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ABSTRACT

It is important to consider the non-linear behavior of the soil in evaluating the seismic behavior of the ground during the large ground motion. Pore water pressures, in the order of 75% of the initial mean confining pressures, were observed at the liquefaction observation sites near the Lake Utonai in Hokkaido, Japan during the 1993 Kushiro-oki earthquake. In the current study, effective stress analysis and total stress non-linear analysis were carried out incorporating both strain-dependent non-linearity and non-linear built-up of pore pressures. The following conclusions were reached: (1) Seismic behavior of the ground, acceleration of the surface ground, transfer functions etc, obtained from the effective analysis were sufficient to predict the observed records; (2) It was found from these analyses that shear strain was reached to 1 or $2x10^{-3}$ and pore water pressure ratio was built up to between 0.2 and 0.4 during the earthquake; (3) The amplitude and phase of the acceleration at the ground surface by effective and total stress analyses agreed well; and (4) The influence of the excess pore water pressure on the seismic behavior of the ground surface is not so significant when the excess pore water pressure ratio was less than 0.4 in general.

INTRODUCTION

It is important to consider the non-linear behavior of the soil in evaluating the seismic behavior of the ground during the large ground motion. Particularly in the case of loose saturated sand layers present at the site, it is necessary to consider the non-linear effects due to both shear strain-dependent soil modulus and the non-linearity caused by built-up of pore water pressures due to cyclic loading conditions (Mori et al., 1993). To evaluate the amplification of ground including the effect of excess pore water pressure, so-called liquefaction vertical array observations, in which piezometers were installed at the same site in addition to the vertical strong motion array have been conducted at several sites (e.g., Ishihara et al. 1981, Ishihara et al. 1989, Yanagisawa et al., 1986). Many kinds of studies were conducted by using observed records. However, there have only been two sites at which records of excess pore water pressure building up to the initial mean confining pressure were obtained (Holzer et al., 1989, Shen et al., 1991). Therefore, liquefaction array observation should be continued to obtain another large ground motion with high pore water pressure for evaluating ground behavior during such condition. The authors carried out the liquefaction array observation at the soft ground near Lake Utonai in Tomakomai, Hokkaido, the northernmost island of Japan, to evaluate the seismic behavior of the ground and effect of sand compaction pile method for protecting the highway road embankment from liquefaction (Nishikawa et al. 1994, Hayashi et al. 1998, Hayashi et al. 2000). In this region, liquefaction was confirmed to have occurred during 1968 Tokachi-oki earthquake (Wakamatsu et al., 1991), and 1982 Urakawa-oki earthquake (Saito et al., 1986). The Kushiro-oki Earthquake on January 15, 1993, caused extensive damage in eastern

Hokkaido, as well as a number of liquefaction incidents (Mori, 1993). The Port and Harbor Research Institute of the Ministry of Transport obtained a record of acceleration, which is considered to have been affected by the liquefaction in a wide sense of the word, at the Kushiro West Port directly above the hypocenter (Iai et al., 1995). At the array observation sites near Lake Utonai as well, not only acceleration records but also valuable records of excess pore pressure were obtained (Odajima et al. 1993), though there was no serious damage around Tomakomai. The pore water pressure ratio was built up to 75% of the initial mean confining pressure. In the current study, effective stress analysis and total stress non-linear analysis were carried out incorporating both strain-dependent non-linearity and non-linear built-up of pore pressures, and compared with the observed records to evaluate the effect of excess pore water pressure on the seismic amplification of the ground.

OBSERVATION SITE AND OBSERVED RECORDS

Figure 1 shows the locations of the sites, and Figure 2 shows an NW-SE geological profile across the observation sites. To the west (Left) of Fig. 2 is a plateau of volcanic ash from the Shikotsu Volcano. The western half of this figure therefore mainly consists of secondary deposits of volcanic ash and pumice eroded and transported from this plateau. The eastern half comprises beach sand and cohesive soil from the hinterland swamp. These volcanic ash (upper) and sand strata are easily liquefiable because they are loose or medium loose. Actually, liquefaction was confirmed to have occurred in this region during 1968 Tokachi-oki earthquake (Wakamatsu et al., 1991), and 1982 Urakawa-oki earthquake (Saito et al., 1986). At the level of G.L.-30 to -32 m is a surface of widespread gravel bed. Because of these soil conditions, the ground was improved by means of the sand compaction pile method. With this as a background, liquefaction array observation has been conducted at Site A in the volcanic ash ground in the west since 1991 and at Site B in the sand ground in the east since 1990, to investigate the liquefaction characteristics of ground and the effectiveness of the countermeasures against liquefaction. In this study, we focused on sand site (Site B), where the record obtained during the 1993 Kushiro-oki earthquake. Figure 3 shows the soil profile and the instrumentation at sand site. The surface layer is filled with volcanic ash. Between G.L.-3 m and G.L.-9 m are the strata of peat and volcanic ash, under which lies a loose sand layer between G.L.-9 m and G.L.-15 m. A silty sand layer appears under the loose sand, followed by strata of medium loose sand and sandy gravel layer. Dense sand or gravel layer, which has high shear wave velocity more than 400 m/s appears at G.L.-30 m. Around the site, surface layers are assumed to be horizontally stratified.

The accelerometers are buried at three levels: G.L.-2 m, -17 m, and -35 m in unimproved ground. At G.L.-2m and G.L.-35m, One accelerometer has three components: one vertical and two horizontal. That at the intermediate depths G.L.-17m has only horizontal components. The accelerometer at G.L.-35 m is located in the dense sandy gravel bed, which can be

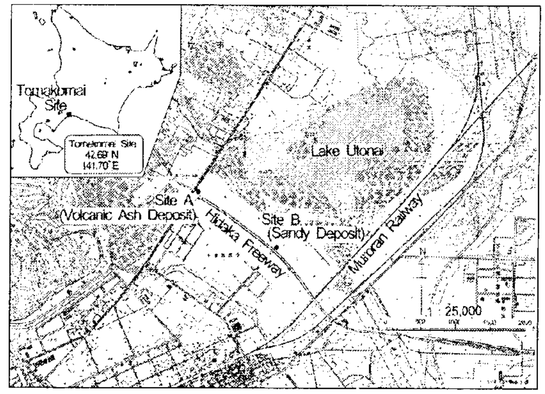


Fig. 1. Location of liquefaction array observation site.

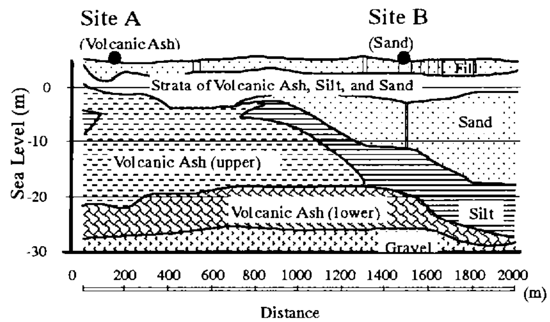
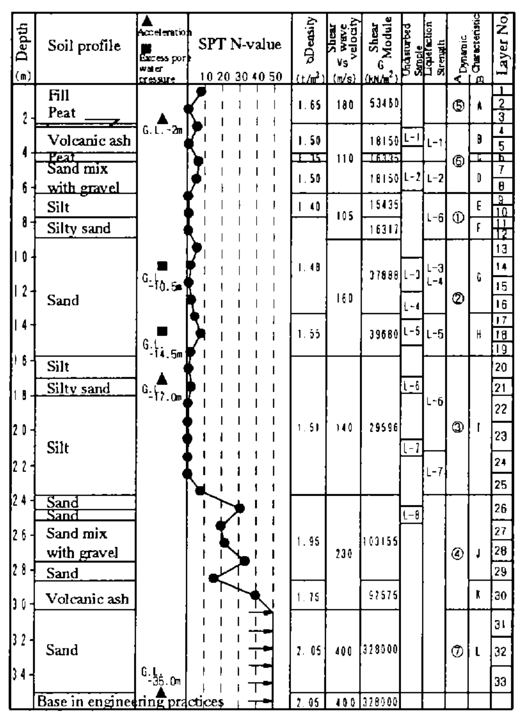


Fig. 2. Cross section of NW-SE around observation sites.



A: Dynamic Characteristics by test
B: Dynamic Characteristics for analysis

Fig. 3. Soil profile and instrumentation at Site B.

regarded as the base in engineering practice. Piezometers are buried at two depths in loose sand layer of between G.L.-9m and G.L.-15.7m where liquefaction is likely. They are buried at the depths of G.L.-10.5 m and -14.5 m, both in the improved and in the unimproved ground for evaluating the effect of improvement. Because orientation errors of accelerometer should be an important problem in discussing the amplification of the ground by bore hole array records, the orientation error was examined and corrected (Ikeda et al., 1998). In this study, the corrected records were used. The performance of the accelerometer and piezometer, system of the array observation, and method of placing the piezometer had been described in detail by Nishikawa (Nishikawa et al., 1994). The effect of the road embankment on the seismic behavior of the ground at the liquefaction array observation sites was small, as revealed by Hayashi (Hayashi et al., 1998). Records of nine earthquakes have been observed at the site, in

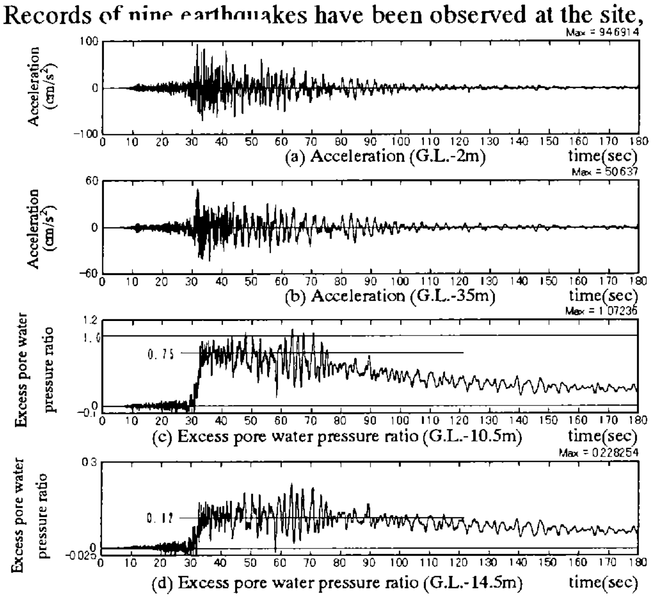


Fig. 4. Observed time histories of 1993 Kushiro-oki earthquake.

which the maximum acceleration records were observed during the 1993 Kushiro-oki earthquake. Table 1 shows the observed earthquakes and maximum acceleration of each record. During the 1993 Kushiro-oki earthquake, maximum accelerations between $50 cm/s^2$ and $70 cm/s^2$ were observed at the base, and almost $100 cm/s^2$ at ground surface (G.L.-2 m). Unfortunately, no records were obtained at site A (volcanic ash ground).

Figure 4 shows the acceleration time history at the depth of G.L.-2 m and G.L.-35 m in N042E direction, which is normal to the road embankment. Also, time histories of the excess pore water pressure ratio (the excess pore water pressure divided by the initial effective mean confining pressure) at the depth of G.L.-10.5m and G.L.-14.5m were shown in Fig. 4. Maximum acceleration was doubled between G.L.-35 m and G.L.-2 m, the values being $50 cm/s^2$ and $94 cm/s^2$ respectively. The excess pore water pressure ratio at the depth of G.L.-10.5 m built up to more than 1 in maximum value and approximately 0.75 excluding the vibration component, which was remarkable rise, though full liquefaction did not occur. On the other hand, excess pore water pressure ratio at the depth of G.L.-14.5 m was relatively small at about 0.12, which was less than that of G.L.-10.5 m. In the waveforms of acceleration time history at G.L.-2 m, the extension of predominant period, which should be appear as a result of large nonlinearity in the ground, or spike-shaped waves, which represents the cyclic mobility, were not found clearly. These suggests that the ground did not reach the state of full liquefaction, and these observation agrees with the fact that no traces of liquefaction, such as sand boils, were found in the reconnaissance just after the quake. In the current study, the influence of the nonlinearity depend on shear strain and the influence of the nonlinearity depend on excess pore water pressure were examined quantitatively by using effective stress analysis and total stress non-linear analysis.

Table. 1. Profiles of earthquake.

	Profile of earthquakes					Profiles of observed records											
	Earthquake	Mj 1)	Epi-center	De-pth	JMA Seismic Intensity (Upper: Tomakomai / Lower: Maximum)	Hipocentral distance / Epicentral distance (km)	Site (A/B)	Start time	Duration time (s)	Maximum acceleration (cm/sec^2) G.L.-2m NS	G.L.-2m EW	G.L.-2m UD	G.L.-17m NS	G.L.-17m EW	G.L.-35m NS	G.L.-35m EW	G.L.-35m UD
1	Urakawa-oki	6.4	42°00'	67	III	118.0	Volcanic ash	91.11.27 04:41:48.73	217	11.6	13.1	4.4	6.9	9.7	6.0	8.8	3.9
	1991.11.27 04:40		142°42'		IV: Urakawa,Hiroo	97.1	Sand	91.11.27 04:41:26.31	249	13.6	17.6	8.4	9.0	11.3	7.8	9.1	3.3
2	Kushiro-oki	7.8	42°51'	107	IV	234.5	Volcanic ash	Trouble of machine	-	-	-	-	-	-	-	-	-
	1993.01.15 20:06		144°23'		VI: Kushiro	208.7	Sand	93.01.15 20:06:47.87	309	100.3	103.9	46.6	68.0	66.7	63.2	53.2	25.0
3	Hokkaido-nansei-oki	7.8	42°51'	34	IV	216.0	Volcanic ash	93.07.12 22:18:11.46	283	15.0	13.8	5.9	11.9	12.7	10.5	10.7	4.4
	1993.07.12 22:17		139°12'		V: Esashi,Otaru,Fukaura	213.3	Sand	93.07.12 22:18:05.25	294	16.7	20.8	6.7	13.4	15.8	10.4	12.7	5.2
4	Hokkaido-nansei-oki	6.5	41°57'	26	III	220.0	Volcanic ash	93.08.08 04:43:49.01	193	8.7	8.9	2.5	5.1	5.7	3.8	4.4	2.3
	1993.08.08 04:42		139°53'		V: Okushiri	218.5	Sand	93.08.08 04:43:41.52	203	7.7	12.3	2.7	4.4	7.7	3.7	3.5	2.2
5	Tomakomai-oki	5.5	41°44'	79	IV	160.2	Volcanic ash	93.12.04 18:30:52.76	195	14.4	9.9	7.9	7.5	7.2	4.4	5.1	2.8
	1993.12.04 18:30		141°59'		IV: Tomakomai,Mutsu	139.4	Sand	93.12.04 18:30:49.47	204	23.4 4)	- 2)	5.1	10.7	8.7	7.6	8.1	2.5
6	Hokkaido-toho-oki	8.1	43°22'	30	IV	469.8	Volcanic ash	94.10.04 22:24:29.88	343	44.3	51.2	24.8	29.5	23.5	20.3	20.9	11.2
	1994.10.04 22:23		147°40'		VI: Kushiro,Akkeshi	468.9	Sand	94.10.04 22:24:21.31	356	86.8	79.8	36.3	43.8	39.7	34.7 3)	- 2)	16.0
7	Sanriku-haruka-oki	7.5	40°27'	0	IV	305.4	Volcanic ash	94.12.28 21:21:11.90	305	61.4	61.2	10.0	32.5	30.8	27.9	21.6	7.8
	1994.12.28 21:19		143°43'		VI: Hachinohe	305.4	Sand	94.12.28 21:21:10.74	296	61.6	51.3	15.5	39.1	36.4	28.0 3)	- 2)	7.7
8	Iwateken-oki	6.9	40°18'	30	III	284.3	Volcanic ash	95.01.07 07:38:53.33	258	15.9	23.6	4.9	10.9	13.2	8.2	11.1	3.8
	1995.01.07 07:37		142°24'		V: Hachinohe,Morioka	282.7	Sand	95.01.07 07:38:54.18	260	16.8 4)	- 2)	6.0	11.8	13.8	10.1 3)	- 2)	3.5
9	Urakawa-oki	5.6	41°45'	45	III	149.1	Volcanic ash	Below trigger level	-	-	-	-	-	-	-	-	-
	1997.02.20 16:55		142°52'		V: Urakawa	142.1	Sand	97.02.20 16:55:45.58	196	6.1	5.7	3.1	5.3	4.0	3.1	2.7	1.5

1) Mj: magnitude of JMA , 2) Troubleof machine, 3) N026E, 4) N340E

CONDITION OF THE NONLINEAR ANALYSIS

Methods for analysis

The authors conducted effective stress analyses using two methods. One is a strain space plasticity model for cyclic mobility based on a multiple shear mechanism defined in a strain space (analysis code: FLIP) (Iai et al. 1992). The other is a model based on experienced effective stress path model of Ishihara and Towhata (analysis code: YUSAYUSA) (Ishihara et al. 1981). In both cases, analysis was carried out with and without the excess pore water pressure rise (effective stress analysis and total stress non-linear analysis, respectively). The relationship on the nonlinearity of shear stress and shear strain was modeled into a modified Hardin-Drnevich model, in which skeleton curve was modeled into Hardin-Drnevich model, and the Masing rule was applied to the hysteresis rule. In the code of FLIP, the Masing rule was modified to permit control of the hysteresis loop size.

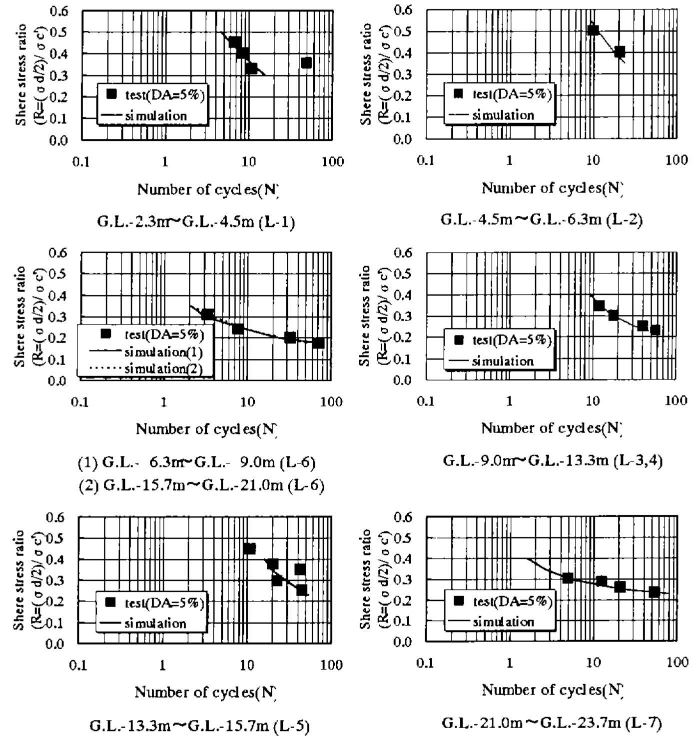
Model for analysis

Because the ground around the site was horizontally stratified and the road embankment near the liquefaction array observation site barely affected the seismic behavior of the ground at the site, the ground was modeled into a one-dimensional model. Figure 3 shows the model of the ground. The model parameters for the analysis were determined by referring to past tests on samples from the site. The strong motion records at G.L.-35m were rotated to N042E, which was normal to the road embankment and were used for the input motions as the total of incident/reflection wave (E+F). Figure 4 shows the input motion.

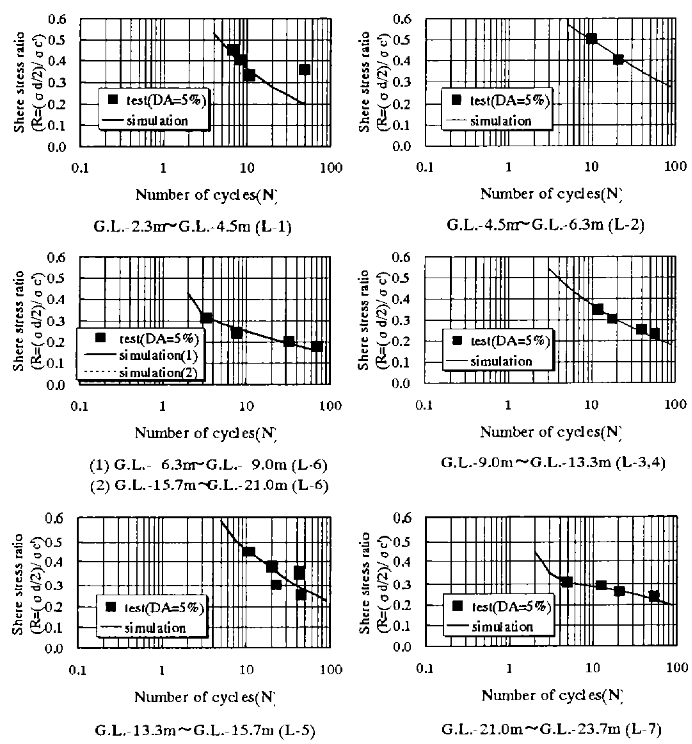
Liquefaction strength

Because the layers between G.L.-2.3m and G.L. -23.7m below water level were mainly consist of sand, though with certain amount of fines, these layers were thought to be liquefiable.

Liquefaction strengths were basically determined by cyclic triaxial tests on so-called undisturbed samples taken by triple-tube sampling from each layer. The cyclic stress ratios, i.e., the strength required to cause 5% double amplitude axial strain in 20 cycles, obtained by the tests are shown in Figure 5. The parameters of liquefaction strength of soil were determined to fit the liquefaction strength obtained by the elemental simulation analysis of simple shear testing using "FLIP" or "YUSAYUSA" to that of the soil obtained by laboratory tests. The liquefaction resistance curve calculated from the simulation analysis is superimposed on Figure 5. The liquefaction resistance curve agrees well with the liquefaction strength obtained by the cyclic triaxial tests with the validity of the parameter setting being considered. However, because it was reported that liquefaction strength of sand from the so-called undisturbed samples by triple tube sampling are



(a) Element simulation by FLIP



(b) Element simulation by YUSAYUSA

Fig. 5. Liquefaction strength and computed liquefaction resistance curves.

severely affected by its disturbance, there is still room for discussion as to how the liquefaction strength should be set. Table 2 shows the liquefaction parameters for the effective stress analysis.

Nonlinear characteristics

The nonlinear characteristics of maximum shear stress (τmax) and maximum damping factor (hmax: only for FLIP) were evaluated by referring to cyclic triaxial tests, triaxial test in CD condition, and past test results. Maximum shear stresses (τmax) for the layer for which the build-up of the excess pore water pressure was considered were determined by the angle of internal friction by CD testing. The maximum shear stresses (τmax) for the layer for which the build-up of the excess pore water pressure was not considered were assumed to be a parameter of the relationship between the shear stress and the shear strain, and set to fit the G/G_0-γ relationship of the Modified Hardin-Drnevich model to that obtained by the laboratory test. The maximum damping factor (hmax) for FLIP was determined at 0.24 by referring the past researches. In the analysis code of YUSAYUSA, the maximum damping factors were defined as $2/\pi$, because the Modified Hardin-Drnevich model was used. In other words, the damping factors of YUSAYUSA in this study were set at 2.6 times as large as those of FLIP. It is thought that the influence of the damping factors on the response is strong at high strain levels. Figure 6 (a) shows the nonlinear characteristics by cyclic triaxial tests and Figures 6 (b) and (c) show the nonlinear characteristics defined for the analysis.

COMPARISON BETWEEN RESULTS OF ANALYSIS AND OBSERVED RECORDS

Maximum responses

Figure 7 shows the maximum response distribution of acceleration, relative displacement, shear stress, shear strain, and excess pore water pressure obtained from the analysis compared with the observed records. Also, the results of equivalent linear analysis previously conducted (Hayashi et al., 1998) are shown in Fig. 7 for comparison. The relative displacements of observed record were obtained by integration from the acceleration.

The accelerations of effective stress analyses agreed well with the observed records. The displacement and shear strain were larger by FLIP and smaller by YUSAYUSA. This may be because the damping factor of FLIP was small and that of YUSAYUSA was large. However, shear stresses were almost same in both analyses. There were no significant difference between effective stress analysis and total stress analysis in maximum responses.

Regarding the excess pore water pressure, though results of analysis agreed with that of observed at G.L.-14.5m, the calculations were smaller than the measurements at G.L.-10.5m. The excess pore water pressure of the silty layer between G.L.-15.7 and G.L.-21m was the largest by both

Table. 2. Model parameters for analysis.

a) Parameters

Symbol	Type of mechanism	Parameter designation
K_a	elastic volumetric	rebound modulus
G_{ma}	elastic shear	shear modulus
ϕ_f	plastic shear	shear resistance angle
ϕ_p	plastic dilatancy	phase transformation angle
H_m	plastic shear	upper bound for hysteretic damping factor
σ'_{m0}		initial mean effective confining pressure
p_1	plastic dilatancy	initial phase of cumulative dilatancy
p_2	plastic dilatancy	final phase of cumulative dilatancy
w_1	plastic dilatancy	overall cumulative dilatancy
S_1	plastic dilatancy	ultimate limit of dilatancy
c_1	plastic dilatancy	threshold limit for dilatancy
θ_s		phase transformation angle
Bp		parameter of layers below the ground water surface
Bu		parameter of layers below the ground water surface
κ		parameter to determine the lower limit of cyclic stress ratio
$\sigma v'_0$		initial effective overburden pressure
$\sigma v'_{min}$		minimum overburden pressure

b) Model parameters for FLIP

	$\sigma m_0'$ (kN/m^2)	Gm_0 (kN/m^2)	Km_0 (kN/m^2)	ρ (kN/m^3)	hmax	ϕf (deg.)	$\phi p'$ (deg.)	s1	w1	p1	p2	c1
Fill	13.9	53500	285337	1.646	0.24	90						
Peat	31.0	18200	96873	1.274	0.24	37	28	0.005	25.0	0.5	1.00	1.00
Volcanic ash	34.8	16300	87181	1.352	0.24	37	28	0.005	28.0	0.5	1.00	1.00
Sand mix with gravel	38.7	18200	96873	1.499	0.24	37	28	0.005	55.0	0.5	1.00	2.00
Silt	44.1	15400	82379	1.401	0.24	37	28	0.005	5.0	1.2	1.00	1.75
Silt	48.5	16300	87093	1.480	0.24	37	28	0.005	5.0	1.2	1.00	1.75
Sand	58.3	37900	202223	1.480	0.24	37	28	0.005	21.0	0.5	0.95	2.30
Sand	70.8	39700	211788	1.548	0.24	37	28	0.005	32.5	0.5	0.95	2.30
Silt	85.6	29600	206319	1.509	0.24	37	28	0.005	6.5	0.5	1.02	2.05
Silt	100.5	29600	157966	1.509	0.24	37	28	0.005	8.0	0.8	1.00	2.50
Sand	122.7	103200	550574	1.950	0.24	90	28					
Volcanic ash	144.5	92600	494106	1.754	0.24	70	28					
Sand	167.3	328000	1750643	2.048	0.24	90	28					

c) Model parameters for YUSAYUSA

	$\sigma v'_0$ (kN/m^2)	Gm_0 (kN/m^2)	mv (kN/m^2)	ρ (kN/m^3)	θs (deg.)	Bp	Bu	κ	$\sigma v'_{min}$
Fill	18.600	53500	0.0	1.646					
Peat	41.356	18200	0.0	1.274	23.7	0.3	0.5	0.06	0.03
Volcanic ash	46.383	16300	0.0	1.352	23.7	0.3	0.5	0.06	0.03
Sand mix with gravel	51.646	18200	0.0	1.499	23.7	0.5	0.1	0.06	0.03
Silt	58.800	15400	0.0	1.401	23.7	5.0	0.3	0.06	0.03
Silt	64.602	16300	0.0	1.480	23.7	5.0	0.3	0.06	0.03
Sand	77.773	37900	0.0	1.480	23.7	1.0	0.3	0.06	0.03
Sand	94.354	39700	0.0	1.548	23.7	0.15	0.2	0.06	0.03
Silt	114.072	29600	0.0	1.509	23.7	5.0	0.3	0.06	0.03
Silt	134.064	29600	0.0	1.509	23.7	4.5	0.1	0.06	0.03
Sand	163.278	103200	0.0	1.950					
Volcanic ash	192.678	92600	0.0	1.754					
Sand	223.107	328000	0.0	2.048					

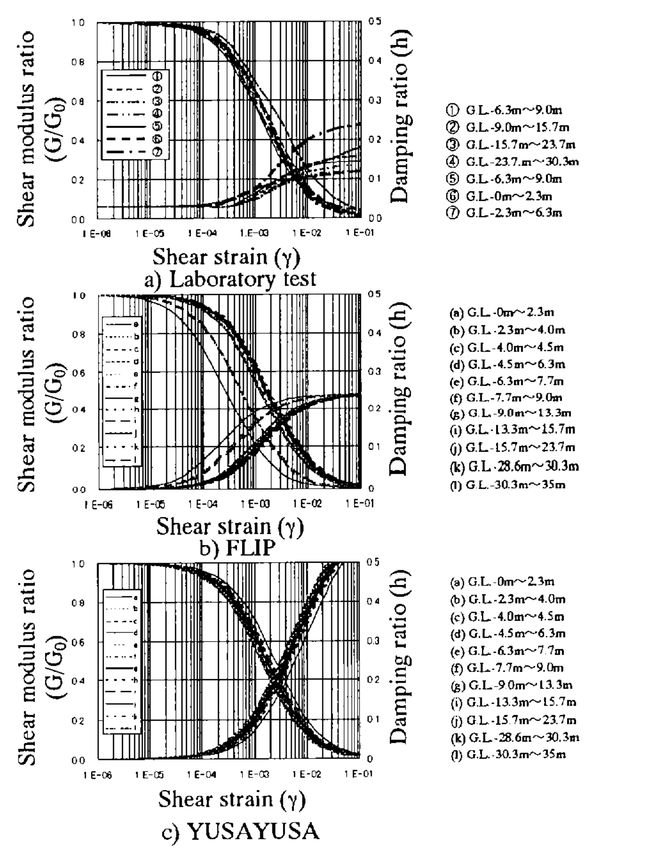


Fig. 6. Characteristics for dynamic deformation.

analysis methods. These ratios were between 0.21 and 0.25 by FLIP, and slightly larger, between 0.36 and 0.40, by YUSAYUSA. The averaged value for the liquefiable layer evaluated by equation (1) was 0.1 in FLIP and 0.25 in YUSAYUSA.

$$PWPR_{ave} = \Sigma\ (Hi \times PWPRi)\ /\ \Sigma Hi \qquad (1)$$

where

$PWPR_{ave}$:Averaged excess pore water pressure ratio
PWPRi :Excess pore water pressure ratio of Layer i
Hi :Thickness of Layer i

Because of agreement of analysis and observation, it is considered that results of analysis explain the seismic behavior of the ground. The shear strain was estimated to reach to 1 or 2 x 10^{-3} and excess pore water pressure ratio was built up to between 0.2 and 0.4, approximately 0.25 on average in liquefiable layers, during the 1993 Kushiro-oki earthquake from these analysis.

Comparing the equivalent linear analysis and nonlinear analysis, no marked differences are observed when the excess pore water pressure ratio is less than 0.4 in general. In detailed comparison, however, the equivalent linear analysis evaluated the maximum acceleration and maximum shear stress larger, and the displacement and shear strain, smaller. It is conservative for stress analysis, but care should be exercised, as it can be on the risky side for strain analysis.

Amplification and time histories

Figure 8 shows the transfer functions between the input motion at the base and G.L.-2m obtained from the analyses compared with that of observed records. The transfer function was smoothed by a Parzen window with bandwidth of 0.2Hz. Not only the fundamental mode, but also the second and the third mode agreed with observed one in both frequency and amplitude by FLIP. On the contrary, though the frequencies agreed in these modes, the amplitude was low by YUSAYUSA. This may be because the damping factor was larger in YUSAYSA. There were no significant difference between the results of effective stress analysis and the total stress nonlinear analysis, though the amplitude was slightly different.

Figure 9 (a) shows the time histories of the acceleration at G.L.-2m obtained from the analyses, compared with that of the observed one. The results of both methods of FLIP and YUSAYUSA agreed well with the measurements. In detail, the phase of the effective stress analysis was delayed compared with that by the total stress analysis. It is considered that the shear modulus was reduced more by the build-up of the pore water pressure in the effective stress analysis. The amplitude by FLIP is larger and that by YUSAYUSA was smaller than the observed value, because of the difference in the damping factor.

Figure 9 (b) shows the analyzed time histories of the excess pore water pressure at G.L.-10.5m and 14.5m compared with the measurements. Excepting the vibration component during the rise of pore water pressure caused by the vertical motion (Mori et al., 1996), and the influence of surface wave in later phases, the results of both methods of FLIP and YUSAYUSA agreed well with the observed values of the in rising time of pore pressure and level of build-up. On the contrary, the level of the pore water pressure build-up was small at G.L.-10.5m compared with the observed value. It is considered that the estimations of liquefaction strength in small areas is an important problem for detailed analysis. There were differences in the processes of pore water pressure build-up by the two analysis methods, because of the difference of the models for evaluation on pore water pressure.

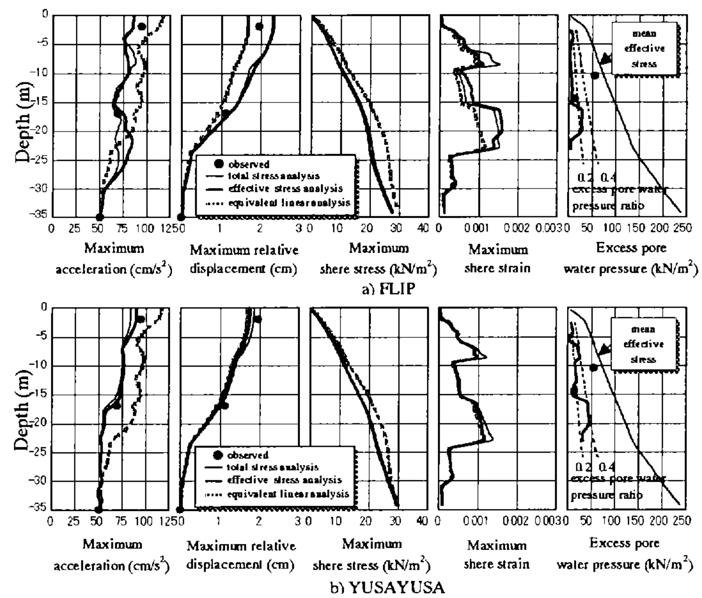


Fig. 7. Distribution of maximum responses.

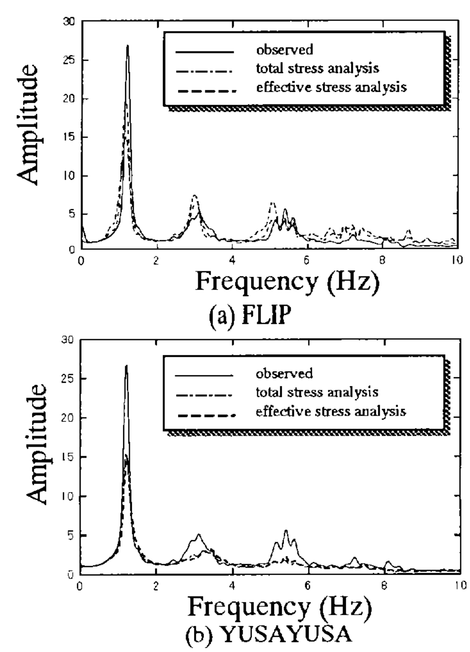


Fig. 8. Comparison of the transfer function between observed record and analysis.

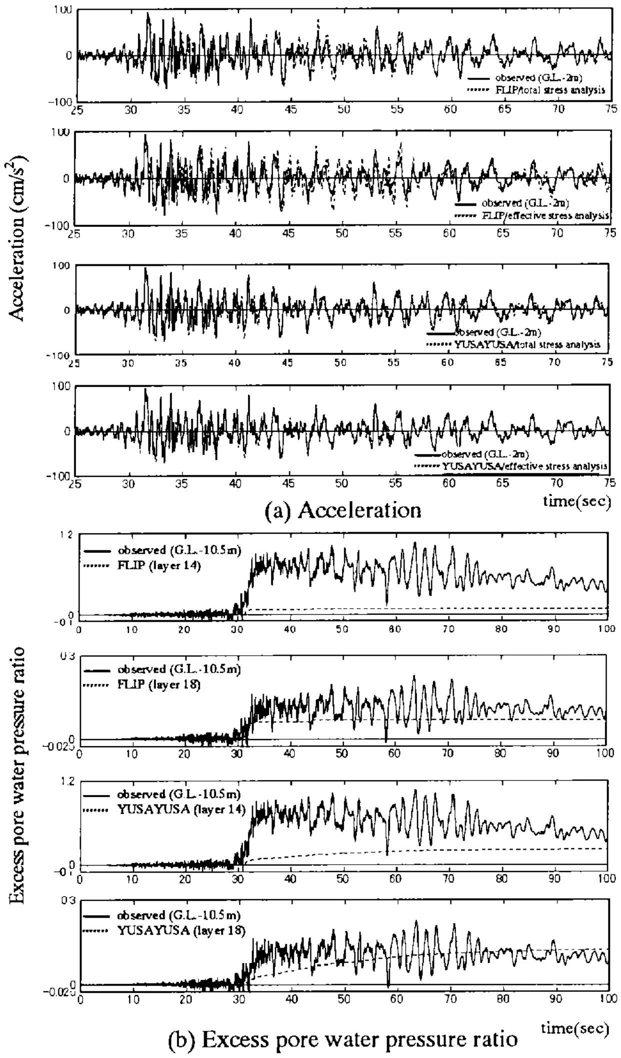


Fig. 9. Comparison of the time histories between observed record and analysis.

General comparison

Seismic behavior of the ground, such as maximum response distribution, acceleration time histories of the surface ground, and transfer functions between the base and the surface of the ground, obtained from the effective analyses agreed well with the observed records, though the excess pore water pressure at G.L.-10.5m was 1/6 to 1/9 lower than the observed records. Therefore, the effective stress methods are thought to be sufficiently effective for seismic response analysis. Because analyses results agreed well with the observed records, and the excess pore water pressure ratio from the analysis built up to between 0.2 and 0.4, approximately 0.25 on average, which was not so large, it is supposed that the high rise of the excess pore water pressure at G.L.-10.5m a localized value, and that only the pore pressure of very loose sand at the upper part of the sand layer under low confining pressure rose high.

It is thought that the influence of the excess pore water pressure on the seismic behavior of the ground surface is not so significant when the excess pore water pressure ratio is less than 0.4 in general. The total stress analysis methods are thought to be also sufficiently effective for seismic response analysis when the excess pore water pressure ratio is less than 0.4 in general. .

Comparing the results of two analysis methods (FLIP and YUSAYUSA), the distribution of the shear stress and time history of the ground surface coincided well with each other. Regarding the transfer function between the base and the ground surface, the amplitude by FLIP agreed well including the high predominant frequencies. However, that by YUSAYUSA was estimated lower, which is presumably due to the difference in the damping factors. In line with the fact, the maximum strain was larger by FLIP. The amplitude of the acceleration time history of FLIP was larger than that of observed record, on the contrary, that of YUSAYUSA was smaller than that of the observed value. The phase by FLIP was slightly delayed, though that by YUSAYUSA gained from FLIP and agreed with the observed one. Concerning the pore pressure, because of the difference of the model, the properties of the excess pore water pressure build-up were different.

CONCLUSION

The following conclusions were reached:

1) The seismic behavior of the ground, such as maximum response distribution, acceleration time histories of the surface ground and transfer functions between the base and surface of the ground, obtained from the effective analyses agreed well with the observed records, though the excess pore water pressure at G.L.-10.5m was 1/6 to 1/9 lower than the observed records. The effective stress methods are thought to be sufficiently effective for seismic response analysis.

2) These analyses revealed that the shear strain reached 1 or 2 x 10^{-3} and that the excess pore water pressure ratio built up to between 0.2 and 0.4, approximately 0.25 on average, in liquefiable layers during the 1993 Kushiro-oki earthquake. Because of the agreement of analyses results and observed records, it is supposed that the high rise of excess pore water pressure at G.L.-10.5m was localized and limited to the pore pressure of very loose sand at the upper part of the sand layer under low confining pressure.

3) The amplitude and phase of the accelerations at the ground surface by effective and total stress analyses agreed well. Though differences were found in the distribution of maximum acceleration and shear strain and the delay of phase in acceleration time history, the differences were marginal. It is thought that the influence of the excess pore water pressure on the seismic behavior of the ground surface is not so significant when the excess pore water pressure ratio is less than 0.4 in general.

4) Comparing the results of two analysis methods, the distribution of shear stress and time history of ground surface coincided well with each other. However, regarding the transfer function between the base and the ground surface, the amplitude by FLIP agreed well including high predominant frequencies, whereas that by YUSAYUSA was lower, presumably due to the difference in the damping factor. In line

with the fact, the maximum strain was larger by FLIP. The amplitude of acceleration time history of FLIP was larger than that of the observed record, but that of YUSAYUSA was smaller. The phase by FLIP was slightly delayed, though that by YUSAYUSA gained from FLIP and agreed with the observed phase. Concerning the pore pressure, because of the difference of the model, the performance of the excess pore water pressure build-up was different.

5) Comparing the equivalent linear analysis and nonlinear analysis, no appreciable differences are observed between the responses when the excess pore water pressure ratio is less than 0.4 in general. In detailed comparison, however, the maximum acceleration and maximum shear stress are evaluated larger, and the displacement and shear strain are smaller by the equivalent linear analysis. It is conservative for stress analysis, but care should be exercised for strain analysis, as the result can be on the risky side.

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