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SYNOPSIS: To study the mechanism of permanent ground displacement due to liquefaction, shaking table tests were conducted. Moreover, vane tests and cyclic torsional shear tests were carried out to measure the rate of decrease of the elastic modulus and the shear strength due to liquefaction. Based on these tests results, the authors proposed a simplified procedure for the prediction of permanent ground displacement. Measures to counter permanent ground displacement were also discussed based on the analysis.

INTRODUCTION

Recently, Hamada et al.(1986) clarified that extremely large ground displacements, up to several meters, occurred in ground liquefied during the 1964 Niigata Earthquake and the 1983 Nihonkai-chubu Earthquake though the ground surface was almost flat. The authors conducted shaking table tests, vane tests and cyclic shear tests to study the mechanism of ground displacement and to ascertain the rate of decrease of the elastic modulus and the shear strength. Based on these tests, a simplified procedure for forecasting permanent ground displacement was proposed. To confirm the accuracy of this method, it was applied to models of the shaking table tests and to two typical cross sections in Niigata City, and the analytical results were compared with the actually observed measurements. Finally, countermeasures against permanent ground displacement were discussed.

SHAKING TABLE TESTS

Shaking table tests were conducted on 19 soil models to study the mechanism of permanent ground displacement due to soil liquefaction. The shaking table used was 1 m in length and 1 m in width in plane. The soil container was 80 cm in length , 65 cm in depth and 60 cm in width , as shown in Fig.1. The front wall of the container was made of glass, through which the deformation of the soil could be observed. Foam rubber of 5 cm in thickness was inserted inside both walls to induce uniform cyclic shear strain in a soil model during shaking.

A soil model consisted of two sand layers : an upper layer of loose sand which would liquefy during shaking, and a lower layer of dense sand which would not liquefy during shaking. The same kind of sand was used for both sand layers ; however, the method of compaction used for each layer was different . First, the lower sand layer was compacted by shaking at 300 gals of acceleration for two minutes. Then, the upper sand layer , which will be called the "liquefied layer" hereafter, was passed through a sieve in air or in water.

Two types of sand were used for the tests: (1) very clean sand taken from a beach in Yamaguchi Prefecture and passed through a sieve with a mesh of 0.42 mm, and (2) silty sand taken from a hill in Chiba Prefecture. Grain size distribution curves of the sands are shown in Fig.2. The fine contents of the two sands were 0 % and 6 % ,respectively.

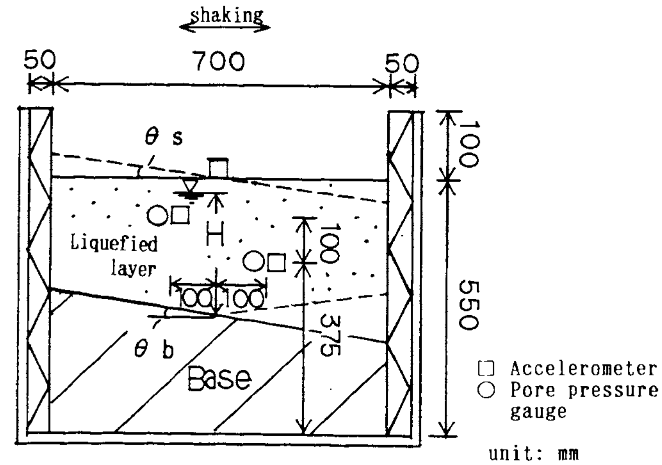

Fig.1 Soil container and model of shaking table test

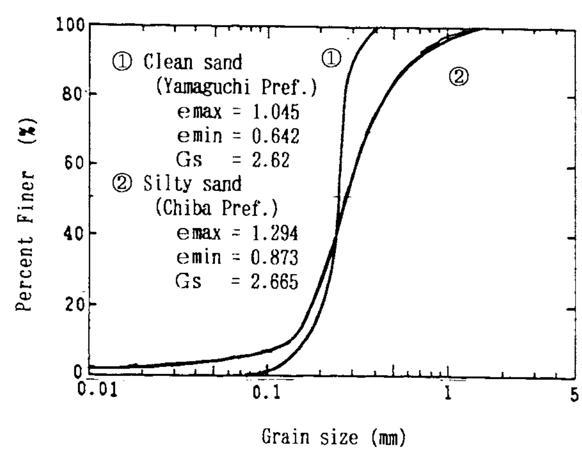

Fig.2 Grain size distribution curves two sands

Models tested were classified into five series, as shown in Fig.3. :

A. the ground surface is flat , but the bottom surface of the liquefied layer is sloped,

B. both the ground surface and the bottom surface of the liquefied layer are sloped in the same direction,

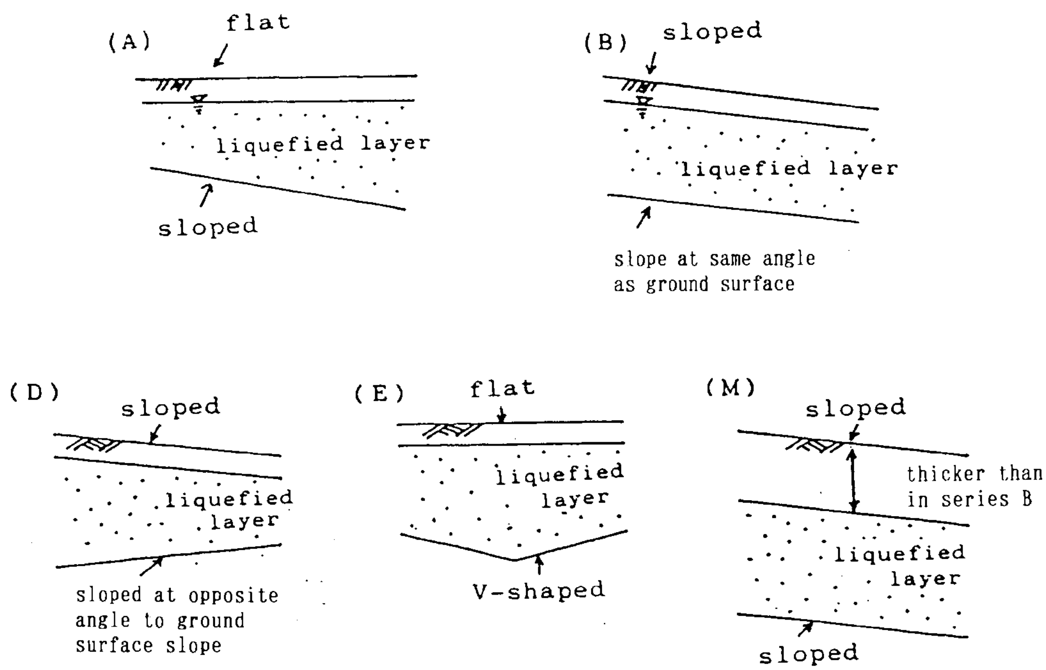

Fig.3 Model types

Table 1 Test conditions for series A, B and C

Case No.	Soil	H (cm)	θ_b (%)	Dr (%)	Density
A-2	Clean sand	17.5	5	47	medium dense
A-3	Clean sand	17.5	10	52	medium dense
A-4	Clean sand	7.5	5	50	medium dense
A-5	Clean sand	27.5	5	56	medium dense
B-2	Clean sand	17.5	5	66	medium dense
B-3	Clean sand	17.5	10	56	medium dense
B-4	Clean sand	7.5	5	66	medium dense
B-5	Clean sand	27.5	5	50	medium dense
B-2-L	Clean sand	17.5	5	-37	very loose
B-2-M	Clean sand	17.5	5	45	medium dense
C-2	Silty sand	17.5	5	62	medium dense
C-2-L	Silty sand	17.5	5	0	very loose
C-2-M	Silty sand	17.5	5	46	loose

H : Thickness of the liquefied layer
θ_b : Slope of the bottom surface of the liquefied layer

Table 2 Test conditions for series D, E and M

Case No.	Soil	θ_b (%)	θ_s (%)	Dr (%)	Density
D-1	Clean sand	10	0	52	medium dense
D-2	Clean sand	10	2	89	medium dense
D-3	Clean sand	10	10	67	medium dense
E-1	Clean sand	5	0	-34	very loose
E-2	Clean sand	10	0	-43	very loose
M-1	Clean sand	5	5	8	loose

θ_s : Slope of the ground surface

D. the ground surface and the bottom surface of the liquefied layer are sloped in opposite directions,
E. the ground surface is flat and the bottom surface of the liquefied layer is sloped toward the center, and
M. the ground surface and the bottom surface of the liquefied layer are sloped in the same direction, but the top surface of the liquefied layer is deep.

Several slopes of the surface or the bottom surface of the liquefied layer and two densities were adopted for each series. Test conditions for all models are shown in Table 1 and Table 2.

Shaking motion was applied in one direction parallel to the horizontal axis in Fig.1 , at a frequency of 3 Hz and with appropriate accelerations to induce liquefaction after several cycles of shaking. The shaking was finished 10 seconds after liquefaction. Acceleration and excess pore water pressure during the shaking were measured at two points, as shown in Fig. 1. Displacement at several points in the liquefied layer was measured by two methods : (1) deformations of nine noodles, placed vertically in the soil at the front of the model, just behind the glass, were measured by photo at intervals of 2.5 seconds during shaking, and (2) displacement of 30 pins on the ground surface was measured by a scale after stopping the shaking.

Fig.4 shows typical deformations of noodles 10 seconds after liquefaction. These displacements must be correlated with real displacement of soil. Among the five series, the largest displacements were observed in series B, as shown in Fig.4(b). This figure shows that displacement developed gradually after the occurrence of liquefaction. The displacement increased linearly in a vertical direction from zero at the bottom surface of the liquefied layer to a maximum value at the ground surface. This means that displacement did not occur at the boundary between the liquefied layer and the non-liquefied layer, but occurred with a constant shear strain in the liquefied layer. In series A, shown in Fig.4(a), displacements were not so large. However , it

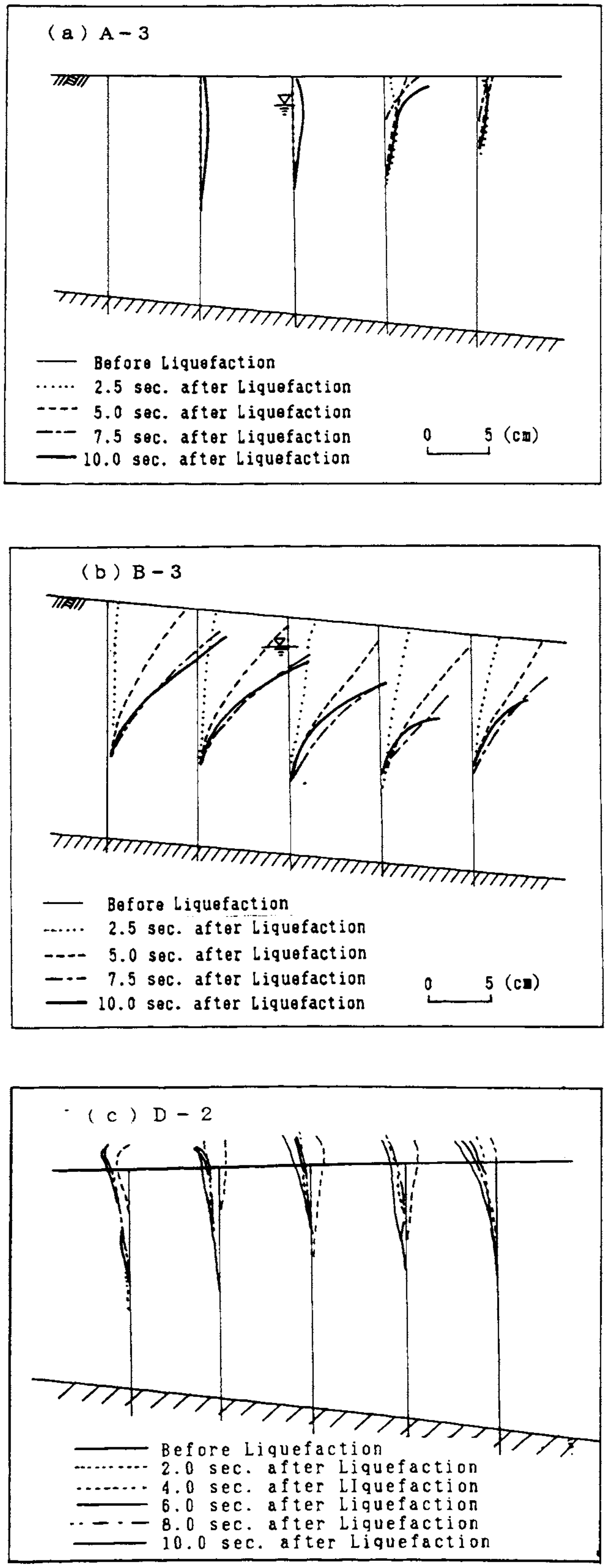

Fig.4 Deformation of the noodles

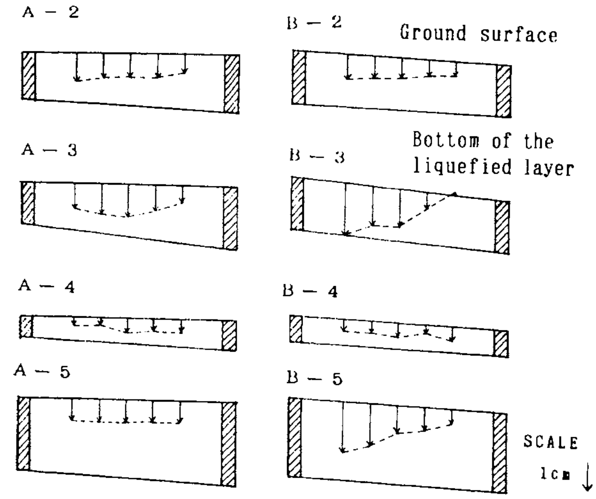

Fig.5 Settlement of the ground surface

seems that displacement was induced from the bottom to the middle part of the liquefied layer, toward the direction of slope of the bottom surface of the liquefied layer. Displacements near the ground surface were small. On the contrary , in series D, displacements toward the slope of the ground surface occurred. In series E, though not shown in Fig.4, it seemed that displacements concentrated slightly toward the central part ; however, this tendency was not clear , because the amounts were small. In series M, displacements of the ground surface were smaller than those in series B. Fig.5 shows the settlement of the ground surface after stopping the shaking. In series B, the settlement at the lower part of the surface was less than that at the upper part. This means that settlement, due to compaction, and uplift of the ground surface was induced in the lower part, due to movement of soil from the upper part. Fig.6 and Fig.7 show the relationship among the thickness of the liquefied layer H, slope of the ground surface θ_s , slope of the bottom surface of the liquefied layer θ_b , and displacement on the ground surface D, which is an average value in the central part of the ground surface. Solid lines in the figures show the trends of the relationship for the clean and medium-dense sand. D increases with H or θ_s , θ_b . According to Fig.6 and Fig.7 , the displacement of loose sand was greater than that of medium dense sand , and the displacement of silty sand was less than that of clean sand. According to Hamada et al. (1986) , the following relationship was derived from observations of the 1964 Niigata Earthquake and the 1983 Nihonkai-chubu Earthquake :

$$D = 0.75 \sqrt{H} \sqrt[3]{\theta} \tag{1}$$

Comparing the effect of H or θ on D,as calculated by Eq.(1), with the solid lines in Fig.6 and Fig.7, the effect of H or θ measured in this test was stronger than expected from Eq.(1). However,the soil container used was small.

According to these results, it is surmised that permanent displacement of the ground would not occur at the surface of a rupture due to a slide, but would occur through out a liquefied layer because of a fall of shear strength due to liquefaction.

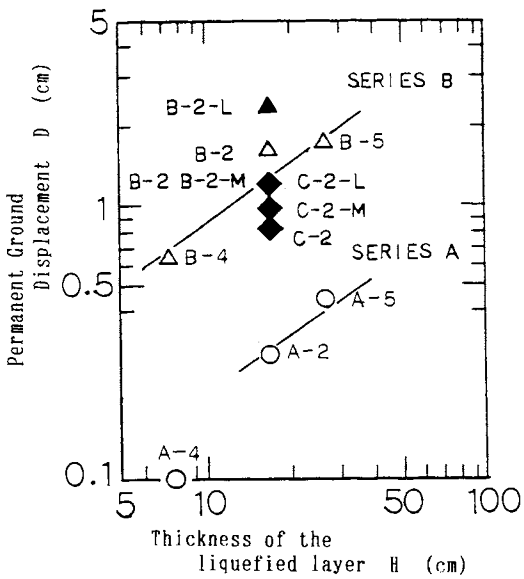

Fig.6 Relationships between D and H

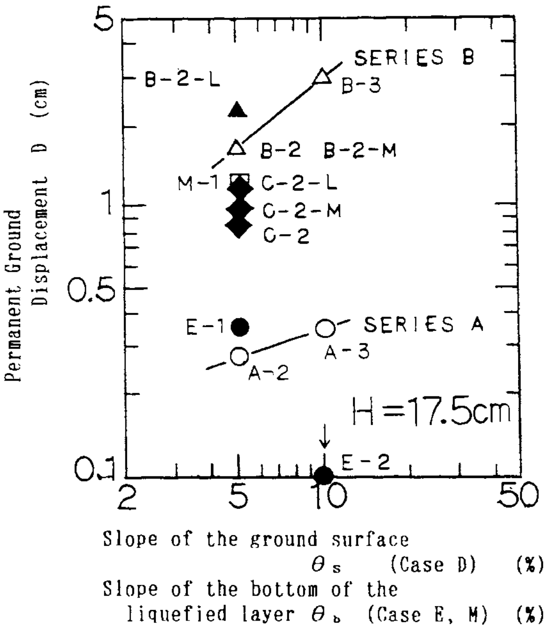

Fig.7 Relationships between D and θ

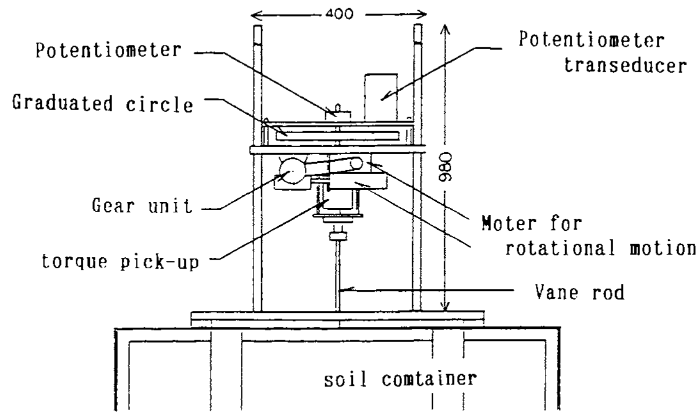

Fig.8 Vane shear test apparatus

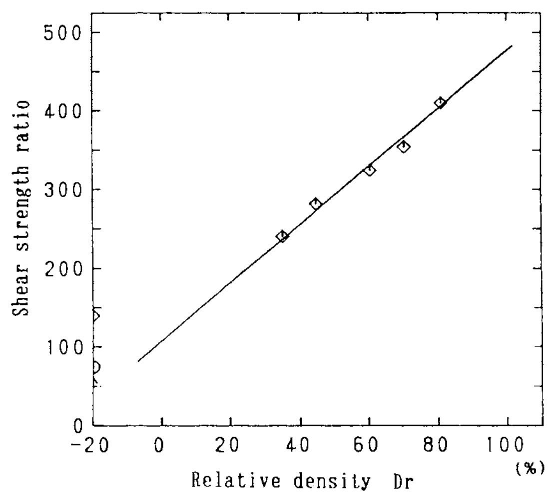

Fig.9 Rate of reduction of shear strength

VANE SHEAR TESTS

Vane shear tests and cyclic torsional shear tests were conducted to evaluate the rate of reduction of the shear strength and the elastic modulus in the process of liquefaction.

The vane shear test apparatus was set on the top of the same soil container as used in the shaking table tests, as shown in Fig.8, and the soil container was vibrated by the shaking table to liquefy the sample. The vane used was 5 cm in height and 2.5 cm in width, and the sand used was Toyoura sand. Samples were prepared by dropping the sand from different heights into the container to obtain several relative densities. The vane shear tests were carried out at a rotational speed of 6 degrees per second in the strain control method. On each sample, tests were conducted before shaking and during liquefaction and the test results were compared.

Fig.9 shows the relationship between the rate of reduction of shear strength due to liquefaction and relative density. According to the test results, the shear strength decreased to about 1/100 to 1/500 of the original strength due to liquefaction.

CYCLIC TORSIONAL SHEAR TESTS

Cyclic torsional shear tests were conducted to clarify the rate of decrease of the elastic modulus due to liquefaction. The sand used for these tests was also Toyoura Sand. All tests were carried out at relative densities of 30 % to 40 % and at an effective isotropical confining pressure , σ_v' , of 0.5 kgf/cm . Hollow cylindrical specimens of 10 cm in height, 10 cm in outer diameter and 6 cm in inner diameter were used.

After fully saturated specimens were consolidated, as shown in Fig.10, a cyclic shear stress of 0.1 Hz was applied until the excess pore pressure ratio ,$\Delta u/\sigma_v'$, reached a prescribed amount. Finally, static shear stress was applied under a constant strain rate of 0.1 % per minute. The prescribed amount of the excess pore pressure ratio, $\Delta u/\sigma_v'$,was changed for each specimen from 0 to 1.0. A specimen of $\Delta u/\sigma_v'$ = 1.0 is a fully liquefied specimen.

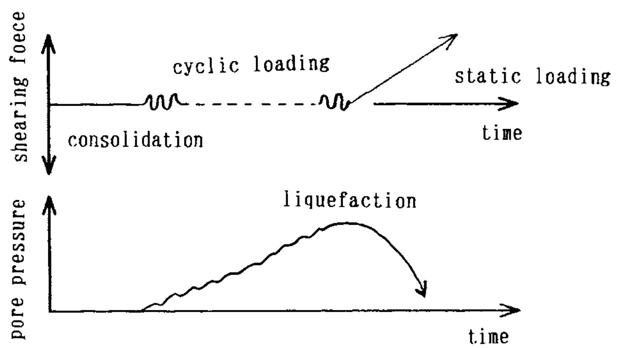

Fig.10 Typical pattern of Test

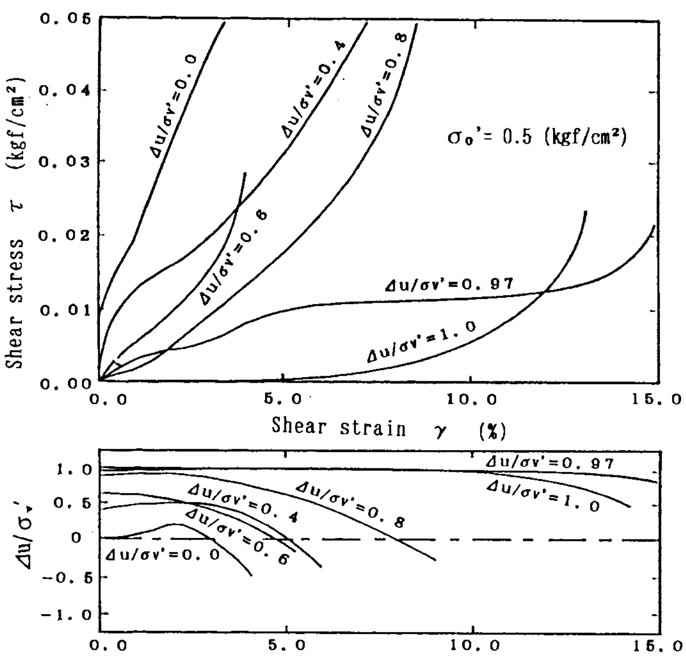

Fig.11 Stress-strain relationships during static tests after cyclic loading

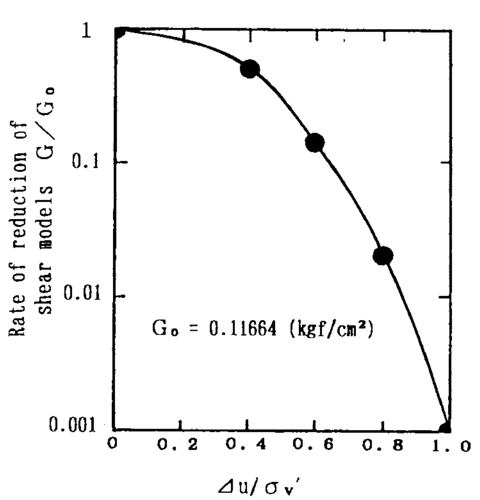

Fig.12 Rate of reduction of shear models

Fig.11 shows the test results during the application of static shear stress. It is clear that if the excess pore pressure ratio increases, the shear strain under the same shear stress increases extremely. This means that the inclination of the stress-strain curve, which can be viewed as an elastic modulus, decreases with the excess pore pressure. The relationship between the excess pore pressure ratio and decreasing ratio of the elastic modulus in small strain, G/G_0, in which G_0 is the elastic modulus of $\Delta u/\sigma_v' = 0$, is plotted in Fig.12. The elastic modulus in small strain, G/G_0 decreases rapidly with an increase in excess pore pressure ratio, $\Delta u/\sigma_v'$, and reaches a very small value, almost 0.001, if a specimen is liquefied fully. However, though the specimen is fully liquefied, the elastic modulus increases under very large strain, such as almost 10% of shear strain, due to dilatancy, as shown in Fig.11.

A SIMPLIFIED PROCEDURE FOR THE ANALYSIS OF THE PERMANENT GROUND DISPLACEMENT

Based on the test results mentioned above and case studies conducted by Hamada et al. (1986), the authors proposed a simplified procedure for the analysis of permanent ground displacement. In this procedure, the authors assumed that permanent ground displacement would occur in liquefied softened ground due to shear stress present before liquefaction. The finite element method was applied twice as follows :

(1) In the first stage, the distribution of stresses in the ground is calculated by the finite element method using the elastic modulus before the earthquake. In the calculation, model layers must be made in several steps, because the soil layers in a natural ground have filled gradually.

(2) Then, holding the stress constant, the finite element method is conducted again using the the decreased modulus due to liquefaction by the earthquake.

(3) The difference in deformation measured by the two analyses is supposed to equal the permanent ground displacement.

To confirm the accuracy of this procedure, several analyses of the soil models used for the shaking table tests were conducted. In these analyses, the shear modulus was determined by a triaxial test. Poisson's ratio during filling, Poisson's ratio during an earthquake and the rate of decrease in the elastic modulus, G/G_0, due to liquefaction, were assumed as 0.35, 0.499 and 1/1000, respectively. Fig.13 shows the result of analysis of Model B-3. Deformation towards

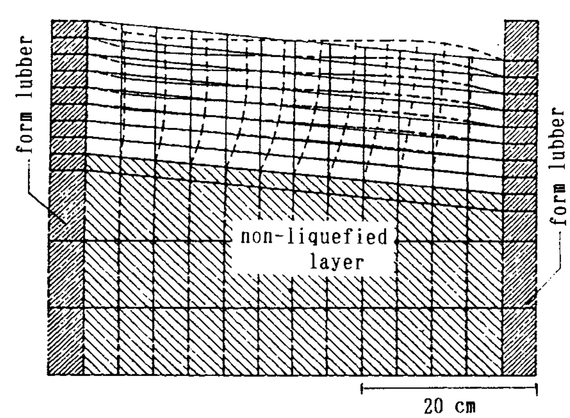

Fig.13 Analized deformation of model grounds. (B－3)

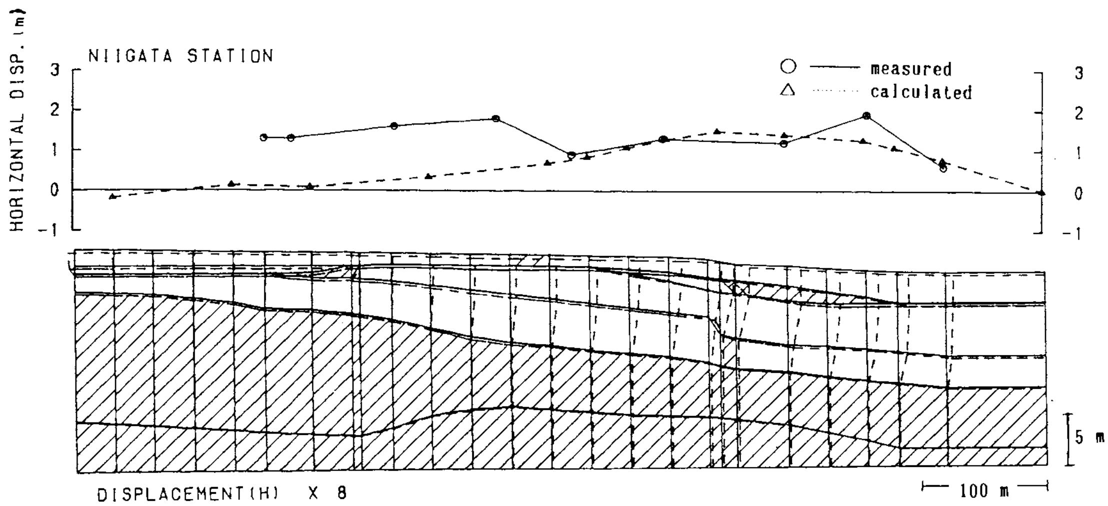

Fig.14 Comparison between calculated displacements and measured displacements. (Niigata Railway Station)

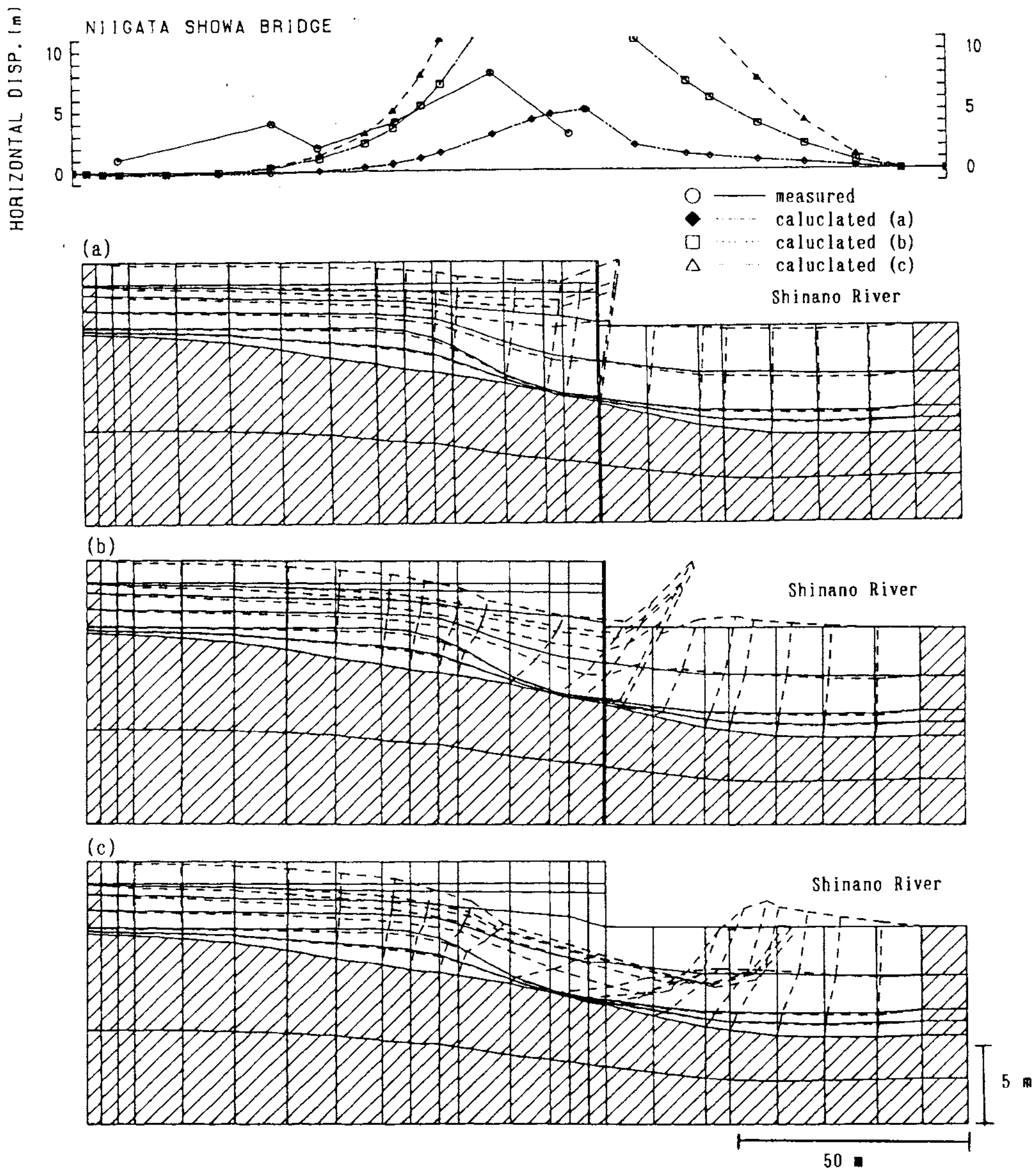

Fig.15 Comparison between calculated displacements and measured displacements. (Niigata Showa Bridge)

the right side occurred in the liquefied layer and displacement increased in a vertical direction from zero at the bottom surface of the liquefied layer to a maximum value at the ground surface. These tendencies coincide fairly well with the test results in Fig.4.

In the next step, permanent ground displacements at the site of Showa Bridge and around Niigata Railway Station in Niigata City during the 1964 Niigata Earthquake were analyzed. These results were compared with the results measured by Hamada et al(1986). In these analyses, Young's modulus ,E, was estimated from SPT N-value by using the formula , E=28N. Poisson's ratios during filling and during an earthquake were assumed as 0.35 and 0.499, respectively, as in the analyses of soil models used for shaking table tests. Three rates of elastic modulus decrease were assumed: 1/1000, 1/500, and 1/2000. Fig.14 compares the results of analysis of ground around Niigata Railway Station with the displacements measured in that area. In Fig.14(a), analyzed displacements and measured displacements have the same tendency: displacement increases with the thickness of the liquefied layer. By comparing the amount of deformation analyzed with that measured, it can be said that the analysis assuming an elastic modulus of decrease rate of almost 1/1000 is appropriate. This decreasing ratio coincides well with the result of the cyclic torsional shear test in Fig.12.

According to a civil engineer in Niigata City, retaining walls made by steel sheet along the banks of Shinano River near Showa Bridge were twisted and fell into the river due to the Niigata Earthquake, and he could not find the retaining walls after the earthquake. Therefore,in the analysis of ground at Showa Bridge, the following three models of the retaining wall were supposed, as shown in Fig.15 :

(1) the retaining wall had been installed below the bottom of the liquefied layer, and was not twisted during the earthquake (Fig.15(b)) ,
(2) the retaining wall had been installed up to the bottom of the liquefied layer, and was twisted during the earthquake (Fig.15(c)) , and
(3) the retaining wall had been installed only up to the middle of the liquefied layer, and fell entirely into the river (Fig.15(d)).

According to the results of analysis, shown in Fig.15(b) , the deformation is not so large if the retaining wall is not twisted. However, if the retaining wall is twisted, extremely large deformation occurs near the river edge, as shown in Fig.15(c) and (d).

This suggests that vertical walls are partially effective in decreasing permanent ground displacement. By comparing the displacements predicted by analysis with the measured displacements on Fig.15(a), it can be seen that the measured values fall between the values of analysis based on the assumptions (1) and (2). As the depth of the retaining wall at this site is not clear, more detailed discussion on the accuracy of the analysis can not be continued.

PROPOSAL OF COUNTERMEASURES AND DISCUSSION OF THEIR EFFECTIVENESS

It is not clear what kind of countermeasures are effective against permanent ground displacement due to liquefaction, because no countermeasures have been applied. However,based on the tests, analyses and case studies, the following three categories of countermeasures seem to be effective : (1) improving the

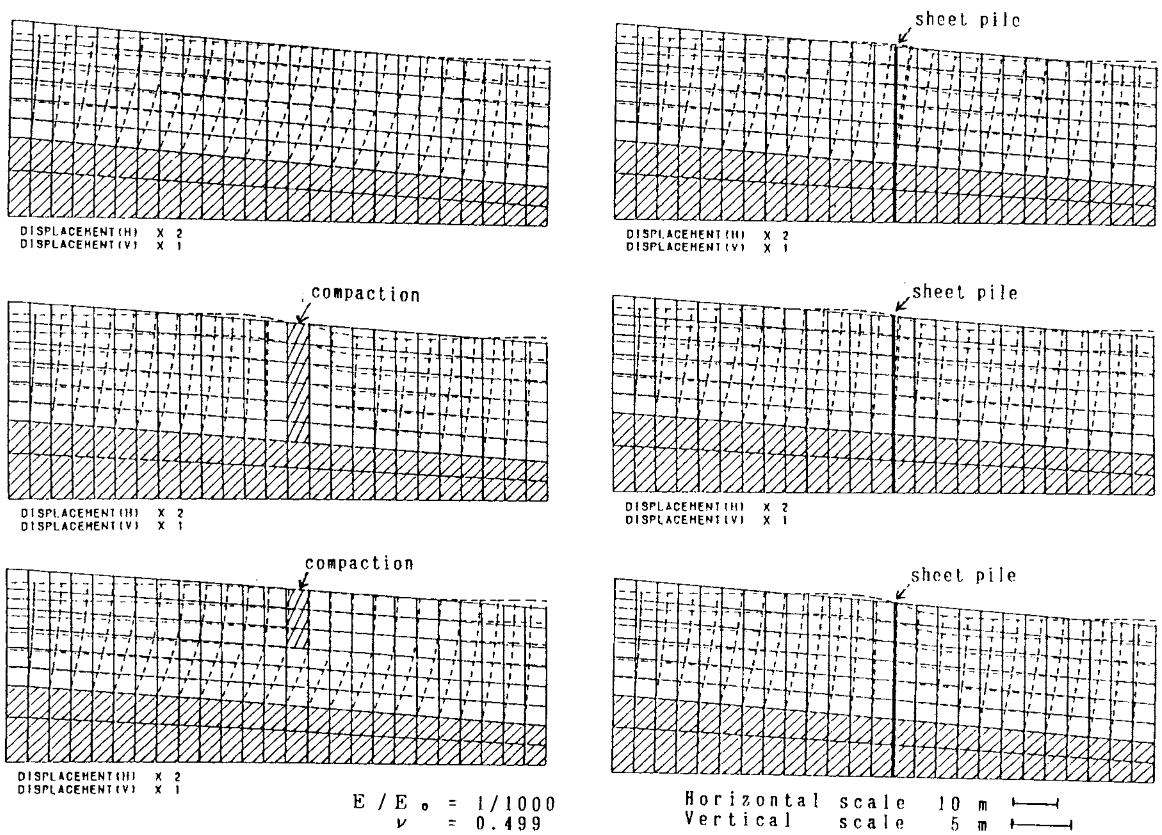

Fig.16 Analized deformation of model grounds with Countermeasure

ground by densification to prevent liquefaction, (2) strengthening structures to prevent their collapse, and (3) strengthening the ground with walls to prevent large ground displacement if liquefaction occurs. Ground densification is generally considered uneconomical, because it must be applied to a wide area. Different methods must be used to strengthen different structures making this approach somewhat impractical. Therefore, strengthening the ground with walls is discussed in this paper.

According to the analyses of ground at Showa Bridge ,the effectiveness of the retaining wall has been suggested, as shown in Fig.15. Therefore, the installation of sheet piles or continuous walls into the ground and the ground compaction were studied, as shown in Fig.1. Several analyses were performed, assuming different countermeasure parameters, on a ground model of 100m in length, with a liquefied layer 10m in thickness and a 3 % slope of the ground surface. The SPT-N values of liquefied layer and the non-liquefied layer were assumed as 3 and 10, respectively. The rate of decrease of the elastic modulus due to liquefaction was supposed as 1/1000.

Six of the results of analysis are shown in Fig.16. Analysis showed that the amount of ground displacement was decreased by installing sheet pile or continuous wall, or by compacting the ground. Moreover, the effectiveness of the countermeasures decreases if the compacted zone does not reach the bottom of the liquefied layer, or if the sheet pile is weak. These relationships are summarized in Fig.17 and Fig.18.

CONCLUSIONS

To study the mechanism of two permanent ground displacements due to liquefaction, shaking table tests, vane shear tests and cyclic torsional shear tests were conducted. In the shaking table tests, displacement did not occur at the boundary between the liquefied layer and the non-liquefied layer, but occurred with a constant shear strain in the liquefied layer. By the cyclic torsional shear tests, it was clarified that the elastic modulus decreased to a very small value, almost 1/1000, due to liquefaction. Based on these results, a simplified procedure for the analysis of permanent ground displacement was proposed. In applying this method to the models of the shaking table tests and to two typical cross sections in Niigata City, the accuracy of the method was confirmed. This method seems to be effective not only to predict permanent ground displacement during a future earthquake, but also to evaluate the effectiveness of countermeasures.

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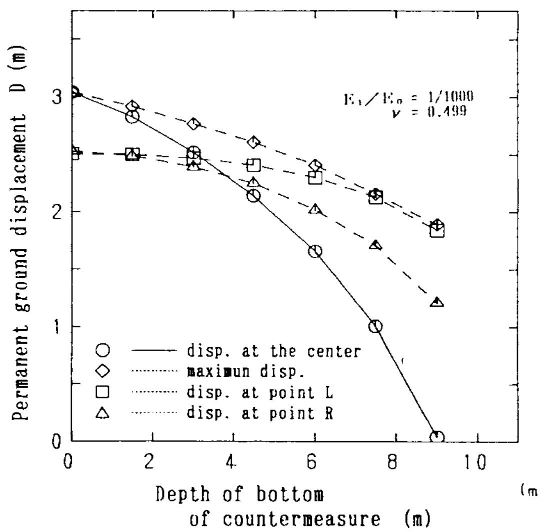

Fig.17 Relationships between D and depth of the bottom of countermeasure

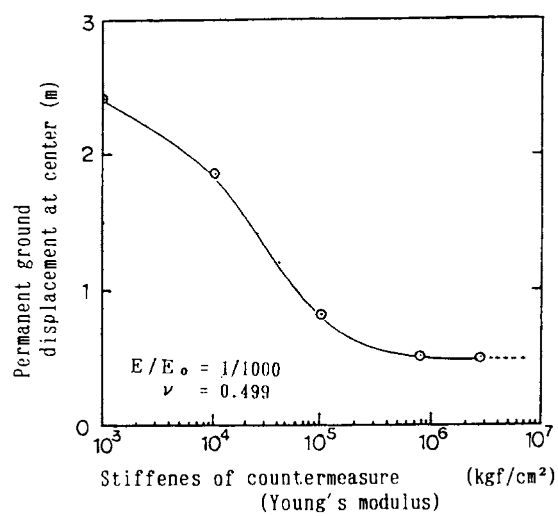

Fig.18 Relationship between D and the stiffness of countermeasure

REFERENCE

Hamada,M., Yasuda,S., Isoyama,R. and Emoto,K. (1986), "Study on Liquefaction Induced Permanent Ground Displacement", Association for the Development of Earthquake Prediction.