DYNAMIC CENTRIFUGE TEST OF PILE FOUNDATION STRUCTURE PART ONE : BEHAVIOR OF FREE GROUND DURING EXTREME EARTHQUAKE CONDITIONS

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SUMMARY

Dynamic centrifuge model tests were carried out to investigate the behavior of structures housing nuclear power facilities at sites where the bedrock is deeply located. There were two sets of tests: one set using a free ground model, and the other using a ground model with a model structure representing a turbines building set on pile foundations. This part discusses the results of the study of free ground. The models were of a scale of 1/100 to accord with the similarity rule for a centrifugal acceleration of 100g. Earthquake simulation tests were performed using sinusoidal waves (S1 test , S2 test) equivalent to a maximum design earthquake (S1) and an extreme design earthquake (S2). Measurements were made of the degree of acceleration, pore water pressure and earth pressure applied to the building during an earthquake. In the model tests, an S2 level earthquake prompted liquefaction in sand layer. In S1 earthquake conditions, liquefaction did not occur as shear strain was low. Using these test results for comparison, the suitability of simulations based on two types of numerical analysis, equivalent linear analysis and effective stress analysis, was studied. The results indicate that either method of analysis produces appropriate values provided that the soil constants are correctly appraised, and can thus be used in the design of structures and ground profiles. Equivalent linear analysis is suitable for ground layers other than those liable to liquefaction. Effective stress analysis is also of a level close to practical application.

INTRODUCTION

In Japan, where large earthquakes are frequent, the foundations of the main structures at nuclear power generating stations are, in principle, built directly onto hard bearing strata. However due to diverse ground characteristics, newly added facilities are sometimes located on sites where the bedrock is at a deep level. Figure 1 shows a hypothetical example. In such locations, it is difficult for all building foundations to be set directly into the bedrock due to time and cost limitations. Thus, some buildings including power generation facilities are supported using piles and diaphragm walls. Until now there has been hardly any use of piles to support main structures at nuclear power generating stations. Thus it became necessary to examine non-linearity of soil generated by seismic forces, and the effect of this non-linearity on structural foundations, for use in the design of nuclear power generating facilities. Figure 2 shows the hypothetical soil profile used in the

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studies. The suitability of a design in practice depends on the degree to which analytical methods can simulate actual phenomena. Models of hypothetical ground were created and dynamic centrifuge tests at 100g were carried out. From these test results, the behavior of ground with a thick layer of deposits, in circumstances of S1 and S2 earthquakes was obtained. Then for the purpose of verifying the suitability of analytical methods, simulation analyses were performed. Two different models were used, one a free ground model, and the other a ground model of the same soil profile that included a turbine building on pile foundations. First, the behavior of free ground during an earthquake was studied both through tests and simulation analyses, using the free ground model. Next, the same tests and analyses were performed on the model with the structure. The influence of the ground in terms of its effect on the structure was examined.

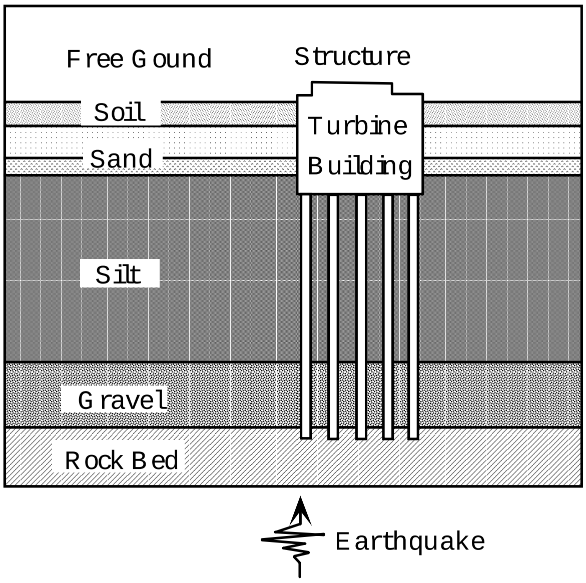

Figure 1 : Overall view

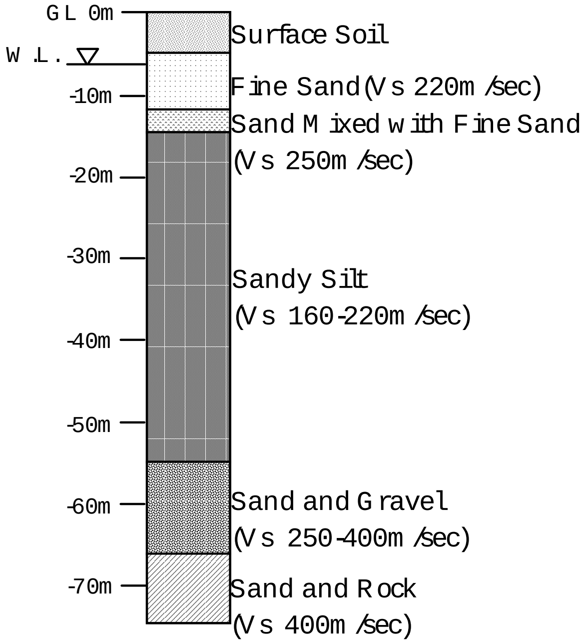

Figure 2 : Hypothetical soil profile

DYNAMIC CENTRIFUGE TESTS

Test Method

The model ground was constructed to represent the hypothetical ground shown in Figure 2, to a depth of 53m below ground level.

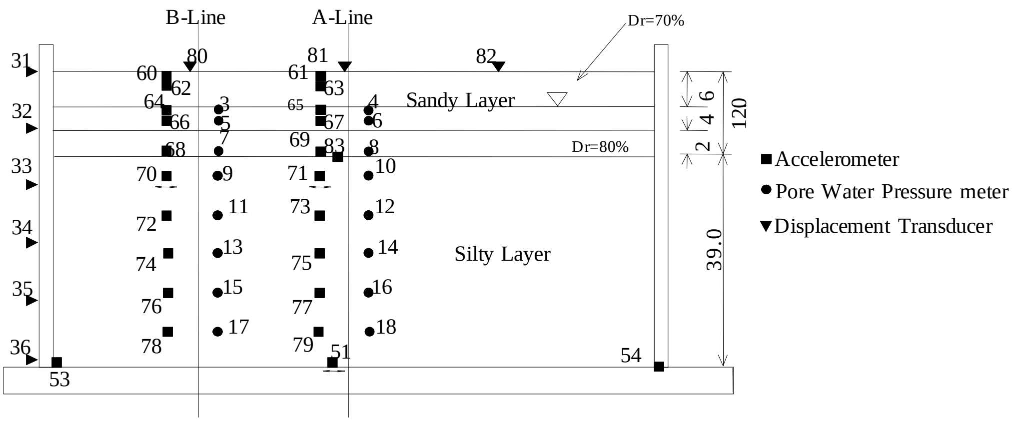

Figure 3 : Centrifuge test model

Figure 3 shows the model ground. It was built in a laminar container 100cm long, 30cm wide and 55.2cm high. The container consisted of thirty-three 15mm thick aluminum laminar rings with roller bearings placed between them. Centrifuge tests were performed at a centrifugal acceleration of 100g.

For the model, a single silty layer was created between GL -12m and GL -53m. To represent the layer of reclaimed land and fine sand between GL +0m and GL -10m, a uniform sand layer was made. Toyoura sand was used, as abundant data on its physical and dynamic properties is available. Alluvial silt from the upper Tokyo layer was used for the silty layer. The relative density, Dr, of the sandy layer was taken to be that determined from the N value of the hypothetical sandy layer using Meyerhof's equation.[2] Table 1 shows the physical characteristics of the Toyoura sand and alluvial silt used in the tests. The silty layer was consolidated normally using seepage force. The sandy layer was created by the air pluviation method using dry sand.

In the earthquake simulation tests, the acceleration and excess pore water pressure in the model ground, the vertical displacement of the soil surface, and the horizontal displacement of the model ground were measured, as shown in Figure 3.

Sinusoidal waves equivalent to a maximum design earthquake and an extreme design earthquake were used as the base input accelerations. The shaking table is controlled through servo hydraulic technology. The amplitude and number of cycles of the input acceleration were set as the amplitudes and numbers of cycles equivalent to a maximum design earthquake and an extreme design earthquake.[3),4)] The frequency of the input acceleration was 1.4Hz. Table 2 shows the input motion conditions.

Table 1 : Physical properties of silt and sand

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Sample Name	Silt	Sand
Soil Particle Density(g/cm^3)	2.667	2.638
Grain-Size Characteristics		
Sand(%)	11.1	98.4
Silt(%)	56.0	1.6
Clay(%)	32.9	0.0
Liquid Limit(%)	41.78	
Plastic Limit(%)	26.48	
Plasticity Index	15.30	
Max. Density(g/cm^3)		1.659
Min. Density(g/cm^3)		2.341

Table 2 : Test case

	S1 Test	S2 Test
Input Motion	Sinusoidal Wave	Sinusoidal Wave
Number of Circles	20	20
Wave(gal)	100	180
Frequency(Hz)	1.4	1.4

Test Result

Figure 4 shows the acceleration time history for the S2 test. Figure 5 shows the relationship between maximum response acceleration and depth in the S1 and S2 tests. In the S2 test, liquefaction caused the wave form to become lagged and brought about a gradual decline in the response acceleration in the sandy layer at depths of less than GL -9.0m. Maximum response acceleration slowly increased to the layer of sand with a relative density of Dr = 80%, but greatly decreased in the sandy layer at depth of less than GL -10m due to liquefaction. In the S1 test, the maximum response acceleration increased to the middle of the silty layer but declined through the upper half of the silty layer. As there was no liquefaction in the sandy layer, the maximum response acceleration increased through to its surface.

Figure 6 shows the excess pore water pressure time history for the S2 test. Liquefaction occurred in the S2 test at depths of around GL -9m due to the relatively low density of the sandy layer, so the response acceleration decreased, as shown in Figure 4. However, even though the maximum pore water pressure reached the overburden pressure line in the dense sand at GL -11m, no decrease was observed in the response acceleration, as shown in the CH-69 acceleration time history in Figure 4. This is thought to be due to the occurrence of cyclic mobility because the sand was dense. No increase was seen in the excess pore water pressure in the silty layer during shaking. A figure has not been included but in the S1 test there was little increase in excess pore water pressure, and no liquefaction was observed.

Figure 7 shows the relationship between maximum shear strain and depth in S1 and S2 tests. Shear strain was determined by calculating the relative displacement between the accelerometers from the measured acceleration and then dividing this by the distance between the upper and lower accelerometers. In the S2 test, in the sandy layer at depths below GL -10m, shear strain values were lower than 0.1% due to liquefaction. In the silt layer, however, shear strain was 0.2% to 0.5%. In the S1 test, shear strain values were lower than in the S2 test, at below 0.2% in both the sandy and silty layers.

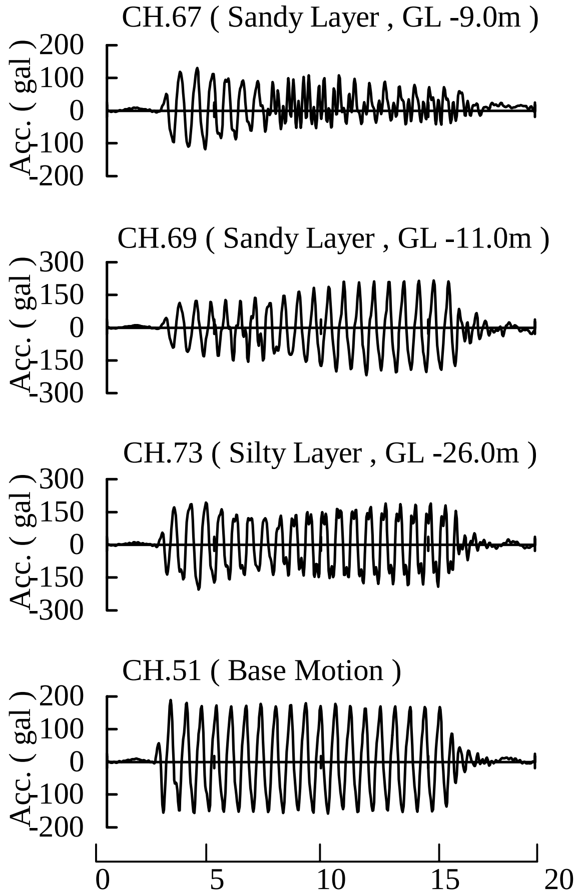

Figure 4 : Time histories of acceleration (S2 test)

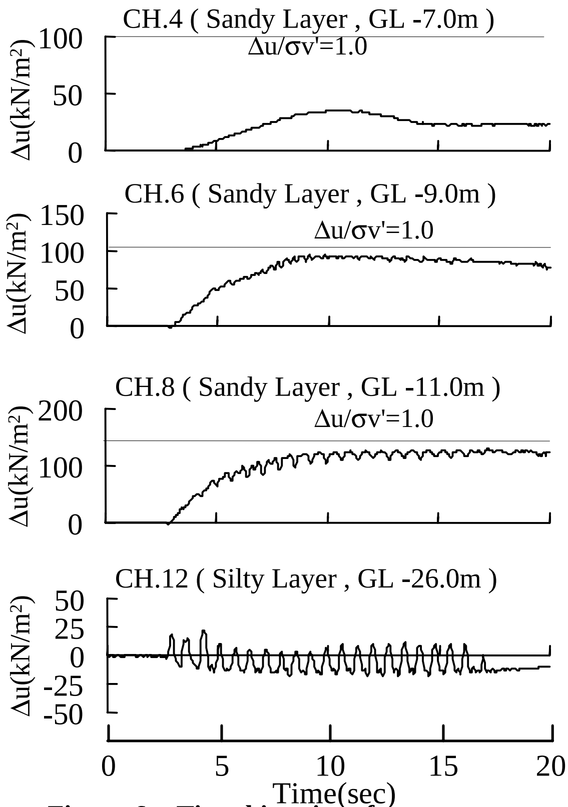

Figure 6 : Time histories of excess pore water pressure (S2 test)

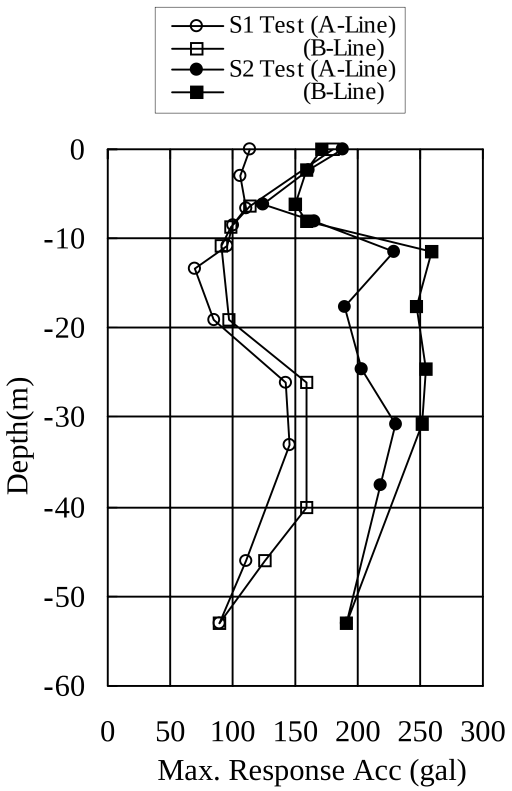

Figure 5 : Distribution of maximum response acceleration

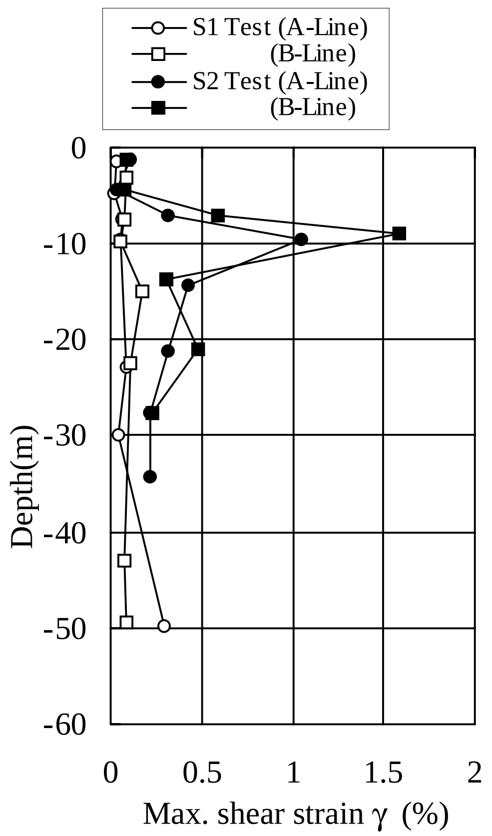

Figure 7 : Distribution of maximum shear strain

NUMERICAL ANALYSIS

Analytical Method

Two analytical methods were used: equivalent linear analysis and effective stress analysis. In equivalent linear analysis, the relationship between the stress and strain in the ground is evaluated from the decline of shear modules due to strain, and then linear analysis is performed using a secant modulus equivalent to the actual strain. The code-word used for this analysis is SHAKE.[5)]

For the effective stress analysis, a constitutive equation for soil in which liquefaction can be simulated was incorporated into the effective stress analysis method proposed by Biot, taking into consideration the elastic-plastic behavior of the soil and the increase and dispersion of the pore water pressure. A densification model was used for the constitutive equation showing liquefaction. The code-word used for this analysis is that developed by TAKENAKA CORPORATION, MuDIAN.[6)]

A one-dimensional analytical model was used that included the weight of laminar container. Taking into consideration the thicknesses of the soil layers and the rings, the number of separate elements was nineteen for the equivalent linear analysis, and thirty-three for the effective stress analysis. Input base motion was used as the input earthquake motion for both S1 and S2 tests.

Values determined from the results of laboratory soil tests were used for the soil constants in the analyses.

They are shown in Table 3. In the equivalent linear analysis, the results of laboratory torsional tests of hollow cylindrical specimens were used for the relationship G and γ, which shows the non-linearity of the soil. In the effective stress analysis, the Ranberg-Osgood model was used for the G and γ relationship, and parameters were set so as to agree with the analysis of the results of laboratory torsional tests of hollow cylindrical specimens. Figure 8 shows a comparison of the G and γ curve of the laboratory tests and Ramberg-Osward Model for the silty layer in each analysis.

Table 3 : Laboratory soil test results

GL±0.0		Density ρ (tf/m3)	Shear Modulus G0 (kN/m2)	Porosity e	Coefficient of permeability k(m/sec)	Effective stress c'	Effective stress ϕ'
-6.0	DR-70	1.546	61500	0.427	1.55E-06		
-10.0		1.890	107000	0.427	1.55E-06	0.940	36.7
-12.0	DR-80	1.910	116000	0.421	1.34E-06	0.920	37.8
-18.0		1.820	58000	0.551	□	0.208	36.1
-32.0	Silt Layer	1.900	73000	0.551	□	0.208	36.1
-47.0		2.015	118000	0.551	□	0.208	36.1
-49.8							

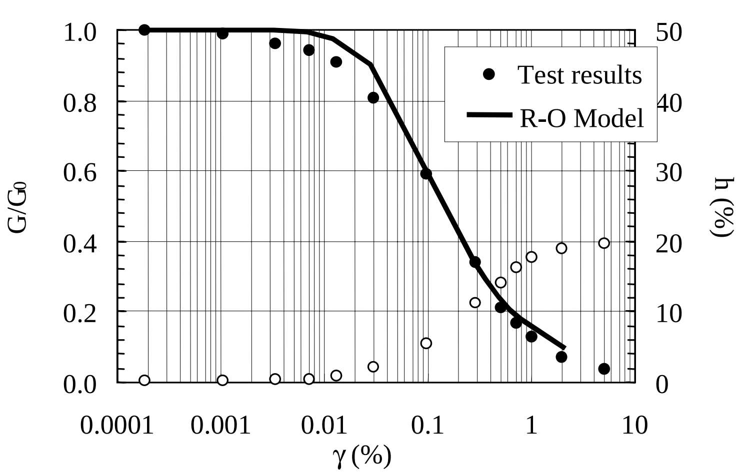

Figure 8 : Relationship between G/G$_0$ and shear strain

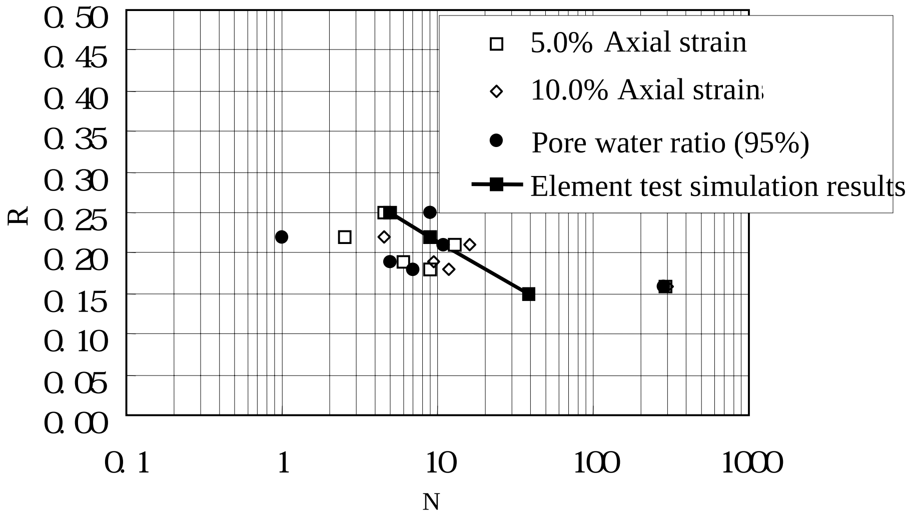

Figure 9 : Relationship between stress ratio and number of repetitions

As shown in Figure 9, the liquefaction parameters in the effective stress analysis were set so that the results of a simulation analysis of element tests matched the results of laboratory liquefaction tests. Rayleigh damping was used as the damping coefficient, and the coefficients were set assuming a 2% damping from the primary and secondary natural frequencies (primary 97Hz, secondary: 250Hz). The initial stress in the ground was calculated assuming a coefficient of earth pressure at rest K0=1.0, since a model ground was restrained by the laminar container.

Analysis Results (S1 Test)

The S1 test results give a low value of shear strain in the ground of 0.07% ~ 0.2%. Of the analytical methods, only equivalent linear analysis was used because no liquefaction occurred in the sandy layer.

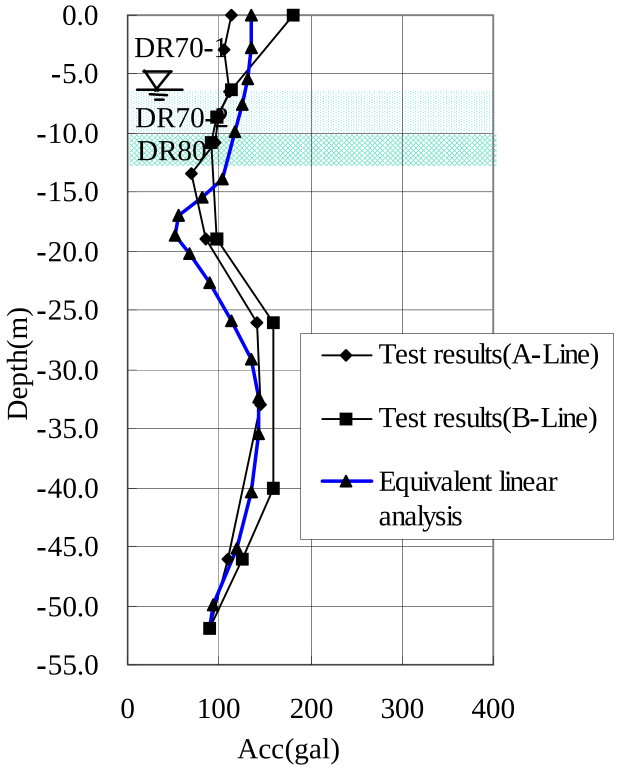

Figure 10 : Relationship between maximum response acceleration and depth for free ground (S1 test)

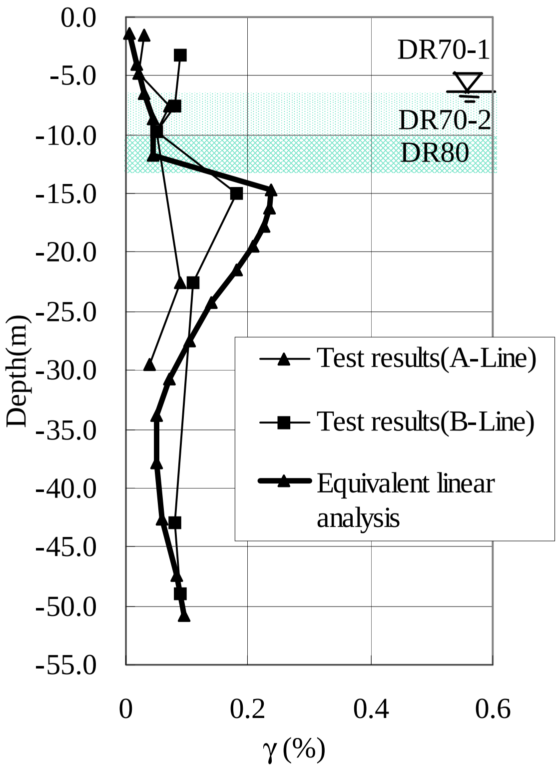

Figure 11 : Relationship between maximum shear strain and depth for free ground (S1 test)

Figure 10 shows the relationship between maximum response acceleration and depth.
A comparison of the results of the tests and analysis shows a close agreement for both the sandy and silty layers, although in the upper part of the silty layer the analytical results are slightly lower than those of the tests. Figure 11 shows the relationship between maximum shear strain and depth. A comparison of the analytical and test results shows a close agreement for both sandy and silty layers. Determining the degree of decline in shear modulus (G/G_0) in the silty layer from Figure 8 gives values from 30%~50%. In terms of maximum response acceleration and maximum shear strain, the results of the equivalent linear analysis closely match those of the tests, indicating that equivalent linear analysis can be used with confidence.

Analysis Results (S2 Test)

Figure 12 shows the relationship between maximum response acceleration and depth. A comparison of the test results and analyses shows that for the silty layer the results of both effective stress analysis and equivalent linear analysis accord closely with the test results. In the sandy layer the occurrence of liquefaction causes the results of the effective stress analysis to be slightly higher than the test results, but it simulates the decline in the ratio of the maximum response acceleration.

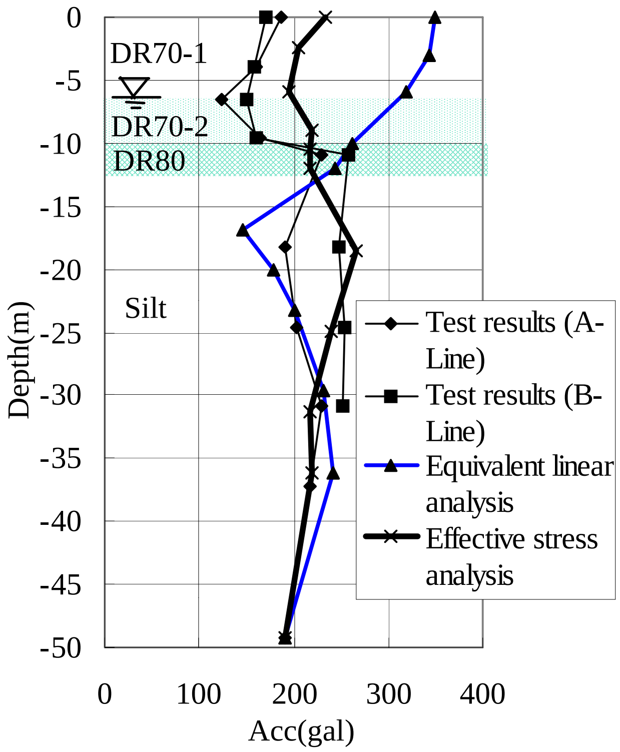

Figure 12 : Relationship between maximum response acceleration and depth for free ground (S2 test)

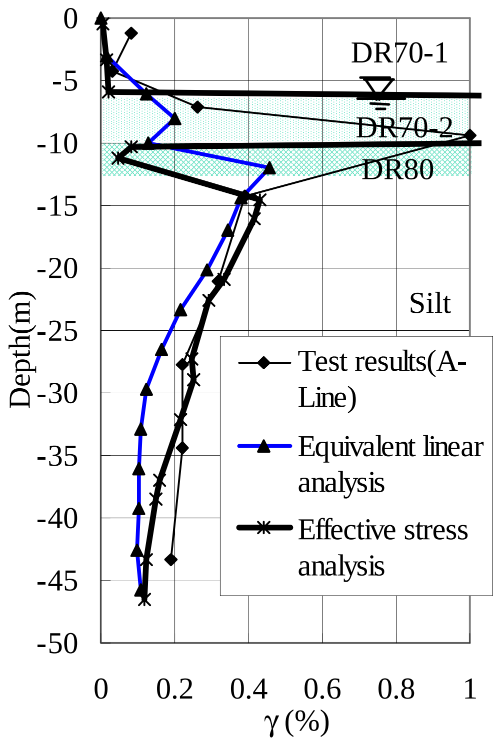

Figure 13 : Relationship between maximum shear strain and depth for free ground (S2 test)

Figure 13 shows the relationship between maximum shear strain and depth. A comparison of the test and analytical results shows that effective stress analysis has simulated the occurrence of liquefaction in the sand layer causing considerable strain. In the silty layer, maximum shear stress through both equivalent linear analysis and effective stress analysis accorded very well with the test results. Determining the degree of decline in shear modulus (G/G_0) in the silt layer in equivalent linear analysis from Figure 8 gives a value between 50%~80%. In terms of maximum response acceleration and maximum shear strain, the results of the equivalent linear analysis of the silt layer closely match those of the tests. Equivalent linear analysis can thus be used with confidence for silt layers where there is no liquefaction or massive strain. Figure 14 shows time history of excess pore water pressure for layers of sand, of Dr=70% at GL -9m. According to effective stress analysis, the sand layer of Dr=70% at GL - 9m is subject to liquefaction. The analytical results closely simulate the time changes leading to liquefaction in the test results.

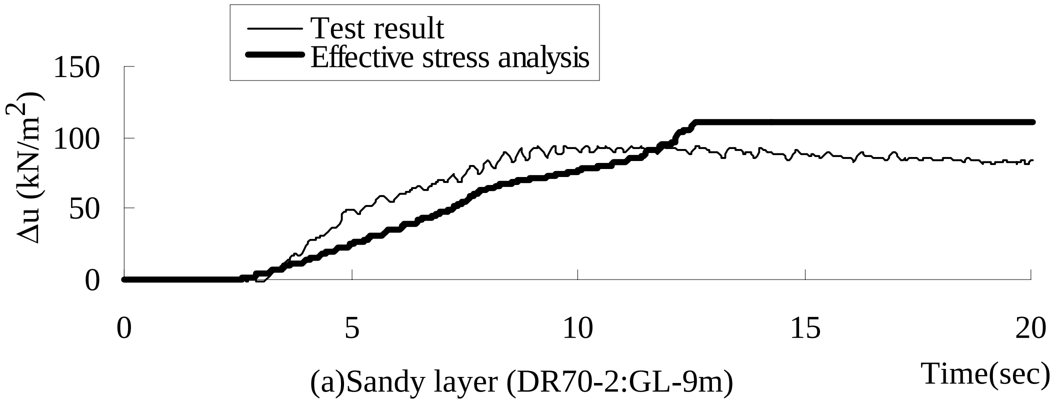

Figure 14 : Time histories of excessive pore water pressure (S2 Test)

CONCLUSIONS

In order to clarify the behavior during a maximum earthquake or an extreme earthquake of hypothetical ground consisting of a sandy layer liable to liquefaction overlaying a thick layer of silt, dynamic centrifuge tests were performed and numerical simulation analyses were made. The following are the main results obtained.

1. In the model tests, an S2 level earthquake prompted liquefaction in the sand layer. In S1 earthquake conditions, liquefaction did not occur as shear strain was low and no increase in excess pore water pressure was observed. It can thus be assumed that in model tests, liquefaction does not occur in the hypothetical ground under circumstances of an S1 level earthquake. However, it is highly likely that liquefaction will occur in the sand layer during an S2 level earthquake and more detailed study is necessary to confirm safety levels.
2. In the model tests, shear strain in the silty layer was 0.2% ~ 0.5% during an extreme earthquake, but for a maximum earthquake the value was lower than 0.2%. The shear strain in this silt layer could be simulated through both equivalent linear analysis and effective stress analysis. The shear modulus of the silt layer is estimated to fall by 50% ~ 80% during an extreme earthquake, and by less than 60% in a maximum earthquake, due to the occurrence of this shear strain.
3. Using effective stress analysis, it was possible to simulate liquefaction of the sandy layer, shear strain in the silt layer and response accelerations in both layers. Effective stress analysis is a valid analytical method for S2 level earthquake conditions, in which shear strain in the ground is relatively large. In the future, it will be possible to use this method for detailed response analysis to S2 earthquake conditions for actual ground in which liquefaction may occur.
4. The behavior of the hypothetical ground during an earthquake can be adequately determined using equivalent linear analysis provided that the shear strain is of an S1 earthquake level.

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