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A COMPARATIVE STUDY OF TWO PRACTICAL METHODS FOR PREDICTING LIQUEFACTION BASED ON OCCURRENCE OF LIQUEFACTION DURING THE 1983 NIHONKAI-CHUBU EARTHQUAKE

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ABSTRACT

This paper compares of two practical methods for predicting liquefaction based on the occurrence of liquefaction during the 1983 Nihonkai-Chubu Earthquake. Based on the results of various measures including borings, standard penetration tests, grain size analyses, and interviews with residents, comparative calculations have been attempted using two practical methods for predicting liquefaction : a procedure from the Japan Highway Bridge Code and a method which utilizes residual pore water pressure data. A comparison of the surveys with the analyses showed a coincidence rate of 28% when applying the procedure of the Japan Highway Bridge Code and 31% for the method utilizing pore water pressure data at $k_{so}=0.15$, the value which is usually used at present in two practical methods. However, using $k_{so}=0.22$, which is the ratio of maximum acceleration to gravity acceleration obtained at the ground surface during the 1983 Nihonkai-chubu Earthguake, the results were 81% identical and 80% respectively. Thus, the occurrence of liquefaction can be accurately estimated by the two practical methods provided there is close agreement between k_{so} and the ratio of maximum acceleration to gravity acceleration at the ground surface during a massive earthguake.

KEYWORDS

Liquefaction ; Practical methods, Damage to houses, Comparative study, 1983 Nihonkai-Chubu Earthguake.

OUTLINE OF TWO PRACTICAL METHODS FOR PREDICTING LIQUEFACTION

A procedure from the Japan Highway Bridge Code

The ability of a soil element at an arbitrary depth below the ground surface to resist liguefaction can be expressed by the factor of liquefaction resistance (F_L), as follows :

$$F_L=R/L \tag{1}$$

R represents in situ resistance (i.e. undrained cyclic strength) of a soil element cyclic loads and can be simply evaluated, based on the results of the undrained cyclic shear test, as follows :

$$R_1=0.0882\sqrt{\frac{N}{\sigma'_v+0.7}} \tag{2}$$

$$R_2=\begin{cases} 0.19 & (0.02\ \text{mm}\leq D_{50}\leq 0.05\ \text{mm}) \\ 0.225\log_{10}(0.35/D_{50}) & (0.05\ \text{mm}<D_{50}\leq 0.6\ \text{mm}) \\ -0.05 & (0.6\ \text{mm}<D_{50}\leq 2.0\ \text{mm}) \end{cases} \tag{3}$$

$$R_3 = \begin{cases} 0.0 & (0\% \leq FC \leq 40\%) \\ 0.004\,FC - 0.16 & (40\% < FC \leq 100\%) \end{cases} \quad (4)$$

where N is blow count measured by the Japanese standard penetration test, σ'_v is the effective overburden stress (in kgf/cm²), D_{50} is the mean particle diameter (in mm), and FC is the fine content in percentage.

L in Eq. (1) is the dynamic load in a soil element encountered through seismic motion and can be expressed in Eq. (5)

$$L = \frac{\tau_{max}}{\sigma'_v} = \frac{\alpha_{max}}{g} \cdot \frac{\sigma_v}{\sigma'_v} \cdot r_d = k_s \cdot \frac{\sigma_v}{\sigma'_v} \cdot r_d \quad (5)$$

where τ_{max} is the maximum shear stress (in kgf/cm²), σ_{max} is the maximum acceleration on the ground surface (in gal), g is the acceleration of gravity (in cm/sec²), σ_v is the total overburden stress (in kgf/cm²), k_s is the horizontal design seismic coefficient, and r_d is the reduction factor of dynamic shear stress.

In Eq. 5, k_s, σ_v and r_d are expressed by Eq. (6), (7), and (8) respectively,

$$k_s = c_Z \cdot c_G \cdot c_I \cdot k_{SO} \quad (6)$$

$$\sigma_v = \{\gamma_{t1} - \gamma_{t2}(X - hw)/10\} \quad (7)$$

$$\gamma_d = 1 - 0.015X \quad (8)$$

where c_Z, c_G, and c_I are the corrections for zone, ground, and importance classification respectively, k_{SO} is the standard horizontal design seismic coefficient, γ_{t1} and γ_{t2} are the unit weights above and below the ground water level (in gf/cm³), h_w is the depth of the ground water table below the ground surface in m, and X is the depth in m.

Finally, as seen in Eq. (1) liquefaction takes place at some depth in the sandy soil layer when F_L is less than or equal to 1.0. Conversely, liquefaction does not occur in a case where F_L is greater than 1.0.

A method utilizing residual pore water pressure data

Maximum dynamic shear stress τ_{max}, at an arbitrary depth under the ground surface is expressed by Eq. (9),

$$\tau_{max} = \frac{\alpha_{max}}{g} \cdot r_d \cdot \sum_{i=1}^{i} (\gamma_i \cdot h_i) \quad (9)$$

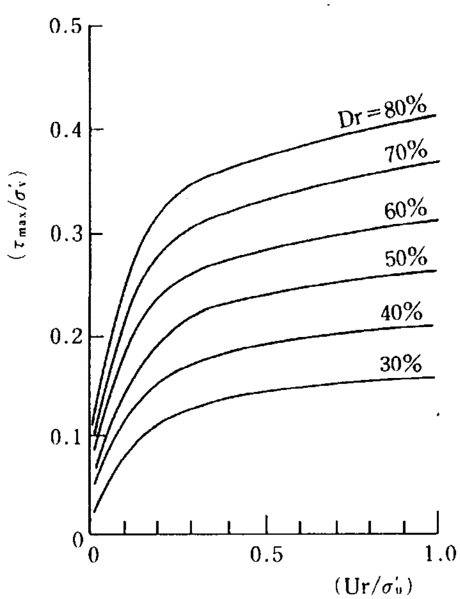

Fig. 1. $\tau_{max}/\sigma'_v \sim Ur/\sigma'_v$

where γ_j and h_j are the unit weight of soil in the j layer (in tf/m^3) and the thickness of the j layer (in m) respectively. Fig. 1 shows the relationship between the ratio of residual pore water pressure (U_r) to the effective overburden stress (σ'_v) and τ_{max}/σ'_v reported by Ishihara. From this relationship, U_r can easily estimated for various values of the relative density D_r mentioned below,

$$D_r = N\sqrt{\{(\sigma'_v/10)+0.70} \qquad (10)$$

where N is the blow count measured by the Japanese Standard penetration test.
Finally, the sand layer can be perfectly liquefied at some depth under conditions where the U_r equals 1.0, whereas in the case that U_r is less than 1.0, the sand layer can only be partially liquefied.

PROBLEMS IN APPLYING TWO PRACTICAL METHODS PREDICTING LIQUEFACTION-INDUCED DAMAGE TO HOUSES

For applying these methods to predict liquefaction-induced damage to houses, proper assessment of the unit weight of soil (ρ_t), the maximum acceleration at the ground surface (α_{max})., and the relationship between the thickness of the non-liguefied layer (H_1) and that of the liquefied layer (H_2) are necessary. This should be done as follows:

(1) This study employed a formula based on the quantitative theory for estimating density with the aid of a method where ρ_t is assessed using the surveyed-N values in addition to geology and soil types as follows:

$$\rho_t\ (\text{gf/cm}^2) = 1.70 \cdot N^{0.027} \cdot H^{0.018} \cdot \left[\begin{array}{l} 1.00: \text{Alluvial deposit} \\ 0.14: \text{Diluvial deposit} \\ 1.03: \text{Tertiary deposit} \end{array}\right] \cdot \left[\begin{array}{l} 1.00: \text{Clay} \\ 1.01: \text{Silt} \\ 1.06: \text{Sand} \\ 1.12: \text{Gravel} \end{array}\right] \qquad (11)$$

where N is the blow count from the standard penetration test and H is the depth in meters.

(2) Horizontal design seismic coefficient (k_s) as expressed in Eq. (6), that is the ratio of maximum acceleration (α_{max}) to gravity acceleration (g), was estimated by the following procedure:

(2.1) Soil profiles, in which N values ranged from 1 to 50, at 26 locations in Wakami Town were prepared for the estimation.

(2.2) The predominant period in the ground (T_G) was obtained by Eq. (12),

$$T_G = 4\sum_{i=1}^{n} \frac{H_i}{V_{si}} \qquad (12)$$

where H_i is the thickness of the ith layer in meters and V_{si} is the velocity of the S-wave in the ith layer in (m/sec), which is expressed for sandy soils as follows:

$$V_{si} = 80N_i^{\frac{1}{2}}\ (1 \leq N_i \leq 50) \qquad (13)$$

where N_i is the blow count from the standard penetration test.

(2.3) From Table 1, ground classification was decided according to the corresponding value of T_G, from Eq. (11), and then from Table 2 the revised coefficient of ground classification C_G was obtained according to the ground classification.

Table 1. Ground classification

Ground Classification	Predominant period of the ground (sec)
First class	$T_G < 0.2$
Second class	$0.2 \leq T_G < 0.6$
Third class	$0.6 \leq T_G$

Table 2. Revised coefficient of ground classification C_G

Ground Classification	First class	Second class	Third class
Revised coefficient C_G	0.8	1.0	1.2

(3) Based on the following relationship between the thickness of the non-liquefied layer (H_1) and the liquefied layer (H_2), obtained from the results of the surveys as shown in Fig. 2, liquefaction-induced damage

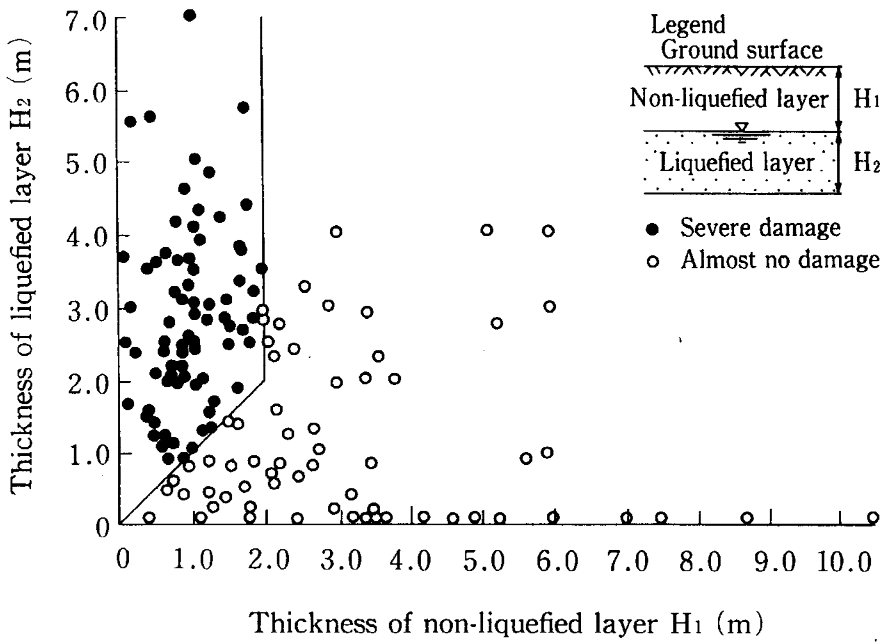

Fig. 2. Relationship between Thickness of the Non-liquefied Layer (H_1) and Thickness of the Liquefied Layer (H_2)

to the house can be confirmed as follows :

(3.1) Liquefaction-induced damage to the house did not occur when it was on a layer of sand with H_1 more than 2.0 m.

(3.2) However, on a sand layer H_1 was less than 2.0 m and $H_1 < H_2$, earthquake damage due to liquefaction occurred, in the part when the N values was less than 15. However, damage to house did not when the layer of sand with $H_1 > H_2$.

COMPARISONS OF THE SURVEYS WITH THE ANALYSIS

Results of estimations using two practical methods for predicting liquefaction are compared with the occurrence of liquefaction-induced damage to houses during the 1983 Nihonkai-Chubu Earthquake. The estimations were based on a k_{so} of 0.15 and 0.22. A k_{so} of 0.15 is usually used in a procedure according to the Japan Highway Bridge Code and k_{so} of 0.22 is the maximum acceleration value estimated by using the observed earthquake data recorded during the 1983 Nihonkai-Chubu Earthquake.

In the case of applying the standard design seismic coefficient k_{so} of 0.15.

Table 3 illustrates the results from comparisons between the surveys and analyses from 26 locations in Wakami Town in Akita Prefecture. Explanations of items in the columns as shown in Table. 3 are described in detail :

(1) In the first column, boring numbers measured at 26 locations in Tamanoike, Yanagihara, Ishidagawara and Gomyoko in Wakami Town are indicated.

(2) Results from the survey are shown in the second column. They are the thickness between the subsurface layers and the bedrock, the depth of ground water table below the groud surface, the average N value in the subsurface layers, and the occurrence of liquefaction-induced damage to houses during the earthquake. In the last column, a circle indicates severe liquefaction-induced damage to houses, and crosses represent almost no damage to a house.

(3) H_1, H_2, and the occurrence of liquefaction-induced damage to houses, are shown in the third and the fourth column, respectively. The last column indicates the degree of the agreement between the surveys

Table 3. Comparison of Results from Surveys and Analyses by Two Practical Methods for Predicting Liquefaction ($k_{so}=0.15$)

Area	① Boring. No.	② Results from surveys & interviews				③ Results estimated by Japan Highway Bridge Code			
		Depth of bedrock (m)	Ground water table	Averaged N Values	Liquefaction-induced damage	H_1 (m)	H_2 (m)	Occurrence of liquefaction-induced damage	Agreement (yes or no)
Tamanoike, etc.	Ta-59- 2	9	0.50	29	○	10	0	×	no
	〃 60- 2	7	0.88	16	○	7	0	×	no
	〃 60- 4	9	1.38	24	×	9	0	×	yes
	〃 63- 1	10	0.90	18	○	10	0	×	no
	〃 63- 2	9	0.80	18	○	9	0	×	no
	〃 63- 9	10	0.95	19	○	10	0	×	no
	〃 63-10	9	1.80	23	×	10	0	×	yes
	〃 63-12	9	1.10	20	×	9	0	×	yes
	〃 6- 3	10	0.70	22	○	10	0	×	no
	〃 6- 8	10	1.20	24	○	10	0	×	no
Gomyōkō	G_D-58- 1	7	0.90	16	○	9	1	×	no
	〃 59- 1	9	1.48	23	○	9	0	×	no
	〃 59- 3	8	1.10	19	○	8	0	×	no
	〃 59- 4	6	3.40	18	×	3	1	×	yes
	〃 63- 1	8	1.10	19	○	8	0	×	no
	〃 63- 2	6	0.58	14	○	3	1	×	no
	〃 63- 3	8	1.05	16	○	8	0	×	no
	〃 6- 1	9	4.15	21	×	10	0	×	yes
	〃 6- 2	9	3.30	11	×	3	1	×	yes
	〃 6- 3	9	1.25	28	○	10	0	×	no
	〃 6- 4	8	1.60	21	○	10	0	×	no
	〃 6- 5	8	1.70	24	○	2	1	×	no
	〃 6- 6	8	1.10	21	○	10	0	×	no
	〃 6- 7	7	1.05	19	○	10	0	×	no
	〃 6- 8	7	0.70	22	○	10	0	×	no
	〃 6-10	10	1.30	18	○	10	0	×	no

Area	④ Results estimated by utilizing pore water pressure data				⑤ Horizontal design seismic coefficient (k_s)					
	H_1 (m)	H_2 (m)	Occurrence of liquefaction-induced damage	Agreement (yes or no)	T_G (s)	Ground classi-fication	C_Z	C_G	C_I	k_s
Tamanoike, etc.	2	1	×	no	0.2	2	0.85	1.0	1.0	0.13
	4	1	×	no	〃	〃	〃	〃	〃	〃
	9	0	×	yes	〃	〃	〃	〃	〃	〃
	10	0	×	no	〃	〃	〃	〃	〃	〃
	1	1	○	yes	〃	〃	〃	〃	〃	〃
	10	0	×	no	〃	〃	〃	〃	〃	〃
	9	0	×	yes	〃	〃	〃	〃	〃	〃
	1	1	○	yes	〃	〃	〃	〃	〃	〃
	1	1	○	yes	〃	〃	〃	〃	〃	〃
	10	0	×	no	〃	〃	〃	〃	〃	〃
Gomyōkō	7	0	×	no	〃	〃	〃	〃	〃	〃
	9	0	×	no	〃	〃	〃	〃	〃	〃
	2	1	×	no	〃	〃	〃	〃	〃	〃
	6	0	○	yes	0.1	1	〃	0.8	〃	0.10
	8	0	×	no	0.2	2	〃	1.0	〃	0.13
	1	2	×	yes	0.1	1	〃	0.8	〃	0.10
	8	0	×	no	0.2	2	〃	1.0	〃	0.13
	10	0	○	yes	〃	〃	〃	〃	〃	〃
	0	4	○	no	0.3	〃	〃	〃	〃	〃
	10	0	×	no	〃	〃	〃	〃	〃	〃
	2	1	×	no	〃	〃	〃	〃	〃	〃
	2	1	×	no	〃	〃	〃	〃	〃	〃
	3	1	×	no	〃	〃	〃	〃	〃	〃
	10	0	×	no	〃	〃	〃	〃	〃	〃
	3	1	×	no	0.1	1	〃	0.8	〃	0.10
	4	2	×	no	0.2	2	〃	1.0	〃	0.13

and the estimations.
(4) The horizontal design seismic coefficient (k_s) was estimated in the fifth column. The predominant period in the subsurface layers (T_G) was obtained by Eq. (12) and ground classification was decided from Table 1. Hence, revised ground coefficient (c_G) was derived as shown in Table 2. Furthermore, the revised coefficient of zone classification (c_Z) of 0.85 was used, based on a procedure from the Japan Highway Bridge Code, where a classification (c_i) of 1.0 was proposed to ensure house stability during an earthquake. Finally, applying a k_{SO} of 0.15, the comparison of the surveys with the analyses showed a coincidence rate of 23% when using procedure from the Japan Highway Bridge Code, and a coincidence rate of 31% when using the method which utilized residual pore water pressure data.

In the case of applying the standard design seismic coefficient k_{SO} of 0.22.

A comparison of the surveys with the analyses showed that the prediction of liquefaction-induced damage during an earthquake could not be accurately done by using existing methods. In order to predict with a high accuracy liquefaction-induced damage, some values and conditions in the two practical methods need to be corrected as follows:
(1) We observed a strong motion record from the 1983 Nihonkai-Chubu Earthquake at the location of FD−7+425 of the Hachirogata polder dike as shown in Fig. 3. The records of three components measured by the SMAC type of accelerograph are shown in Fig. 4. Based on the recorded N-S component as shown in the lowest part of Fig. 4, seismic response analysis was attempted in order to estimate a maximum response acceleration on the crest of the dike at WC−13+650, which is located less than one thousand meters from Wakami Town. Fig. 5 showns a soil profile, including velocities of the S wave and unit weights of soils obtained from the surveys at WC−13+650 of the Hachirogata western dike, and they were utilized for response analysis. A maximum response acceleration of 224 gal was obtained from the response analysis, as shown in the uppr part in Fig. 6. Therefore, we attempted to use a standard horizontal seismic coefficient k_{SO} of 0.22 in stead of a k_{SO} of 0.15.
(2) The revised coefficient or region classification (cz) of 0.85 in Akita Prefecture should be corrected to c_z of 1.00, because a great number of houses and housing sites suffered serious damage due to liquefaction during the 1983 Nihonkai-Chubu Earthquake.

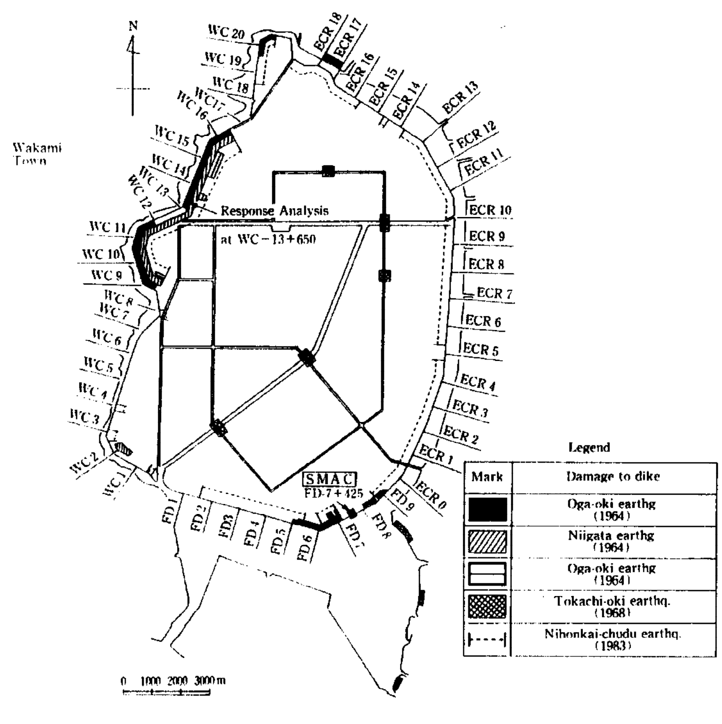

Fig. 3. Plan of Hachirogata Polder Dike

Depth (m)	Soil types	Vs (m/s)	γt (gf/cm³)
0	Fs	130	1.85
	As	95	1.85
	Ac	95	1.50
20	As	140	1.85
	Ds	190	1.95
40			
	Dc	290	1.70
	Ds	330	1.95
	Dc	290	1.70
60	Ds	330	1.95
	Dc	290	1.70
80			
	(T)	500	2.20

Fig. 5. Soil Profile at WC−13+650.

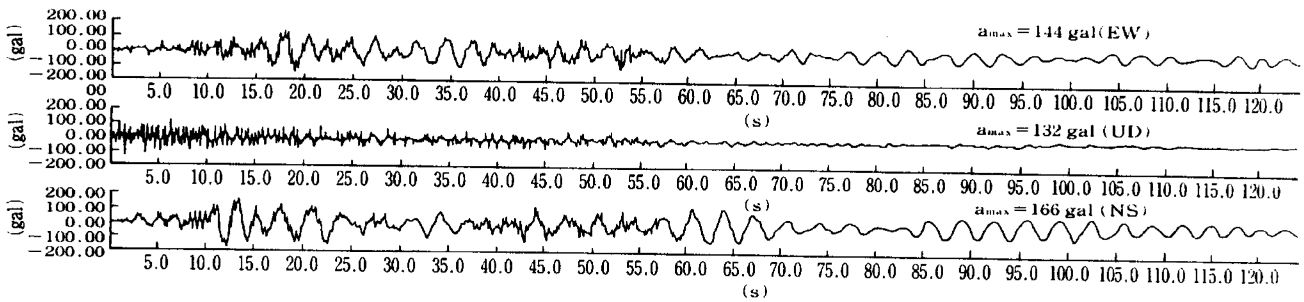

Fig. 4. Earthquake Records measured by SMAC during the 1983 Nihonkai-Chubu Earthquake at FD−7+425 on Hachirogata Polder Dikes.

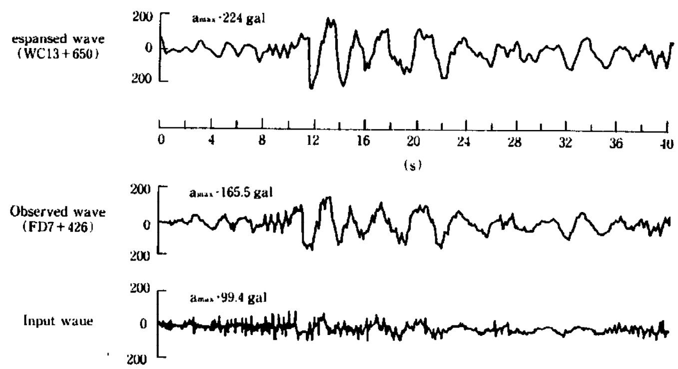

Fig. 6. Response wave, Observed wave, and Input wave.

(3) The ground surface over ground water table was not liquefied during the earthquake because there was no water at the surface. Therefore, the F_L values for this calculation should be realer than 1.0.
By taking these corrections into consideration, the comparison of the surveys with the analyses showed a coincidence rate of 81% using the procedure of Japan Highway Bridge Code and 80% for the method which utilized pore water pressure data (see Table 4). Thus, liquefaction-induced damage to a house can be accurately predicted by the two practical methods provided there is a close agreement between the k_{so} value and a ratio of maximum acceleration to gravity acceleration.

A PRACTICAL METHOD FOR PREDICTING LIQUEFACTION-INDUCED DAMAGE TO HOUSE

Using the relationship between H_1 and H_2, the extent of the damage to houses due to liquefaction during an earthquake on the east side of the sand dunes along the North Japan seaside with a similar origin and magnitude as the 1983 Nihonkai-Chubu Earthquake can be predicted. The following steps can be followed to determine the risk of liquefaction-induced damage to houses :
(1) Immediately before building a house, it is recommended that auger drilling be carried out the building site. If the ground water table is at a depth of at least 2.0m, the potential for liquefaction-induced damage to the house is reduced.
(2) If the ground water table is at a depth of less than 2.0m, boring, including a standard penetration test, shoud be performed on an on-going basis based on these results, likelihood of liquefaction-induced damage to houses can be predicted as follows : (i) Severe liquefaction-induced damage will likely occur to houses built on a layer of sandy soil with H_1 less than H_2, and N is less than 15. (ii) However, little liquefaction-induced damage will likely occur to houses built on a layer of sandy soil with H_1 greater than H_2. If the likelihood of liquefaction-induced damage can be assessed, some measures can be taken to improve the foundation of the house or the loose sandy layer.

Table 4. Comparison of Results from Surveys and Analyses by Two Practical Methods for Predicting Liquefaction ($k_{so}=0.22$)

Area	① Boring. No.	② Results from surveys & interviews				③ Results estimated by Japan Highway Bridge Code			
		Depth of bedrock (m)	Ground water table	Averaged N Values	Liquefaction-induced damage	H_1 (m)	H_2 (m)	Occurrence of liquefaction-induced damage	Agreement (yes or no)
Tamanoike, etc.	Ta-59- 2	9	0.50	29	○	0.5	0.5	○	yes
	〃 60- 2	7	0.88	16	○	0.88	2.12	○	yes
	〃 60- 4	9	1.38	24	×	10	0	×	yes
	〃 63- 1	10	0.90	18	○	0.9	2.1	○	yes
	〃 63- 2	9	0.80	18	○	0.8	4.2	○	yes
	〃 63- 9	10	0.95	19	○	0.95	3.05	○	yes
	〃 63-10	9	1.80	23	×	10	0	×	yes
	〃 63-12	9	1.10	20	×	5	1	×	yes
	〃 6- 3	10	0.70	22	○	0.7	5.3	○	yes
	〃 6- 8	10	1.20	24	○	1.2	0.8	×	yes
Gomyōkō	GD-58- 1	7	0.90	16	○	0.91	4.1	○	yes
	〃 59- 1	9	1.48	23	○	3	1	×	no
	〃 59- 3	8	1.10	19	○	1.10	1.90	○	yes
	〃 59- 4	6	3.40	18	×	10	0	×	yes
	〃 63- 1	8	1.10	19	○	1.10	3.9	○	yes
	〃 63- 2	6	0.58	14	○	0.58	2.42	○	yes
	〃 63- 3	8	1.05	16	○	1.05	1.95	○	yes
	〃 6- 1	9	4.15	21	×	10	0	×	yes
	〃 6- 2	9	3.30	11	×	4	1	×	yes
	〃 6- 3	9	1.25	28	○	10	0	×	no
	〃 6- 4	8	1.60	21	○	1.6	2.4	○	yes
	〃 6- 5	8	1.70	24	○	2	2	○	yes
	〃 6- 6	8	1.10	21	○	3	1	×	no
	〃 6- 7	7	1.05	19	○	2	2	○	yes
	〃 6- 8	7	0.70	22	○	0.7	0.3	×	no
	〃 6-10	10	1.30	18	○	2	5	○	yes

Area	④ Results estimated by utilizing pore water pressure data				⑤ Horizontal design seismic coefficient (K_s)					
	H_1 (m)	H_2 (m)	Occurrence of liquefaction-induced damage	Agreement (yes or no)	T_G (s)	Ground classi-fication	C_Z	C_G	C_I	k_s
Tamanoike, etc.	0.5	0.5	○	yes	0.2	2	1.00	1.0	1.0	0.22
	0.88	5.12	○	yes	〃	〃	〃	〃	〃	〃
	10	0	×	yes	〃	〃	〃	〃	〃	〃
	2	2	○	yes	〃	〃	〃	〃	〃	〃
	0.8	7.2	○	yes	〃	〃	〃	〃	〃	〃
	0.95	8.05	○	yes	〃	〃	〃	〃	〃	〃
	10	0	×	yes	〃	〃	〃	〃	〃	〃
	5	1	×	yes	〃	〃	〃	〃	〃	〃
	0.7	7.3	○	yes	〃	〃	〃	〃	〃	〃
	1.2	0.8	×	no	〃	〃	〃	〃	〃	〃
Gomyōkō	0.9	5.1	○	yes	〃	〃	〃	〃	〃	〃
	1.48	2.52	○	yes	〃	〃	〃	〃	〃	〃
	1.10	1.90	○	yes	〃	〃	〃	〃	〃	〃
	10	0	×	yes	0.1	1	〃	0.8	〃	0.18
	3	2	×	no	0.2	2	〃	1.0	〃	0.22
	0.58	3.42	○	yes	0.1	1	〃	0.8	〃	0.18
	1.05	1.95	○	yes	0.2	2	〃	1.0	〃	0.22
	4.15	0.85	×	yes	〃	〃	〃	〃	〃	〃
	3.3	3.7	○	no	0.3	〃	〃	〃	〃	〃
	3	2	×	no	0.2	〃	〃	〃	〃	〃
	1.6	2.4	○	yes	〃	〃	〃	〃	〃	〃
	1.7	2.3	○	yes	〃	〃	〃	〃	〃	〃
	1.10	0	×	no	〃	〃	〃	〃	〃	〃
	2	3	○	yes	〃	〃	〃	〃	〃	〃
	0.7	0.3	×	no	0.1	1	〃	0.8	〃	0.18
	2	6	○	yes	0.2	2	〃	1.0	〃	0.22

REFERENCES

Kotoda, K. and Wakamatsu, K. (1984). Geomorphological consideration on site of liquefidsides during Nihonkai-Chubu Earthquake. *Soils and Foundations, vol. 32, No. 9*, 59~63.

Masaki, K. Taniguchi, H. and Iida, S. (1982). Empirical equations for the estimation of shear were velocity and density in soils by meana of boring data. *17th Anual Meeting, JSSMFE*, 1541~1544.

Ito, T, Asada, A. Konno, T. Kurihara, M. and Hotta, A. (1992). A study of damage to sandy soils due to liquefaction induced by earthquaks and of methods for predicting liquefaction. *Proceedings of 10th World Conference on Earthquake Engineering*. 1343~1347.