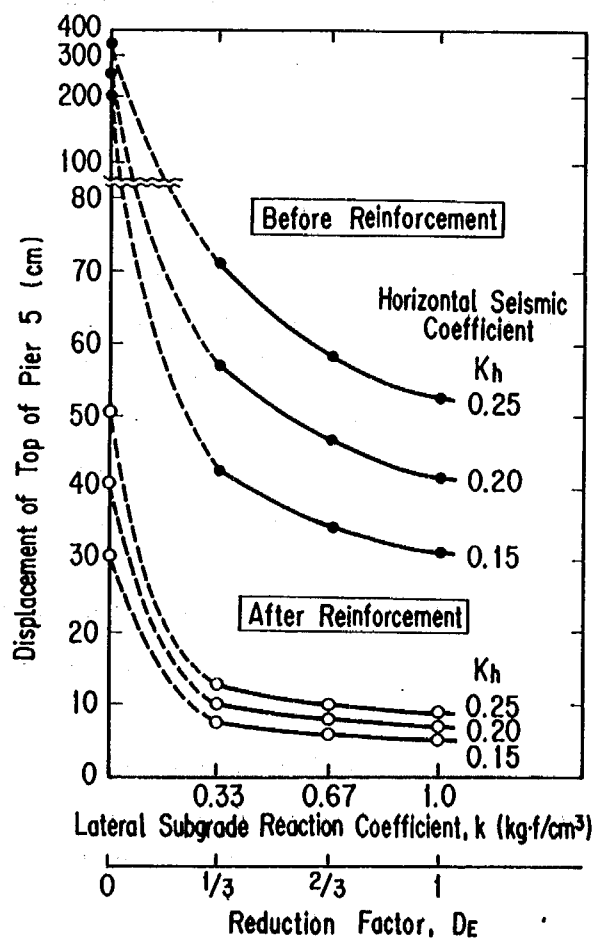


Fig. 94 Relationship between Maximum Displacement of Top of Pier 5 and Reduction Factor, D_E or Lateral Subgrade Reaction Coefficient of Grounds, k (kgf/cm³)

6.4 Harbor Structures

Soil Liquefaction Assessment for Harbor Structures

In designing harbor structures, Technical Standards of Harbor Facilities (1979)⁵⁵ are generally employed. In this manual, a procedure for assessing soil liquefaction potential is described.

Liquefaction potential may be evaluated from particle size distribution curves and SPT (Standard Penetration Test) N-values. First of all, a particle size distribution curve of the soils below the water table can be compared with the range of grain size distribution curves of sands which have high possibility to cause liquefaction and are shown in Fig. 2 of 1.2. When the particle size distribution curve under consideration is in the range of "Easily liquefy" in Fig. 2, the SPT N-values for soils below the water table at the site will be compared with the critical N-values for the assessment of liquefaction occurrence. Critical SPT N-values are shown in Fig. 3 of 1.2. If the N-value under consideration is smaller than the critical N-value, the soils under consideration will be judged to liquefy during earthquakes. It should be noted that only those soils which are assessed to liquefy from the particle size distribution curve and also the N-value will be judged to liquefy for the design purposes.

Example of Practical Use

In designing harbor structures, liquefaction potential of sandy soils is generally evaluated and some countermeasures are executed when necessary.

One of the typical examples of the assessment of liquefaction potential is seen in the design and construction of Sinsuna Gate in Tokyo Port. This gate has been constructed in order to protect the low land area of Tokyo from flooding due to storm surges. The seismic design coefficient for the gate was taken as 0.25.

The subsoil at this gate site is very soft cohesive soil, and the bearing capacity is very low. Therefore, it was decided that the soil of the sea bottom with a thickness of 12 meters be dredged and the soil be replaced by clean sands. A schematic cross-section of the foundation following the soil replacement is shown in Fig. 95.

The liquefaction potential of the sand layer made by the soil replacement was assessed in the following way.

The grain size distribution curve of the sand to be replaced was compared with ranges of curves for sands which have a possibility of liquefaction occurrence. The comparison is illustrated in Fig. 96. It is seen from the

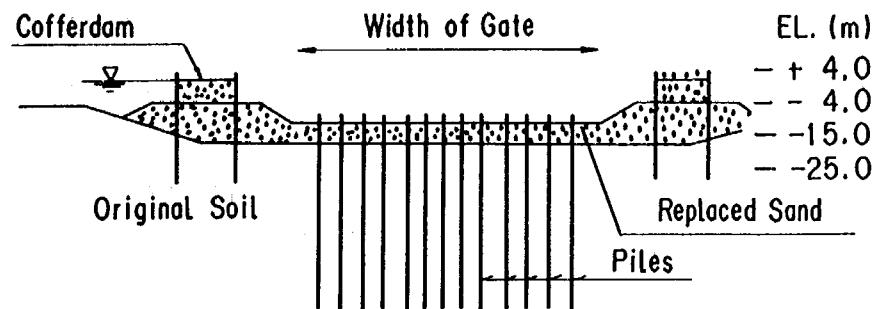


Fig. 95 Cross Section of a Site after Soil Replacement

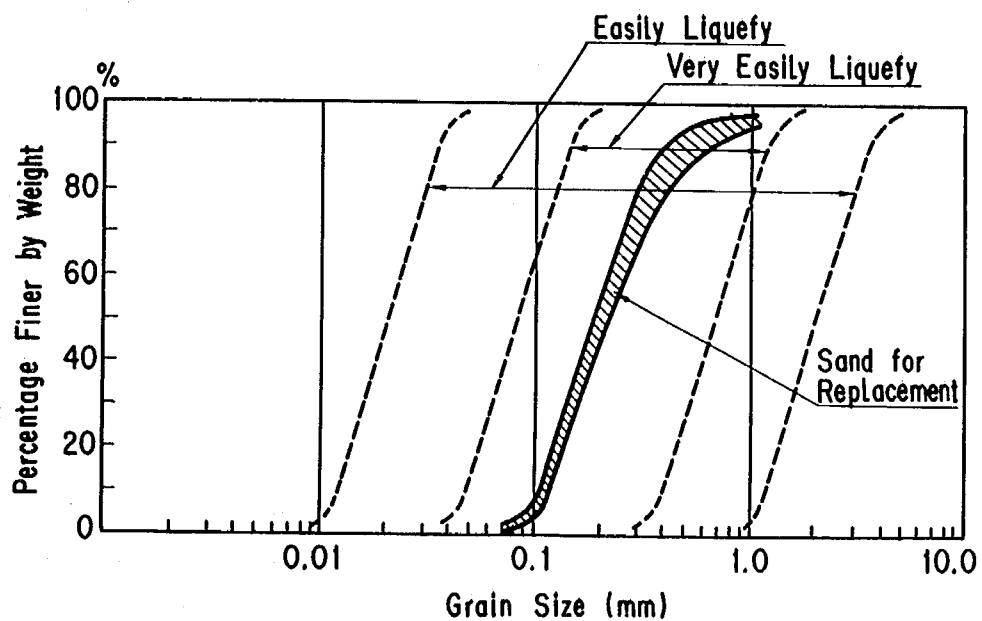


Fig. 96 Grain Size Distribution Curve of Sand for Replacement Comparison with Distribution of Liquefaction Vulnerability

figure that the particle size distribution curve is in the range of "very easily liquefy."

Then, the critical SPT N-values causing liquefaction was estimated from Fig. 3. Correlation between the design seismic coefficient and the peak acceleration of earthquake ground motion was uncertain. It will be conservative, however, to consider that the peak acceleration is not less than the product of the gravity acceleration and the design seismic coefficient. Accordingly, it seems that the acceleration of 250 gals can be the lowest value of the peak ground acceleration for an earthquake corresponding to the design seismic coefficient of 0.25. A critical SPT N-value for this ground acceleration is obtained about 14 from Fig. 3.

The SPT N-value of the sand layer was predicted based on the experiences of similar replacement works. It is concluded that the SPT N-value of that layer would be considerably smaller than the critical N-value of 14. Therefore, the sand layer was judged to cause liquefaction under the design earthquake.

Shaking table tests were performed using sands to be used for the replacement. The tests also indicated that the sand in loose condition would liquefy when the sand was subjected to ground motions to be considered in the design. The sand layer to be filled following dredging was compacted in order to make it stable against liquefaction phenomena. After the compaction, undisturbed sand sampling and the standard penetration tests were carried out to confirm that the compaction was satisfactory.

6.5 Tailing Dams

In view of the experience of the failure of Mochikoshi Tailing Dam which sustained heavy damage during the Izu-Oshima Kinkai Earthquake of 1978 (see 2.3 of this paper), the Ministry of International Trade and Industry amended in 1980 "Specifications for Construction of Tailing Dams."⁵⁶ The provisions on soil liquefaction in the specifications may be summarized as follows:

1. Saturated cohesionless soil layers which exist in the depth within 20 meters from the surface and have standard penetration test N-values of 20 or less, shall have potential to cause liquefaction during earthquakes. For those layers with liquefaction potential, the increase in pore-water pressures due to soil liquefaction shall be considered in the seismic stability analysis. The conventional seismic coefficient method assuming circular slippage slices can be employed in the seismic design.

2. The increase in pore-water pressure (U_ℓ) due to soil liquefaction shall be estimated by the following equation:

$$U_\ell = \begin{cases} 0 & \text{for } F_L > 1.25 \\ 0.3\sigma'_v & \text{for } 1.0 \leq F_L \leq 1.25 \\ \sigma'_v & \text{for } F_L < 1.0 \end{cases} \quad (25)$$

where σ'_v is the effective overburden pressure and F_L is the liquefaction resistance factor.

3. The liquefaction resistance factor (F_L) shall be evaluated by the equation

$$F_L = \frac{R}{L} \quad (26)$$

where R denotes the in situ resistance of soil elements to dynamic loads, and L denotes the dynamic loads to the soil elements induced by the earthquake motion.

4. The in situ soil resistance (R) can be obtained by

$$R = 1.2 R_\ell \quad (27)$$

where R_ℓ is the laboratory shear strength ratio to cause liquefaction and can be determined by the cyclic triaxial compression test for undisturbed soil specimens sampled from the field. When the use of the cyclic triaxial compression test is impossible, the following equations can be used to estimate the values of R_ℓ :

- (a) For tailings, dam materials, and deposits made from mining materials of hard rocks such as black ore,

$$R_\ell = 0.088 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.2. \quad (28)$$

- (b) For tailings, dam materials, and deposits made from mining materials other than the above (a),

$$R_\ell = 0.088 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.85 \log_{10} \frac{0.50}{D_{50}} \quad (29)$$

(c) For sedimentary deposits (silts),

$$R_{\ell} = 0.088 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.10 \quad (30)$$

In Eq. (29) the minimum value of R_{ℓ} can be 0.15, and D_{50} (mean particle diameter) can take 0.004 mm for $D_{50} < 0.004^{\text{mm}}$, and 0.4^{mm} for $D_{50} > 0.4^{\text{mm}}$. Furthermore, in Eq. (30) the value of R_{ℓ} shall be zero for $N < 1$.

5. The dynamic load (L) shall be obtained either by the earthquake response analysis or the following empirical equation.

$$L = \frac{4}{3} k_h \left(\frac{\sigma_v}{\sigma'_v} \right) (1 - 0.025z) \quad (31)$$

where k_h denotes the seismic design coefficient of the tailing dam specified in the Specifications ($k_h = 0.1$ to 0.15 depending on the construction site and the type of the tailing dam), σ_v and σ'_v are the total and effective overburden pressures ($\text{kg}\cdot\text{f}/\text{cm}^2$), respectively, and z is the depth (m).

6.6 Oil Tanks

The Fire Defence Agency⁵⁷ specifies the following on soil liquefaction in the Notification Specifying Detailed Provisions of Technical Criteria for Regulating Dangerous Articles such as oil tanks.

1. Saturated sandy soils which have the mean grain size of D_{50} of 2.0^{mm} or less and have the standard penetration test N-value less than the critical values shall be judged to cause liquefaction. The critical values are indicated in Table 20. In the table Case A is applicable to sands within an area of a circle having a center at the center of the base of an oil tank and a radius equal to the radius of the tank minus 5 m. Case B is applicable to sands outside an area defined in the above, and inside an area of a circle having the same center and a radius equal to the radius of the tank plus L, where $L (= 2/3\ell)$ and shall be between 5 and 10 m. ℓ (the depth from the ground surface) is equal to 15 m for a horizontal soil layer and the necessitated depth for the other cases.

Table 20 Critical SPT N-Values for Assessment of Liquefaction
for Dangerous Articles such as Oil Tanks (After Fire
Defence Agency)

CONTENTS OF FINE PARTICLES (< 0.075 mm), S	CRITICAL SPT N-VALUE	
	CASE A	CASE B
$S < 5\%$	12	15
$5 \leq S \leq 10$	8	12
$10 \leq S \leq 35$	6	7

2. Those sandy layers which are judged to cause liquefaction should be appropriately strengthened.

6.7 Earth Works for Roads (JRA)

In the Guidelines for Earth Works on Soft Grounds (a part of Earth Works for Roads) issued by the Japan Road Association,⁵⁸ consideration of seismic resistance of embankments on loose sandy grounds is required.

1. It is stated that liquefaction potential of sandy grounds depends on the following factors:
 - (1) Intensity and Duration of Earthquake Ground Motions. Liquefaction of grounds can be caused by strong ground motions capable of generating dynamic shearing stresses in the grounds higher than a certain limit determined by the soil conditions.
 - (2) Density of Sands (Void Ratio, Relative Density, or SPT N-Value). It is known that the higher the density of the soil, the less likely liquefaction is to occur, and liquefaction hardly ever happens in the ground with the relative density over 75 to 80%. SPT N-values of the ground which have liquefaction possibility will vary with the groundwater level and the magnitude of surcharge. A figure indicating relations between the critical SPT N-values and the depth from the ground surface is given.
 - (3) Grain Size Distribution.
The grain size distribution of sandy soils has a remarkable influence on liquefaction potential. A figure indicating the range of grain size distribution of soils in which liquefaction is likely to occur is given.
 - (4) Surcharge, Groundwater Level, and Soil Characteristics.
Dynamic shearing stresses to induce liquefaction increase with an increase in the initial confining pressure. Therefore the liquefaction possibility will decrease at deeper positions in the ground, and also in ground just underneath large embankments. Occurrence of liquefaction varies remarkably with soil characteristics such as layering and uniformity of ground soil.

2. Two principal procedures for assessing liquefaction potential are proposed:

- (1) Procedure based on laboratory cyclic triaxial test,
- (2) Procedure based on in situ soil conditions.

As an example of the second procedure, the procedure employed in the specifications for highway bridges (mentioned in 6.3 in this paper) is introduced.

Liquefaction of sandy layers is influenced by complicated factors, and so assessment of liquefaction should be made with a careful examination using available information and damage experiences of soil liquefaction.

3. When a sandy ground is judged to have a possibility of liquefaction at road embankment construction sites, appropriate countermeasures which can be determined in view of the characteristics of the road, the scale of embankment, and the environmental conditions shall be executed.

6.8 Common Utility Ducts

The Research Center for Land Development Technology recently issued the Seismic Design Manual for Common Utility Ducts⁵⁹ with the commission from the Ministry of Construction. Common utility ducts are newly constructed under main streets in large cities, with the aim to install public utility facilities such as water supply pipes, gas supply pipes, electric wires, and telephone wires.

It is stipulated in the Manual that the assessment of liquefaction shall be made in accordance with the procedures described in 3.2 and 3.3 of this paper. A characteristic feature of the Manual is that it provides an article on safety of common utility ducts against uplift due to soil liquefaction during earthquakes, because it is concluded that the uplift of ducts may cause serious damage to ducts as well as roads. The provisions can be summarized as follows:

1. The factor of safety (F_u) against uplift due to soil liquefaction shall be calculated by (See Fig. 97)

$$F_u = \frac{W_s + W_B + Q_s + Q_B}{U_w + U_\alpha} \quad (32)$$

where

- W_s : Overburden soil load acting on the top of duct (t.f)
- W_B : Dead weight of duct (including inside utility facilities) (t.f)
- Q_s : Shear resistance of overburden soil (t.f)
- Q_B : Friction resistance between the side walls of duct and side soils (t.f)
- U_w : Uplift due to hydrostatic pressure acting on the base of duct (t.f)
- U_α : Uplift due to excessive pore-water pressure acting on the base of duct during earthquakes (t.f)

$$U_\alpha = L_u \cdot B \cdot \sigma'_v \quad (33)$$

- L_u : Excessive pore-water pressure ratio
($L_u = \Delta u / \sigma'_v$)
- B : Width of the base of duct (m)
- σ'_v : Effective overburden pressures at the level of the base of duct (t.f/m²)
- Δu : Excessive pore-water ratio (t.f/m²)

All quantities in the above shall be obtained for unit length of the duct. For soil layers which will be judged to liquefy (or liquefaction resistance factor, $F_L < 1.0$), the values of Q_s and Q_B shall be zero. For estimating L_u , Fig. 98 shows a relation between F_L (liquefaction resistance factor) and L_u , which was obtained from shaking table experiments of model sands conducted at PWRI.⁶⁰

2. The factor of safety (F_u) of duct against uplift shall be greater than unity. When the factor is less than unity, appropriate counter-measures shall be taken. In the Manual several examples of counter-measures including soil improvement measures and structural improvement measures are shown.

3. For several sites of common utility ducts to be constructed in Tokyo, Nagoya, and Osaka areas, factors of safety against uplift were calculated according to Eq. (32). In the calculation, the liquefaction resistance factor (F_L) and the liquefaction potential index (P_L) described in 3.2 and 3.3, respectively, were also obtained. As a result, Fig. 99 illustrates a relation between P_L and F_u . It is seen from this figure that the liquefaction potential index (P_L) can be used as an indicator for the safety of ducts against uplift due to soil liquefaction. It seems appropriate that a duct at a site with a P_L -value of 15 or higher has a high potential to cause uplift during an earthquake, and therefore its safety against uplift due to soil liquefaction shall be carefully investigated and effective countermeasures shall be conducted.

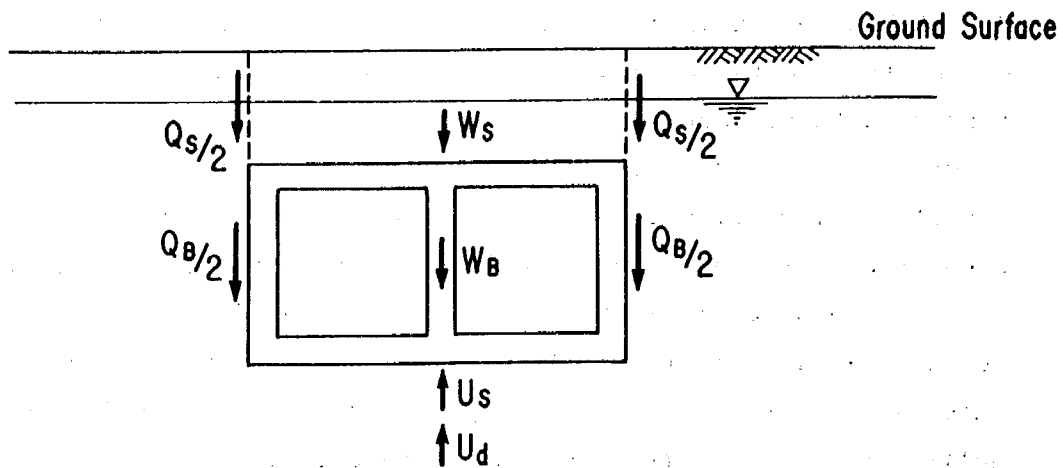


Fig. 97 Factor of Safety against Uplift due to Soil Liquefaction (F_u)

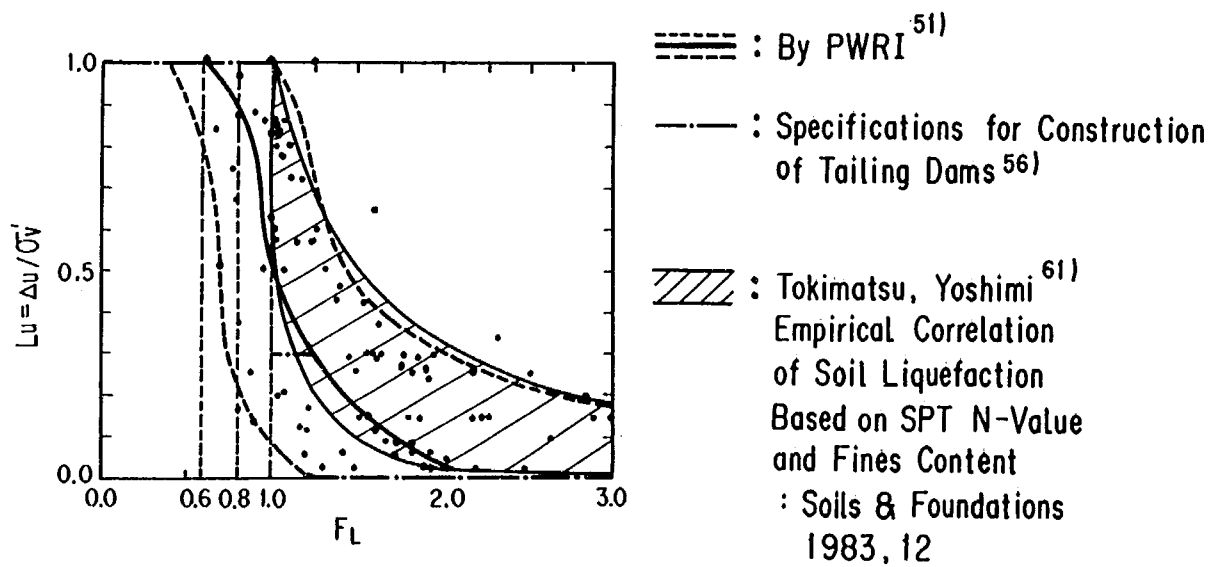


Fig. 98 Relation between Liquefaction Resistance Factor (F_L) and Excessive Pore-Water Ratio ($L_u = \Delta u / \sigma_v'$)

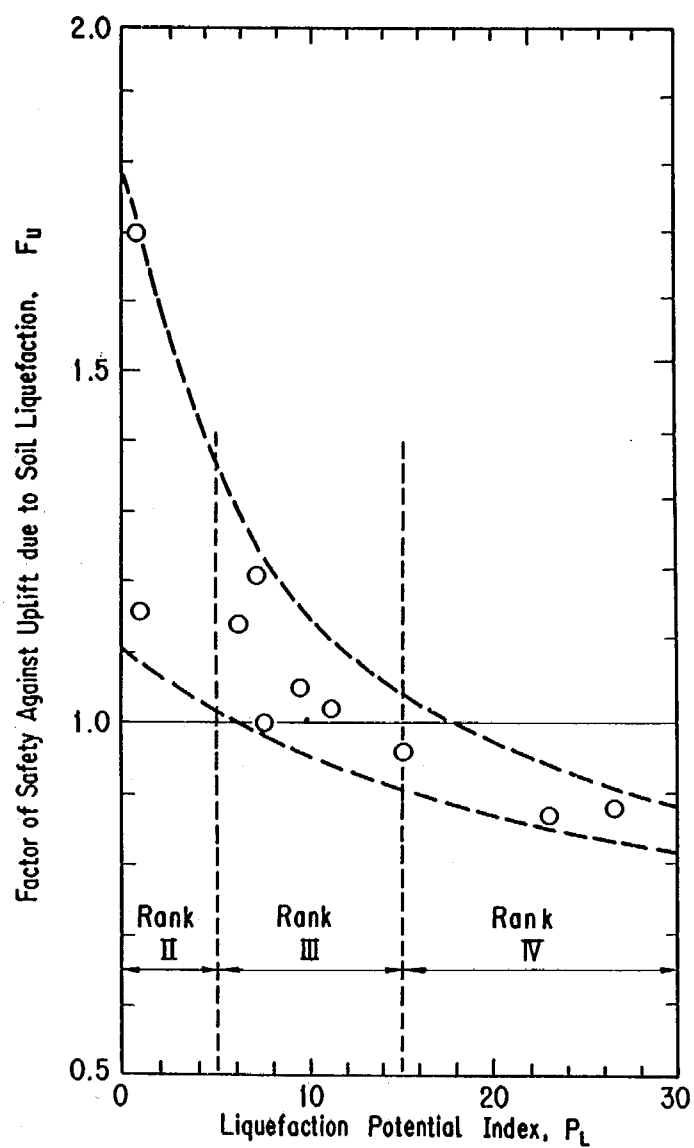


Fig. 99 Relation between Liquefaction Potential Index (P_L) and Factor of Safety against Uplift due to Soil Liquefaction (F_u)

7. CONCLUSIONS

In this state-of-the-art paper, the author attempts to describe some important results of earthquake-induced soil liquefaction studies performed in Japan, with emphasis on experimental studies and practical applications. From this comprehensive study, the following may be concluded:

1. From an extensive survey of soil liquefaction during past earthquakes, it is found that soil liquefaction causes serious damage to various civil engineering structures. It is also disclosed that re-liquefaction is likely to happen at sites where liquefaction took place in the past. Intensities of ground shaking, geographical and geological conditions, geotechnical conditions, and water tables are key factors causing soil liquefaction. There is an approximate relation between the earthquake magnitude and the maximum epicentral distance to cause liquefaction.
2. Simplified procedures in which the liquefaction resistance factor (F_L) and the liquefaction potential index (P_L) are evaluated in terms of SPT N-values, soil overburden pressures, soil mean particle diameters, and the peak surface acceleration can be effectively employed to assess soil liquefaction potential and to account for the effects of liquefaction.
3. Shaking table tests using models of sandy soils and structures provided a better understanding of the effects of soil liquefaction on the behavior and stability of pile foundations and soil embankments.
4. Countermeasures to reduce structural damages caused by soil liquefaction can be classified into two groups, namely soil improvement countermeasures and structural improvement countermeasures. Several effective countermeasures currently used for various types of structures are introduced and discussed.
5. Provisions of design codes for various structures such as buildings, highway bridges, port and harbor structures, etc. are described. It is found that each design code has some content different from others, by taking into account the unique characteristics of each type of structures.
6. From the results described in previous chapters, it is clarified that studies on soil liquefaction are still far from the completed

stage, therefore further investigations should be anticipated. The following subjects might be future principal studies on earthquake-induced soil liquefaction problems:

A. Dynamic Shear Strength of Various Sandy Soils Causing Liquefaction.

Relating to procedures for estimating dynamic shear strength of sandy soils causing liquefaction, the following studies shall be performed.

- a. Dynamic shear strength of silty sands with high percentage of fine particles,
- b. Dynamic shear strength of coarse sands and gravelly sands,
- c. Dynamic shear strength of soil layers with high SPT N-values and high relative densities,
- d. Dynamic shear strength of soil layers with initial shear stresses under static loadings,
- e. Comparison between in situ shear strength of soils and laboratory shear strength of specimens, and development of an accurate testing apparatus to correctly obtain in situ shear strength,
- f. Dynamic shear strength of sandy soils when subjected to random seismic loadings, and
- g. Probabilistic approaches to estimate liquefaction potential of sandy soils.

B. Seismic Response Analysis Considering Soil Liquefaction

In order to clarify the liquefaction characteristics of soils and the effects of soil liquefaction on seismic structural behavior during the course of outbreak of liquefaction, appropriate procedures of seismic response analysis of soils and structures, in which generation and vanishment of excessive pore-water pressures can be taken into account, shall be developed. In this regard several procedures have been proposed up to date, yet further investigations are anticipated to precisely clarify the effects of soil liquefaction.

C. Seismic Design Methods of Structures Considering Soil Liquefaction.

In Chapter 6 of the paper provisions on soil liquefaction specified in the design codes of various structures are described. Since those provisions are made in view of limited information on liquefaction, they are not necessarily comprehensive. Therefore, it is recommended that more reasonable design methods shall be established by identifying the increase in excessive pore-water pressures, the bearing capacity of liquefying soils, and the relation between the degree of liquefaction and the extent of structural damages.

D. Countermeasures Against Soil Liquefaction

As indicated in Chapter 5 of this paper, several concepts of countermeasures are proposed to reduce the structural damages due to soil liquefaction during earthquakes. Those countermeasures can be classified into two categories: One is the countermeasure of structural strength without improving soils, and the other is the countermeasure to directly avoid liquefaction outbreak by increasing soil strength. In selecting appropriate countermeasures, the cost of construction is one of the key factors as well as the effectiveness and the method of construction. In this connection, the determination of the area and the depth of soil layers to be improved becomes necessary to be investigated. Therefore, it will become important to study the area and the depth of soil layers to be improved in accordance with the soil improvement method and the structural type. In order to solve this problem, laboratory vibration tests using a large-size shaking table and seismic analyses on dynamic interactions between soils and structures will be principal means.

E. Microzonation Maps of Soil Liquefaction Potential.

It may be possible using the proposed procedures shown in this paper to assess liquefaction potential at a given site if ample information on geotechnical engineering properties and seismic ground motions at that site are available. Furthermore, it is necessitated to have an effective procedure to make microzonation maps indicating liquefaction potential in wider regions where information on geotechnical engineering properties

is not sufficient. In the procedure, microtopographical conditions, soil profiles, and earthquake ground motions may be jointly employed for assessing liquefaction potential. It is very meaningful for the administration to have liquefaction microzonation maps in their respective regions so that effective land use planning and city planning can be implemented.

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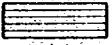
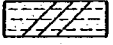
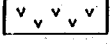
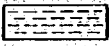
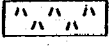
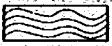
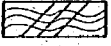
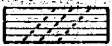
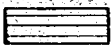
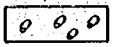
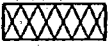
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APPENDIX

In the Appendix, soil profiles, SPT N-values and other data for the 64 liquefied sites and the 23 non-liquefied sites are shown for the reference of the reader. On the bottom of each figure information on settlement of the ground surface is also provided. (Refer to Section 3.3 on page 56)

Legend for Soil Profiles

	Top Soil		Clay		Clayey Silt		Humus
	Fine Sand		Silt		Sandy Silt		Rock
	Medium Sand		Loam		Silty Sand		Clayey Loam
	Coarse Sand		Silty Clay		Silty Gravel		Sandy Loam
	Gravel		Sandy Clay		Shell		Peat

[Note] * (at D_{50}) : Measured Value
 D_s : Settlement of Ground Surface

No.	L - 1					L - 2					L - 3					
Site	Shinano River Railroad Bridge (SP.1)					Shinano River Railroad Bridge (SP.2)					Higashi-Kosen Bridge Br. 1					
Depth (m)	Profile	N-Value		Depth (m)	N-Value	D50 (mm)	Profile	N-Value		Depth (m)	N-Value	D50 (mm)	Profile	N-Value		D50 (mm)
		Distribution	20 40					Distribution	20 40					Distribution	20 40	
5.0						0.33 0.27 0.26 0.20 0.17 0.07						0.20		1,15 2,5 3,4 4,15	4 6 3 9	0.02 0.15
10.0						0.36 0.41 0.33 0.38 0.38 0.25						0.25		5,5 6,4 7,4 8,4 9,4 10,4	6 6 20 29 33 40	 0.23
15.0												0.20		12,15 13,4 14,3 15,15	23 25 23 25	 0.22
														16,5 17,4 18,2 19,15	29 24 23 24	 0.11

Ds : Unknown

Ds : Unknown

Ds : 38.2cm and 32.4cm at Pier and Abutment, Respectively

Appendix (1)

No.	L - 4						L - 5						L - 6					
Site	Higashi - Kosen Bridge Br. 2						Higashi - Kosen Bridge Br. 4						Bandal Bridge Br. 6					
Depth (m)	Profile	N - Value			D50 (mm)	Profile	N - Value			D50 (mm)	Profile	N - Value			D50 (mm)			
		Distribution	Depth (m)	N- Value			Distribution	Depth (m)	N- Value			Distribution	Depth (m)	N- Value				
																20	40	20
5.0		1.2	6	0.032 0.132 0.25 . 														

Ds : 37.3cm and 40.2cm at Piers

Ds : Nothing

Ds : 1.2m at Abutment

Appendix (2)

No.	L - 7					L - 8					L - 9												
Site	Yochiyo Bridge Br. 1					Yochiyo Bridge Br. 5					Yochiyo Bridge Br. 7												
Depth (m)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)								
		Distribution 20 ▽ 40	Depth (m)	N- Value			Distribution 20 ▽ 40	Depth (m)	N- Value			Distribution 20 ▽ 40	Depth (m)	N- Value									
5.0			1.0	6	0.25			1.0	4	0.30			0.5	7	0.25								
			2.0	5	0.02			2.0	1	0.30			1.2	4	0.25								
			3.0	4	0.184 *			3.0	0.5	0.212 *			3.5	5	0.25								
			4.0	5	0.25			4.0	0.5	0.30			4.5	5	0.25								
			5.0	5	.			5.0	1	0.30			5.5	3.5	0.368 *								
			6.0	6	.			6.0	2	0.25			6.5	7	0.25								
			7.0	7	.			7.0	4	.			7.5	6	.								
			8.0	9	.			8.0	3	.			8.5	10	.								
			9.0	8	.			9.0	6	.			9.5	11	.								
			10.0	10	0.20			10.0	4	.			10.5	15	0.368 *								
10.0					11.0			10	.					11.0	7		.			11.5	14.5	0.20	
					12.0			35	.					12.0	11		0.20			12.5	27	.	
					13.0			34	.					13.0	19		.			13.5	31	.	
					14.0			29	.					14.0	32		.			14.5	24	.	
					15.0			23	.					15.0	21		.			15.5	30	.	
15.0					15.0	23	.				16.0	16		.							.		

Ds : 1.0~1.1m at Retaining Wall

Ds : Unknown

Ds : 1.0~1.1m at Retaining Wall

Appendix (3)

No.	L - 10					L - 11					L - 12				
Site	Shin-Matsuhama Bridge Br. 2					Taihei Bridge Br. 1					Taihei Bridge Br. 2				
Depth (m)	Profile	N - Value		D ₅₀ (mm)	Profile	N - Value		D ₅₀ (mm)	Profile	N - Value		D ₅₀ (mm)			
		Distribution	Depth			Distribution	Depth			Distribution	Depth				
		20 ∇ 40	(m)			20 ∇ 40	(m)			20 ∇ 40	(m)				
5.0			0.5	4	0.288			0.3	3			0.5	5	0.528	
			2.5	6	0.288			2.0	3			0.374	1.0	7	0.528
			4.0	5	0.436			4.0	5			0.374	3.5	6	0.528
			5.5	6	0.436			6.0	4			0.746	5.0	7	0.312
			7.0	8	0.42			7.0	10			0.746	6.5	25	0.424
			9.0	10	0.42			8.5	5			0.288	7.5	27	.
			11.0	9	0.42			10.5	12			0.352	9.0	17	.
			13.0	23	0.176			12.0	13			0.352	10.0	29	.
			14.0	25	.			13.0	42			0.504	11.0	44	1.536
			15.0	24	.			14.5	50			.	12.5	39	.
10.0			16.5	38	.			15.5	44	.			13.5	36	.
			18.0	7	0.288			16.5	56	.			15.0	46	.
			19.0	29	0.122			18.0	54	.			16.0	55	.
			20.0	37	0.122			18.5	48	.			17.5	52	.
								19.0	57	.			18.5	54	.
								19.5	49	.			19.0	38	.
													20.0	49	.

Ds : 1.1~2.3cm at Piers

Ds : Unknown

Ds : Unknown

Appendix (4)

No.	L - 13					L - 14					L - 15				
Site	Showa Bridge Br. 1 (After E.)					Showa Bridge Br. 2 (After E.)					Showa Bridge Br. 3 (After E.)				
Depth (m)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)
		Distribution 20 40	Depth (m)	N- Value			Distribution 20 40	Depth (m)	N- Value			Distribution 20 40	Depth (m)	N- Value	
5.0			1.3	2	0.30			2.31	4	0.15			2.0	5	0.15
			3.3	3	.			4.30	10	0.31			3.5	7	0.26
			5.3	8	.			6.32	2	0.03			5.2	10	0.43
			7.3	15	.			8.31	11	0.38			7.5	11	0.37
			8.3	17	.			10.3	18	0.46			9.2	12	0.06
10.0			11.3	22	.			12.30	33	0.18			10.2	10	0.46
			12.3	14	.			14.3	34	0.19			11.2	15	0.27
			13.3	16	.			16.3	27	0.11			12.2	18	0.18
			14.3	30	0.25			18.3	47	0.11			13.2	27	0.18
			15.3	34	.			19.3	35	0.12			14.2	23	0.19
15.0			16.3	29	0.20			16.3	27	0.11			15.2	36	0.37
			17.3	29	.			18.3	47	0.11			17.2	43	0.28
			18.3	34	.			19.3	35	0.12			18.2	55	0.10
			19.3	42	.								19.2	55	0.12

Ds : 2.5~5.2cm at Abutment and Pier

Ds : 1.1~9.2cm at Piers

Ds : 1.1~9.0cm at Piers

Appendix (5)

No.	L - 16					L - 17					L - 18						
Site	Showa Bridge Br. 2 (Before E.)					Showa Bridge Br. 3 (Before E.)					Niigata Airport						
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)			
		Distribution 20 40	Depth (m)				N- Value	Distribution 20 40				Depth (m)	N- Value		Distribution 20 40	Depth (m)	N- Value
5.0			2.5	2	0.18			2.6	4	0.296			1.0	5	0.25		
			3.0	4	0.22			3.3	6	0.308			2.0	6	.		
			3.4	4	0.26			4.3	6	0.258			3.0	7	.		
			4.5	4	0.33			5.3	5	0.288			4.0	5	0.30		
			5.1	4	0.39			6.3	5	0.228			5.0	5	.		
			6.5	5	0.04			7.3	5	0.184			6.0	4	.		
			7.15	5	0.96			8.3	6	0.208			7.0	8	.		
			7.5	5	0.30			9.3	8	0.216			8.0	14	0.25		
			8.3	8	0.43			10.3	8	0.296			9.0	11	.		
			9.3	7	0.11			11.3	18	0.304			10.0	9	.		
10.5			10	0.43	12.3			27	0.216	11.0			17	.			
11.5			14.5	0.25	13.3			24	0.184	12.0			12	0.30			
12.5			18	0.21	14.3			10	0.226	13.0			9	.			
13.3			23	0.19	15.3			13	0.148	14.0			8	.			
14.5			28	0.188	16.3			22	0.12	15.0			28	0.20			
15.1			31	0.32	17.3			28	0.116	16.0			30	.			
15.3			35.5	0.232	18.3			37	0.14	17.0			28	.			
16.3			36	0.11	19.3			36	0.124	18.0			31	.			
17.1			36	0.264						19.0			40	.			
17.5			38	0.126						20.0			43	.			

Ds : 1.1~9.2cm of Piers

Ds : 1.1~9.0cm of Piers

Ds : 1.0 m of Terminal Building

Appendix (6)

No.	L - 19					L - 20					L - 21				
Site	Sekiyo					Niigata Railroad Hospital (Before E.)					Niigata Railroad Hospital (Before E.)				
Depth (m)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)
		Distribution	Depth (m)	N- Value			Distribution	Depth (m)	N- Value			Distribution	Depth (m)	N- Value	
5.0		0.6m			0.68		1.3m			0.43		1.3m			0.32 *
		2.5	2	0.36			1.3	3	0.45			1.3	2	0.32	
		3.5	3	0.37			2.3	2	0.45			3.3	3	0.15	
		4.5	2	0.18			4.3	3	0.53			4.3	5	0.20	
		5.5	3	0.35			5.3	10	0.22			5.3	8	0.23	
		6.5	11	0.46			6.3	6	0.26			6.3	4	0.17	
		7.5	13	0.45			7.3	9	0.24			7.3	3	0.18	
		8.5	10	0.56			8.3	9	0.21			8.3	11	0.19	
		9.5	9	0.70			9.3	11	0.35			9.3	27	0.20	
		10.5	10	0.68			10.3	14	0.42			10.3	12	0.19	
11.5		8	11.3				14	0.58	11.3			16	0.22		
12.5		7	12.3				17	0.42	12.3			33	0.17		
13.5		24	13.3				11	0.38	13.3			29	0.27		
14.5		15	14.3				15	0.39	14.3			32	0.26		
15.5		5	15.3				14	0.35	15.3			35	0.21		
16.5		27	16.3				30	0.53	16.3			49	0.17		
17.5		26	17.3				60	0.22	17.3			55	0.19		
18.5		35	18.3				53	0.20	18.3			70	0.16		
19.5		32	19.3				52	0.23	19.3			64	0.15		

Ds : Unknown

Ds : 30 ~ 100cm at Building

Ds : ———

Appendix (7)

No.	L - 22						L - 23						L - 24								
Site	Niigata Railroad Hospital Br.1 (After E.)						Niigata Railroad Hospital Br.2 (After E.)						Kawagishi - Cho								
Depth (m)	Profile	N - Value				D ₅₀ (mm)	Profile	N - Value				D ₅₀ (mm)	Profile	N - Value				D ₅₀ (mm)			
		Distribution	Depth (m)	N- Value				Distribution	Depth (m)	N- Value				Distribution	Depth (m)	N- Value					
																			20	40	20
5.0		0.63m		1.15	8	0.29		1.19m		1.15	6	0.40		1.25m		1.32	11	0.41			
				2.15	6	1.20				2.15	7	0.47				2.54	8	0.40			
				3.15	7	0.12				3.15	7	0.032				3.87	3	0.43			
				4.15	8	0.21				4.15	4	0.047				5.01	6	0.48			
				5.15	10	0.23				5.15	8	0.25				6.50	13	0.46			
				6.15	4	0.30				6.15	7	0.26				7.88	12	0.44			
				7.15	7	0.27				7.15	5	0.27				9.52	7	0.42			
				8.15	8	0.28				8.15	9	0.22				11.00	11	0.43			
				9.15	13	0.27				9.15	10	0.34				12.40	24	0.40			
				10.15	11	0.32				10.15	14	0.34				14.01	7	0.29			
10.0					11.15	12		0.36				11.15		12	0.44				15.15	42	0.31
					12.15	18		0.38				12.15		13	0.38						
					13.15	22		0.33				13.15		13	0.38						
					14.15	40		0.30				14.15		12	0.36						
					15.15	41		0.25				15.15		39	0.35						
					16.15	37		0.25				16.15		41	0.55						
					17.15	43		0.23				17.15		31	0.22						
					18.15	40		0.24				18.15		42	0.20						
					19.15	46		0.21				19.15		40	0.24						
			15.0																		

Ds : ———

Ds : ———

Ds : 46 ~ 300 cm at Kawagishi Cho
Apartments

Appendix (8)

No.	L - 25					L - 26					L - 27							
Site	Kawagishi - Cho Br. 2					Kawagishi - Cho Br. 3					Kawagishi - Cho Br. 4							
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)				
		Distribution	Depth (m)				N- Value	Distribution				Depth (m)	N- Value			Distribution	Depth (m)	N- Value
5.0				1.30	6	0.44	*	1.50	8	0.38	*	1.51	8	0.41	♦			
				2.51	3	0.44		2.51	7	0.35		2.52	9	0.42				
				4.56	3	0.45		4.02	5	0.31		4.00	9	0.43				
				5.86	14	0.50		5.51	7	0.46		5.50	6	0.51				
				7.50	13	0.44		6.90	8	0.46		7.00	10	0.50				
10.0				9.00	14	0.44		8.60	9	0.38		8.46	11	0.35				
				10.35	13	0.43		9.95	8	0.35		10.01	11	0.43				
				11.91	10	0.36		11.51	11	0.42		11.43	15	0.25				
				13.50	14	0.25		13.03	9	.		13.01	9	0.41				
15.0				14.85	24	0.26		14.50	22	.		14.35	22	0.43				
				16.50	22	0.23		16.00	21	.		16.00	26	0.39				
								18.85	47	0.19		17.50	22	0.31				
												18.90	46	0.15				

Ds : _____

Ds : _____

Ds : _____

Appendix (9)

No.	L - 28					L - 29					L - 30						
Site	Kawagishi - Cho BC 21-2					Kawagishi - Cho BC 21-3					Kawagishi - Cho BC 104						
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)			
		Distribution	Depth (m)				N- Value	Distribution				Depth (m)	N- Value		Distribution	Depth (m)	N- Value
5.0			1.4	3	0.20			1.4	3	0.02			1.0	2	0.25		
			2.4	4				2.4	3				0.25				
			3.4	7				3.4	4				0.20				
			4.4	6				4.4	5				0.20				
			5.4	9				5.4	6				0.30				
			6.4	8				6.4	8				0.25				
			7.4	9				7.4	7				.				
			8.4	11				8.4	9				.				
			9.4	10				9.4	10				.				
			10.4	12				10.4	7				.				
10.0			11.4	13	0.30			11.4	8	0.30			11.0	10	.		
			12.4	18				12.4	16				.				
			13.4	23				13.4	21				.				
			14.4	26				14.4	26				.				
			15.4	25				15.4	25				.				
			16.4	29				16.4	29				.				
			17.4	34				17.4	34				.				
			18.4	39				18.4	35				.				
			19.4	43				19.4	43				.				
			20.0	43				20.0	43				.				
15.0			12.4	18	0.30			12.4	16	0.30			12.0	9	0.20		
			13.4	23				13.4	21				.				
			14.4	26				14.4	26				.				
			15.4	25				15.4	25				.				
			16.4	29				16.4	29				.				
			17.4	34				17.4	34				.				
			18.4	39				18.4	35				.				
			19.4	43				19.4	43				.				
			20.0	43				20.0	43				.				
			20.0	43				20.0	43				.				
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20.0	43																

Ds : _____

Ds : _____

Ds : _____

Appendix (10)

No.	L - 31						L - 32						L - 33							
Site	Kawagishi - Cho BC 14						Nanae Beach (Br.1)						Nanae Beach (Br.2)							
Depth (m)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)					
		Distribution	Depth (m)	N - Value			Distribution	Depth (m)	N - Value			Distribution	Depth (m)	N - Value						
																20	40	20	40	20
5.0		1.35m			2.2	7	0.25		0.95m			1.2	7	0.10		0.95m			1.2	0.06
		3.2	7	0.25	2.2	10	.		2.2	.										
		4.2	8	0.20	3.2	12	.		3.2	.										
		5.2	7	.	4.2	16	.		4.2	.										
		6.2	7	.	5.2	10	0.09		5.2	0.10										
		7.2	6	.	6.2	11	.		6.2	0.10										
		8.2	7	.	7.2	25	.		7.2	.										
		9.2	9	0.25	8.2	23	.		8.2	.										
		10.2	9	.	9.2	25	.		9.2	.										
		11.2	9	.	10.2	26	.		10.2	.										
10.0		12.2	13	.	11.2	30	.	11.2	.											
		13.2	13	.	12.2	18	0.05	12.2	0.05											
		14.2	13	.	13.2	16	.	13.2	.											
		15.2	14	.	14.2	14	.	14.2	.											
		16.2	33	.	15.2	8	.	15.2	.											
		17.2	31	.	16.2	10	.	16.2	.											
		18.2	31	.	17.2	9	.	17.2	.											
		19.2	32	.	18.2	46	0.15	18.2	0.15											
		19.2	32	.	19.2	41	0.15	19.2	0.15											
		Ds : —————						Ds : Unknown						Ds : Unknown						

Appendix (11)

No.	L - 34						L - 35						L - 36						
Site	Nanae Beach (Br. 3)						Hachinohe City						Gifu City						
Depth (m)	Profile	N - Value				D ₅₀ (mm)	Profile	N - Value				D ₅₀ (mm)	Profile	N - Value				D ₅₀ (mm)	
		Distribution		Depth (m)	N - Value			Distribution		Depth (m)	N - Value			Distribution		Depth (m)	N - Value		
		20	40					20	40					20	40				20
5.0		▽ 1.1m		1.2	7	0.70		▽ 0.57m		1.2	2	0.25		▽ 0.9m		1.20	3	0.29	*
				2.2	13	.				2.3	0.6	0.20				2.15	5	0.21	
				3.2	11	.				3.3	0.4	.				3.17	5	0.18	
				4.2	13	.				4.3	0.3	.				4.15	10	0.30	
				5.2	15	.				5.3	12	.				5.15	8	0.31	
				6.2	16	0.07				6.3	36	.				6.15	8	0.29	
				7.2	16	.				7.3	15	0.10				7.15	11	0.27	
				8.2	18	.				8.1	24	0.20				8.15	8	0.23	
				9.2	21	.				9.1	16	0.10				9.15	9	0.25	
				10.2	23	.				10.1	27	.				10.15	11	0.30	
10.0				11.2	34	.				11.1	18	.				11.15	13	0.32	
				12.2	19	.				12.1	25	0.20				12.15	17	0.60	
				13.2	14	0.03				13.1	20	.				13.15	23	0.47	
				14.2	9	.				14.2	37	.				14.15	22	0.51	
				15.2	11	.				15.1	25	.				15.15	19	0.47	
				16.2	7	.				16.2	28	.				16.15	24	0.48	
				17.2	11	.				17.2	33	.							
				18.2	30	0.08													
				19.2	34	0.08													
Ds : Unknown						Ds : 50cm						Ds : Unknown							

Appendix (12)

No.	L - 37					L - 38					L - 39				
Site	Kagamihara Unuma					Kagamihara Ogaselke					Mangoku, Ohgaki				
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)	
		Distribution	Depth (m)				Distribution	Depth (m)				Distribution	Depth (m)		
		20 40					20 40					20 40			
		0.75m	1.18	4	0.005		2.1m	115	3	0.002		1.31	2	0.004	
			3.15	15	0.80			222	2	0.001		2.4	3	0.03	
5.0			4.15	16	1.20			400	11	0.66		3.1	5	0.07	
			5.15	18	1.60			515	13	0.56		4.15	6	0.32	
			6.1	17	0.73			615	16	1.30		5.17	8	0.36	
			7.15	18	0.57			715	21	1.50		6.15			
			8.15	19	0.60			815	29			7.15			
			9.15	20	0.70			915	29			8.15			
10.0								1015	20			9.15			
								1115	22	1.70		10.17	19	0.45	
								1215	44	1.20		11.15	16	0.72	
								1315	27			12.15	18	0.65	
								1415	33			13.15	18	0.25	
								1515	40			14.15	18	0.27	
15.0								1715	43			15.15	16	0.23	
								180	41			16.15	14	0.28	
								192	49			17.15	11	0.033	
												18.15	8	0.018	
												19.15	8	0.025	

Ds : Unknown

Ds : Unknown

Ds : Unknown

Appendix (13)

No.	L - 40					L - 41					L - 42				
Site	Meikodori					Kohmei					Inaei				
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)	
		Distribution	Depth (m)				Distribution	Depth (m)				Distribution	Depth (m)		
		20 40					20 40					20 40			
		0.6m	1.15	3	0.33		0.9m	1.75	3	Z (m)		0.23m	1.85	3	0.13
			2.15	1	2.6			2.65	3	0.43 (2.65)			4.18	3	0.015
			3.55	1	0.21			4.05	4	0.27 (3.15)			5.15	2	0.01
5.0			4.15	10	0.24			4.8	7	0.60 (4.15)			5.65	3	0.01
			5.15	1	0.006			5.7	8	0.43 (5.15)			6.48	10	0.12
								6.15	10	0.38 (6.15)			7.13	6	0.23
			7.75	1	0.003			7.9	20	0.57 (7.15)			7.13	6	0.23
			9.15	2	0.005			8.4	27	0.36 (8.15)			8.15	5	0.03
10.0			10.15	35	0.25			9.8	37	0.30 (9.15)			9.00	5	
			11.15	50	0.55			10.4	39	0.39 (10.15)			10.15	6	0.024
			12.15	52	0.41			11.55	43	0.40 (11.15)			11.00	6	
			13.15	42	2.3			12.15	40	0.39 (12.15)			12.15	18	0.33
			14.15	30	1.50			13.15	39	0.42 (13.15)			13.15	12	0.25
15.0			15.15	37				14.25	47	0.68 (14.15)			14.15	24	0.33
								15.65	53	0.20 (15.15)			15.15	25	0.33
													16.15	36	0.30
													17.15	39	0.06
													18.15	37	0.15
													19.15	23	0.003

Ds : 1.0 ~ 1.5m of a House

Ds : 30 ~ 50cm

Ds : Unknown

Appendix (14)

No.	L - 43					L - 44					L - 45				
Site	Takayo 45-35					Maruoka No. 2					Takayo 2-168				
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)	
		Distribution	Depth				Distribution	Depth				Distribution	Depth		
		20 40	(m) Value				20 40	(m) Value				20 40	(m) Value		
5.0		1.15	1	0.04	◆		0.9	2	0.13	◆		1.15	1	0.021	◆
		2.15	5	0.24			1.5	4	0.23			2.15	1	0.025	
		3.15	8	0.28			2.4	3				3.15	6	0.05	
		4.15	11	0.20			2.95	2	0.44			4.15	7	0.13	
		5.15	12	0.33			3.9	7				5.15	8	0.35	
		6.15	18	0.55			4.6	17				6.15	27	0.38	
		7.15	18	0.65			5.0	25	4.80			7.15	28	0.54	
		8.15	16	1.50			6.5	32	4.30			8.15	29	0.78	
		9.15	22	0.55			7.65	36	4.50			9.15	29	1.20	
		10.15	15	0.50			8.05	42	0.19			10.15	31	1.00	
10.0		11.15	16	0.50			9.9	5				11.15	6	0.037	
		12.15	31	0.50			10.1	33	0.19			12.15	21	0.14	
		13.15	8	0.55			12.15	36	0.36			13.15	19	0.037	
		14.15	6	0.068			12.9	38				14.15	6	0.042	
		15.15	6	0.15			13.5	37				15.15	10	0.045	
		16.15	6	0.175			14.4	40	0.36			16.15	10	0.16	
		17.15	2	0.022			15.5	45				17.15	9	0.029	
		18.15	2	0.32			16.1	45	0.38			18.15	10	0.028	
		19.15	31	0.025			17.4	33				19.15	8	0.047	
							18.9	45	0.24						
							19.65	44	0.30						

Ds : Unknown

Ds : 50cm (be Covered with Water)
30cm at Temple Building

Ds : Unknown

Appendix (15)

No.	L - 46					L - 47					L - 48				
Site	Abukuma Bridge Br. 4					Mouth of Abukuma River					Yurige - kami Y - 1				
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)	
		Distribution	Depth N-Value				Distribution	Depth N-Value				Distribution	Depth N-Value		
		20 40	(m) Value				20 40	(m) Value				20 40	(m) Value		
5.0		1.30	11	1.30	◆		1.30	5	0.07	◆		2.30	4	0.56	◆
		2.30	14	0.60			2.30	5	0.07			3.30	1	0.025	
		3.30	13	1.40			3.30	7	0.10			4.30	0	0.025	
		4.30	15	1.40			4.30	9	.						
		5.30	17	0.60			5.30	12	.						
							6.30	10	.			6.30	5	0.60	
							7.30	11	.			7.30	7	0.60	
							8.30	12	.			8.30	37	0.15	
							9.30	17	0.10			9.30	32	.	
							10.30	36	.			10.30	68	.	
10.0							11.30	38	.			11.30	79	.	
							12.30	32	.			12.30	58	.	
							13.30	38	.			13.30	71	.	
							14.30	37	.			14.30	94	.	
							15.30	46	.					.	
							16.30	43	.					.	
							17.30	44	.					.	
							18.30	47	.					.	
							19.30	45	.					.	
									.					.	

Ds : 20~30cm (Estimated by Photo.)

Ds : Nothing (Estimated by Photo.)

Ds : 25cm at Toe of River Dyke

Appendix (16)

No.	L - 55					L - 56					L - 57							
Site	Oiri (1) No. 2					Uomachi B - 1					Uomachi B - 2							
Depth (m)	Profile	N - Value				D ₅₀ (mm)	Profile	N - Value				D ₅₀ (mm)	Profile	N - Value				
		Distribution		Depth (m)	N- Value			Distribution		Depth (m)	N- Value			Distribution		Depth (m)	N- Value	
		20	40					20	40					20	40			20
5.0		-2.4 m							1.00	7	0.35				1.00	8	0.35	
									2.00	6	.				2.00	6	.	
									3.00	6	.				3.00	4	.	
									4.00	5	.				4.00	5	.	
									5.00	4	.				5.00	4	.	
10.0				5.30	9	0.40 ♦				6.00	3	0.025				6.00	3	0.025
				6.30	8	0.36 ♦				7.00	2	0.04				7.00	5	0.04
				7.30	12	0.044 ♦				8.00	12	0.35				8.00	6	0.35
				8.30	15	0.023 ♦				9.00	11	0.35				9.00	7	0.35
										10.00	4	0.07				10.00	7	0.07
15.0				12.30	8	0.07				11.00	3	0.07				11.00	2	0.07
				13.30	14	0.07				12.00	1	0.04						
				14.80	38	0.07				13.00	2	0.04						
				15.30	45	0.04				14.00	3	0.025						
										15.00	1	0.04						
										16.00	50	2.00						

Ds : Nothing (Eye Measurement)

Ds : Unknown

Ds : Unknown

Appendix (19)

No.	L - 58					L - 59					L - 60							
Site	Rifu No. 12					Shiomi No. 1					Shiomi No. 2							
Depth (m)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)			
		Distribution		Depth (m)			N- Value	Distribution				Depth (m)	N- Value	Distribution		Depth (m)	N- Value	
		20	40					20	40					20				40
5.0		-2.72m							1.30	9	0.15				1.30	3	0.15	
				3.30	6		0.60			2.30	15		0.15			2.30	4	0.15
				4.30	4		0.60			3.30	16		0.35			3.30	0	0.04
				6.30	7		0.60			5.30	16		0.35			5.30	12	0.15
				8.30	9		0.60			7.30	20		0.15			7.30	12	0.15
10.0				9.30	0	0.04				9.30	14	0.15				9.30	13	0.15
				10.30	0	0.04				11.30	20	0.15				11.30	17	0.15
				11.30	0	0.04				13.30	15	0.15				13.30	16	0.15
				12.30	0	0.04				15.30	4	0.04				15.30	5	0.04
				13.30	5	0.07				17.30	5	0.04				17.30	4	0.025
15.0				14.30	5	0.60				19.30	3	0.04				19.30	3	0.025
				15.30	4	0.60												
				16.30	15	0.60												
				18.30	23	0.35												
				19.30	8	0.35												

Ds : 10 cm

Ds : Nothing (Estimated by Photo.)

Ds : Nothing (Estimated by Photo.)

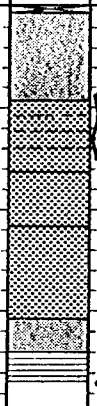
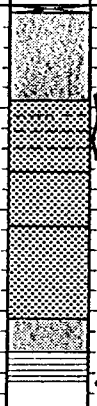
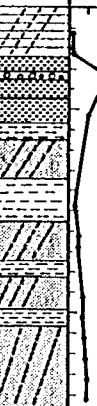
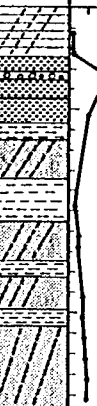
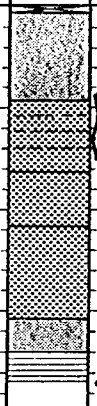
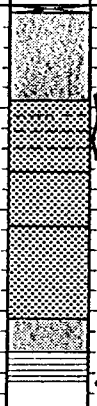
Appendix (20)

No.	NL - 1					NL - 2					NL - 3							
Site	Jindoji					Kogane - Cho					Higashi Kosen Bridge Br.5							
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)				
		Distribution	Depth (m)				N- Value	Distribution				Depth (m)	N- Value			Distribution	Depth (m)	N- Value
5.0		2.3m	1.5 13 2.5 3 3.5 1 4.5 12 5.5 27 6.5 22 7.5 22 8.5 18 9.5 21 10.5 21 11.5 30 12.5 41 13.5 56 14.5 48 15.5 42 16.5 30 17.5 31 18.5 31 19.5 19	0.20			5.2m	1.3 4 2.3 7 3.3 9 4.3 10 5.3 11 6.3 13 7.3 15 8.3 18 9.3 18 10.3 15 11.3 29 12.3 32 13.3 38 14.3 42	0.25			2.5m	1.2 12 2.2 22 3.2 23 4.2 37 5.2 33 6.2 42 7.2 38 8.2 43 9.2 45 10.2 43 11.2 43 12.2 38 13.2 32 14.2 44 15.2 45	0.264 0.25 0.214 0.25 0.132	*			
Ds : Unknown						Ds : Unknown						Ds : No Damage at Piers						

Appendix (23)

No.	NL - 4					NL - 5					NL - 6							
Site	Shin - Matsushima Bridge Br. 1					Omiya 266 K 712 M					Showa Bridge Br.4 (After E.)							
Depth (m)	Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)		Profile	N - Value		D ₅₀ (mm)				
		Distribution	Depth (m)				N - Value	Distribution				Depth (m)	N - Value			Distribution	Depth (m)	N - Value
5.0		20 40	1.5 4 3.5 5 5.0 5 6.0 14 7.0 13 8.0 23 9.5 18 10.5 41 11.5 39 12.0 42 13.0 24 14.0 40 15.0 26 16.0 31 17.0 28 18.0 30 19.0 39 20.0 35	0.154 0.154 0.262 0.432 0.432 0.352 . 0.36 . . . 0.36 0.106 . .	*		20 40	1.4 0 2.3 10 3.3 31 4.3 32 5.3 31 6.3 36 7.3 40 8.3 32 9.3 48 10.3 46 11.3 39 12.3 34 13.3 37 14.3 40 15.3 36 16.3 40 17.3 37 18.3 40 19.3 45	0.30 0.20 0.20 0.27 0.20 0.20 0.20 0.36 0.20	*		20 40	1.6 7 3.5 3 5.5 27 7.5 33 9.7 40 10.7 43 11.5 26 12.5 34 13.5 37 14.5 42 15.5 49 16.5 41 18.8 50 19.8 61	0.25 0.07 0.25 0.25 0.25 0.20 0.20 .				
Ds : 0.5~0.7cm at Piers						Ds : Unknown						Ds : 0.2~0.8cm at Pier and Abutment						

Appendix (24)

No.	NL - 7					NL - 8					NL - 9																						
Site	Nishi Oh-Hata-Cho					Gotanda Bridge Br. 1					Gotanda Bridge Br. 2																						
Depth (m)	Profile	N - Value		Depth (m)	N- Value	D ₅₀ (mm)	Profile	N - Value		Depth (m)	N- Value	D ₅₀ (mm)	Profile	N - Value		Depth (m)	N- Value	D ₅₀ (mm)															
		Distribution						Distribution						Distribution																			
		20	40					20	40					20	40				20	40													
5.0				1.5	7	0.20				1.15	2	0.04				1.3	2	0.012															
				2.5	9					2.2	1					2.3	2																
				3.5	12					3.33	3					3.3	19		5.3	10	0.72												
				4.5	4					4.2	1					6.8	6	0.075					7	0.046									
				5.5	5					7.8	12	0.095																					
				6.5	2											8.3	14																
				7.5	4					9.3	11	0.13				9.85	4	0.009															
				8.5	18					11.3	4								0.005														
				9.5	8					12.5	7	0.034				11.31	5	0.044															
				10.5	13					13.3	7																						
10.0										11.5	17	0.20				14.3	8	0.032				14.3	7	0.10									
										12.5	19					15.3	28								15.3	6	0.052						
										13.5	21					16.3	33	0.20				16.3	8										
										14.5	22					17.3	30							0.04				17.32	9				
										15.5	25					18.3	10	18.3	9														
										16.5	22					19.3	12				19.3	9											
										17.5	20																						
										18.5	5																						
										19.5	20																						

Ds : Unknown

Ds : Unknown

Ds : Unknown

Appendix (25)

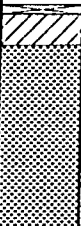
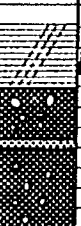
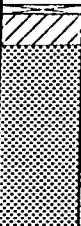
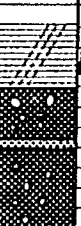
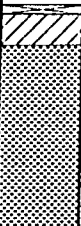
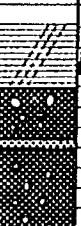
No.	NL - 10						NL - 11						NL - 12					
Site	Maruoka						Nakamura						Nakamura					
Depth (m)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)	Profile	N - Value			D ₅₀ (mm)			
		Distribution	Depth (m)	N- Value			Distribution	Depth (m)	N- Value			Distribution	Depth (m)	N- Value				
																20	40	20
5.0			0.9	4	0.02			1.30	6	0.40			1.30	12	0.32			
			1.65	3	0.025			2.30	11	0.46			2.30	4	0.11			
			2.4	3	0.006			3.30	18	0.38			3.30	8	0.12			
			3.15	2	0.12			4.30	26	0.35			4.30	12	0.33			
			3.9	6	0.36			5.30	25	0.65			5.30	17	0.30			
			4.65	4	0.03			6.30	47	0.60			6.30	34	0.60			
			5.4	7	0.10			7.30	45	0.60			7.30	20	0.60			
			6.15	10	0.125			8.30	50	0.35			8.30	33	0.15			
			6.9	20	0.46								9.30	44	0.15			
			7.65	23	0.28								10.30	19	0.15			
8.4			4	0.009														
9.15			10	0.26														
9.9			11	0.005														
10.65			12	0.017														
11.4			13	0.017														
12.15			9	0.035														
12.9			9	0.035														
13.65			18	0.20														
14.4			33	0.40														
15.15			42	0.45														
15.9			45	0.51														
16.65																		
17.4																		
18.15																		
18.9																		
19.65																		

Ds : Nothing

Ds : Nothing (Eye Measurement)

Ds : Nothing (Eye Measurement)

Appendix (26)

No.	IL - 13						L - 14						NL - 15					
Site	Yuriage - kami Y - 3						Kitakami River Br. 10						Natori River 3.2 km + 100 m					
Depth (m)	Profile	N - Value		Depth (m)	N- Value	D ₅₀ (mm)	Profile	N - Value		Depth (m)	N- Value	D ₅₀ (mm)	Profile	N - Value		Depth (m)	N- Value	D ₅₀ (mm)
		Distribution						Distribution						Distribution				
		20	40					20	40					20	40			
5.0				3.30	9	0.651				2.00	4	0.07				4.20	16	0.60
				4.30	21	0.50				3.00	8	.				5.20	22	0.60
				5.30	19	0.55				4.00	3	.				6.20	18	0.35
				6.30	19	0.70				5.00	14	.				7.20	15	0.60
				7.30	23	0.80				6.00	12	.				8.20	47	.
				8.30	36	0.39				7.00	13	.				9.20	39	.
				9.30	44	0.15				8.00	8	.				10.20	53	.
10.0				10.30	34	0.15				9.00	9	.				11.20	48	0.15
									10.00	10	.				12.20	47	.	
15.0										11.00	11	.				13.20	50	.
									12.00	13	.				14.20	50	.	
									13.00	7	0.025				15.20	50	.	
									14.00	10	.				16.20	50	.	
									15.00	4	.				17.20	50	.	
									16.00	4	.				18.20	50	.	
									17.00	8	.				19.20	50	.	
									18.00	10	.							
									19.00	8	.							

Ds : Nothing

Ds : Nothing

Ds : Nothing

Appendix (27)

No.	NL - 16					NL - 17					NL - 18									
Site	Kin-oh Bridge P ₈					Abukuma Bridge Br.1					Abukuma Bridge Br.2									
Depth (m)	Profile	N - Value		Depth (m)	N - Value	D ₅₀ (mm)	Profile	N - Value		Depth (m)	N - Value	D ₅₀ (mm)	Profile	N - Value		Depth (m)	N - Value	D ₅₀ (mm)		
		Distribution	20 40					Distribution	20 40					Distribution	20 40				Distribution	20 40
																				
5.0		-4.0m ▽		4.30 2	0.072			-4.3m ▽		4.30 11	0.20			-3.4m ▽		4.30 11	0.41			
				5.30 4	0.025					5.30 21	0.35					5.30 18	0.60			
				6.30 5	0.044					6.30 17	0.55					6.30 21	0.40			
				7.30 11	0.044					7.30 15	0.52					7.30 24	0.90			
				8.30 31	2.00					8.30 21	0.60					8.30 39	0.60			
				9.30 27	0.58					9.30 40	0.60					9.30 23	.			
10.0				10.30 19	0.18					10.30 29	0.60					10.30 39	.			
				11.30 17	.					11.79 50	0.60					11.79 39	0.15			
				12.30 19	.					13.30 42	0.15					13.30 16	0.15			
				13.30 7	0.025					14.79 35	0.15					14.79 58	0.15			
15.0				14.30 7	0.025					16.30 50	0.15					16.30 32	0.15			
				15.30 8	0.26					17.30 35	0.15					17.79 18	0.07			
				16.30 16	.					19.30 29	0.15					19.30 50	0.15			
				17.30 17	.															
				18.30 23	.															
				19.30 24	.															

Ds : Unknown

Ds : Nothing (Eye Measurement)

Ds : Nothing (Eye Measurement)

Appendix (28)

SOIL LIQUEFACTION
STUDIES IN JAPAN,
STATE-OF-THE-ART

PWRI TECHNICAL MEMORANDUM
No. 2239
Aug. 1985
Toshio Iwasaki

Key Words: EARTHQUAKE, LIQUEFACTION, SANDY SOILS,
SOIL LIQUEFACTION

Synopsis:

This paper describes the state-of-the-art of studies on
earthquake-induced soil liquefaction, with emphasis
on experimental results and practical applications.



郵便はがき

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SOIL LIQUEFACTION STUDIES IN JAPAN
STATE-OF-THE-ART.