THE MECHANISM AND A SIMPLIFIED PROCEDURE FOR THE ANALYSIS OF PERMANENT GROUND DISPLACEMENT DUE TO LIQUEFACTION

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ABSTRACT

To study the mechanism of permanent ground displacement due to liquefaction, shaking table tests on 24 soil models were conducted. In these models, the slope of the ground surface, the thickness of a liquefied layer, the density of the liquefied layer and some other factors were changed. Permanent ground displacements did not occur at the boundary between the liquefied layer and the lower non-liquefied layer, but occurred in the liquefied layer with a constant shear strain. Relationships between the displacement and the thickness of the liquefied layer or the slope of the ground surface were also obtained. Then, vane shear tests and cyclic torsional shear tests were carried out to measure the rate of reduction of shear strength or shear modulus. By the cyclic torsional shear tests, it was clarified that the shear modulus decreases to a very small value due to liquefaction. Based on these test results, a simplified procedure for the analysis of permanent ground displacement was proposed. To confirm the accuracy of this procedure, it was applied to the models used for the shaking table tests and to typical soil cross sections of soil at Niigata City. Finally, some measures to prevent permanent ground displacement were discussed based on the shaking table tests and analyses.

Key words : deformation, earthquake damage, earthquake resistant, laboratory testing, liquefaction, model test, sandy soil (IGC : E 8/D 7)

INTRODUCTION

Many structures were damaged due to liquefaction during the Niigata Earthquake in 1964 and the Alaska Earthquake in the same year. The settlement of buildings and

tanks, failure of embankments and quay walls, damage of bridges, floating of buried tanks, and other damage was reported by many researchers, including Ohsaki (1966), Watanabe (1966), and Kishida (1966). In addition to these damage, some damage due

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to large permanent ground displacement caused by liquefaction over a wide area have been recognized qualitatively. For example, an expanse of ground at the Hakusan power station, near the Shinano River, was permanently displaced (Japan Society of Civil Engineering, 1966). During the 1983 Nihonkai-chubu Earthquake, some permanent ground displacements were also observed at Noshiro City.

Hamada et al. (1986 a, 1986 b, 1988) measured the permanent ground displacements caused by the 1964 Niigata Earthquake and the 1983 Nihonkai-chubu Earthquake, by pre- and post-earthquake aerial surveys. They clarified that extremely large ground displacements, up to several meters, occurred in the ground liquefied during the two earthquakes, though the ground surface was almost flat.

The authors conducted shaking table tests, vane tests and cyclic shear tests to study the mechanism of the permanent ground displacement and to ascertain the rate of decrease of shear modulus and 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 the models of the shaking table tests and to two typical cross sections at Niigata City, and the analytical results were compared with the actually observed measurements. Finally, appropriate countermeasures against permanent ground displacement were discussed.

SHAKING TABLE TESTS

Shaking table tests were conducted on 24 soil models, of which test conditions are described in Table 1, to study the mechanism of permanent ground displacement due to soil liquefaction. In these tests, series A, B, C, D, E and F were conducted first. Then, other series were continued to confirm the mechanism of permanent ground displacement. Shaking table used was 1 m in length and 1 m in width in plane. The soil container, used for series A, B, C, D, E

Table 1. Test conditions

Model No.	Soil	H (m)	θ_s (%)	θ_b (%)	D_r (%)	Density	D (m)
A-1	CS	0.175	0	5	47	MD	0.003
A-2	CS	0.175	0	10	52	MD	0.004
A-3	CS	0.075	0	5	50	MD	-0.001
A-4	CS	0.275	0	5	56	MD	0.005
B-1	CS	0.175	5	5	66	MD	0.016
B-2	CS	0.175	10	10	56	MD	0.030
B-3	CS	0.075	5	5	66	MD	0.006
B-4	CS	0.275	5	5	50	MD	0.018
B-1-L	CS	0.175	5	5	-37	VL	0.023
C-1	FS	0.175	5	5	62	MD	0.009
C-1-L	FS	0.175	5	5	0	VL	0.012
C-1-M	FS	0.175	5	5	46	L	0.010
D-1	CS	0.175	0	-10	52	MD	0.004
D-2	CS	0.175	2	-10	89	MD	0.022
D-3	CS	0.175	10	-10	67	MD	0.066
E-1	CS	0.175	0	5	-34	VL	0.005
E-2	CS	0.175	0	10	-43	VL	0.001
F-1	CS	0.275	5	5	8	L	0.013
G-1	TS	0.180	0	5	37	MD	0.020
G-2	TS	0.180	0	10	30	MD	0.024
G-3	TS	0.180	5	5	37	MD	0.033
G-4	TS	0.180	10	10	35	MD	0.060
H-1	TS	0.180	0	5	20	MD	0.019
I-1	TS	0.075	5	5	0	VL	0.052
	TS	0.125	5	5	30	MD	

• CS : clean sand, FS : sand with fines, TS : Toyoura sand

• H : thickness of the liquefied layer at the center

• θ_s : slope of the ground surface

• θ_b : slope of the bottom surface of the liquefied layer

• D_r : relative density of the liquefied layer

• MD : medium dense, L : loose, VL : very loose

• D : displacement on the ground surface

• soil container used for series G, H and I is slightly bigger than that for other series

and F, was 80 cm in length, 65 cm in depth and 60 cm in width, as shown in Fig. 1. For series G, H and I, a slightly bigger container, of 100 cm in length, 65 cm in depth and 60 cm in width, was used. The front wall of the container was made of glass, through which the deformation of the soil could be viewed. A foam rubber of 5 cm in thickness was inserted inside both walls to induce uniform cyclic shear strain in a soil model during shaking.

Fig. 1. Soil container and model of shaking table test

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. The liquefied layer was filled horizontally except Model H-1 as men-

Fig. 2. Grain size distribution curves of three sands

tioned later.

Three 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, (2) sand with fines taken from a hill in Chiba Prefecture, and (3) Toyoura sand. Grain size distribution curves of the sands are shown in Fig. 2. The fine contents of the three sands were 0%, 6% and 0%, respectively.

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

A. the ground surface is flat, but the

Fig. 3. Model types

bottom surface of the liquefied layer is sloped,

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

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,

F. 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,

G. the ground condition is almost the same as that in Models A-1, A-2, B-1 and B-2, but the shaking motion was applied in the opposite direction, perpendicular to the horizontal axis,

H. the ground surface is flat and the bottom surface of the liquefied layer is sloped the same as Model A-2, however the "liquefied layer" was filled with a slope of -45° as shown in Fig.3 (H), and

I. both the ground surface and the bottom surface of the liquefied layer are sloped the same as in Model B-1, but the "liquefied layer" is separated into two sublayers; a very loose layer and a medium dense layer.

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

Shaking motion was applied in one direction parallel to the horizontal axis in Fig.1 except series G, 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 three points, as shown in Fig.1. Displacements at several points in the liquefied layer were measured by two methods: (1) deformations of nine to sixteen noodles, placed vertically in the soil at the front of the model, just behind the glass, were measured

Fig. 4. Deformation of the noodles

by photograph at intervals of 2.5 seconds after liquefaction, and (2) displacements of 18 to 30 pins on the ground surface were measured by a scale after stopping the shaking.

Fig.4 shows typical deformations of the noodles up to 10 seconds after liquefaction. These displacements must be correlated with real displacement of soil. Among the first six series, the largest displacements were observed in series B, as shown in Table 1 and Fig.4(b). This figure shows that the displacement developed gradually after the occurrence of liquefaction. The displacement increased almost 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 the displacement did not occur at the boundary between the liquefied layer and the non-liquefied layer, but occurred in the liquefied layer with a constant shear strain. In the series A, shown in Fig.4(a), displacements were not so larger. However, it seems that the displacement was induced toward the direction of slope of the bottom surface of the liquefied layer. The displacements near the ground surface were small. On the contrary, in the series D, the displacements

toward the slope of the ground surface occurred. In the series E, though not shown in Fig. 4, it seemed that the displacements concentrated slightly toward the central part; however, this tendency was not clear, because the amounts were small. In the series F, the displacements of the ground surface were smaller than those in series B.

In Model G-3, the displacement also developed gradually after the occurrence of liquefaction as shown in Fig. 4(d), almost as it did on Model B-1. In Model H-1, the displacement occurred not toward the slope of the bottom surface of the liquefied layer, but toward the slope of the filled layers, as shown in Fig. 4(e). This means that some amount of shear stress with the direction of -45° was sustained in the upper sand layer before shaking, and the displacement occurred during liquefaction because of a fall of shear strength. The displacement in Model I-1 was larger than that in Model B-1, as shown in Table 1.

Fig. 5 shows the settlement of the ground surface measured after stopping the shaking. In the series B, the settlement at the lower part of the surface was less than that at the upper part. This means that not only the settlement due to compaction but also the uplift of the ground surface due to movement of soil from the upper part was induced in the lower part.

Fig. 5. Settlement of the ground surface

Fig. 6 and Fig. 7 show the relationship among the thickness of the liquefied layer H , the slope of the bottom surface of the liquefied layer θ_b , and the 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

Fig. 6. Relationships between D and H

Fig. 7. Relationships between D and θ

trends of the relationship for the clean and medium-dense sand. D increases with H or 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 sand with fines was less than that of clean sand. According to Hamada et al. (1986 a, c), the following relationship was derived from observations of the 1964 Niigata Earthquake, the 1983 Nihonkai-chubu Earthquake and the 1971 San Fernando Earthquake :

$$D = 0.75 \sqrt{H} \sqrt[3]{\theta} \quad (1)$$

where, θ is the larger gradient of the ground surface, θ_s or the bottom surface of the liquefied layer, θ_b .

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, it must be noted that the soil container used was small.

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

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 shear 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

Fig. 8. Vane shear test apparatus

(a) Before shaking

(b) During liquefaction

Fig. 9. Vane shear test results

Fig. 10. Rate of reduction of shear strength

control method. On each sample, tests were conducted before shaking and during liquefaction, and the test results were compared.

The test results are shown in Fig. 9. Shear strength decreases extremely with the occurrence of liquefaction. Fig. 10 shows the relationship between the rate of reduction of shear strength due to liquefaction and relative density, in the same level of strain where peak stresses were measured in the tests conducted before shaking. According to the test results, the shear strength decreased to about 1/200 to 1/400 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 shear 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'_0 of 49 kN/m² (0.5 kgf/cm²). Hollow cylindrical specimens of 10 cm in height, 10 cm in outer diameter and 6 cm in inner diameter were used.

After saturated specimens were consolidated, as shown in Fig. 11, a cyclic shear stress of 0.1Hz was applied until the excess pore pressure ratio, $\Delta u/\sigma'_0$ reached a pre-

Fig. 11. Loading pattern of cyclic torsional tests

Fig. 12. Stress-strain relationships during static tests after cyclic loading

scribed amount. Then, a static shear stress was supplied under a constant strain rate of 0.1% per minute. The prescribed amount of the excess pore pressure ratio, $\Delta u/\sigma'_0$, was changed for each specimen from 0 to 1.0. A specimen of $\Delta u/\sigma'_0=1.0$ is a fully liquefied specimen.

Fig. 12 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 shear modulus,

Fig. 13. Rate of reduction of shear modulus

G decreases with the excess pore pressure. The relationship between the excess pore pressure ratio and the decreasing ratio of the shear modulus in small strain $G_{0,i}/G_{0,i}$, in which $G_{0,i}$ is the shear modulus of $\Delta u/\sigma'_0 = 0$, is plotted in Fig. 13. The rate of reduction of shear modulus, $G_{0,i}/G_{0,i}$ decreases rapidly with an increase in the excess pore pressure ratio, $\Delta u/\sigma'_0$, and reaches a very small value, almost 0.001, if a specimen is fully liquefied. However, though the specimen is fully liquefied, the shear modulus increases under very large strain, such as almost 10% of shear strain, due to dilatancy, as shown in Fig. 12.

A SIMPLIFIED PROCEDURE FOR THE ANALYSIS OF THE PERMANENT GROUND DISPLACEMENT

Based on the test results mentioned above and the case studies conducted by Hamada et al. (1986 a, b, c), 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 been filled gradually.

(2) Then, holding the stress constant, the finite element method is conducted again using 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 elastic modulus was in the first stage 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, $E_{0,i}/E_{0,i}$ due to liquefaction, were assumed as 0.35, 0.499 and 1/1000, according to the results of the cyclic torsional shear test respectively. Fig. 14 shows the result of analysis of Model B-2. Deformation towards the right side occurred in the liquefied layer and displacement increased in a vertical direction from zero at the

Fig. 14. Analyzed deformation of model ground (B-2)

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. Though the result is not shown here, an analysis of Model H-1 was also conducted. The tendency of deformation is in agreement with the test result shown in Fig. 4(e).

In the next step, permanent ground displacements at the site of Showa Bridge and around Niigata Railway Station at Niigata City during the 1964 Niigata Earthquake were analyzed. These results were compared with the results measured by Hamada et al (1986 a, b). In these analyses, the elastic modulus, E , was estimated from SPT N -value by using the formula, $E=28 N$. Poisson's ratios during filling and during the earthquake were assumed as 0.35 and 0.499, respectively, as in the analyses of soil models used for the shaking table tests. Three rates of elastic modulus decrease were assumed: 1/500, 1/1000, and 1/2000. Fig. 15 compares the results of analysis of ground around Niigata Railway Station with the displacements measured in that area. In Fig. 15 (a), analyzed displacements and measured displacements have the same tendency: displacement increases with the thickness of

Fig. 15. Comparison between calculated displacements and measured displacements at Niigata Railway Station

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. 13.

According to a civil engineer at Niigata City, retaining walls made by steel sheet along the banks of Shinano River near Showa Bridge were bent 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

Fig. 16. Comparison between calculated displacements and measured displacements at Showa Bridge in Niigata City

shown in Fig.16 :

(1) the retaining wall had been installed below the bottom of the liquefied layer, and was not bent during the earthquake (Fig.16 (b)),

(2) the retaining wall had been installed up to the bottom of the liquefied layer, and was bent during the earthquake (Fig.16 (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.16(d)).

According to the results of analysis, shown in Fig.16(b), the deformation is not so large if the retaining wall is not bent. However, if the retaining wall is bent, extremely large deformation occurs near the river edge, as shown in Fig.16(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.16(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 present tests, analyses and case studies, the following three categories of countermeasures seem to be effective: (1) improving the ground in all areas by densification to prevent liquefaction, (2) strengthening structures to prevent their collapse, and (3) strengthening the ground with walls, steel piles, sand piles, or densification at a narrow band, to prevent large ground displacements if liquefaction occurs. Ground densification in all areas is generally considered uneconomical, because it must be applied to a wide area. Different methods must be used to strengthen different struc-

(a) Model

(b) Displacement on the ground surface

Fig. 17. Results of test on and analysis of a model with a compacted band

tures making this approach somewhat impractical. Therefore, strengthening the ground by densification through a narrow band was studied by a shaking table test and an analysis.

In the shaking table test, the soil composition, slope and shaking conditions were selected as in Model G-3. A band of 1 cm in thickness was compacted from the ground surface to the base, after filling the liquefiable sand layer, as shown in Fig.17(a). Displacements, ten seconds after the occurrence of liquefaction, measured by pins on the ground surface, are shown in Fig.17(b). In Fig.17(c), values of those displacements averaged for each row are shown. It is clear that the displacement of some area close to the compacted band on the upstream side was decreased due to the countermeasure.

In the same model, the simplified procedure for analysis was carried out. Fig.17(c) compares the displacements on the ground surface estimated by analysis with those measured in the tests, in both cases with and without the compacted band. Though the estimated displacements are slightly smaller than the observed displacements, distribution curves of displacements are almost similar. Then, ratios of displacement with the compacted band, D_c , and displacement without the compacted band, D , are compared in Fig.17(d). The effectiveness of the countermeasure by the compacted band is also appeared in the analysis, the same as in the test.

CONCLUSIONS

To study the mechanism of permanent ground displacements due to liquefaction, a series of shaking table tests, vane shear tests and cyclic torsional shear tests were conducted. In the shaking table tests, the displacement did not occur at the boundary between the liquefied layer and the non-liquefied layer, but occurred in the liquefied layer with a constant shear strain. By the cyclic torsional shear tests, it was clarified

that the elastic modulus decreased to a very small value, almost 1/1000 of the original value, 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 at Niigata City, the accuracy of the method was confirmed. This method seems to be effective not only to predict permanent ground displacements during a future earthquake, but also to evaluate the effectiveness of countermeasures.

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