INTENSITY OF EARTHQUAKE GROUND MOTION AT LIQUEFIED SITES

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ABSTRACT

The critical intensity of earthquake ground motion which separates liquefiable to non-liquefiable conditions is investigated. The peak ground acceleration and velocity are calculated at about 130 liquefied and non-liquefied sites during 19 Japanese earthquakes. These sites are on natural levee, river channel, reclaimed land or drained land, and have high liquefaction susceptibility. In the calculation, the effects of fault rupture and local site condition are considered, to obtain more accurate results than those in previous studies. The results of the calculation indicate that (1) the occurrence of soil liquefaction is better correlated with the peak ground velocity than with the peak ground acceleration, (2) soil liquefaction is likely to occur for the ground with high liquefaction susceptibility, when the peak ground velocity exceeds 15 kins (cm/s), and (3) the degree of soil liquefaction shows no clear correlation with peak ground velocity or acceleration.

Key words : case history, earthquake, earthquake ground motion, liquefaction, peak ground velocity (IGC : E 8)

INTRODUCTION

In recent earthquakes, the damage of wooden houses or life lines, which are important elements in seismic risk evaluation, is mainly caused by soil liquefaction, not by ground shaking (e.g. Taniguchi and Iida, 1986; Satake, 1986). To interpret soil liquefaction during earthquake, several empirical methods have been already proposed (e.g. Seed and Idriss, 1971; Iwasaki et al., 1978; Tokimatsu and Yoshimi, 1983). In

these studies, the soil properties, such as SPT *N*-value or fines content, at the liquefied sites were carefully collected from boring tests, because the liquefaction resistance of soils can be evaluated by these properties. On the other hand, the peak ground accelerations, which is related to the shear stress in soil ground, at the liquefied sites were simply estimated from the empirical attenuation law, the overturning of the simple tombstones or the reported seismic intensity scale. The empirical attenuation law with

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Fig. 1. Peak ground acceleration and velocity observed near liquefied and non-liquefied sites

parameters of earthquake magnitude and distance has the standard deviation of approximately 100% to the mean value. Consequently, the peak ground accelerations used in these studies are only rough estimates.

These rough estimations, however, are obliged by the unavailability of the strong-motion record at or near the liquefied site. Fig. 1 shows the peak ground acceleration and velocity observed near the liquefied and non-liquefied sites during the 1964 Niigata, 1968 Tokachi-Oki, 1978 Miyagiken-Oki, 1980 Chibaken-Chubu, 1982 Urakawa-Oki, and 1983 Nihonkai-Chubu earthquakes. Most of the data were observed at harbors. The number of the data is limited for discussing the critical intensity of ground motion which separates liquefiable to non-liquefiable conditions. It is also difficult to verify that the peak ground acceleration is the most appropriate measure of ground motion for evaluating soil liquefaction potential.

It should be noted that the review of case histories of soil liquefaction in the previous studies tends to be concentrated on the 1964 Niigata Earthquake and the later ones. As most of these earthquakes are due to reverse faulting with low dip angle and their magnitude is 7 1/2 to 8, the proposed methods may

be valid only for earthquakes of the specific type.

In this paper, the intensities of the ground motions at liquefied sites during past earthquakes are calculated, to investigate the critical intensity which separates liquefiable to non-liquefiable conditions. In the calculation, (1) the effects of dimensions of earthquake fault plane, the rupture propagation and local site conditions, in addition to earthquake magnitude and distance, are considered and (2) a large number of earthquakes including those before the Niigata Earthquake is reviewed.

CALCULATION OF INTENSITIES OF EARTHQUAKE GROUND MOTIONS AT LIQUEFIED SITES

Method. The peak ground acceleration and peak ground velocity are calculated by the semi-empirical method which can take the effects of dimensions of earthquake fault plane, the rupture propagation and local site conditions into account (Kobayashi and Midorikawa, 1982). The fault of an earthquake is divided into small elements. The response envelope waveform of SDOF (Single Degree of Freedom) system from each element arriving at a given site is established using empirical formulas derived from statistical analysis of strong-motion records. Then, the response envelope waveform for the entire fault is calculated by superposing the results for the elements. These calculations are performed for each period, and the response spectrum on the seismic bedrock is obtained from the maximum values. The schematic diagram of the superposition is shown in Fig. 2. From the spectrum intensities of the response spectrum, the peak acceleration A_B and velocity V_B on the seismic bedrock are obtained (Midorikawa and Kobayashi, 1978).

In order to take the local site effect into account, the site amplification factor for peak acceleration G_A and that for peak velocity G_V are introduced. The peak ground acceleration PGA and velocity PGV

Fig. 2. Schematic diagram on the method for calculating ground motion (after Kobayashi and Midorikawa, 1982)

Fig. 3. Relation between amplification factor for peak acceleration, G_A , and S-wave velocity of surface layer, V_t (after Midorikawa and Kobayashi, 1980)

are obtained from the products of the peaks on seismic bedrock and the site amplification factors :

$$PGA = G_A \times A_B \quad (1)$$

$$PGV = G_V \times V_B \quad (2)$$

G_A is strongly affected by the S-wave velocity of upper-most layer V_t . As shown in Fig. 3, they show relatively good correlation, nevertheless the thickness of the upper-most layer is not taken into account. Its relation is represented as the following equation (Midorikawa and Kobayashi, 1980) :

$$G_A = \begin{cases} 5.5 & (V_t < 200 \text{ m/s}) \\ 40 \times V_t^{-0.374} & (V_t > 200 \text{ m/s}) \end{cases} \quad (3)$$

where V_t is in m/s. Generally, V_t at liquefied sites is lower than 200 m/s. It follows that

Fig. 4. Relation between amplification factor for peak acceleration, G_A , and depth down to hard soil layer

G_A at liquefied site takes around 5.5. However, the scatter of the data from this value is not small (see Fig.3). To diminish the scatter, the correlation of G_A with the depth down to the hard soil layer whose SPT N -value is greater than 50, is checked for the data with V_t of lower than 200 m/s. The data are selected from those used by Midorikawa and Kobayashi (1980). As shown in Fig. 4, at the depth of 15 meters, the amplification shows the maximum. If the predominant period of the ground is determined by soil layers above the hard layer, the predominant period of the ground with 15 meters depth becomes 0.24 to 0.3 seconds, when the average S-wave velocity of the soil layers is considered to be 200 to 250 m/s. It has been pointed out from observation and analyses that the earthquake motion on the ground with such a predominant period tends to have

Fig. 5. Relation between amplification factor for peak velocity, G_V , and mean S-wave velocity from surface to 30 meters depth, V_m (after Midorikawa, 1987)

larger peak acceleration (e.g. Tajime et al., 1977). Consequently, the correlation of G_A with the depth is physically reasonable. The relation of G_A to the depth is proposed as indicated by the solid line in the figure.

On the other hand, G_V is affected by the deeper layers. As shown in Fig. 5, the mean S-wave velocity from surface to 30 meters depth, V_m , correlates with G_V (Midorikawa, 1987) :

$$\left. \begin{aligned} G_V &= 170 \times V_m^{-0.6} & (V_m < 1100 \text{ m/s}) \\ &= 2.5 & (V_m > 1100 \text{ m/s}) \end{aligned} \right\} \quad (4)$$

Fig. 6. Results of nonlinear analysis of ground amplification

where V_m is in m/s.

These amplification factors are obtained on the basis of the linear response of soil layers, but the effects of the nonlinear response may not be negligible at liquefied sites. The nonlinear analysis by SHAKE (Schnabell et al., 1972) is conducted to check the validity of the results from the linear response. In the analysis, the G/G_0 and the equivalent damping coefficient are set to 0.5 and 15% at the strain level of 10^{-3} , respectively. The record at Taft in 1952 is adopted as base rock input motion. Fig. 6 shows the results

(a) Peak ground acceleration

(b) Peak ground velocity

Fig. 7. Comparison of observed and calculated peaks

Table 1. List of liquefied and non-liquefied sites with calculated peaks of ground motion

Earthquake ⁺	Site Name	PGA cm/s ²	PGV cm/s	Degree [*]
Nobi (1891) M=8.0 L	Ezaki, Gifu City	1380	90	A
	Ginan Town, Gifu Pref.	1480	97	U
	Kasamatsu Town, Gifu Pref.	1380	97	A
	Nishibiwajima Town, Aichi Pref.	550	58	A
	Jinmokuji Town, Aichi Pref.	660	65	A
	Oritoyo, Nagoya City	660	59	A
	Okudashinden, Nishio City	270	30	A
	Nagashima Town, Mie Pref.	380	57	A
	Tanabe Town, Kyoto Pref.	150	11	A
	Oda, Amagasaki City	90	11	C
	Wakayama Port, Wakayama City	70	10	—
Kanto (1923) M=7.9 R	Kurihashi Town, Saitama Pref.	170	31	B
	Washinomiya Town, Saitama Pref.	180	29	B
	Shobu Town, Saitama Pref.	190	25	U
	Sugito Town, Saitama Pref.	200	35	A
	Kurohama, Hasuda City	230	33	A
	Hihori, Kasukabe City	210	40	B
	Nagamiya, Iwatsuki City	220	41	A
	Matsubushi Town, Saitama Pref.	220	33	B
	Fukuroyama, Koshigaya City	240	32	B
	Oosawa, Koshigaya City	240	46	A
	Ho, Yoshikawa, Saitama Pref.	250	46	B
	Yomaki, Misato City	330	34	B
	Tajima, Urawa City	260	44	U
	Kitasuna, Tokyo	350	65	B
	Tsukishima, Tokyo	380	48	B
	Shinagawa, Tokyo	510	52	A
	Oomori, Tokyo	510	47	A
	Maruko, Kawasaki City	610	40	A
	Eiraku, Yokohama City	550	103	B
	Asaba Town, Shizuoka Pref.	160	23	—
Kita-Tango (1927) M=7.3 L	Nodagawa Town, Kyoto Pref.	560	53	A
	Oosaka Port, Oosaka City	80	12	U
	Awara Town, Fukui Pref.	100	14	—
	Kawauchi, Tokushima City	60	7	—
	Wakayama Port, Wakayama City	60	8	—
Kita-Izu (1930) M=7.3 L	Kannami Town, Shizuoka Pref.	710	66	B
	Nirayama Town, Shizuoka Pref.	960	98	C
	Eiraku, Yokohama City	120	18	—
Nishi-Saitama (1931) M=6.9 L	Menuma Town, Saitama Pref.	340	37	U
	Sukage, Hanyu City	300	25	U
	Kazo, Kazo City	250	23	U
	Oosato Vil., Saitama Pref.	630	37	A
	Fukiage Town, Saitama Pref.	610	39	A
	Yoshimi Town, Saitama Pref.	510	30	U
	Washinomiya Town, Saitama Pref.	200	20	U
	Shobu Town, Saitama Pref.	230	19	U
	Shiraoka Town, Saitama Pref.	190	15	U
	Kurohama, Hasuda City	200	18	U
	Hihori, Kasukabe City	150	19	U
	Nagamiya, Iwatsuki City	150	18	U
	Sasakuboshinden, Iwatsuki City	150	14	U
	Oosawa, Koshigaya City	130	17	U
	Miwanoe, Yoshikawa, Saitama Pref.	120	10	U
	Ho, Yoshikawa, Saitama Pref.	120	15	—
	Tajima, Urawa City	160	18	—

Earthquake ⁺	Site Name	PGA cm/s ²	PGV cm/s	Degree [*]
Shizuoka (1935) M=6.4 L	Shimizu Port, Shimizu City	440	24	C
	Kannami Town, Shizuoka Pref.	120	10	—
Tottori (1943) M=7.2 L	Yoshinari, Tottori City	1010	105	A
	Hikona, Yonago City	150	16	A
	Nodagawa Town, Kyoto Pref.	140	14	—
Tonankai (1944) M=7.9 R	Ginan Town, Gifu Pref.	300	27	U
	Takehana, Hashima City	200	31	U
	Saya Town, Aichi Pref.	230	46	A
	Tsukijiguchi, Nagoya City	260	38	A
	Nishitsukiji, Nagoya City	260	36	A
	Inenagashinden, Nagoya City	260	37	A
	Otogawa, Handa City	390	34	A
	Tomonaga, Fukuroi City	580	52	A
	Yokoi, Fukuroi City	420	54	A
	Asaba Town, Shizuoka Pref.	420	56	A
	Oosaka Port, Oosaka City	120	23	C
	Kawauchi, Tokushima City	80	13	—
	Wakayama Port, Wakayama City	100	17	—
Mikawa (1945) M=6.8 R	Komeno, Nishio City	900	84	A
	Kamidomeki, Nishio City	950	93	A
	Inenagashinden, Nagoya City	240	21	A
	Nijo, Nagoya City	250	23	A
	Kamishinden, Tsushima City	170	18	U
	Nagashima Town, Mie Pref.	190	23	—
	Saya Town, Aichi Pref.	170	20	—
	Ichinomiya, Ichinomiya City	220	13	—
Nankai (1946) M=8.0 R	Ujina, Hiroshima City	90	16	A
	Hikona, Yonago City	70	11	B
	Kawauchi, Tokushima City	150	24	C
	Oosaka Port, Oosaka City	90	17	C
	Wakayama Port, Wakayama City	130	21	C
	Okudashinden, Nishio City	80	8	—
	Ginan Town, Gifu Pref.	50	6	—
Fukui (1948) M=7.1 L	Awara Town, Fukui Pref.	910	106	A
	Sakai Town, Fukui Pref.	790	83	A
	Takaya, Fukui City	700	66	A
	Maruoka Town, Fukui Pref.	810	65	A
	Ezaki, Gifu City	150	10	—
Tokachi-Oki (1952) M=8.2 R	Mukawa Town, Hokkaido	190	20	A
	Hakodate Station, Hakodate City	130	13	—
Oga-Oki (1964) M=6.9 R	Frontal Dike, Oogata Vil., Akita Pref.	120	20	A
	Western Dike, Oogata Vil., Akita Pref.	140	19	A
	Aomori Port, Aomori City	90	10	—
Niigata (1964) M=7.5 R	Western Dike, Oogata Vil., Akita Pref.	80	14	A
	Kawagishi, Niigata City	140*	46*	A
	Kyogase Vil., Niigata Pref.	540	38	A
	Kurosaki Vil., Niigata Pref.	370	42	A
	Tagami Vil., Niigata Pref.	260	35	A
	Bunsui Town, Niigata Pref.	210	25	A
	Shinkai, Nagaoka City	170	23	A
	Ishinomaki Port, Ishinomaki City	100	15	—
Tokachi-Oki (1968) M=7.9 R	Mukawa Town, Hokkaido	270	29	A
	Hakodate Station, Hakodate City	220	30	A
	Aomori Port, Aomori City	260*	38*	A
	Nakazato Town, Aomori Pref.	170	31	A
	Frontal Dike, Oogata Vil., Akita Pref.	120	28	A

Earthquake*	Site Name	PGA cm/s ²	PGV cm/s	Degree [#]
Tokachi-Oki (1968) M=7.9 R	Muroran Port, Muroran City	280*	26*	A
	Kushiro Port, Kushiro City	50*	7*	—
	Ishinomaki Port, Ishinomaki City	80	15	—
Nemurohanto-Oki (1973) M=7.4 R	Hanasaki Port, Nemuro City	280	16	A
	Kushiro Port, Kushiro City	200*	27*	A
Miyagiken-Oki (1978) M=7.4 R	Hasama Town, Miyagi Pref.	430	37	A
	Minamikata Town, Miyagi Pref.	360	39	A
	Kanan Town, Miyagi Pref.	510	35	B
	Ishinomaki Port, Ishinomaki City	370	46	A
	Oosato Town, Miyagi Pref.	500	27	A
	Yuriage Bridge, Natori City	360	35	A
	Abukuma Bridge, Iwanuma City	320	34	B
	Aomori Port, Aomori City	30*	5*	—
Urakawa-Oki (1982) M=7.1 R	Atsuma Town, Hokkaido	190	13	A
	Muroran Port, Muroran City	190*	16*	B
	Hakodate Station, Hakodate City	50*	5*	—
	Mukawa Town, Hokkaido	230	15	—
	Kushiro Port, Kushiro City	50	5	—
Nihonkai-Chubu (1983) M=7.7 R	Nakazato Town, Aomori Pref.	230	37	A
	Aomori Port, Aomori City	170*	29*	A
	Gomyoko, Oogata Vil. Akita Pref.	200	31	A
	Frontal Dike, Oogata Vil., Akita Pref.	160	33	A
	Western Dike, Oogata Vil., Akita Pref.	160	31	A
	Akita Port, Akita City	240*	28*	A
	Niigata Port, Niigata City	15*	13*	—
	Hakodate Station, Hakodate City	60*	15*	—

* L: Lateral faulting, R: Reverse faulting

* Observed Value

Degree of Liquefaction A: Heavy, B: Slight, C: Little, U: Unknown, —: None

of the nonlinear analysis for a sandy ground. Although the peak ground acceleration by the nonlinear analysis (broken line) does not increase linearly with the base rock input acceleration in the high amplitude level, the peak ground acceleration by the linear analysis (solid line) shows no significant difference from that by the nonlinear analysis (broken line) when the acceleration is less than 200 gals (cm/s²). The peak ground velocity by the nonlinear analysis (dotted line) increases almost linearly with the input acceleration, even in the high amplitude level. Fortunately, the critical intensity of ground motion which separates liquefiable to non-liquefiable conditions would not be much higher than 200 gals as expected from Fig. 1. Therefore, the calculation based on the linear response of ground should be available for discussing the critical intensity.

To check the accuracy of the results by the above-mentioned method, PGA and PGV

at strong-motion accelerograph sites are calculated for major earthquakes, such as the 1964 Niigata, 1968 Tokachi-Oki, 1978 Miyagiken-Oki, 1980 Izu-Toho-Oki, 1982 Urakawa-Oki and 1983 Nihonkai-Chubu earthquakes. The comparison between the calculated and observed values is shown in Fig. 7. The standard error of the calculated PGA to the observed is approximately 50%. In the case of PGV, the standard error is within 30%. In both cases, the standard error is much smaller than that of the ordinary attenuation laws with parameters of earthquake magnitude and distance.

Results. The 19 Japanese earthquakes, during which the soil liquefaction occurred extensively and whose fault models have been studied and compiled (Okada and Suzuki, 1984; Suzuki, 1985), are selected as the subject of the study (see Table 1). Thirteen earthquakes which occurred before the Niigata earthquake are included. The

magnitude of the earthquakes ranges from 6.4 to 8.2. The peak acceleration and velocity of the seismic bedrock motion at each site are calculated for these earthquakes, considering the respective fault model. The amplification factors of ground at each site are calculated using boring data. The empirical equations in terms of soil indices by Ohta and Goto (1978) are used to estimate the mean S-wave velocity V_m . The calculation is performed to about 130 liquefied and non-liquefied sites whose geomorphological unit is natural levee, river channel, reclaimed land or drained land, because these units have high liquefaction susceptibility among various geomorphological units (Wakamatsu, 1980). As shown in Table 1, the location of these sites is distributed in most part of Japan. In this paper, the non-liquefied site

is defined as the site with liquefaction susceptibility, which did not liquefy during an earthquake. For the liquefied sites, the degree of soil liquefaction is classified into four classes as shown in Table 2.

Figs. 8 and 9 show the plots of the calculated PGA and PGV. In the figures, the observed values near liquefied sites are also plotted. The critical PGA, which separates

Table 2. Degree of liquefaction

Class	Criteria	Symbol in Figs. 8 and 9
Heavy	Sand boils at many points, or with heavy damage of structures	●
Slight	Sand boils at several points, or with slight damage of structures	◐
Little	Sand boil at a point, or with no damage of structures	○
Unknown	Sand boils, but no description on the degree in documents	○
None	No sand boil	+

Fig. 8. Peak ground acceleration at liquefied and non-liquefied sites

Fig. 9. Peak ground velocity at liquefied and non-liquefied sites

sites where liquefaction occurred and sites where liquefaction did not occur, ranges 70 to 250 gals, and the scatter of the critical PGA is not small. The tendency that the critical acceleration decreases with increasing earthquake magnitude is recognized, as has been pointed out by the previous studies (e.g. Seed et al., 1984).

On the other hand, the critical PGV lies within a narrow band of 10 to 20 kins (cm/s), regardless of the earthquake magnitude. On the average, it can be considered as 15 kins for the ground with high liquefaction susceptibility such as natural levee, river channel, reclaimed land or drained land. This implies that no significant difference might be recognized on the liquefaction susceptibility in the same geomorphological unit, and that the liquefaction susceptibility at a site might be classified only by the geomorphological unit, as has been pointed out by Wakamatsu (1980).

Among the whole data, the correlation of the degree of liquefaction with PGA or PGV could not be recognized clearly. The degree of soil liquefaction is sensitive to various ground conditions such as the depth, thickness, inclination or density of sand layers, and the depth of ground water. Generally, these minute conditions are different at site by site, even in the same geomorphological unit (Kotoda and Wakamatsu, 1984). Therefore, to predict the degree of soil liquefaction, various geotechnical data at the site of interest should be indispensable.

MAGNITUDE DEPENDENCE OF THE CRITICAL PEAK GROUND ACCELERATION AND VELOCITY

To check the magnitude dependence of the critical PGA and PGV which separate liquefiable to non-liquefiable conditions, the relationship between earthquake magnitude, M , and maximum source distance to liquefied sites, D , is examined. Fig. 10 shows this relationship for the 19 earthquakes shown in Table 1 and three other Japanese earthquakes. In the figure, the shortest

Fig. 10. Relation between maximum source distance to liquefied site and earthquake magnitude

distance from the fault plane is adopted instead of the epicentral distance, because the epicentral distance has not significant meaning for large earthquakes with large source area. Excepting few cases, the upper bound line for the distance where liquefaction has been observed is represented by the following equation :

$$\log D = 0.6M - 2.4 \quad (5)$$

where D is in kilometer. This relation is slightly different from that proposed by Kuribayashi and Tatsuoka (1975), due to the different definition of distance.

The magnitude-distance relations which gives 100 gals of PGA and 10 kins of PGV are derived from the empirical attenuation laws proposed by Katayama (1974) and Muramatsu (1964), respectively. As shown by the dotted line in the figure, the iso-acceleration line shows the different magnitude dependence to Eq. (5). However, the iso-velocity line shown by the broken line corresponds to Eq. (5) fairly well. This suggests that the soil liquefaction is possible to occur when the peak ground velocity exceeds 10 kins, irrespective of earthquake magnitude. This statistical result is consistent with our calculation mentioned before, and supports that the critical PGV is inde-

pendent of earthquake magnitude.

MEASURE OF GROUND MOTION FOR EVALUATING SOIL LIQUEFACTION POTENTIAL

Usually, in soil liquefaction problems, the peak ground acceleration has been used as the measure of the intensity of the ground motion, because the shear stress which is developed in soil ground during earthquake is related to the acceleration. However, it might be difficult to interpret the soil liquefaction only by the peak ground acceleration, because the ground acceleration is only instantaneous value and may not correspond to effective value.

Here, we assume that the product of the peak ground acceleration and duration is an appropriate measure to evaluate soil liquefaction potential, because the laboratory tests show that the duration of ground motion also affects liquefaction susceptibility. The correlation of the product with the peak ground acceleration or peak ground velocity is plotted in Fig. 11 for the strong-motion records observed at harbors and near bridges across the river. The site conditions of these strong-motion sites are similar to those

of liquefied sites. To define the duration, the integral of the power of the acceleration is calculated. The first 5% and the last 5% amplitudes of the total power is deleted, and the remaining 90% is considered as the significant contribution to the integral of the power of the acceleration. The duration is defined as the time interval during which the significant contribution takes place (Trifunac and Brady, 1975). The product shows better correlation with the velocity than the acceleration. It is plausible that the ground motion with longer duration is generally predominant in longer period range. As the result, the peak velocity of the record with longer duration becomes larger in comparison with its peak acceleration. This could be one of the interpretations that the peak ground velocity is more suitable measure for evaluating soil liquefaction potential than the acceleration.

The other interpretation could be also possible. It has been pointed out that the peak ground velocity is related to the strain level developed in soil layers. According to Tokimatsu and Midorikawa (1981), the shear strain in soil ground increases linearly with increasing PGV when PGV is less than 10 kine, but increases more rapidly when PGV

Fig. 11. Correlation of product of peak ground acceleration and duration with peak ground acceleration or velocity

becomes greater than 15 to 20 kines, because of nonlinearity of soil. It is interesting that the value of 15 kines corresponds to the strain level from which nonlinear soil properties become apparent. Consequently, it could be understood easily that the peak ground velocity is the suitable measure of ground motion for evaluating soil liquefaction potential.

CONCLUSIONS

On the basis of the calculation of ground motions at liquefied sites during past earthquakes in Japan, the following conclusions are obtained:

(1) The occurrence of soil liquefaction is better correlated with the peak ground velocity than with the peak ground acceleration which has been often used in previous studies. The peak ground velocity is considered as more appropriate measure of ground motion for evaluating liquefaction potential.

(2) For the ground with high liquefaction susceptibility, such as natural levee, river channel, reclaimed land or drained land, soil liquefaction is likely to occur when the peak ground velocity exceeds 15 kines.

(3) The degree of soil liquefaction shows no clear correlation with peak ground velocity or acceleration, and it is difficult to predict the degree of liquefaction only from the intensity of ground motion.

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NOTATIONS

PGA=peak ground acceleration

PGV=peak ground velocity

A_B =peak acceleration on seismic bedrock

V_B =peak velocity on seismic bedrock

G_A =site amplification factor for peak accelera-

tion

G_V =site amplification factor for peak velocity

V_s =shear-wave velocity of surface layer

V_m =mean shear-wave velocity from surface to 30 meters depth

D =maximum source distance to liquefied site

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