DYNAMIC CENTRIFUGE TEST OF PILE FOUNDATION STRUCTURE PART TWO : BEHAVIOR OF STRUCTURE AND GROUND DURING EXTREME EARTHQUAKE CONDITIONS

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SUMMARY

Dynamic centrifuge tests were performed to investigate the interaction between a hypothetical pile-supported turbine building of a nuclear power station and the surrounding ground during an extreme earthquake. Numerical simulation analyses were undertaken at the same time using the equivalent linear analysis and effective stress analysis methods to study the suitability of numerical analysis. In the tests the response acceleration of the turbine building, the earth pressure during the earthquake, the bending moment of the piles, the response acceleration and pore water pressure in the surrounding ground were measured. It was found that in the tests, the low level of earth pressure acting on the turbine building model and the reduction of ground deformation brought about by the restraining effect of the structure on the surrounding ground together worked to hold off the occurrence of liquefaction (during S2 level earthquake conditions). It was also found possible to simulate results of tests through numerical analysis provided that the soil constants of the model ground are correctly appraised.

DYNAMIC CENTRIFUGE TESTS

Test Method

Dynamic centrifuge tests were performed by using the model of a turbine building on pile foundations. Figure 1 shows the hypothetical turbine building used as the model. It was a 1/100 scale model centrally positioned in free ground the same as that described in the previous paper, as shown in Figure 2. It was not possible to make a model of the complete building so a model was made of one part of it, as shown in Figure 1, was used as the base for the model. It was a near rigid body of stainless steel 30cm long, 30cm wide and 23cm high. The position of its center of gravity as inertia mass in the shaking direction and vertical load per unit area were equivalent to those of a full size turbine building. The piles were made of 1mm thick duralumin pipe with a diameter of 25mm, designed so that their diameter and bending stiffness would satisfy the similarity rule for dynamic centrifuge tests. The pile tops were rigidly attached to the structure and the pile tips were hinged to the base plate of the container. In building the model, the piles were first set in the container and then a normally consolidated layer of silt was prepared using the seepage force method as previously described. Once the silt layer was in place, part of it was removed to allow the turbine building model to be attached to the pile foundations. Finally, the sand layer was created using the air pluviation method previously described.

In the earthquake simulation tests, the response acceleration, pore water pressure, and horizontal displacement of the model ground were measured, as well as the response acceleration of the model structure and the earth pressure acting on its walls, the bending strain on the piles, and the vertical displacement of the ground surface. The input acceleration through sine waves was at two levels, S1 and S2, as before.

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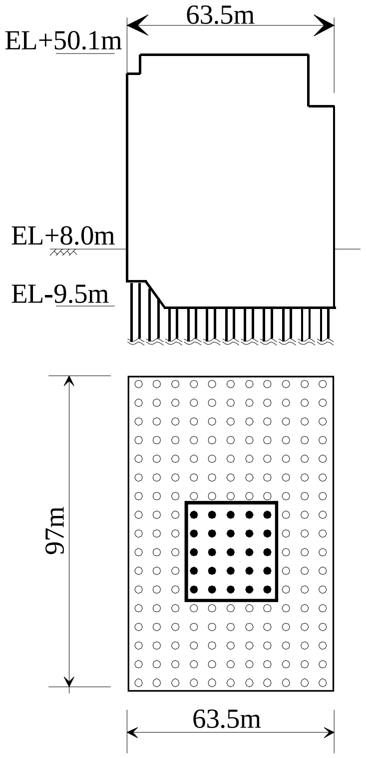

Figure 1 : Hypothetical turbine building used as the model

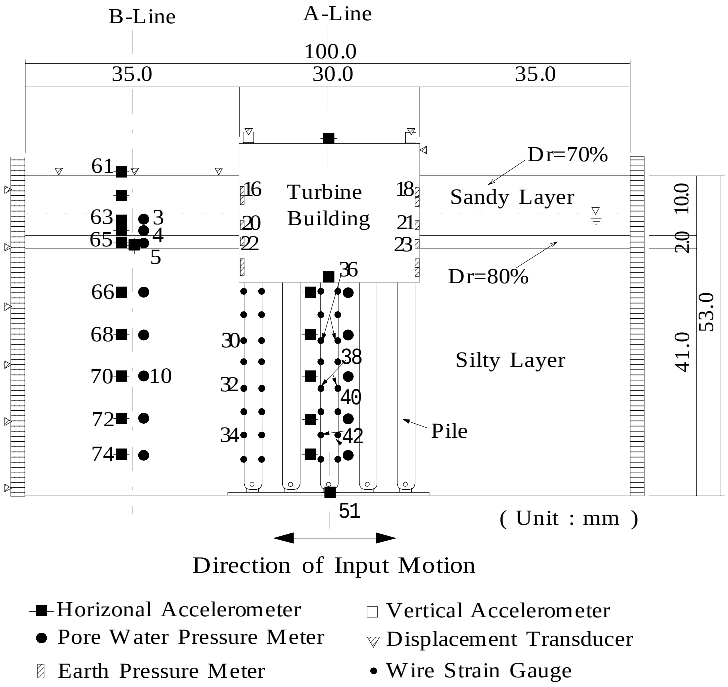

Figure 2 : Centrifuge test model

Test Results

Figure 3 shows the acceleration time history for the sand layer in the S2 test. Figure 4 shows the relationship between maximum response acceleration and depth in S1 and S2 tests. Figure 5 shows the time history of excess pore water pressure for the sandy layer in S2 test. Figure 6 shows the relationship between hydrostatic pressure, maximum pore water pressure and depth for the S1 and S2 tests. No liquefaction was observed in the sandy layer surrounding the model structure, due to the suppression of shear deformation due to the presence of the structure (see Figure 5,6), and there was no decline in response acceleration nor occurrence of lagging in the wave form that were observed in previous S2 test of free ground. In both sets of tests, maximum response acceleration increased from the bottom of the silt layer in the surrounding ground through to its center, but declined from the center to the top of the layer. (As the acceleration in the silt layer was not measured by accelerometers CH66, 68, and 70 in these tests, this cannot be read directly from the figure, but can be assumed from the results of the other tests).

Liquefaction did not occur in the sand layer, so maximum response acceleration increased towards the surface. In the silty layer directly below the model structure maximum response acceleration increased slightly in the lower part but decreased towards the bottom of the structure. In the structure itself, maximum response acceleration increased towards the top but was lower than the input base motion at the top.

Figure 7 shows the relationship between maximum shear strain and depth in S1 and S2 tests. Maximum shear strain in the sand layer was smaller than that of free ground model (part one)[1)] since liquefaction did not occur in this model.

Figure 8 shows the time history of increased earth pressure during shaking in the S2 test. During shaking, the increased earth pressure acted on the two sides of the structure in opposing phases, and the earth pressure at the front of the structure facing the direction of shaking decreased, while that at the back increased.

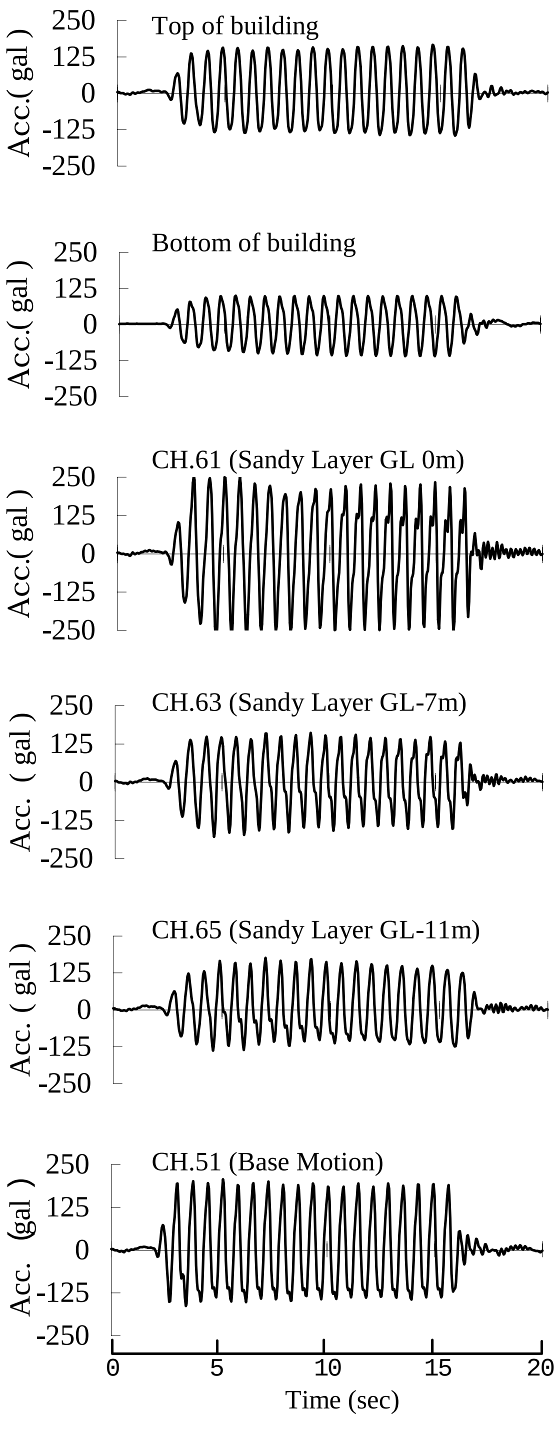

Figure 3 : Time histories of acceleration (S2 test)

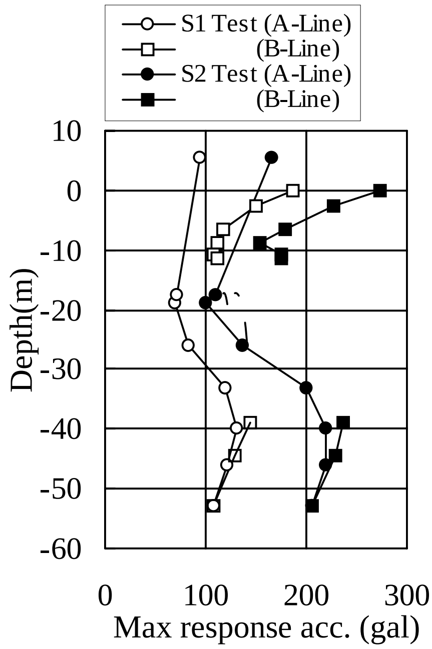

Figure 4 : Distribution of maximum response acceleration

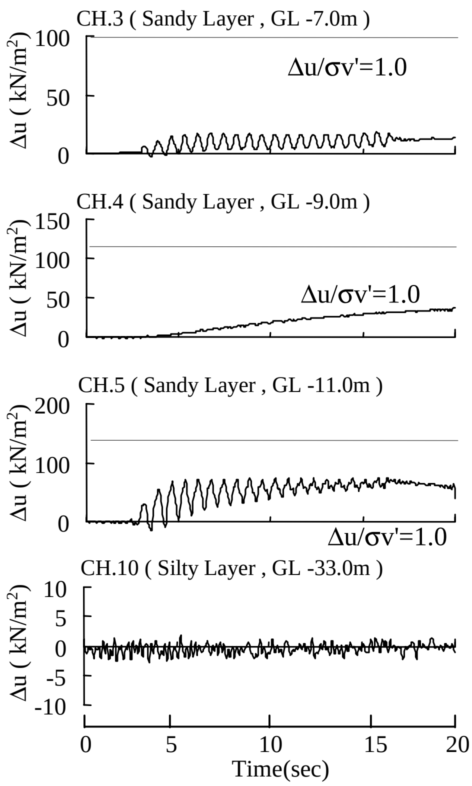

Figure 5 : Time histories of excess pore water pressure (S2 test)

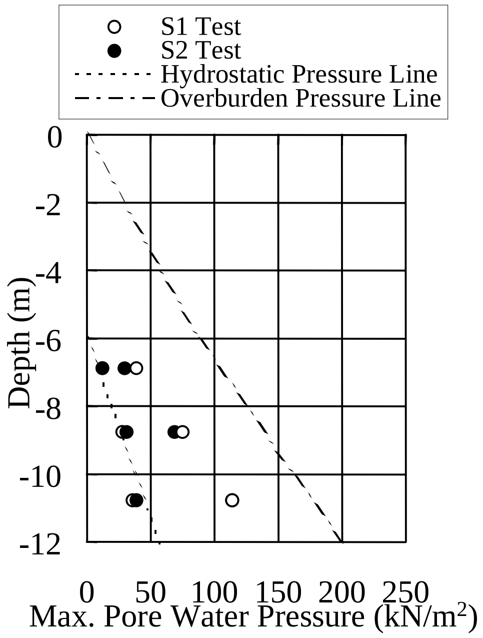

Figure 6 : Distribution of pore water pressure of sandy layer

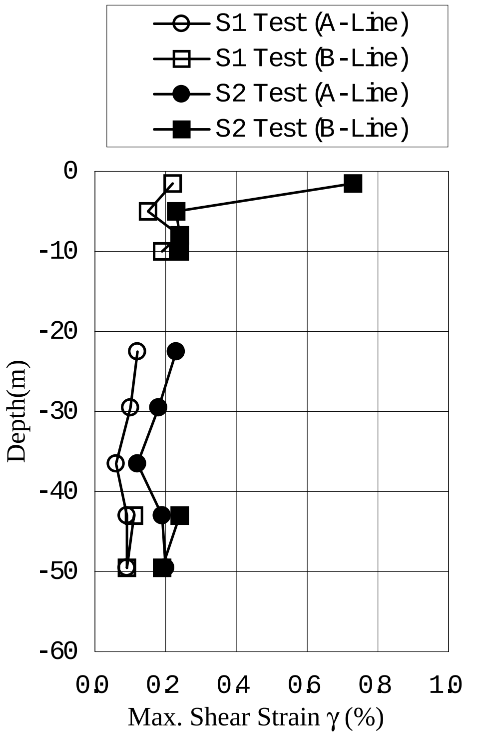

Figure 7 : Distribution of maximum shear strain

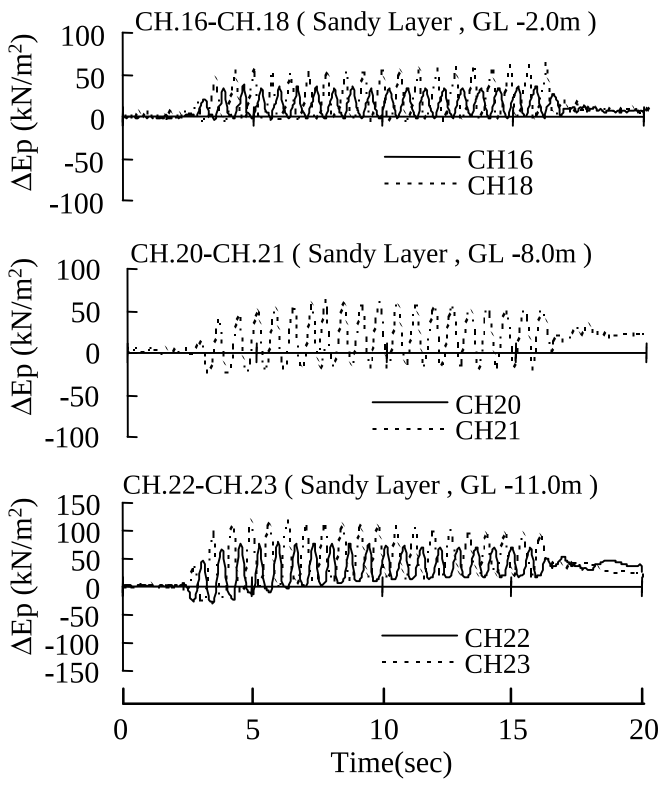

Figure 8 : Time histories of increased earth pressure (S2 test)

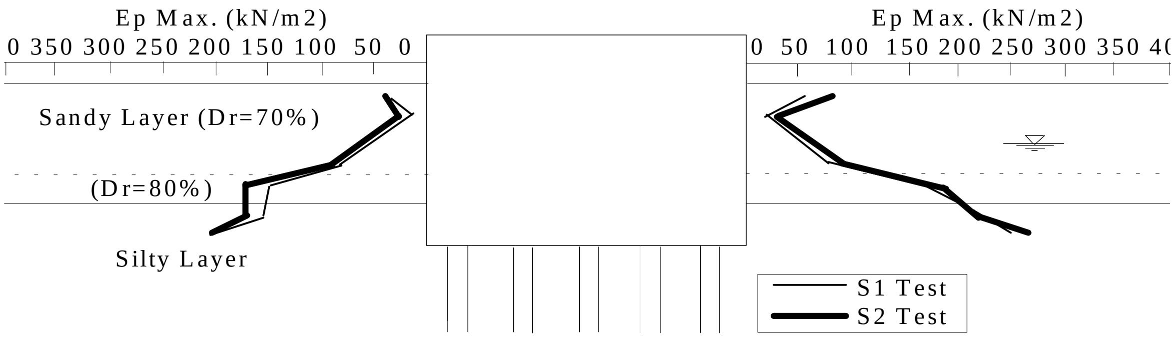

Figure 9 : Distribution of maximum earth pressure

Figure 9 shows the distribution of maximum earth pressure during shaking. Maximum earth pressure during shaking, as shown in the figure, are lower than the passive earth pressure during earthquakes of Mononobe and Okabe using acceleration values obtained in test.

Figure 10 shows the time history of the bending moment on the piles in the S2 test. Figure 11 shows a distribution of the maximum bending moment on the piles in the S2 test. The highest values for bending moment on the piles were at the tops. At the same depth, bending moment on piles around the outer periphery were greater than those on piles at the center. This is thought to indicate the effect of the pile group.

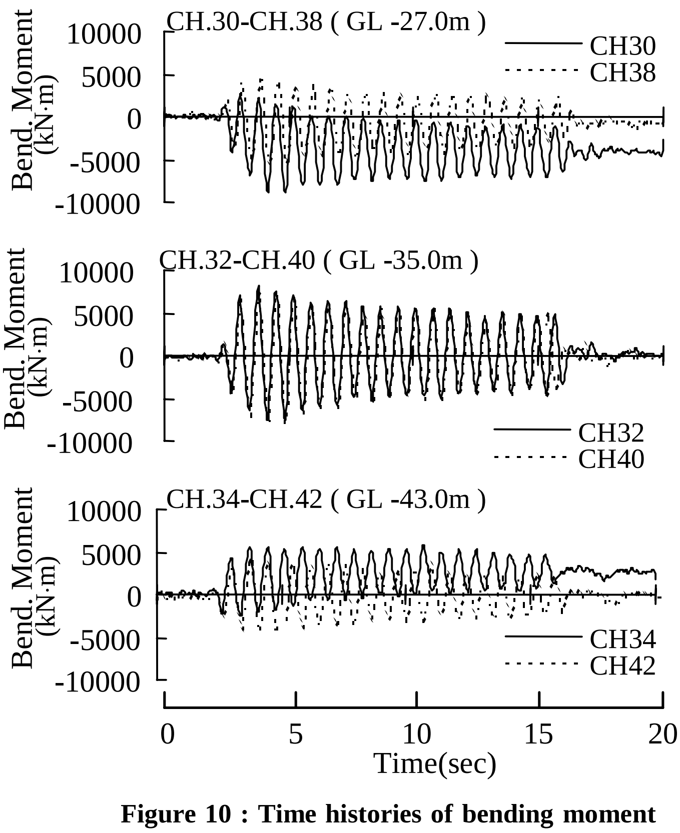

Figure 10 : Time histories of bending moment of piles (S2 test)

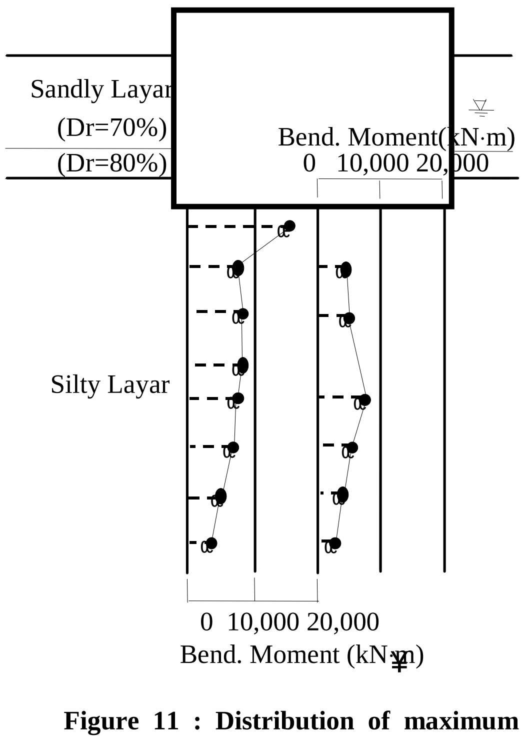

Figure 11 : Distribution of maximum bending moment of piles (S2 test)

NUMERICAL ANALYSIS

Analytical Conditions

The two analytical methods of equivalent linear analysis and effective stress analysis were used as before. The code used for the equivalent linear analysis was SUPER-FLUSH/2D[2), while that used for the effective stress analysis was MuDIAN. Both are two-dimensional FEM analytical methods.

Figure 12 shows the two-dimensional FEM analysis model that includes a building with pile foundations, and the boundary conditions. For modeling, plane strain elements were used for the ground, beam elements were used for the piles, and both plane and beam elements were used for the structure. The mass of the laminar container was replaced by the equivalent mass of boundary nodal points. For the boundary conditions of the analytical model, the bottom plate fixed to the shaking table was taken as a fixed condition, and the boundaries of the laminar container were taken as the horizontal rollers.

The input seismic motion was the input base motion in the S2 test previously described.

The same values were used as in the previous paper for the soil constants and analytical parameters. This paper describes the results of S2 test. In the S2 test, a low level of shear strain, 0.2% ~ 0.4%, was recorded in the ground and there was no occurrence of liquefaction, and so numerical analysis simulation was confined to S2 earthquake conditions.

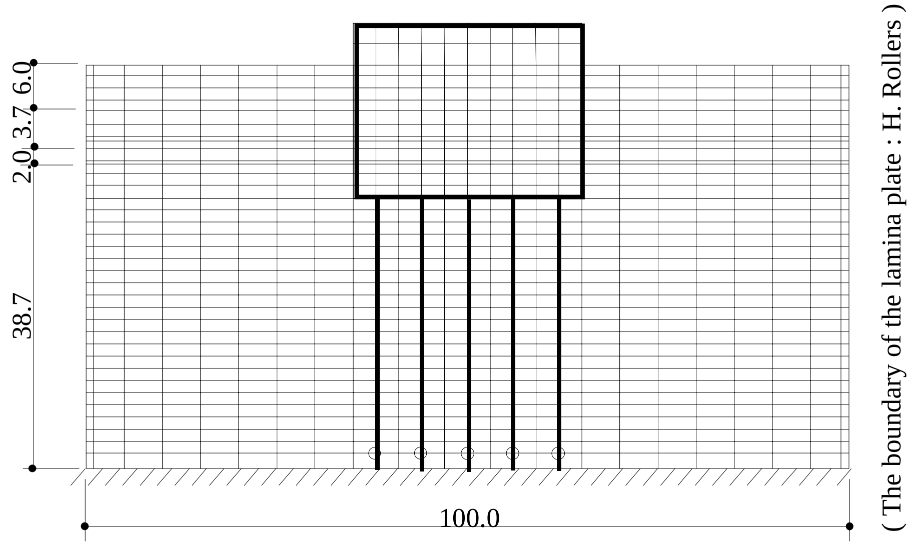

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Figure 12 : Numerical analysis model

Analysis Results (S2 Test)

Figure 13 shows the relationship between maximum response acceleration and depth. A comparison of the test and analysis results for the silt layer shows good agreement for both equivalent linear analysis and effective stress analysis. The results of effective stress analysis agree closely with the test results for the sandy layer.

Figure 14 shows the relationship between maximum shear strain and depth. The test results indicated 0.2% for the sand layer and 0.2% ~ 0.4% for the silt layer, and both equivalent linear analysis and effective stress analysis results matched them closely.

Figure 15 shows the relationship between pile bending moment and depth. A comparison of the test and analysis results shows good agreement for the pile tops. For the mid sections of the piles, the results of the effective stress analysis agree well with the test results. Equivalent linear analysis, however, gave lower values.

Figure 16 shows a distribution of maximum increased earth pressure on the two sides of the structure. The results obtained from both equivalent linear analysis and effective stress analysis gave higher values than those obtained from the tests. This is thought to be because in the tests it was assumed that slippage between the structure sides and the ground would occur during shaking, but in the analysis the boundary between the building and ground uses the same nodal points, and so this phenomenon cannot be simulated.

Figure 17 shows the time history of excess pore water pressure in the sand layer (Dr=70%: GL -9m). In the tests there was no liquefaction and the maximum pore water pressure ratio was 0.4 at a depth of GL -9m. The results of effective stress analysis agreed well with the test results.

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.

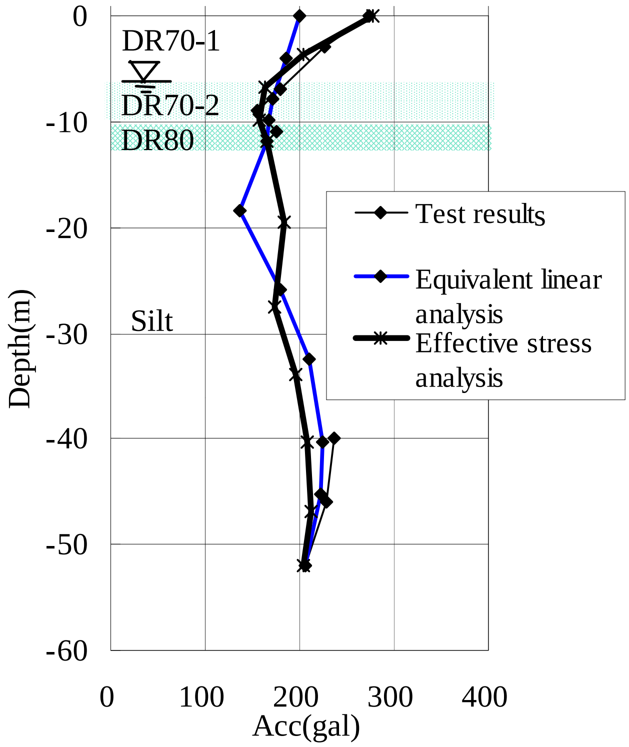

Figure 13 : Relationship between maximum response acceleration and depth for free ground (S2 test)

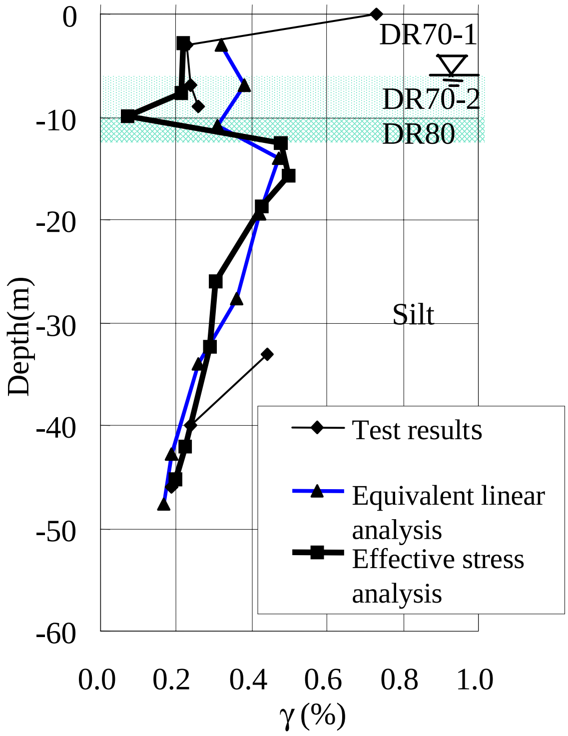

Figure 14 : Relationship between maximum shear strain and depth for free ground (S2 test)

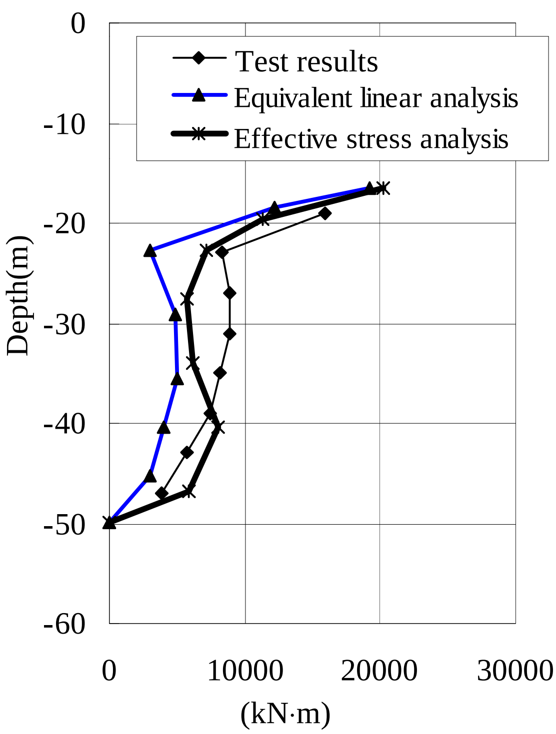

Figure 15 : Relationship between maximum pile bending moment and depth (S2 test)

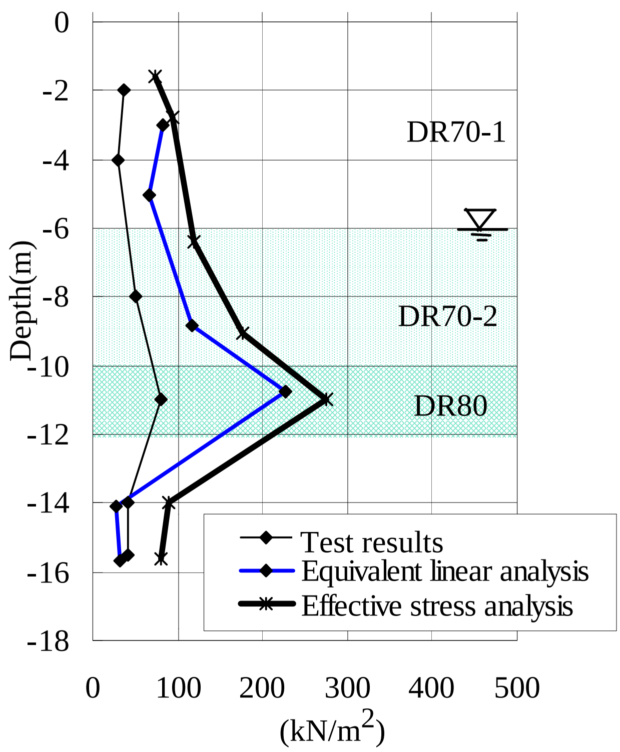

Figure 16 : Relationship between building side increased earth pressure and depth (S2 test)

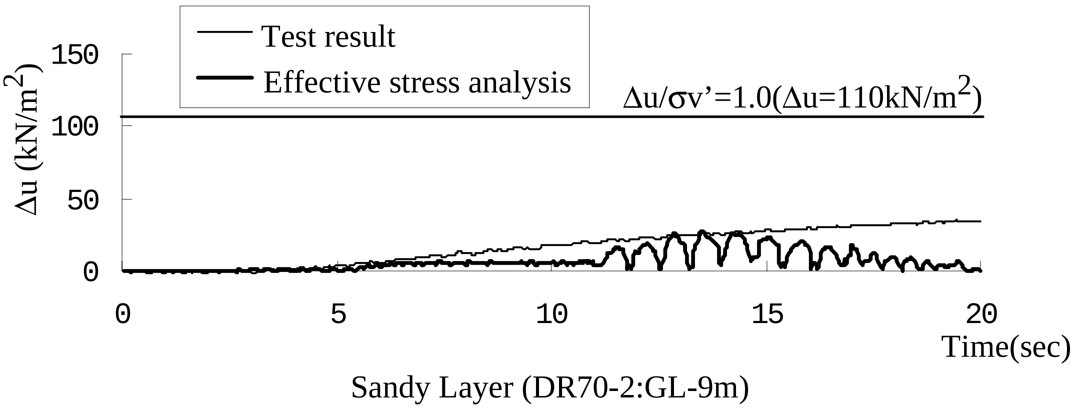

Figure 17 : Time histories of excessive pore water pressure (S2 Test)

CONCLUSIONS

This study assumed the construction of a turbine building of a nuclear power station, set on a pile foundation in ground consisting of a thick layer of deposited silt and a layer of sand, as described in the previous paper. Dynamic centrifuge tests were performed and numerical simulation analyses undertaken. The main results obtained are as follows:

1. No liquefaction was observed in the sandy layer in the tests under S2 level earthquake conditions. This is due to the structure's suppression of shear strain deformation in the sand layer. The same results were obtained through effective stress analysis. It may also be assumed that in actual ground, no liquefaction will occur in a sandy layer surrounding a structure.
2. For response acceleration, shear strain and excess pore water pressure of the model ground, results of the tests and analyses agreed well. This is because the ground was within the elastic behavioral range, and the same conclusion was reached from the results for an S1 level earthquake on free ground.
3. The earth pressure results during earthquakes obtained from the tests were lower than the results from both analytical methods and results obtained using the Mononobe-Okabe equation. This confirmed that these methods of calculating pressure can safely be used in the design of the structure.
4. The pile bending moments obtained in the tests agreed with the values obtained through effective stress analysis. The equivalent linear analysis results gave lower values overall.
5. The results of the equivalent linear analysis and effective stress analysis largely agreed. Either method of analysis is suitable for ground where there is no liquefaction or major deformation, as in these tests.

REFERENCES

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2. Namikawa, T., Togashi, K., Nakafusa, S., Babasaki, R., Hashiba, T., 2000. "Dynamic centrifuge test of pile foundation structure, Part One: Behavior of structure and ground during extreme earthquake conditions", *12th World Conference on Earthquake Engineering*