Evaluation of Liquefaction Potential of Gravelly Soil Layer Based on Field Performance Data

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ABSTRACT

In general, gravelly soil layers are considered to be less susceptible to liquefy during earthquakes than sandy soil layers. However, in 1995 Hyogoken-Nambu Earthquake, sand ejection due to liquefaction was observed in Port Island and Rokko Island, which were man-made islands reclaimed by gravelly soils. In this paper, firstly, site investigation including sampling by freezing technique was conducted in Rokko island where lots of ejected sand was not observed during the earthquake to know liquefaction strengths of the reclaimed deposits. Secondly, earthquake response analyses were conducted for a site in Rokko Island as well as a site in Port Island to evaluate maximum stress ratios which were generated in the reclaimed deposits during the earthquake. Finally, relationships between maximum shear stress ratios during earthquakes and penetration resistance were investigated for liquefied sites and nonliquefied sites in the man-made islands.

INTRODUCTION

In general, gravelly soil layers are considered to be less susceptible to liquefy during earthquakes than sandy soil layers. However, traces of liquefaction which occurred in gravelly soil layers are discovered by archaeological excavation conducted recently. Furthermore, some gravelly ground liquefied in recent large earthquakes. In 1995 Hyogoken-Nambu Earthquake, sand ejection due to liquefaction was observed in Port Island and Rokko Island, which were man-made islands reclaimed by gravelly soils, as shown in Fig.1. However, a closer look at Fig.1 seems to reveal that severity of the sand ejection varies largely with the location in the man-made islands. In the northern area of Rokko island together with Port Island(Phase I), the ground surfaces are mostly covered by the ejected sands, whereas there are a few traces of sand ejection on the ground surfaces of the southern area of Rokko Island and Port Island(Phase II). Residual inclination and subsidence of buildings with spread foundation in Port Island(Phase I) are reportedly larger than those in the southern area of Rokko Island(Kakurai et al., 1996). Though the causes for the difference in sand ejection as well as damage to the buildings are already listed in other literature(Yasuda et al., 1996 ; Ohta et al., 1997), quantitative assessment of the causes are not done yet.

In this paper, firstly, site investigation including sampling by freezing technique was conducted in Rokko island where lots of ejected sand was not observed during 1995 Hyogoken-Nambu Earthquake to know liquefaction strengths of the reclaimed deposits. Secondly, earthquake response analyses were conducted for a site in Rokko Island as well as a site in Port Island to evaluate maximum stress ratios which were generated in the reclaimed deposits during 1995 Hyogoken-Nambu Earthquake. Finally, relationships between maximum shear stress ratios during earthquakes and penetration resistance were investigated for liquefied sites and nonliquefied sites in the man-made islands.

SITE INVESTIGATION

In the whole area of Port Island together with the northern area of Rokko Island, land reclamation was conducted mainly using granite-origin gravelly soil called Masa, whereas in the southern area of Rokko Island, land reclamation was conducted mainly using mud stone-origin and tuff-origin gravelly soils excavated from Kobe layers. Since results of liquefaction tests using Masa samples procured by sampling by in situ freezing technique were already published(Zen et al., 1996 ; Hatanaka et al., 1997), sampling by the in situ freezing technique was conducted for reclaimed soils in the southern area of Rokko Island. The location of sampling site is shown in Fig.1 as Site G. Since Site G is located in the area improved by sand drains, sampling by in situ freezing technique was conducted at a spot among sand drain columns.

Figure 2 shows the grain size distributions of samples indicating that the grain size distributions of samples procured by the in situ freezing technique are in harmony with those by

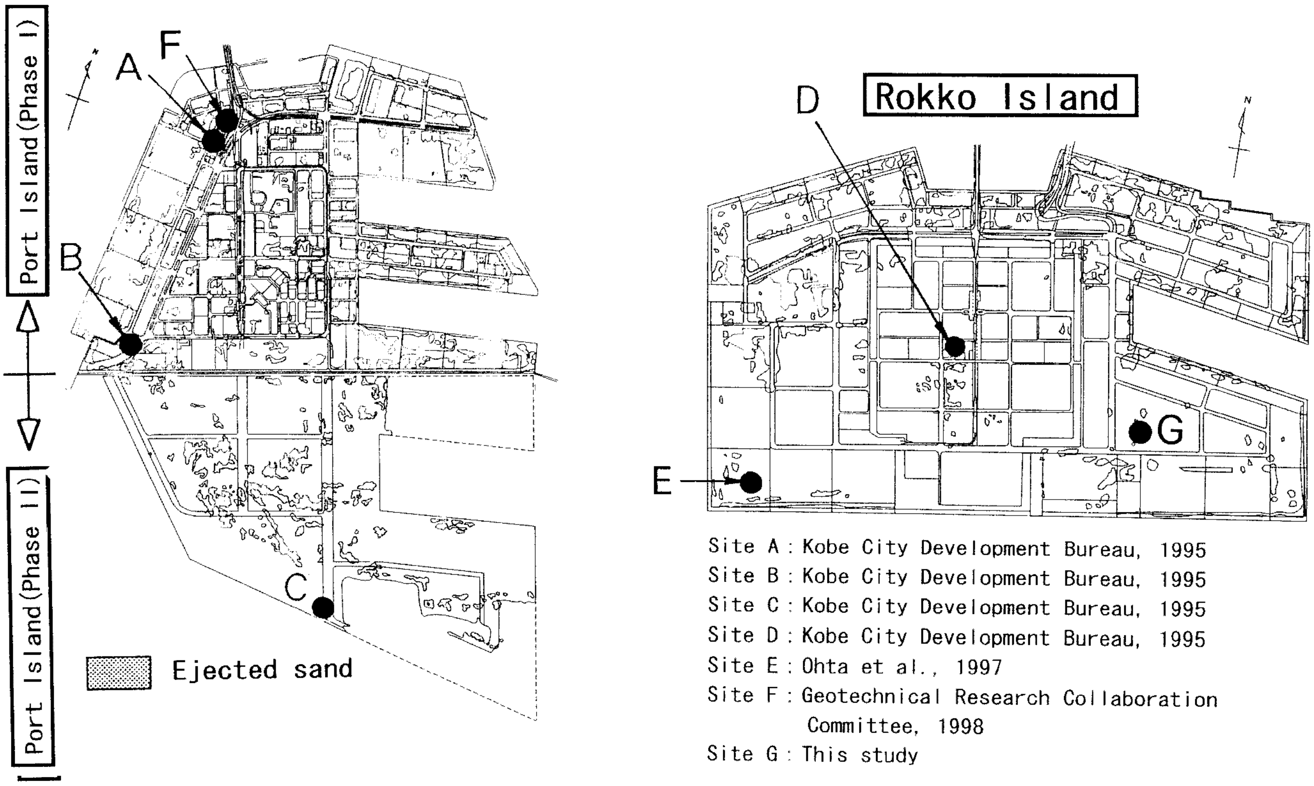


Fig.1 Distribution of ejected sand and locations of Sites A, B, C, D, E, F, G in the man-made islands

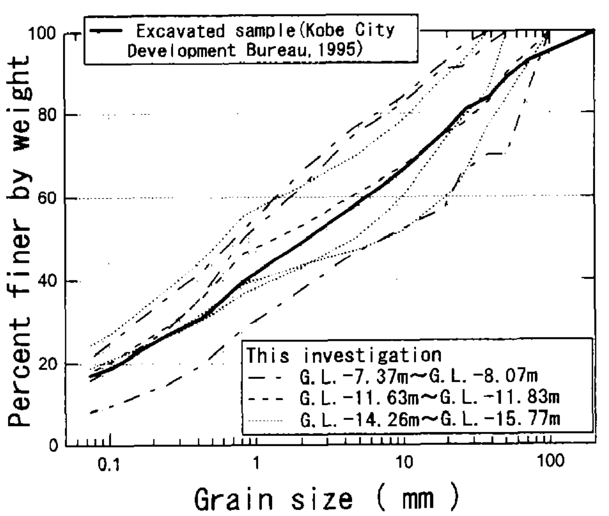


Fig.2 Grain size distributions of samples from Rokko Island

excavation near the surface of the ground. Figure 2 also shows that the contents of gravel and fines are about 30 to 60 % and 10 to 30 %, respectively. Table 1 shows the conditions and results of undrained cyclic triaxial tests conducted for the samples procured by the in situ freezing technique. Figure 3 shows the relationships between stress ratios and number of cycles. Undrained cyclic strengths defined as stress ratios required to reach 5% double axial strain amplitude ,DA, in 20 loading cycles ,N_c, of the upper layer, intermediate layer and lower layer are about 0.32, 0.28, 0.25, respectively. Since the undrained cyclic strengths of Masa samples are reportedly 0.25 or 0.20(Zen et al., 1996 ; Hatanaka et al., 1997), the undrained cyclic strengths of the samples from Site G are not smaller than those of the Masa samples. This difference is possibly attributable to the difference in SPT N-value as discussed in the later part of this paper.

EARTHQUAKE RESPONSE ANALYSIS OF SITE A AND SITE E

Ground conditions and input earthquake motions

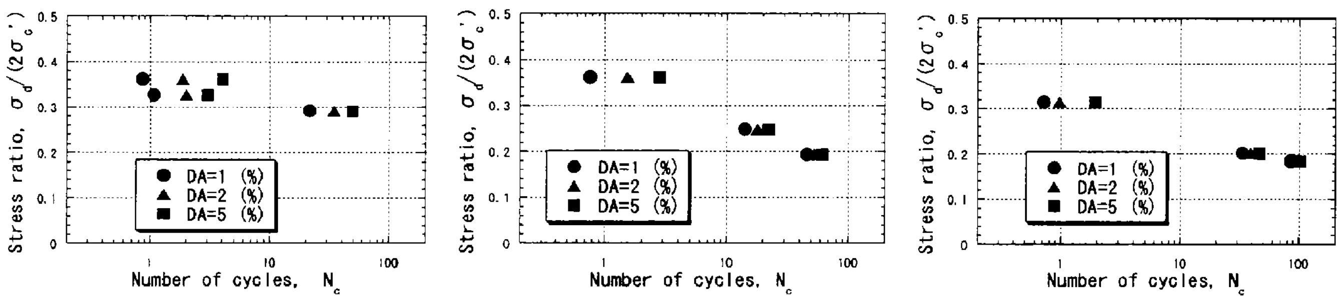
Earthquake response analyses were conducted for Site A and Site E, whose locations were shown in Fig.1. Soil profiles of Site A and Site E are shown in Fig. 4. Four kinds of earthquake motions were used for input motions of this earthquake response analysis. Two of them are NS component and EW component of the observed waves of Site A recorded at a depth of G.L. -83.4 m. The others are NS component and EW component of the waves obtained by modifying the earthquake motions recorded at New Kobe transformer substation of the Kansai Electric Power Company. The methodology of modification is described in the literature (Tanaka et al., 1999).

Methodology and models for earthquake response analysis

Table 1 Conditions and results of liquefaction tests

Layer	No. of Specimen	Depth G.L. (m)	ρ_d g/cm^3	σ'_c kPa	σ_d / $2\sigma'_c$	N_c				G_0 MPa
						DA= 1%	DA= 2%	DA= 5%	DA= 10%	
Upper	1	-7.87 ~ -8.07	1.77	98	0.362	0.86	1.86	4.04	5.96	108
	2	-8.54 ~ -8.74	1.81	98	0.291	21.5	34	49	60	127
	3	-7.37 ~ -7.57	1.88	98	0.327	1.08	2	3	3	167
Intermediate	4	-11.63 ~ -11.83	1.76	196	0.194	46	53	62	73.5	201
	5	-11.46 ~ -11.66	1.72	196	0.248	14	18	22	26	167
	6	-13.05 ~ -13.25	1.88	196	0.362	0.76	1.52	2.77	4.53	147
Lower	7	-15.57 ~ -15.77	1.62	245	0.202	33	39	46	52	127
	8	-14.26 ~ -14.46	1.63	245	0.315	0.71	0.97	1.9	2.86	176
	9	-14.50 ~ -14.70	1.70	245	0.185	84.5	92	101	109.5	274

ρ_d : Dry density, σ'_c : Effective confining pressure, G_0 : Initial shear modulus, Nc : Number of loading cycles
σ_d / $2\sigma'_c$: Stress ratio, σ_d : Axial stress amplitude, DA : Double amplitude of axial strain



(a) G.L. -7.37 m ~ -8.74 m *(b) G.L. -11.46 m ~ -13.25 m* *(c) G.L. -14.26 m ~ -15.77 m*

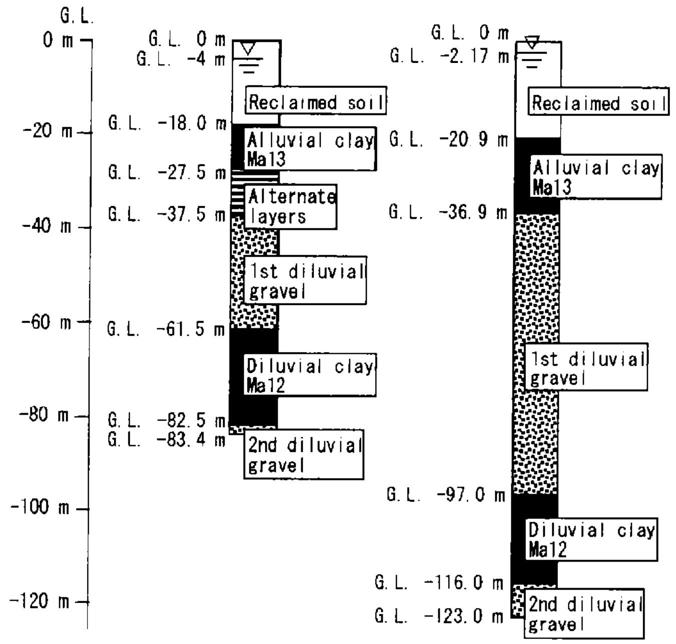
Fig.3 Results of liquefaction tests conducted for samples from Site G procured by in situ freezing technique

One-dimensional effective stress analyses were conducted. Stress-strain relationships of reclaimed soil layers at Site A and Site E and alternate layers at Site A are modeled by an effective stress model based nonlinear constitutive law(Kanatani et al., 1994). Ma13 layer of an alluvial clay deposit, which might have considerable effects on the earthquake response of the reclaimed layers, is modeled by Ramberg-Osgood model (R.O. model) and modified Hardin-Drnevich model (M.H.D model)(Tanaka et al., 1985). Both models adopt the Masing law so that they can be used for the earthquake response analysis. The M.H.D model is modified from H.D. model in order that shear strength τ_f, initial shear modulus G_0 and reference strain γ_r , which is defined as shear strain amplitude at $G/G_0 = 0.5$, can be determined independently. τ_f of Ma13 layer is determined by halving unconfined compression strength q_u. Model parameters for earthquake response analyses of Site A are determined by the data used for earthquake response analyses described in the literature(Kobe City Development Bureau, 1995). $G/G_0 \sim \gamma$ and $h \sim \gamma$ relationships and liquefaction strength of the reclaimed soil layer at Site E are determined by the results of triaxial tests of the samples from Site G procured by the in situ freezing technique in this study. Other model parameters for earthquake response analyses of Site E are determined by the data used for earthquake response analyses described in the literature(Ohta et al., 1997).

Results of analysis

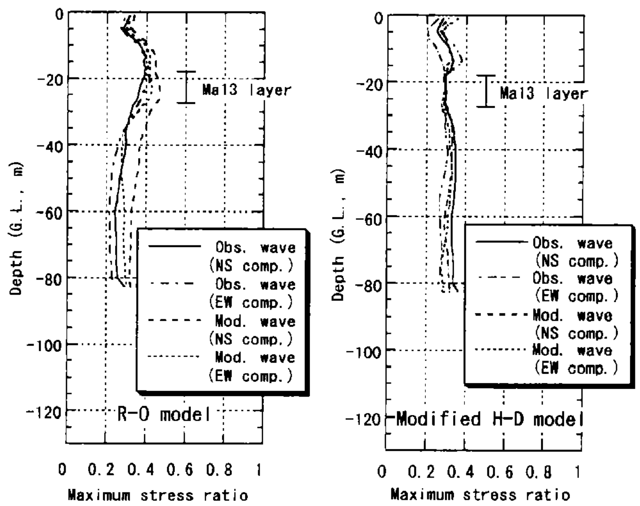
Figure 5 shows the distribution of maximum stress ratio $L_{max}(=\tau_{h,max} / \sigma'_{v0}$, $\tau_{h,max}$: maximum horizontal shear stress, σ'_{v0} : effective overburden pressure). Since the change of L_{max} in the reclaimed soil layer is not so large, L_{max} of the reclaimed soil layer equals that of Ma13 layer approximately. This fact can be clearly seen in Figs. 6(a) and 6(b). Since L_{max} of Ma13 layer never exceeds shear strength ratio τ_f / σ'_{v0}, L_{max} of the reclaimed soil layer induced by very large earthquakes, such as the 1995 Hyogoken-Nambu Earthquake, approximately equals τ_f / σ'_{v0} of the Ma13 layer. The reason that L_{max} approximately equals τ_f / σ'_{v0} of the Ma13 layer is that shear strain induced by the earthquake motion concentrates on the Ma13 layer resulting in loss of stiffness of the Ma13 layer as illustrated in Fig.7.

EVALUATION OF LIQUEFACTION POTENTIAL AT SITE A AND SITE E



(a) Site A *(b) Site E*

Fig.4 Soil profiles at Site A and Site E

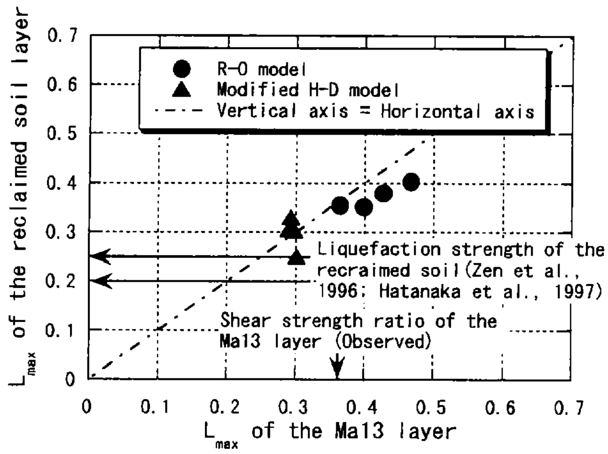


(a) R-O model *(b) Modified H-D model*

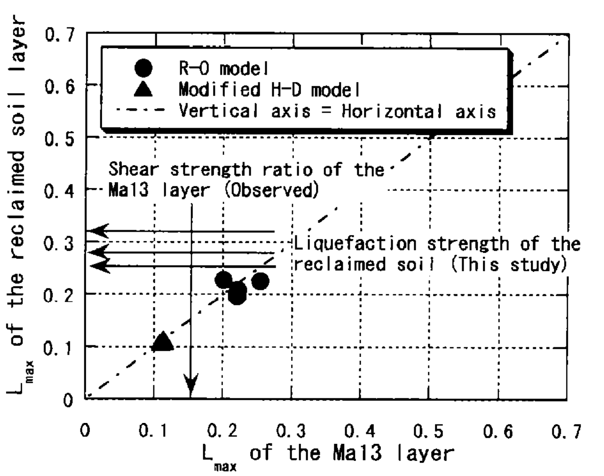
Fig.5 Distributions of maximum stress ratio, L_{max}

As shown in Fig.6(b), L_{max} of reclaimed soil layer at Site E is lower than liquefaction strengths. This fact coincides with the fact that there are a few traces of sand ejection on the ground surfaces near Site E. Contrary to this, L_{max} of reclaimed soil layer at Site A is larger than liquefaction strengths. This fact coincides with the fact that there are a lot of traces of sand ejection on the ground surfaces near Site A.

RELATIONSHIP BETWEEN SPT N-VALUE AND L_{max} IN SITES A, B, C, D, E, AND F



(a) Site A



(b) Site E

Fig.6 Relationships between L_{max} in the Ma13 soil layer and L_{max} in the reclaimed soil layer

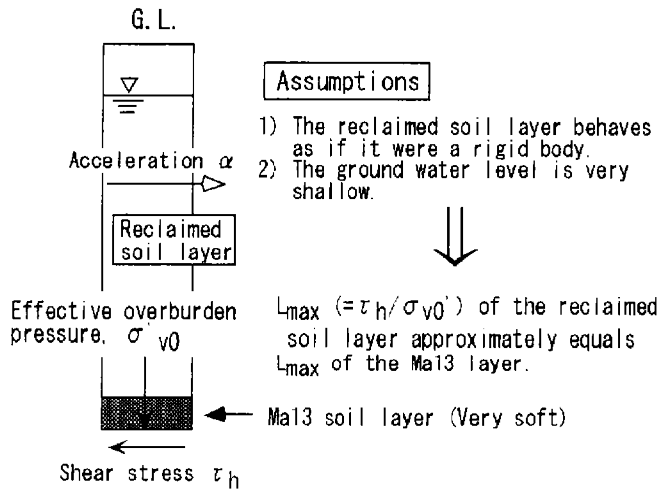


Fig.7 Effect of the Ma13 layer on the earthquake response of the reclaimed soil layer

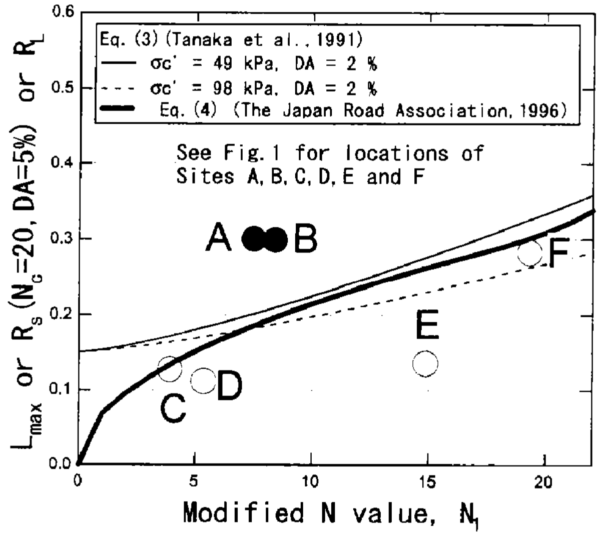


Fig.8 Relationships between N_1 and L_{max} or $R_s(N_c=20, DA=2\%)$

As shown in Fig.6, L_{max} of the Ma13 layer by the M.H.D model never exceeds τ_f / σ'_{v0} of the Ma13 layer, whereas L_{max} of the Ma13 layer by the R.O model exceeds τ_f / σ'_{v0} of the Ma13 layer. As mentioned previously, since L_{max} of Ma13 layer never exceeds shear strength ratio τ_f / σ'_{v0}, results of analyses by the M.H.D. model are more reliable than those by the R.O model. Furthermore, a closer look at the calculated results by the M.H.D. model in Figs.6(a) and 6(b) seems to reveal that the relationships between L_{max} of the reclaimed soil layer and τ_f / σ'_{v0} of the Ma13 layer can be approximated by the following equation:

$$L_{max} = \tau_{h,max}/\sigma'_{v0} = 0.85\times(\tau_f/\sigma'_{v0}) = 0.85\times(q_u/2)/\sigma'_{v0} \qquad (1)$$

Therefore L_{max} of the reclaimed soil layer was evaluated by Eq.(1) in this paper.

Solid circles and open circles in Fig.8 denote the relationships between L_{max} and N_1 of the saturated reclaimed soil layers in Sites A, B, C, D, E and F. In that case, L_{max} was calculated by Eq.(1) using unconfined compression strengths described in literature(Kobe City Development Bureau, 1995 ; Ohta et al., 1997). N_1 was modified from N in terms of σ'_{v0} using the following equation : .

$$N_1 = 1.7N/(0.7+\sigma'_{v0}/P_1) \qquad (2)$$

Where, P_1=98 (kPa)

Calculated results by the following equation , which was already proposed for gravelly soils(Tanaka et al., 1992), were also plotted in Fig.8.

$$R_s(N_c=20, DA=2\%) = 0.15+0.0059\left\{\frac{N_1}{2}\cdot\left(\frac{\sigma'_c}{P_1}\right)^{-0.5}\right\}^{1.3} \qquad (3)$$

Where, $R_s(N_c=20, DA=2\%)$: stress ratio which is required to reach 2% double axial strain amplitude, DA, in 20 loading cycles, N_c. σ'_c : Effective confining pressure (kPa)

Equation (3) was obtained by modifying the original formula by substituting modified LPT (Large Penetration Test) blow count, N_{L1}, for N_1 based on the test results reported by Yoshida et al.(1988).

On the other hand, The Japanese Road Association proposed the following equation for liquefaction assessment (The Japan Road Association, 1996).

$$R_L = 0.0882\cdot\sqrt{N_a/1.7} \qquad (N_a < 14) \qquad (4a)$$

$$R_L = 0.0882\cdot\sqrt{N_a/1.7} + 1.6\times10^{-6}\cdot(N_a-14)^{4.5} \qquad (N_a \geq 14) \qquad (4b)$$

where,

$$N_a = \{1-0.36\cdot\log_{10}(D_{50}/2)\}\cdot N_1 \qquad (5)$$

D_{50} : Mean grain size (mm)

Calculated results by Eq. (3) of σ_c' = 49, 98 kPa are plotted in Fig.8 as a thin solid line and a thin dotted line, respectively. A thick solid line denotes calculated results by Eqs. (4a) and (4b) of D_{50}=1.9 mm. The difference between the calculated results by Eq.(3) and those by Eqs. (4a) and (4b) seems small when N_1 is in the region of 5 ~ 20.

According to Fig. 1, there is a lot of ejected sand near Sites A, B, whereas there is little ejected sand near Sites C, D, E . Thus we consider Site A, B liquefied sites and Sites C, D, E nonliquefied sites. Fig.8 reveals that liquefaction assessment for Sites A, B, C, D, and E was conducted by Eqs.(3), (4a) and (4b) successfully.

Though there seems to be a lot of ejected sand near Sites F in Fig.1, Site F is reportedly improved by the rod compaction method(Geotechnical Research Collaboration Committee on the Hanshin-Awaji Earthquake, 1998). Small damage to the structure on the ground surface at Site F was reported, whereas settlement of the ground of about 30 cm was reported. Thus, we can think that serious liquefaction did not occur at Site F, though excess pore water pressure generated to some extent. In Fig.8, an open circle which denotes Site F is plotted among calculated results by Eqs.(3), (4a) and (4b). This coincides with the phenomena which was observed at Site F.

Therefore, it can be said that liquefaction behavior of the reclaimed soil layers in Port Island and Rokko Island can be assessed by Eqs.(3), (4a) and (4b) successfully.

CONCLUSIONS

The following conclusions were drawn through this study. (1) Maximum shear stress ratio of a liquefied site during 1995 Hyogoken-Nambu Earthquake is larger than liquefaction strength of the liquefied site, whereas maximum shear stress ratio of a nonliquefied site is smaller than liquefaction strength of the nonliquefied site. (2) The relationship between the maximum shear stress ratio and modified SPT N-value, N_1 of the boundary between liquefied sites and nonliquefied sites can be approximated by existing equations by which liquefaction strength of gravelly soils can be evaluated.

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