COUNTERMEASURES AGAINST LIQUEFACTION INDUCED SETTLEMENT FOR POWER TRANSMISSION TOWERS

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SUMMARY

After the 1995 Hyogoken Nambu earthquake, it has been recognized that restoring service facility from the damage is critical for urgent recovery of city function. Among lifelines, power transmission towers are highly important electrical components, some of which are required to survive with only minor damage even after a strong earthquake. In the past earthquakes such as 1964 Niigata earthquake, transmission towers were inclined due to the effects of soil liquefaction.

It is therefore important to develop a method for estimating seismic performance of transmission towers during and after earthquakes. A series of dynamic centrifuge tests were conducted by modeling an isolated footing of a tower on sand. In these tests, four isolated footings were modeled together on a liquefiable sand layer and the effectiveness of three types of countermeasures against settlement due to liquefaction was examined. These countermeasures were designed to connect the four footings to each other, and the types of connections modeled were: 1) a plate at the base of the footing, 2) a plate at soil surface level, and 3) a surface plate with surrounding sheet piles. The seismic performances of these countermeasures were compared with that of a tower without any countermeasures. The results show that the base plate can prevent the settlement by as much as 70 %, and that even the surface plate can prevent the settlement by 30 %. The test results also show that, in the modeled conditions, the effectiveness of the surrounding sheet pile walls was relatively small. This result was considered to be a consequence of an inadequate length of the sheet piles compared with the width of the foundation.

INTRODUCTION

It has been recognized that power transmission towers are very important facilities which affect the post-earthquake recovery of city services. The damages of towers due to soil liquefaction were reported after Niigata earthquake in 1964, as shown in Figure 1. The inclination and the settlement of the towers were the main damages according to the report. The 1995 Hyogoken Nambu Earthquake revealed that much wider types of soil might liquefy due to strong earthquake motions. Power transmission towers sometimes use ‘isolated footing’ type foundations, with each of the four legs of the tower supported on an independent footing. Given the importance of these towers it is important to develop a method for evaluating the settlement of such isolated footings on liquefiable soils during and after earthquakes.

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The authors conducted a series of dynamic centrifuge tests to examine the settlement of power transmission towers during and after liquefaction, as already reported by Kawasaki et al [1][2]. In the first phase of this study, attention was placed on finding factors which govern the settlement of isolated footings. The four main variables in the tests were relative density of soil, magnitude of input motion, number of cycles of motion, and width of the footing. A typical isolated footing designed to support one of the legs of a power transmission tower was extensively tested. Finally, based on the results of the centrifuge tests, a simple method to estimate the settlement was presented. The predicted settlement was expressed in a conventional form by using a number of deviating factors which relate to the above four factors. From the results of more than 30 tests, the following empirical equation was proposed for estimating the settlement of the transmission towers:

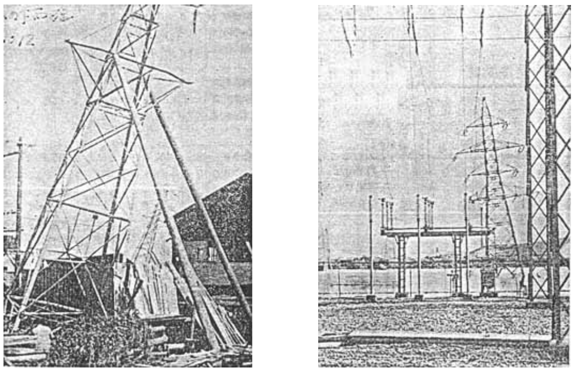


Figure 1: Inclined power transmission towers seen after Niigata earthquake, 1964

$$S = S_o \cdot C1 \cdot C2 \cdots\cdots Cm \cdot D1 \cdot D2 \cdots\cdots Dn \tag{1}$$

where, S is the settlement of transmission tower, S_o is the settlement of tower under particular standard conditions defined below, *C1-Cm* are deviating factors from S_o relevant to the actual ground condition and input motion, and *D1-Dn* are deviating factors from S_o relevant to the specific structure under consideration. In the proposed method, the factors taken into account were: thickness of liquefiable layer (*C1*), thickness of unliquefiable layer (*C2*), relative density (*C3*), grain size (*C4*), factor related to degree of liquefaction (*C5*), width of footing (*D1*) and load intensity (*D2*).

Each deviating factor is shown graphically in Figure 2. In the tests, the conditions set as the standard conditions were:

C1 : 13.2 m, *C2* : 0 m, *C3* : 40 %, *C4* : D_{50} = 0.161 mm (Toyoura sand), *C5* : F_L = 1,
D1 : 3.2 m and *D2* : 28.4 kPa.

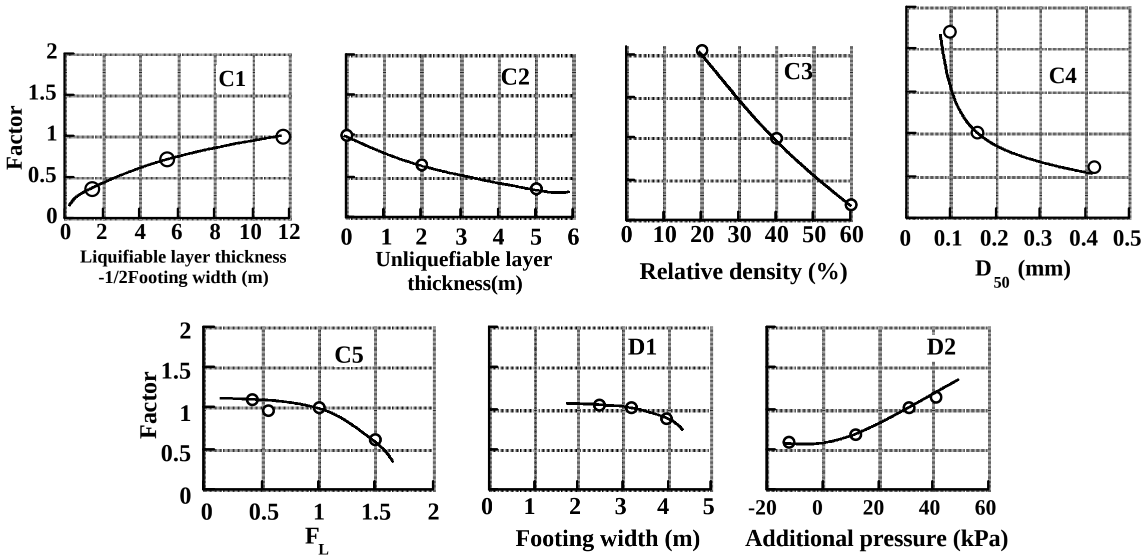


Figure 2: Deviating factors obtained a series of centrifuge tests

It should be noted that, a more accurate estimation of the settlement requires an additional, extensive review of the field observations and an effort is currently being made for refining the proposed method, which will be presented in the near future.

After examining the factors which govern the footing settlement, the authors conducted another series of dynamic centrifuge tests to examine the effectiveness of three methods for reducing the settlement of the

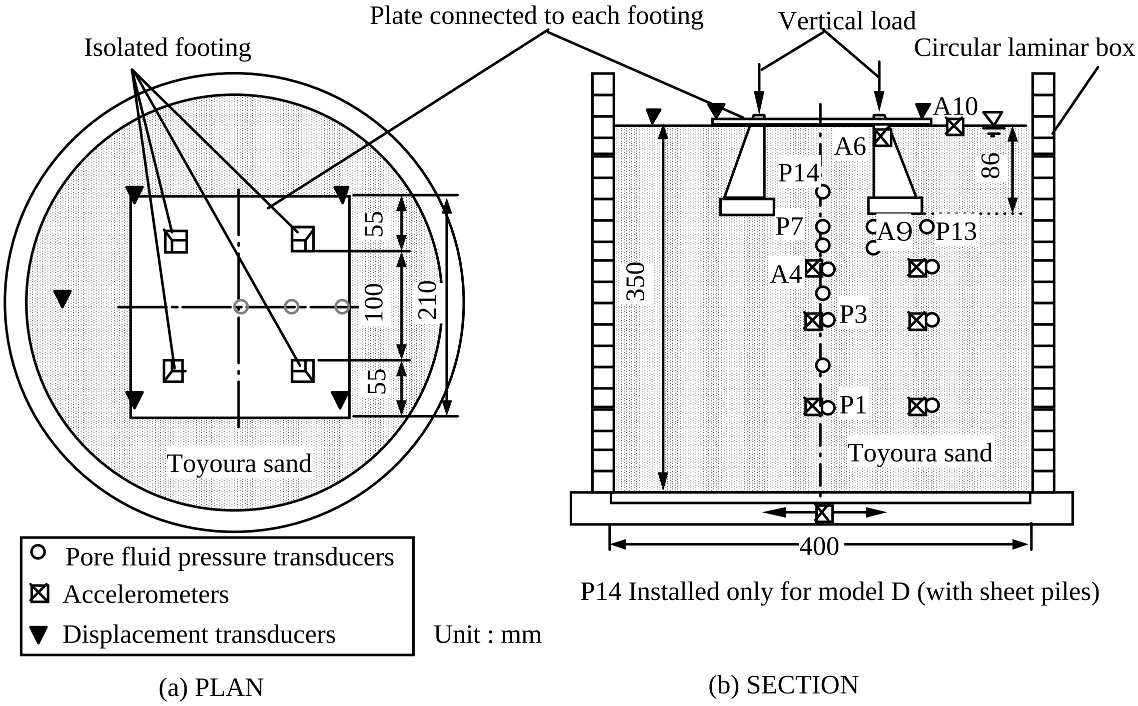


Figure 3: Schematic figure of centrifuge model

footings. The results obtained from the second series of tests are presented in this paper. It should be noted that countermeasures for reducing the settlement considered in this paper are for already existing isolated tower footings, rather than for newly constructed footings.

MODEL DESIGNS AND TEST PROCEDURES

The details of the centrifuge machine used in this study were described by Nagura et al. [3]. The machine has an effective radius of 2.65 m and has an electrohydraulic servo-controlled shaking table designed for conducting dynamic tests. A centrifugal acceleration of 50 g was applied to a 1/50 model (here, 'g' denotes gravity).

Figure 3 shows a schematic figure of the centrifuge setup designed for the study. Four independent footings, which support a transmission tower, were modeled. The model shown in the figure corresponds to 'Model C' in this paper (see Figure 4 in the next section). A circular laminar box with an inside diameter of 400 mm was used as the soil box. The material of the model footing was made by mixing cement, sand and iron powder. The width of the model footing was 50 mm (2.5 m at prototype scale) and a weight of 18.3 tons per footing at prototype scale was applied vertically. The distance between two footings in the model was set at 100 mm (5.0 m). Toyoura sand with a height of 350 mm (17.5 m) was used as a soil sample. The properties of Toyoura sand are summarized in Table 1.

Table 1: Properties of Toyoura sand

Density of soil particle (g/cm^3)	Maximum density (g/cm^3)	Minimum density (g/cm^3)	Mean grain size (mm)
2.661	1.654	1.349	0.162

Dry Toyoura sand was first poured into the soil box. After placing four model footings on the sand and placing the surface sand at the prescribed level, the sand was saturated with silicon oil with a viscosity 50 times higher than that of water. The relative density of the soil was set at about 40-45 % prior to the shaking. Before starting the main shaking test, the model was spun to observe the initial settlement due to a centrifugal acceleration of 50 g. The test models with and without countermeasures for reducing the settlement are shown in Figure 4. The models tested were:

Model A: Without any countermeasures.
Model B: Connecting the base of all footings by a concrete plate.
Model C: Connecting the four footings by a concrete plate at ground surface level.

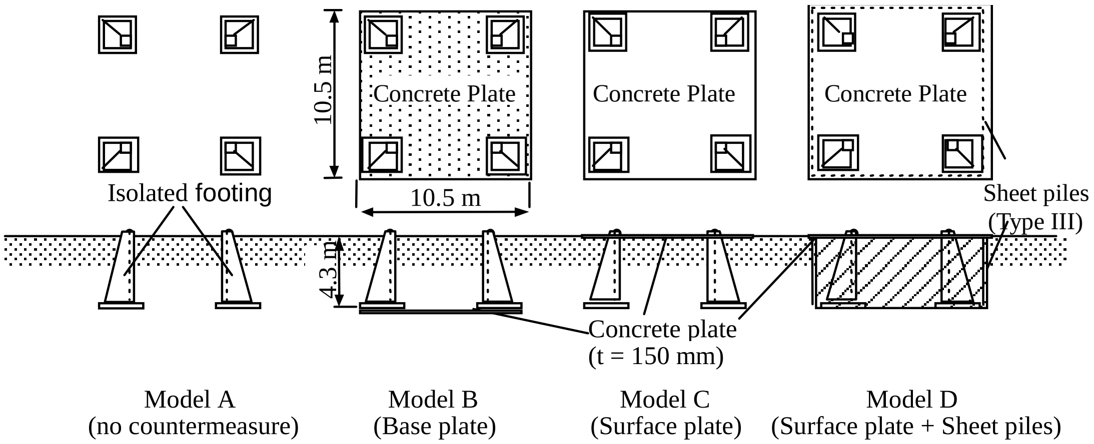


Figure 4: Countermeasures modeled in the study

Model D: In addition to connecting the four footings by a concrete plate at ground surface level, as in Model C, the footings were surrounded by sheet pile walls.

The models A and B were tested and reported by Sakemi et al [4]. The base and surface plates in the models were made by plastic plate with a thickness of 5 mm, which corresponds to a concrete plate of about 150 mm at prototype scale. Sandpaper was attached on the surface of the plastic plate to create a rough surface. The model B was thought to be the most effective as the four footings behave as one single foundation with respect to the soil above the base plate. However, constructing such a base plate of footings of already existing towers during the tower operation is not only difficult but may also be unrealistic.

Other two models C and D were proposed as feasible alternatives to the model B. A photograph of model C is shown in Figure 5. In both models C and D, the plate connecting the four isolated footings was attached at the ground surface level, which is not only much easier but is, also, more economic. It was considered that the use of a surface plate would reduce the stress transferred to the isolated footings as the plate itself would provide additional resistance against settlement of the tower. Model D is similar to model C but in addition to the surface plate, sheet pile walls were placed around the four footings. The bending rigidity of the sheet piles was set equivalent to Type III sheet piles in Japan.

The above models were excited by a sinusoidal horizontal wave with 20 cycles at 50 Hz which is equivalent to 20 cycles at 1 Hz for the prototype model. The motion shown in Figure 6 was applied to the base of the model. The peak acceleration corresponding to the motion was set at 4.5 g at model scale (90 gal at prototype scale). During the tests, pore fluid pressure, acceleration and the settlement of the soil surface were measured as well as the settlement of the model footing. The locations of the transducers are shown in Figure 3.

TEST RESULTS

Acceleration Response:

The observed acceleration responses for the models A and C are compared in Figures 7 to 9. Figure 7 shows the response of the accelerometer A10 (at the soil surface, see Figure 3), Figure 8 shows the response of A9 (on soil below near the center of the model), and Figure 9 shows the response of A6 (in the model footing). In all three figures, the acceleration response did not fade away right up to the end of the shaking even though the soil liquefied significantly.

Figure 5: Photograph of Model C

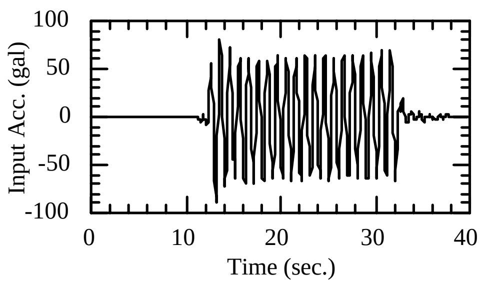


Figure 6: Input acceleration

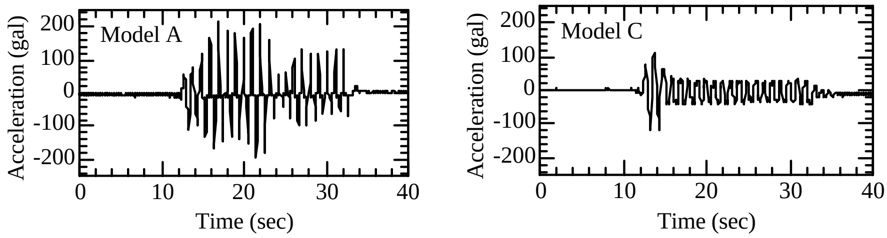


Figure 7: Acceleration response observed at soil surface (Models A & C, A10)

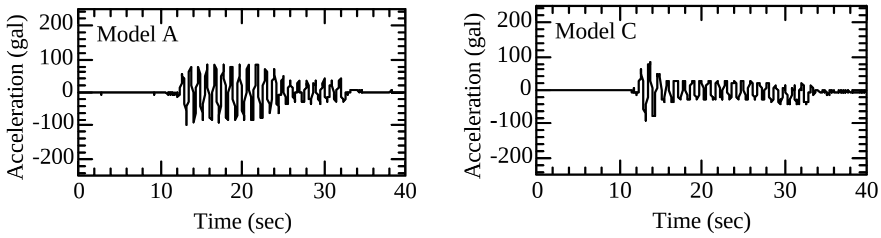


Figure 8: Acceleration response observed below the center of footing (Models A & C, A9)

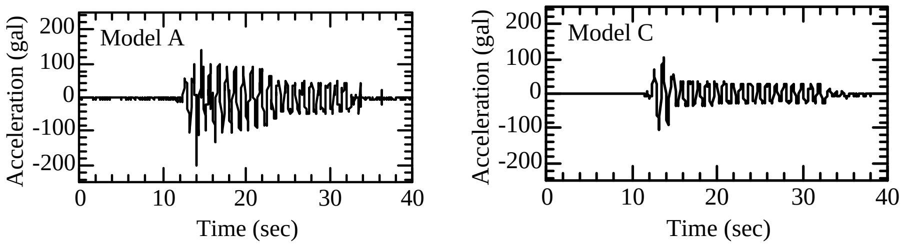
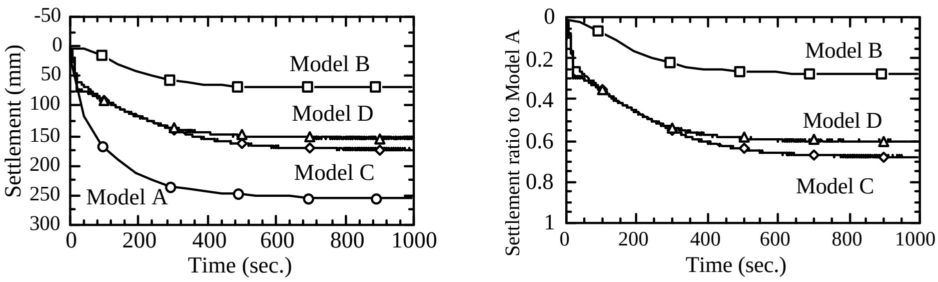


Figure 9: Acceleration response in the model footing (Models A & C, A6)

Figures 7 and 8 show clearly the difference between the acceleration responses of models A and C. The response of the model D was close to that of the model C. The magnitude of the acceleration of model A was higher than those observed in the other three models, and the response of model A was also more complex. Since each footing was set independently in model A, the acceleration response of the four footings was considered not to be completely the same, and this might influence the acceleration response of the soil around the model. It was also considered that, models B, C, and D, in which the four footings were connected together, behaved as a single unit, which could lead to a much higher effective mass in these models than in model A.



(a) Observed settlement at prototype scale (b) Settlement ratio to Model A

Figure 10: Settlement of model footings

Settlement

Figure 10 and Table 2 summarize the settlement of the models during and after the shaking at prototype scale. The settlement shown in the figures and the table is the average of the four footings settlements. In the figure, only the initial 20 seconds corresponds to the shaking period, i.e. the settlement continued after the shaking. The following observations can be made from Figure 10:

- As expected, model B, in which the connecting plate was attached at the base of the footings, was the most effective way of reducing the settlement. The final settlement measured in model B was as low as 80 mm, compared with that of 250 mm in model A (without countermeasure), i.e. the settlement reduced by about 70 %. The settlement during the shaking period itself was significantly smaller in model B than those observed in the other models. An explanation of this behavior is the much smaller pore fluid pressure ratio beneath the model, as shown later in this paper.

- Although the measured settlements of models C and D were larger than that of model B, their settlements were about 2/3 of that of model A, i.e. the settlement was reduced by about 30 % by connecting the four independent footings at the ground surface level.
- The results of the tests in this study showed the effect of the surrounding sheet piles on the settlement to be small. The final settlement in model D was about 61% of model A. It was considered that the relatively short length of the sheet piles used in model D could be the reason for the small difference in the final settlements of models C and D.

Table 2: Settlement of model footings

Model	Relative Density (%)	Maximum input acceleration (gal)	Settlement during shaking (mm)	Final settlement	
				Footing (mm)	Soil surface (mm)
A: Without any countermeasures	39.6	91	200	250	Not measured
B: Base plate	43.7	93	75	75	80
C: Surface plate	45.5	88	70	174	81
D: Surface plate + Sheet piles	45.1	86	64	154	55

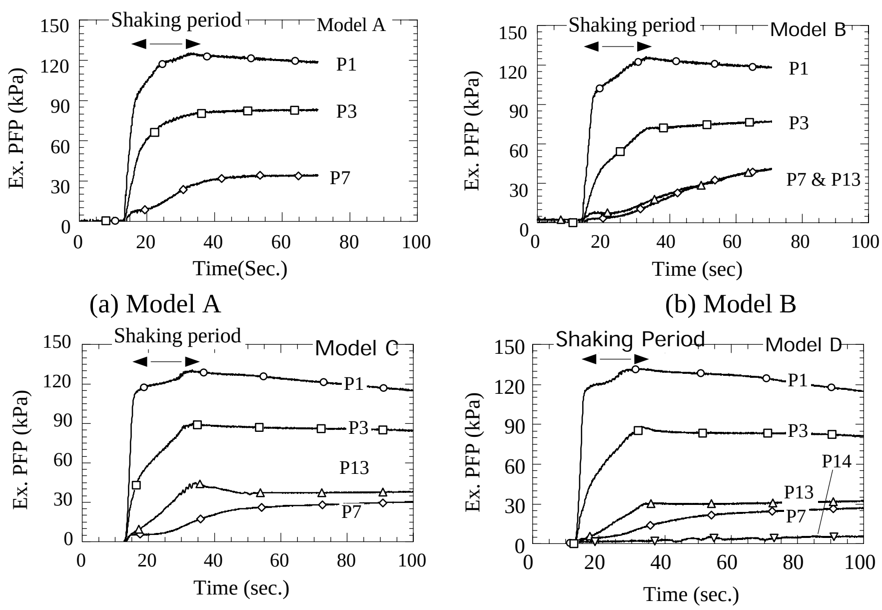
Input acceleration and settlement are shown at prototype scale

Pore Fluid Pressure

The pore fluid pressure responses of the four models at various locations are shown in Figure 11. The figures show that the rate of increase in the pore fluid pressure to be relatively slower in models A and B compared to models C and D. This difference in behavior was considered to be the result of the slight difference in the shape of the created input motions. The following observations can be made from the figures:

- As shown later, the excess pore fluid ratio outside the influenced area seemed to increase close to unity, indicating that the soil in this area liquefied significantly.
- Although the instrumented depths for transducers P7 and P13 were the same, the increase in the excess pore fluid pressure was smaller for P7 in the models C and D. In the case of model B, the rate of increase in the pore fluid pressure was slightly slower compared with the other models. This might be related to the very small settlement of the footing during and after the shaking, which was caused by higher confining pressure.
- The increase in the pore fluid pressure for P14 in model D was considerably small. This may be as a result of the sheet pile walls around the model footing which prevented an increase.

Figure 12 shows the pore fluid pressure ratio at a depth just beneath the model footing for models A, B and D. The effective overburden pressure was estimated by using Boussinesq solution. Note that, for model D, the



(a) Model A (b) Model B

(c) Model C (d) Model D

Figure 11: Pore fluid pressure response observed for Models A - D

effects of the load transferred directly from the surface plate were ignored for simplicity in calculating the initial overburden pressure. The peak pressure ratio for model A (without any countermeasures) was close to unity and model B shows the lowest value of about 0.5. Figure 12 clearly shows that the lower the pore fluid pressure ratio below the model footings was, the smaller the footing settlement became.

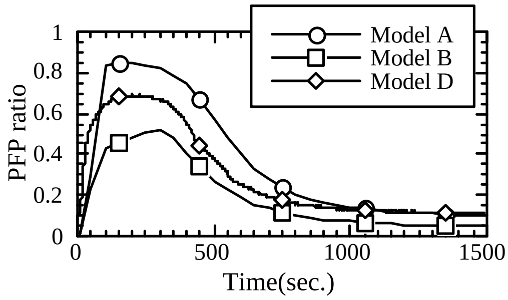
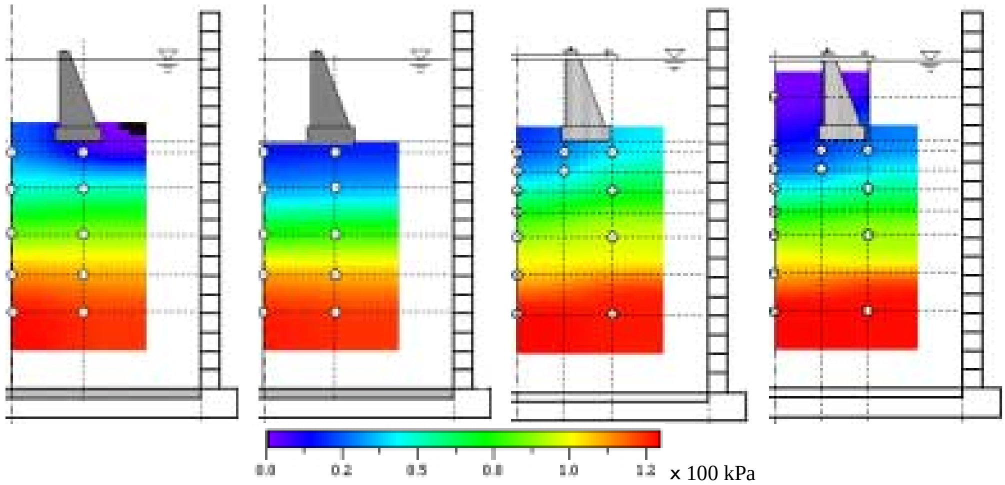


Figure 12: Pore fluid pressure ratio observed beneath the model footings

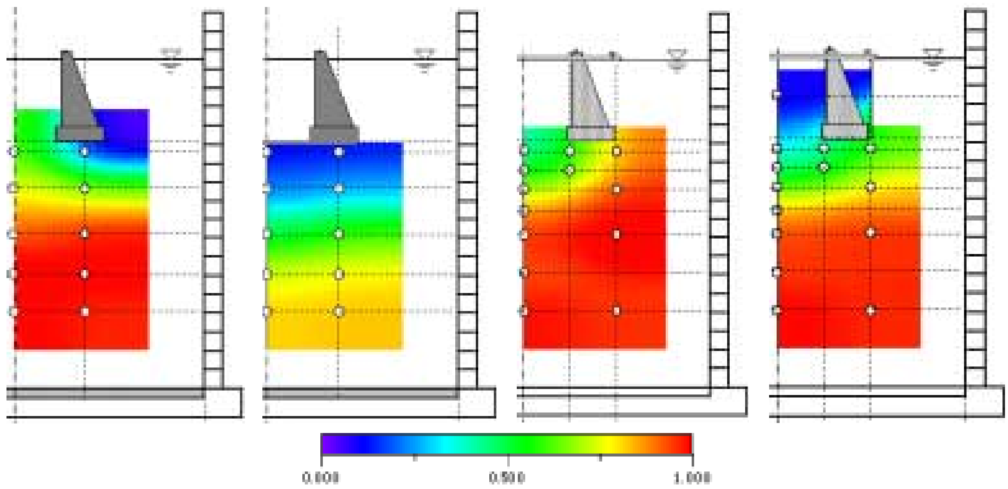
It is also very important to relate the pore fluid pressure response with the observed settlement behavior. The distribution of pore fluid pressure around the model footing just after the shaking is drawn for each model in Figure 13. The corresponding pressure ratio is also drawn in Figure 14. A point of caution regarding these figures, the contours were created from a limited number of pressure transducer readings so that the profiles far from the location of the transducers might not be correct. The small circles shown in the figures indicate the location of the pore pressure transducers. Figure 12 indicates that in all four models the soil near the bottom of the box liquefied significantly. The magnitude of the pore fluid pressure ratio of model B in Figure 14 can be seen to be very small due to the effects of the load transferred through the base plate of the model. The pore pressure ratios for models C and D showed a more centered area with a low ratio as there was significant load transfer through the connecting plate at the surface.

The distribution of pore fluid pressure ratio when the settlement increase rate decreased is shown in Figure 15. During these tests, it was found that, after the pore fluid pressure ratio decreased to about 0.6 - 0.7, there was a



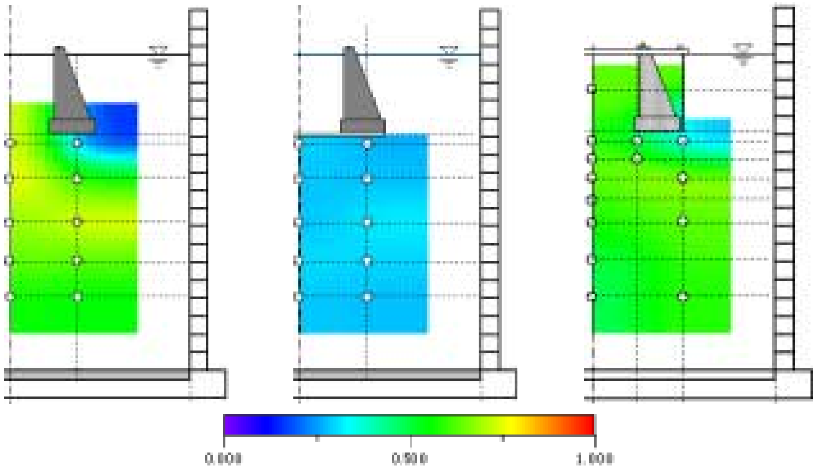
(a) Model A (b) Model B (c) Model C (d) Model D

Figure 13: Contour of pore fluid pressure just after the end of shaking



(a) Model A (b) Model B (c) Model C (d) Model D

Figure 14: Contour of pore fluid pressure ratio just after the end of shaking



(a) Model A (b) Model B (c) Model D

Figure 15: Contour of pore fluid pressure ratio when the degree of settlement increase dropped

decrease in the rate at which settlement increased. This trend was consistently observed throughout all the tests including the first series in this study.

CONCLUSIONS

A series of dynamic centrifuge tests were conducted on isolated foundations of power transmission towers to examine the effectiveness of countermeasures against the settlement due to liquefaction. Three types of countermeasures were tested: in the first the four footings were connected at base level, in the second the connection was made at ground surface level and in the third the footings were connected at ground surface level and in addition sheet piles were installed around the footings. The following conclusions were drawn:

1. The most effective method of reducing the settlement of the footing was to connect the four footings at base level. The settlement was reduced by as much as 70 % compared to the case without any countermeasures. In this method, the soil inside the four footings possibly behaved as one unit together as like one large footing, and the pressure transferred to the soil beneath the plates worked effectively to reduce the pore fluid ratios in the soil.
2. Even using a method to connect four footings at ground surface level can reduce the settlement by about 30 %. This was because the surface plate could be a resistant structure against the settlement, and thus the load transferred to the soil through the footings reduced. The pore fluid pressure ratios inside the four footings might also be smaller, which led to the model behavior more likely as a unit together with sand.
3. Under the testing conditions in this paper, the effects of the sheet piles on the settlement reduction were small. It may be necessary to examine the effects of the length of the sheet piles on the settlement.
4. Considering actual site conditions which are generally seen in Japan, the existence of an unliquefiable layer at the soil surface may influence the settlement behavior, especially when a surface plate is used as a countermeasure as used in models C and D. These effects will also be examined in the next stage.

ACKNOWLEDGEMENTS

The authors would like to express sincere appreciation to Mr. T. Fujiwara of Taisei Corporation for his great assistance during the tests. This study was organized by the Association for Development of Earthquake Prediction. The authors also would like to express sincere appreciation to the members in the committee for the fruitful discussions and suggestions.

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