GROUND SEISMIC BEHAVIOR FROM LIQUEFACTION ARRAY OBSERVATION

Hirochika HAYASHI[1], Jun'ichi NISHIKAWA[2], Takuya EGAWA[3], Shigeru MIWA[4] And Takaaki IKEDA[5]

SUMMARY

Liquefaction array observations to measure ground acceleration and pore water pressure during earthquakes simultaneously have been conducted on soft ground on the south side of Lake Utonai, Hokkaido, Japan. The observation sites are adjacent to a road embankment. It was therefore believed that the observation records might be affected by the road embankment.

The effects of the road embankment were examined through one-dimensional equivalent linear analysis and two-dimensional equivalent linear FEM analysis. As a result, it was found that the dynamic characteristics of the ground are not affected by the existence of the road embankment. The above results almost reproduced the results of an observation in which non-linear characteristics, such as 10^{-3} of maximum shear strain, appeared when shear modulus decreased to 75%. It was thus revealed that equivalent linear analysis is effective in elucidating the seismic behavior of such ground.

INTRODUCTION

The ground on the south side of Lake Utonai, Hokkaido, Japan, consists of loose secondary volcanic ash and sand deposits, whose N value is 10 or less at a G.L. (ground level) of -15 to -20 m. Liquefaction occurred in this area during the Tokachi-Oki Earthquake in 1968 and Urakawa-Oki Earthquake in 1982 [Wakamatsu 1991, Saito 1986]. Liquefaction array observations have therefore been conducted at two sites in this area to measure the ground acceleration and pore water pressure during earthquakes simultaneously [Hayashi 1991, Odajima et al. 1992, Nishikawa et al. 1994]. The two observation sites are the section with a dominant sand layer and the section with a dominant volcanic ash layer in the ground. Measurements for nine earthquakes have been collected since the start of observation (volcanic ash ground: 1991, sandy ground: 1990). Ground seismic behavior was examined in past studies [Hayashi et al. 1997, Taniguchi 1997, Ikeda et al. 1997].

Because there is a road embankment adjacent to the observation sites, it was thought that the measurements would be affected by the road embankment. One-dimensional equivalent linear analysis (one-dimensional dynamic analysis) and two-dimensional equivalent linear FEM analysis (two-dimensional dynamic analysis) were therefore conducted on sandy ground observation sites. The transfer functions and time history responses of the ground were compared and the effects of the road embankment on dynamic characteristics at the observation sites were investigated.

With the equivalent linearization method used for analysis, nonlinear characteristics of the ground material can be examined equivalent linearly. This method is often used for dynamic analysis of the ground because of its easy modeling, short calculation time, considerable research cases in the past and other reasons. However, it is generally thought to be effective only for strain levels with up to 10^{-3} of shear strain [Yoshida 1994]. The applicability of the equivalent linearization method for ground seismic behavior with relatively large shear strain was therefore examined.

[1] Civil Engineering Research Institute (CERI) , Hokkaido Development Bureau, Sapporo, Japan E-mail: hayashi@ceri.go.jp
[2] Civil Engineering Research Institute (CERI) , Hokkaido Development Bureau, Sapporo, Japan
[3] Civil Engineering Research Institute (CERI) , Hokkaido Development Bureau, Sapporo, Japan
[4] Technical Research Institute, Tobishima Corporation, Sekiyado, Japan
[5] Technical Research Institute, Tobishima Corporation, Sekiyado, Japan E-mail: takaaki_ikeda@tobishima.co.jp

DESCRIPTION OF THE LIQUEFACTION ARRAY OBSERVATION METHOD

Figure 1 shows a map of liquefaction array observation sites and Fig. 2 shows a plan view for the installation of seismographs on the target ground. This study focused on observation sites on sandy ground. As mentioned before, there is a road embankment adjacent to the positions where seismographs were installed. Each road embankment section is 7.35 m in height, 25 m in crown width and 1:1.8 in slop gradient. Ground improvement has been conducted on the outside of the toe of slope using the sand compaction pile method to prevent liquefaction. Figure 3 shows a cross section of the road embankment.

Seismographs were installed at G.L. -2 m, -17 m and -35 m in the unimproved ground [Nishikawa et al. 1994]. Seismograph at G.L. -35 m was installed in the gravel layer which spreads over a wide area of the section and has a high shear wave velocity compared with the upper surface layer. This gravel layer was regarded as a layer equivalent to an engineering basement because Vs = 400 m/s. Pore pressure meters were buried in sand layers with a possibility of liquefaction at G.L. -10.5 and -14.5 m of the unimproved and improved ground.

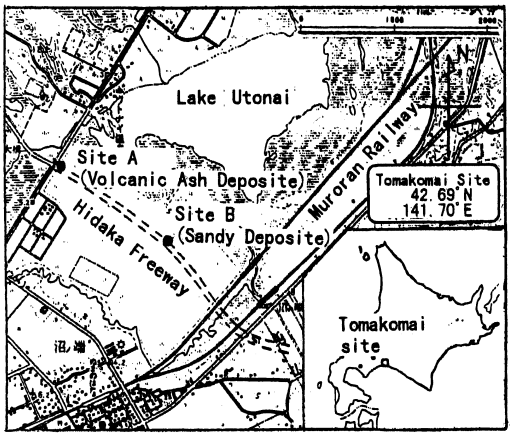


Fig. 1 Location of liquefaction arrey site

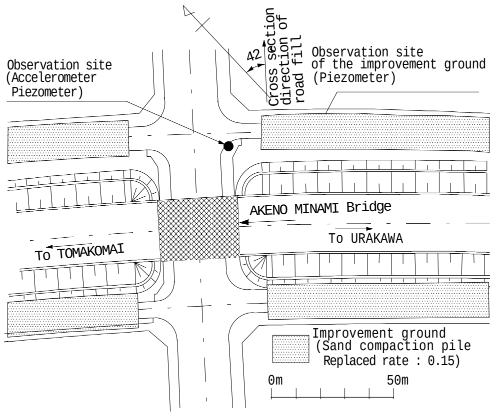


Fig2 Liquefaction Arrey Observation Site (Sandy Deposit Site)

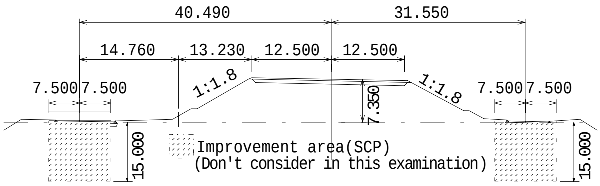


Fig.3 Cross section of road embankment

EXAMINATION CONDITIONS

Examination Method

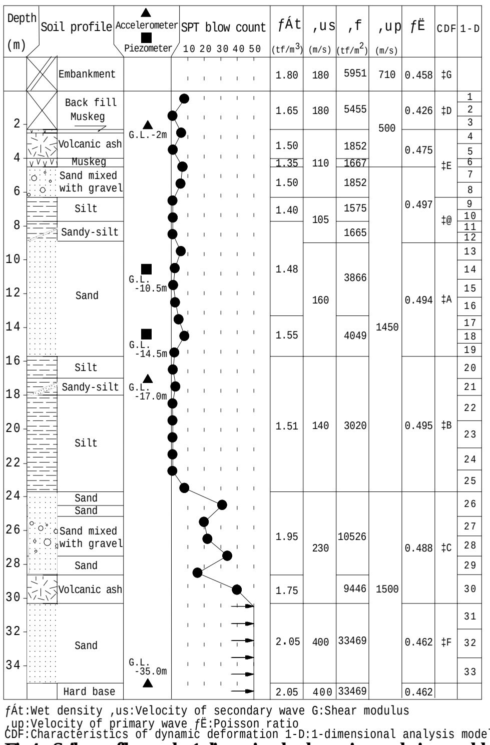
The effects of the road embankment on the liquefaction array observation sites were examined by conducting one-dimensional dynamic analysis only for the ground. Two-dimensional dynamic analysis was conducted for both the ground and road embankment to compare the transfer functions and the time history responses of the two. Regarding the applicability of the equivalent linearization method for ground under a relatively large amount of strain, the observation results and the transfer functions, the time history responses and other factors obtained from one-dimensional dynamic analysis were compared.

As analysis codes, SHAKE2 based on the multiple reflection theory was used for one-dimensional dynamic analysis and TFLUSH for two-dimensional analysis. For both codes, functions were enhanced from the original SHAKE and FLUSH codes [Schnabel et al. 1972, Lysmer et al. 1975]. Analysis was conducted for the cross section of the embankment. For both types of analysis, examination was conducted at a frequency range of 0 to 10 Hz. The effective strain coefficient used for the equivalent linearization method was 0.4. The convergence criterion was met when the errors of new and old shear modulus and damping constant were both within 5%. For the effective strain coefficient, 0.65 suggested by Schnabel et al. has often been used[Schnabel et al. 1972]. However, it has also been found from studies based on observation records that the optimum value is around 0.4, because the optimum effective strain coefficient varies by earthquake motion[Ohsaki 1982, Takura 1987]. Based on these data indicating the best value for reproducing observation records after examining effective strain coefficient, 0.4 was used for this study.

Analysis Model

Figure 4 shows the soil boring log and physical properties of the ground at the sites of the examination. The layer distribution under the ground surface in the figure corresponds to the one-dimensional analysis model. Physical properties of the ground were established based on the results of on-site ground surveys and indoor tests on undisturbed samples. Because observations were conducted on unimproved ground, ground improvement was not taken into account. Figure 5 shows dynamic deformation characteristics (ratio of shear modulus and strain dependency curve of the damping coefficient). Regarding the strain dependency curve of the damping coefficient, the damping coefficient obtained from indoor tests at the time of small strain was approximately 0. However, according to an examination using observation records obtained by Nishikawa et al. at the same observation sites, there is damping of approximately 3% for seismic behavior within a linear range [Nishikawa et al. 1994]. Therefore 3% was regarded as the lower limit of the damping coefficient. Although eight types of dynamic deformation characteristics were determined based on the results of the ground survey, they were almost equivalent regardless of differences in soil property.

Figure 6 shows a two-dimensional analysis model. As seen in Fig. 2, the observation sites were located in a viaduct. Because the road embankment adjoins the sites, modeling in this study was conducted on the assumption that the road embankment continue to the viaduct part. The ground was thought to be stratified horizontally around the study sites. Half sections were analyzed using symmetric boundaries and the initial physical properties were made uniform at the same depth. In the ground underneath the embankment, the shear modulus of the ground may increase due to the confining pressure of the embankment. However, the effect of this pressure was not taken into account because the shear wave velocity distribution obtained by PS logging was



ƒÁt:Wet density ,us:Velocity of secondary wave G:Shear modulus
,up:Velocity of primary wave ƒÈ:Poisson ratio
CDF:Characteristics of dynamic deformation 1-D:1-dimensional analysis model

Fig.4 Soil profile and 1-dimensional dynamic analysis model

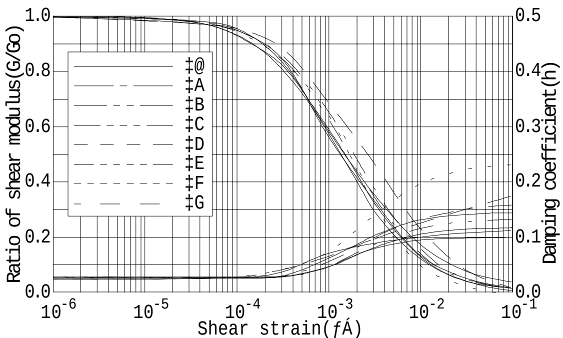


Fig.5 Characteristics of dynamic deformation

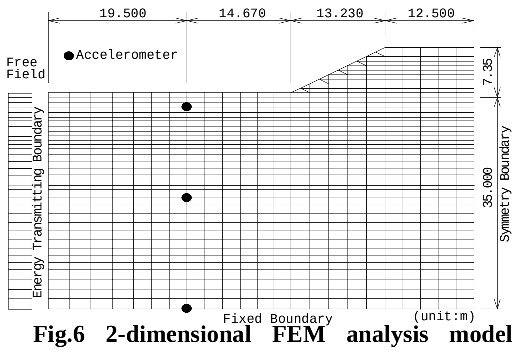


Fig.6 2-dimensional FEM analysis model

almost equivalent to that of the other parts of the ground. The right side of the analysis model (center of the embankment) was regarded as the symmetric boundary (horizontal roller), left side as the energy transmission boundary, and the lower part as the fixed boundary. Although there is a side embankment (approx. 2.8 m high and 8.0 m wide) on the north side of the road embankment, it was not modeled because it does not affect the seismic behavior of the road embankment [Taniguchi et al. 1998]. The improved section of the ground on the southeast side of the observation sites was not taken into consideration either because its effect was thought to be small.

Input Earthquake Motion

The Kushiro-Oki Earthquake (January 15, 1993, JMA intensity scale M = 7.8), with the largest earthquake motions, and the Urakawa-Oki Earthquake (February 20, 1997, JMA intensity scale M = 5.6), with relatively small earthquake motions, were examined. Ground behavior during the Kushiro-Oki Earthquake was thought to be nonlinear earthquake motion although it was unremarkable, while that during the Urakawa-Oki Earthquake was thought to be linear. In both cases, bearing errors of the buried seismographs were corrected and the orientation was changed to the direction of the cross section of the embankment. For input earthquake motion, records at G.L. -3.5 m, which was thought to be equivalent to the engineering basement, were used. The maximum input earthquake motion was 57.8 cm/s^2 in the Kushiro-Oki Earthquake and 2.9 cm/s^2 in the Urakawa-Oki Earthquake.

EFFECTS OF ROAD EMBANKMENT ON DYNAMIC BEHAVIOR AT THE OBSERVATION SITES

Comparison of Transfer Functions

Figure 7 shows a comparison of transfer functions of G.L. -2 m for G.L. -35 m and obtained from one and two-dimensional dynamic analysis for the Kushiro-Oki Earthquake. The two-dimensional analysis represents transfer functions at the installation positions of seismographs shown in Figs. 2 and 6. For both one and two-dimensional analysis, three dominant peaks were seen. At the first peak, the dominant frequency was smaller and the amplification factor was larger in the two-dimensional dynamic analysis. At the second peak, on the contrary, the amplification factor was larger in the one-dimensional analysis. However, the difference in amplification factors was small and the two transfer functions were thought to be almost equivalent.

Figure 8 shows a comparison of transfer functions of G.L. -2 m for G.L. -35 m obtained from one and two-dimensional dynamic analysis for the Urakawa-Oki Earthquake. Results were similar to those of the Kushiro-Oki Earthquake. There were three dominant peaks. The dominant frequency was smaller and the amplification factor was larger in the two-dimensional dynamic analysis at the first peak, and the amplification factor was

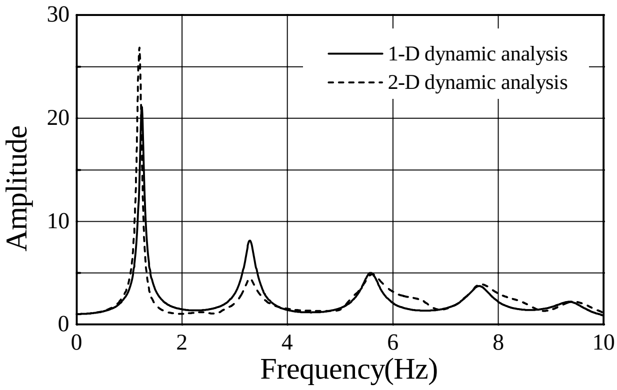


Fig.7 Comparison Between 1-D Dynamic Analysis And 2-D Dynamic Analysis For Transfer Function Of G.L.-2m For G.L.-35m(1993.1.15 Kushiro-Oki Earthquake)

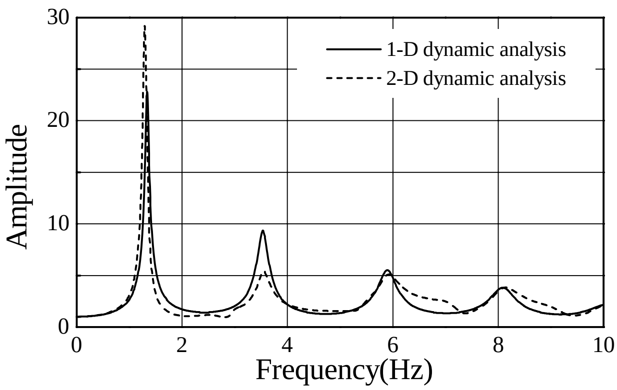


Fig.8 Comparison Between 1-D Dynamic Analysis And 2-D Dynamic Analysis For Transfer Function Of G.L.-2m For G.L.-35m(1997.2.20 Urakawa-Oki Earthquake)

larger in the one-dimensional analysis at the second peak.

The transfer functions of the Kushiro-Oki Earthquake and the Urakawa-Oki Earthquake were also compared. The dominant frequency was smaller in the Kushiro-Oki Earthquake than in the Urakawa-Oki Earthquake, meaning that nonlinear characteristics appeared of the ground and extended the period of the ground's transfer function.

Comparison of The Depth Distribution of The Maximum Response

Figures 9 (a) and 9 (b) show the depth distribution of the maximum acceleration, shear strain and shear stress obtained from the two analyses for the Kushiro-Oki and Urakawa-Oki earthquakes. The depth distribution was almost equivalent in the one and two-dimensional dynamic analyses in the Kushiro-Oki Earthquake. The appearance of nonlinear characteristics, such as increasing shear strain at G.L. -6.3, -9.0, -15.7 and -23.7 m in the silt layer, showed particular correspondence. During the Urakawa-Oki Earthquake, there were relative differences in shear strain and shear stress. However, the absolute quantity of the differences was small and their distribution profiles were almost equivalent. It was therefore assumed that the depth distribution was almost equivalent for the one and two-dimensional dynamic analyses. The maximum acceleration observed is marked in the figure. It can be seen that the analysis results almost corresponded to the observation results for both earthquake motions.

In this way, differences by analysis method were not seen in the transfer functions and the depth distribution of maximum values at the liquefaction array observation sites. It is thus thought that the road embankment hardly affected the liquefaction array observation sites.

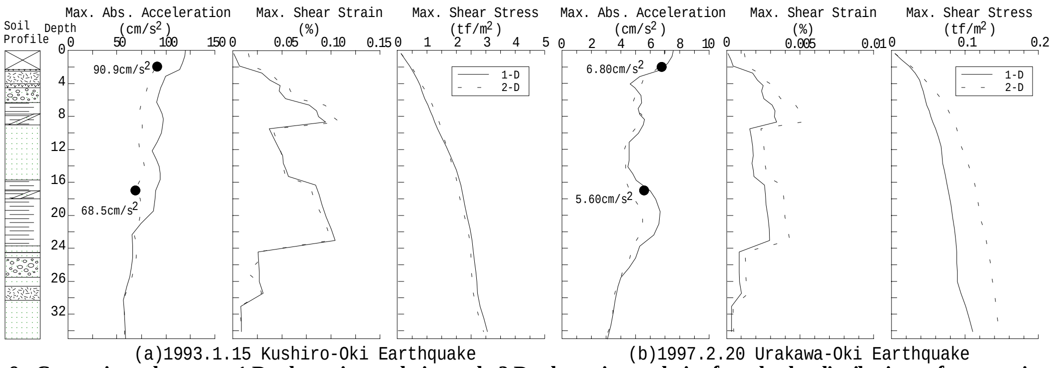


Fig.9 Comparison between 1-D dynamic analysis and 2-D dynamic analysis for depth distribution of a maximum response

EXAMINATION OF THE APPLICABILITY OF THE EQUIVALENT LINEARIZATION METHOD

Comparison of transfer functions

Figure 10 shows a comparison of transfer functions of G.L. -2 m for G.L. -35 m obtained from one-dimensional dynamic analysis for the Kushiro-Oki Earthquake. The transfer functions in the observation results underwent smoothing in the Parzen window with he window width of 0.2 Hz. In the same way as with the analysis results, three dominant peaks were seen in the transfer functions obtained from observation results. The amplification factor at the second peak was larger in the analysis results, while at the third peak was slightly larger in the observation results. Seen from a broader viewpoint, however, the dominant frequency was almost equivalent in the observation and analysis results. Although the dominant frequency in the analysis results was slightly higher at the first peak, the results corresponded well.

Figure 11 shows a similar comparison as in Fig. 10 for the Urakawa-Oki Earthquake. At the first peak, the amplification factor was larger and the dominant frequency was slightly lower in the analysis results. At the second and third peaks, however, both the dominant frequency and amplification factor were correspondent. Transfer functions in the observation results of the Kushiro-Oki and Urakawa-Oki earthquakes were compared. The dominant frequency was lower in the Kushiro-Oki than in Urakawa-Oki earthquake. It was thus thought that nonlinear characteristics appeared in the ground in the Kushiro-Oki Earthquake. With the Urakawa-Oki Earthquake as a reference, the dominant frequency rate was found by Rf = fi {Kushiro-Oki}/fi {Urakawa-Oki}. Where, fi is the dominant frequency of peak i. The dominant frequency rate of the first to third peaks was around Rf = 0.88. The average apparent modules change obtained by Rf^2 was 0.78.

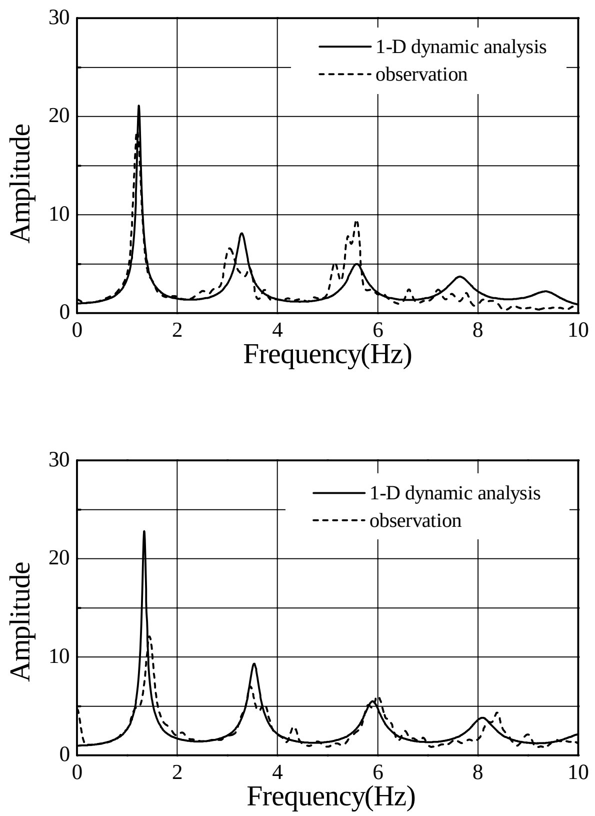


Fig.11 Comparison between 1-D dynamic analysis and observation for transfer function of G.L.-2m for G.L.-35m(1997.2.20 Urakawa-Oki Earthquake)

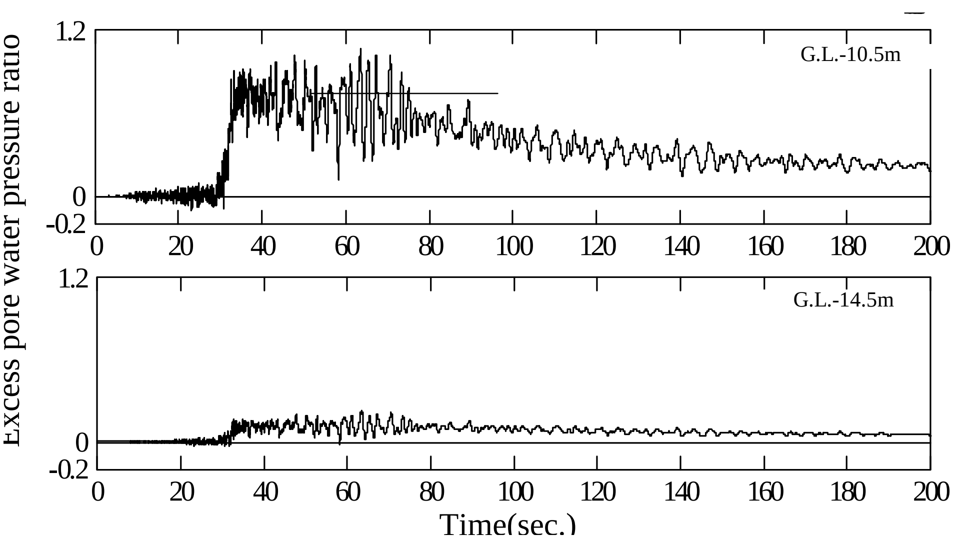


Fig. 12 Excess pore water pressure time history measured in sandy deposit (1993.1.15 Kushiro-Oki Earthquake)

Figure 12 shows the time history of excess pore water pressure ratio measured with pore water pressure gauges installed at G.L. -10.5 and -14.5 m in the unimproved ground during the Kushiro-Oki Earthquake. Where, the excess pore water pressure ratio is value found by dividing the excess pore water pressure by the effective confining pressure. At G.L. -10.5 m, the peak value reached 1.0. The elevation excluding vibrational component was as large as 0.75, while it was as small as 0.15 at G.L. -14.5 m. However, the decrease in the apparent shear modules was not large although the excess pore water pressure ratio rose to 0.75 at G.L. -10.5 m. It was thus thought that the excess pore water pressure did not increase considerably throughout the sand layer. In other words, it rose in limited areas in the upper part of the sand layer and this area as relatively thin compared with the thickness of the entire ground. This supposition is based on the fact that the rise of excess pore water pressure ratio was small at G.L. -14.5 m.

Comparison of acceleration time histories

Figures 13 show comparisons of the observation records and the results of a one-dimensional dynamic analysis with regard to the entire acceleration time history (0 to 100 sec.) and its main dynamic part (30 to 45 sec.) at G.L. -2 m during the Kushiro-Oki Earthquake. Both the amplitudes and phases of the entire wave forms corresponded almost perfectly with each other.

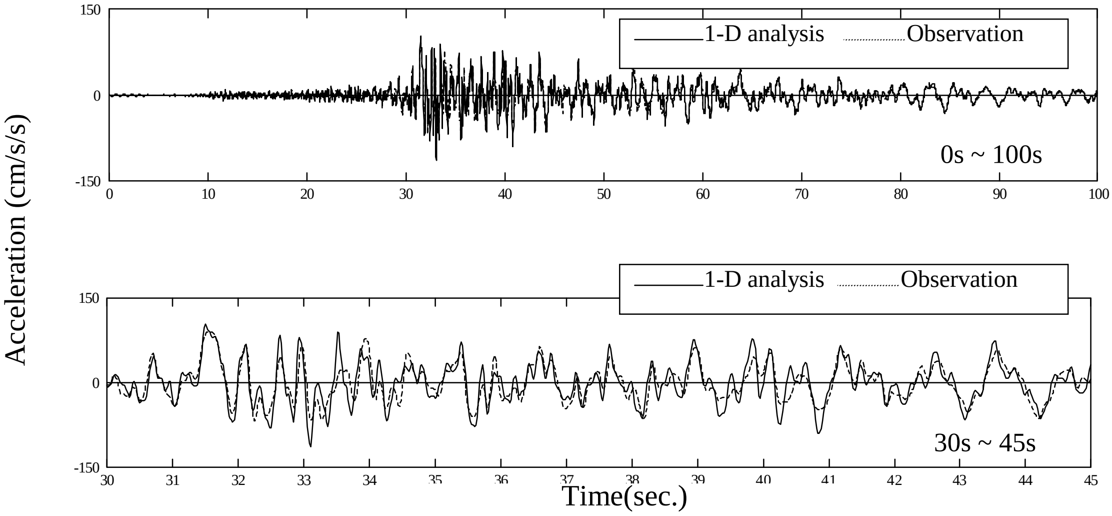


Fig. 13 Comparison Between 1-D Dynamic Analysis And Observation For Acceleration Time History Of Surface (G.L.-2m)

Depth Distribution of Maximum Responses

Figure 14 shows the depth distribution of the maximum values of acceleration, relative velocity, relative displacement, shear strain and shear stress during the Kushiro-Oki Earthquake derived from the one-dimensional dynamic analysis results. Relative velocity and displacement values are expressed relative to the basement. Observation results are marked in the distribution map for acceleration, relative velocity and relative displacement. The velocity and displacement in the observation results were calculated by removing invalid elements from the long-frequency side of the acceleration time history and integrating them with FFT.
Although the maximum acceleration was larger in the analysis results in the case of the Kushiro-Oki Earthquake due to the reasons indicated in the previous section, both the relative velocity and displacement were thought to almost perfectly correspond with the observation results. The maximum shear strain which occurred in the ground was approximately 1 x 10^{-3} in the Kushiro-Oki Earthquake. The ratio of shear modulus, found by dividing the shear modulus at the time when the equivalent linear analysis converged by the initial shear modulus, is shown at the far right of Fig. 14. During the Kushiro-Oki Earthquake, which featured large

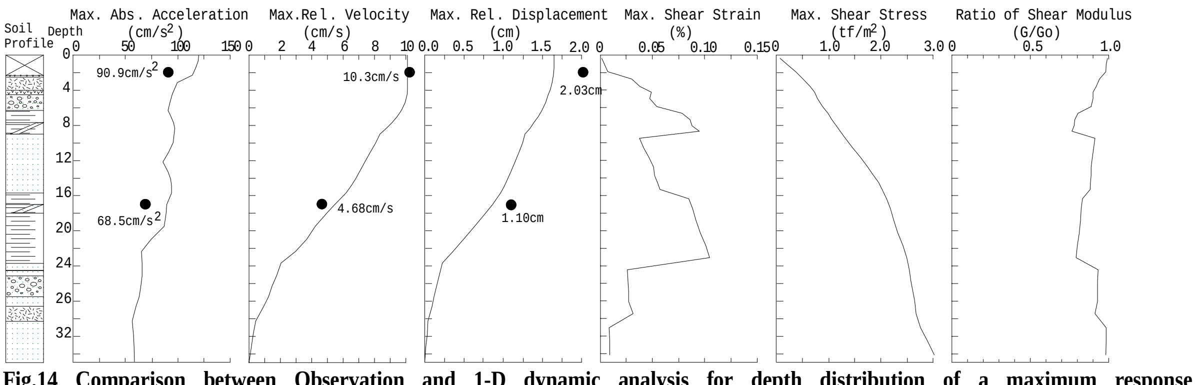


Fig.14 Comparison between Observation and 1-D dynamic analysis for depth distribution of a maximum response (1993 Kushiro-Oki Earthquake)

earthquake motions, maximum shear modulus decreased to 75% of the initial shear modulus. The result was almost consistent with the percentage of decrease in shear modulus calculated from the observation results. In the Urakawa-Oki Earthquake, the maximum shear strain which occurred in the ground was around 3 x 10^{-5}. A decrease in shear modulus was not seen in the equivalent linear analysis.

CONCLUSIONS

One-dimensional dynamic analysis which did not take into account the adjoining road embankment and two-dimensional dynamic analysis which did take the embankment into account were conducted at liquefaction array observation sites. The following results were found from the analyses:
1) No clear differences were seen in the comparison of the transfer function and maximum value distribution of the ground. It is therefore thought that the road embankment barely affects the seismic behavior at the liquefaction array observation sites.
2) In the Kushiro-Oki Earthquake, shear modulus decreased to 75% of the initial shear modulus and caused shear strain as large as 1 x 10^{-3} in the ground. In this case, the observation and analysis results almost corresponded with each other with regard to the transfer function and acceleration time history of the ground.
3) If the decrease in shear modulus is around 75% of the initial shear modulus and the shear strain is around 1 x 10^{-3}, the equivalent linearization method is thought to be effective for seismic response analysis. This case includes the effect of excess ore water pressure, which is equivalent to 75% of the effective confining pressure, in some parts of the ground.

REFERENCES

Hayashi,H.(1991), "Setting of Automatic Liquefaction Observation System -Hidaka National Express Way, Hokkaido-", *Tsuchi-to-Kiso*, Vol.39, No.1, JGS, pp117-118.(in Japanese)

Hayashi,H., Nishikawa,J. and Taniguchi,K.(1997), "Array Observation of Ground and Road Embankment", *Proceedings of The 24th JSCE Earthquake Engineering Symposium*, JSCE, pp117-120. (in Japanese)

Ikeda,T., Mori,S., Miwa,S., Nishikawa,J. Hayashi,H. and Taniguchi,K.(1997), "Application of Equivalent Linear Analysis toward Records from Liquefaction Array Observation", *Proceedings of The 24th JSCE Earthquake Engineering Symposium*, JSCE, pp125-128. (in Japanese)

Lysmer,J., Udaka,T., Tsai,C.-F. and Seed,H.B (1975), *FLUSH a computer program for approximate 3-D analysis of soil-structure interaction problem*, Earthquake Engineering Research Center, University of California, Barkeley.

Nishikawa,J., Odajima,H., Mori,S. and Miwa,S.(1994), "Array Observation of Liquefaction during Kushiro-Oki Earthquake of January 15, 1993", *Tsuchi-to-Kiso*, Vol.42, No.3, JGS, pp57-62.(in Japanese)

Ohsaki,Y.(1982), "Dynamic Nonlinear Model and One-Dimensional Nonlinear Response of Soil Deposits", *Research Report*, Department of Architecture Faculty of Engineering, University of Tokyo.

Odajima,H., Noto,S. and Hayashi,H.(1992), "Observation System of Pore Water Pressure in Volcanic Ash Ground", *Proceedings of The 28th Annual Conference*, *Vol.Ⅲ*, JSCE, pp806-807. (in Japanese)

Saito,K.(1986), "Liquefaction Characteristic of Reclaimed Ground using Dredged Soils during Urakawa-Oki Earthquake", *Proceedings of The 21st Japan National Conference*, JGS, pp843-844. (in Japanese)

Schnabel,P.B., Lysmer,J. and Seed,H.B. (1972), *SHAKE a computer program for earthquake response analysis of horizontally layered sites*, Earthquake Engineering Research Center, University of California, Barkeley.

Takura,T., Sato,M., Shimazu,K., Oyama,K. and Watanabe,O.(1987), "A Study on Influence of Stratum Division and Effective Strain Coefficient on Results of Nonlinear Response Analysis", *Proceedings of The 22th Japan National Conference*, JGS, pp655-658. (in Japanese)

Taniguchi,K., Nishikawa,J. and Hayashi,H.(1997), "Influence of Approached Road Embankment in Array Observation of Liquefaction, *Proceedings of The 24th JSCE Earthquake Engineering Symposium*, JSCE, pp121-124. (in Japanese)

Taniguchi,K., Nishikawa,J., Hayashi,H., Ikeda,T., Miwa,S. and Mori,S.(1998), "Influence of Earthfill Shape upon Array Observation of Liquefaction", *Proceedings of The 33rd Japan National Conference*, JGS, pp911-912. (in Japanese)

Wakamatsu,K.(1991), *Maps for Historic Liquefaction Sites in Japan*, Tokai University press, Tokyo, pp33. (in Japanese)

Yoshida,N.(1994), "Application of Practical Program, SHAKE", *Proceedings of Ground Motion Amplification in Soft Deposit Symposium*, JSG, pp14-31. (in Japanese)