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Appropriate countermeasures against permanent ground displacement due to liquefaction

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ABSTRACT: Permanent ground displacements due to liquefaction caused by the 1991 Terile-Limón Earthquake were studied. Then, appropriate countermeasures against liquefaction were studied based on shaking table tests and analyses. Four types of countermeasures were compared. Among them, the continuous wall method was the most effective. Moreover, earth pressure acting on quay walls at the onset of liquefaction was studied by shaking table tests to utilize it to the seismic design of quay walls.

1 INTRODUCTION

Recently, permanent ground displacements caused by the 1964 Niigata Earthquake and the 1983 Nihonkai-chubu Earthquake were measured by pre- and post-earthquake aerial surveys (Hamada et al.1986,1987), and 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. Moreover, large permanent ground displacements occurred during the 1990 Luzon Earthquake and the 1991 Terile-Limón Earthquake in Philippines and in Costa Rica, respectively. Those permanent ground displacements were classified into two groups: ① lateral flow at shores of a river or a sea, and ② large displacement along a gentle slope.

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 the shear modulus and the shear strength (Yasuda et al.1992). Based on these tests, a simplified procedure for forecasting permanent ground displacement was proposed.

In the next step, appropriate countermeasures against the permanent ground displacement were studied based on shaking table tests and analyses. In case of the large displacement along a gentle slope, countermeasures by strengthening the ground with sand piles, steel piles, densification at a narrow band or continuous walls were studied. The effectiveness and the limitation were clarified. Moreover, effectiveness of the countermeasure in the full scale ground was studied by some analyses. In case of the lateral flow, increasing of earth pressure due to liquefaction were studied.

In this paper, the damages of three bridges due to lateral flow during the 1991 Terile-Limón Earthquake are introduced. Then, shaking table tests and analyses for countermeasures are shown.

2. LATERAL FLOW INDUCED BY THE 1991 TERILE-LIMÓN EARTHQUAKE

The Terile-Limón Earthquake, with a magnitude of 7.4 (Ms), occurred on April 22, 1991 in southeast Costa Rica. Liquefaction and landslides were induced at many locations in lowlands and mountains, respectively, as shown in Fig.1. Watanabe, Yoshida and one of the authors conducted site investigation of liquefied sites (Watanabe et al, 1992), and classified the liquefaction sites into three groups: ① sand dune and back marsh along the Caribbean Sea, ② shores of a river, and ③ inland alluvial plain. Many lateral flow were observed in the first and the second sites.

Fig.2(a) shows a schematic cross section perpendicular to the coast of Caribbean Sea at the first group. Wide cracks and boiled sands were observed on the top of the sand dune. The height of the sand dune is almost 2 to 3 m. Though soil condition is not clear because no soil investigation have been conducted, liquefied layer is estimated as shown in Fig.2(a). And lateral flow must be induced from the top of the sand dune to

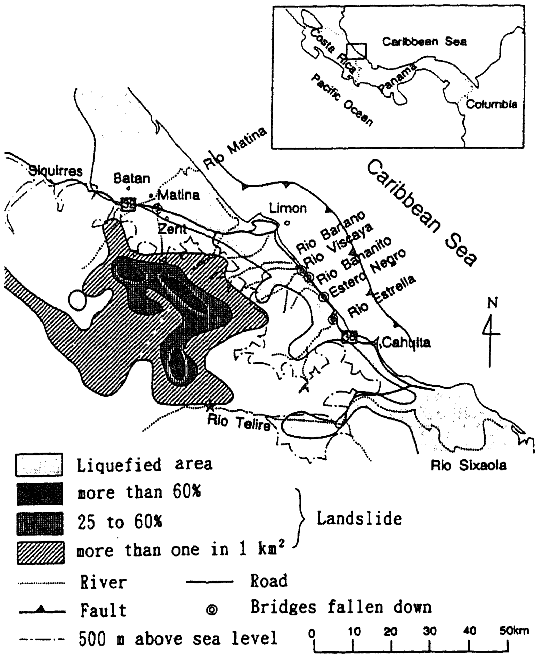
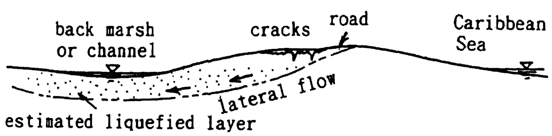
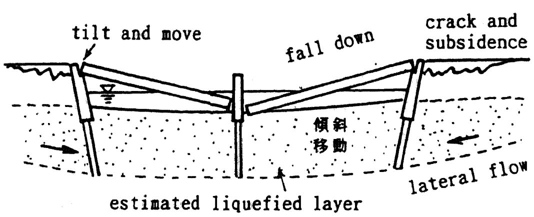


Fig. 1 Distribution of Sites of Liquefaction and Landslide Caused by the 1991 Terile-Limón Earthquake (Mora, 1991)



(a) Schematic cross section perpendicular to the coast



(b) Schematic illustration of the damaged bridge

Fig. 2 Schematic Illustration of Lateral Flow (Watanabe et al. 1992)

a back marsh or a channel.

Five road bridges fell down by the earthquake as shown in Fig.1. Three of them are estimated to have damage due to lateral flow of shores of rivers. Photo 1 shows the damage of the bridge over the Bananito River. Two spans with simply supported girders fell down into river beds. Abutments in both sides were tilted and moved towards the river center. Many wide cracks, parallel to the river, were induced on the shores of the river. The abutments had been supported with piles of about 15 m in length. However, it is estimated that the depth of liquefied layer was equal or deeper than the depth of points of the piles. Fig.2(b) shows a schematic illustration of the damage. The abutments were estimated to be tilted and moved due to the lateral flow of the liquefied layer.

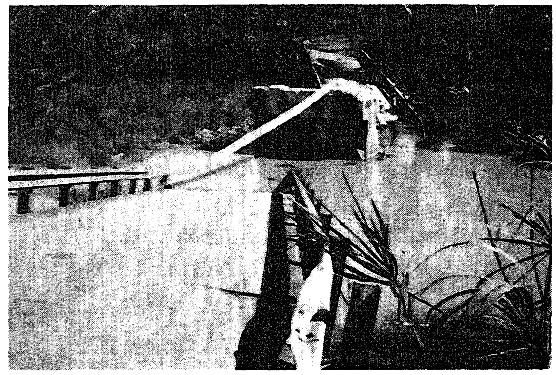


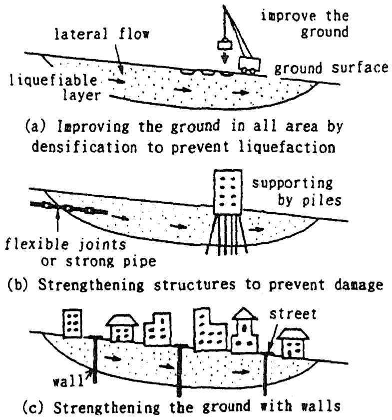
Photo 1 Damage of the Bridge over the Bananito River

3 KIND OF COUNTERMEASURES AGAINST PERMANENT GROUND DISPLACEMENT

It is not clear what kind of countermeasures are effective against the permanent ground displacement due to liquefaction on a gentle slope, because no countermeasures have been applied. However, based on the tests, analyses and case studies, the following three categories of countermeasures, as shown in Fig.3, seem to be effective : (1) improving the ground in all area by densification to prevent liquefaction, (2) strengthening structures to prevent damage, and (3) strengthening the ground with walls or steel piles, sand piles, densification at narrow bands, to prevent large ground displacement if liquefaction occurs.

Ground densification in all area 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 by walls or steel piles, sand piles, densification through a narrow band was studied by shaking table tests and analyses.

In case of the lateral flow at shores of a river, the following two categories of countermeasures, as shown in Fig.4, seems to be effective: (1) improving the ground of shores of a river or a sea, and (2)

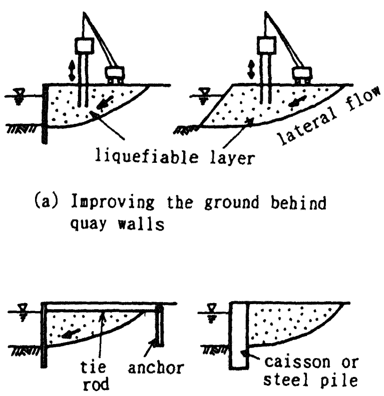


(a) Improving the ground in all area by densification to prevent liquefaction

(b) Strengthening structures to prevent damage

(c) Strengthening the ground with walls or steel piles, sand piles, densification at narrow bands, to prevent large ground displacement

Fig. 3 Three Categolies of Countermeasures against Permanent Ground Displacement on Gentle Slope



(a) Improving the ground behind quay walls

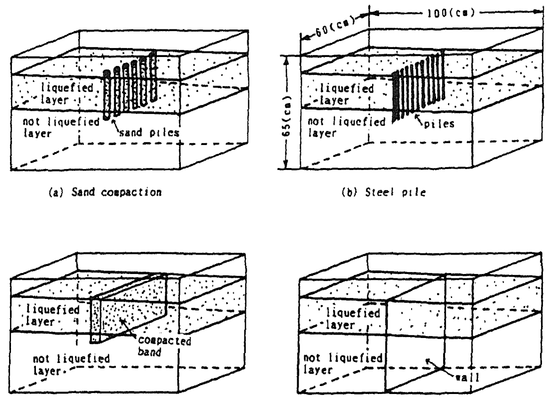
(b) Strengthening the quay walls

Fig. 4 Two Categories of Countermeasures against Lateral Flow at River Shore

strengthening quay walls to prevent damage. In the second method, earth pressure during liquefaction must be considered into the design of the quay wall. Therefore, increment of earth pressure acting on a quay wall due to liquefaction was studied by shaking table tests.

4 SHAKING TABLE TESTS ON COUNTERMEASURES AGAINST PERMANENT GROUND DISPLACEMENT ON GENTLE SLOPES

Shaking table tests to ascertain effective countermeasures against permanent ground displacement due to liquefaction on gentle slopes were carried out by using a soil container shown in Fig.5. Sand used was Toyoura Sand, which is a clean sand, and the relative density of the loose layer, which



(a) Sand compaction

(b) Steel pile

(c) Compaction of the ground with a band

(d) Continuous underground concrete or steel wall

Fig. 5 Models of Countermeasures in Shaking Table Tests

is the liquefied layer, was arranged as 30%. Four types of countermeasures were applied to the model ground, (1) sand compaction, (2) steel piles, (3) compaction of the ground with a band, and (4) continuous underground concrete or steel wall. The following models were used for the four types of countermeasures in considering scale effects:

(1) In the sand compaction method, aluminum piles of 2 cm in outer diameter were stood in the dense layer, which is the not-liquefied layer, with a depth of 5 cm. Then the loose layer, which is the liquefied layer, was filled with the pipes erect. After filling the loose layer, the pipes were pulled out and some Toyoura Sand was poured into the holes. The sand in the holes was compacted by pushing a rod to a relative density of almost 90 % to 100 %. Tests were conducted under three conditions. The number of the compacted sand piles and rate of replacement in each case is shown in Table 1(a).

(2) In the steel pile method, vinyl chloride piles of 1.8 cm in outer diameter and 2.5 mm in thickness were used. The method of

Table 1 Test Conditions of Countermeasures

(a) Sand Compaction

Case No.	Number of piles	Rate of replacement As (%)
S-1	6	3.1
S-2	8	5.6
S-3	10	8.7

(b) Steel pile

Case No.	Number of piles	Pitch of piles(cm)	Number of rows
P -1	10	6	1
P -2	12	5	1
P -3	15	4	1
P -4	20	3	1
PT-1	15	7.5	2
PT-2	20	5.8	2

(c) Compaction of the ground with a band

Case No.	Thickness of the compacted band (cm)
W-1	0.5
W-2	1.0
W-3	1.5
W-4	2.0

(d) Continuous underground concrete or steel wall

Case No.	Thickness of the wall (cm)
A-1	0.2
A-2	0.3

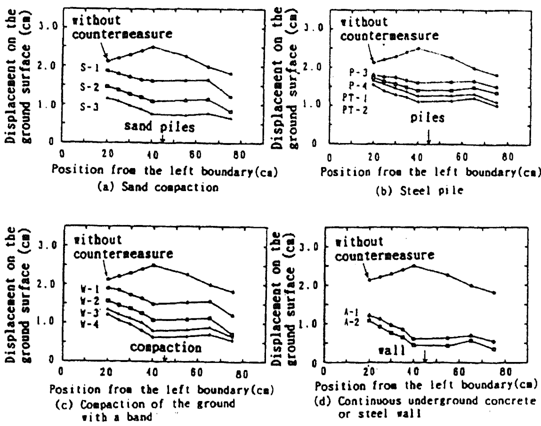


Fig. 6 Measured Displacement in Shaking Table Tests

erecting the piles and of filling the loose layer were also the same. Young's modulus of the piles was 32,000 kgf/cm^2. In this method, piles were stood in one row or in two rows with triangle alignment. Test conditions are shown in Table 1(b).

(3) Instead of vinyl chloride pipes, two sheets of walls made of aluminum, with a thickness of 2 mm, were used in the ground compaction with a band method. The depth of installation of the walls, method of filling loose layer, and method of compacting the sand in the trench after pulling out the walls were the same as in the sand compaction method. Four thicknesses of the compaction band were tested, as shown in Table 1(c).

(4) In the continuous underground concrete or steel wall method, an acrylic wall of 2 mm or 3 mm in thickness was used, as shown in Table 1(d). The wall was stood on the bottom surface of the soil container. Eight pieces of strain gauges were pasted on the wall to measure the bending strain of the wall due to earth pressure. Installation of the wall and method of filling loose layer were the same as in the steel pile method.

In all tests, the thickness of the loose layer was 20 cm, and slopes of the ground surface and bottom surface of the loose layer were 3 %. Models were shaken in the perpendicular direction to the horizontal axis, according to a 3 Hz sine wave up to 10 seconds after the occurrence of liquefaction.

Fig.6 shows the measured displacements on the ground surface after stopping the shaking. Without countermeasures, displacements of 2 to 2.5 cm occurred on the ground surface, with the maximum value at the center. In contrast, displacements with countermeasures decreased to 2 cm to 2 mm, with the minimum value on the upper side, on the left side in the figure of the countermeasures. In the steel pile method, an alignment with two rows was more effective than an alignment with one row if the numbers of piles were the same. In case of the continuous underground wall method, the distribution of earth pressure acting on the wall was estimated as shown in Fig.7 based on the measured strain and Young's modulus of the wall. The distribution curve was almost triangular.

Displacements with an underground wall were the smallest among the four types of countermeasures, as shown in Fig.6. In the sand compaction method or steel pile method, some soil-flow through the piles was

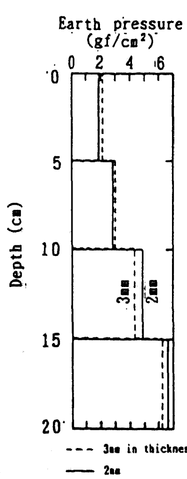


Fig. 7 Distribution of Earth Pressure

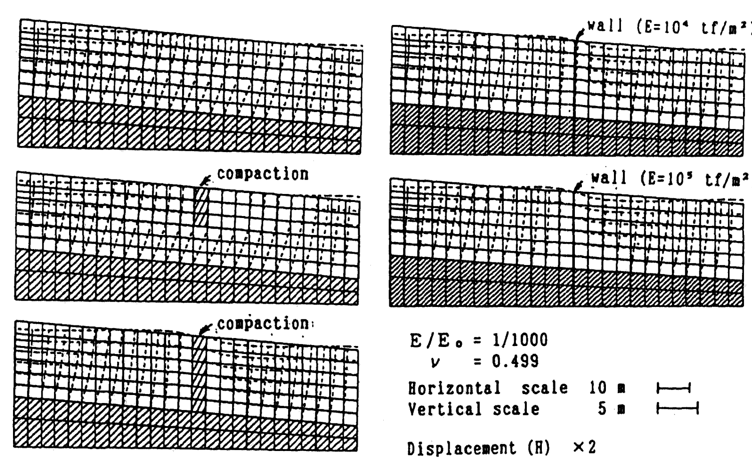


Fig. 8 Analyzed Deformation of Model Grounds with Countermeasure

induced. And, in the ground compaction with a band, some bending of the compacted band occurred due to inadequate stiffness of the compacted band. Therefore, it can be said that the continuous wall method is the most effective among the four methods. However, stress induced in the wall must be evaluated during the design of the wall.

5 ANALYSES FOR THE EFFECTIVENESS OF COUNTERMEASURES IN THE GROUND

To know the effectiveness of the countermeasures, mentioned above, in the ground, several analyses were performed based on a simple method proposed by the authors (1992), assuming different countermeasure parameters, on a ground model of 100 m in length, with a liquefied layer 10 m in thickness and a 3 % slope of the ground surface. Among the four types of countermeasures by the continuous underground concrete or steel wall and the compaction of the ground with a band were selected for the analyses. The SPT-N values of liquefied layer and the non-liquefied layer were assumed as 3 and 30, respectively. The rate of decrease of the elastic modulus due to liquefaction was supposed as 1/1000.

Five of the results of analysis are shown in Fig.8. Analysis showed that the amount of ground displacement was decreased by installing 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 continuous wall is weak.

6 SHAKING TABLE TESTS ON EARTH PRESSURE

According to Tsuchida (1968), the coefficient of earth pressure acting on a quay wall increases to about 1.0 if the ground behind the quay wall is liquefied.

In view of the findings of shaking table tests conducted by the authors (1991), if the bottom surface of the liquefied layer behind the quay wall is sloped toward the sea or river, as shown in Fig.9(a), it is supposed that the earth pressure acting on the quay wall as a result of liquefaction is greater than the earth pressure when $\theta_b = 0$: On the contrary, if the bottom surface is sloped in the opposite direction, as shown in Fig.9(b), the earth pressure is assumed to be smaller than the pressure when $\theta_b = 0$: In general, the slope of the bottom surface of the liquefied layer toward the sea or river is considered to be more common in-

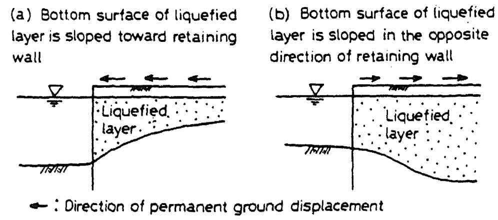


Fig. 9 Schematic Illustrations of Two Types of the Liquefied Layer

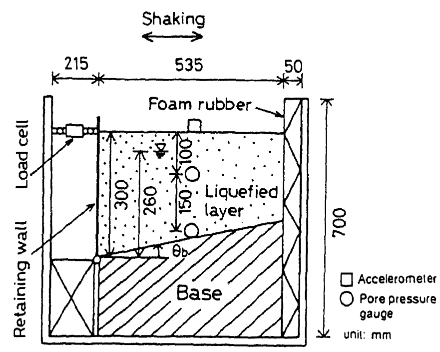


Fig. 10 Soil Containers and Models of Shaking Table Test

situ than the opposite case, because most of the ground in the vicinity of quay walls is reclaimed land. However, it is considered possible that the bottom surface of the liquefied layer may slope away from the sea or river if the quay wall is constructed at a site with different ground conditions. Therefore, shaking table tests were conducted using both patterns.

In the tests, a container, of 80 cm in length, 70 cm in depth and 50 cm in width shown in Fig.10 was used. In the container, a retaining wall of aluminum plate was installed with reinforcement.

Total earth pressure was measured using a loadcell, assuming that the retaining wall of the model is hard enough to resist deformation by earth pressure. In this case, it was supposed that the earth pressure was distributed in the shape of a right angled triangle with depth. This assumption is absolutely valid. Therefore, two series of tests were performed; one was conducted with the hinge connection at the bottom edge and the load cell at the top edge; the other was the opposite case. In these tests, the slope of the bottom of the liquefied layer θ_b was varied from -10 % to 10 %. The soils used for the tests were clean sand and sand with 6 % of fines.

The coefficient of earth pressure was measured at three points during the test:

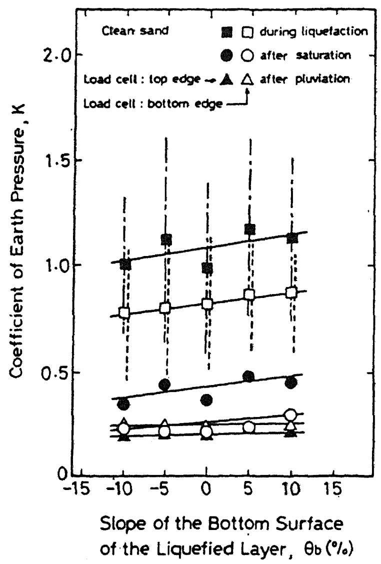


Fig.11 Relationships between K and θ_b of Clean Sand

(1) after the dry sand was poured into the soil container through a sieve in air, (2) at the time the ground water level was raised to a depth of 4 cm from the ground surface, (3) during shaking.

The earth pressure increases as the excess pore water pressure builds up, and it reaches a maximum value at the onset of initial liquefaction. After the shaking, the earth pressure decreases as the excess pore water pressure dissipates.

Fig.11 indicates the coefficient of the earth pressure, K, obtained by assuming that the earth pressure is distributed in the shape of a right-angled triangle with depth, as described above. After the dry sand is poured into the soil container, the value of the coefficient K is 0.2 to 0.3, irrespective of the slope of the bottom of the liquefied layer θ_b. When the sand layer is saturated by raising the ground water level to a desired depth, the value of K is nearly 0.4 and varies with θ_b. At the onset of liquefaction, K reaches about 1.0 and varies with θ_b. The coefficient of earth pressure measured by a loadcell at the top edge differs slightly from the coefficient measured at the bottom edge. The distribution of the earth pressure is not precisely in the shape of a right-angled triangle. The dashed lines and the chain lines in Fig.11 denote amplitudes of the dynamic component of the earth pressure. Averages of the maximum and minimum value of these amplitude are plotted in Fig.11 using the marks □ and ■.

7 CONCLUSIONS

Permanent ground displacement due to soil liquefaction brings severe damages to many structures. To study the effectiveness of countermeasures on a gentle slope by strengthening the ground, shaking table tests and analyses were conducted. Four type of strengthening method were selected: (1) sand compaction, (2) steel pile, (3) compaction of the ground with a band, and (4) continuous underground concrete or steel wall. In all shaking table tests, the amount of the ground displacement of some area close to the countermeasures was decreased. The most effective method was the continuous wall method. Analyses on a ground model of 100 m in length also showed that the amount of the displacement was decreased by installing the continuous wall. Moreover, increment of earth pressure acting on a quay wall was studied by shaking table tests. Test results showed that the coefficient of the earth pressure reached to about 1.0 at the onset of liquefaction, and the coefficient increased slightly with the slope of the bottom surface of the liquefied layer.

REFERENCES

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

Hamada,M., Yasuda,S. and Isoyama,R. 1987. Liquefaction-induced Permanent Ground Displacement During Earthquakes:Proc. of the Pacific Conf. on Earthquake Eng., Vol.3, pp.37-47.

Mora,S. 1991. Personal Communication.

Tsuchida,H. 1968. Study on Lateral Earth Pressure and Dynamic Pore Water Pressure of Water Saturated Sand during Vibration: Soil Mechanics and Foundation Engineering, Vol.16, No.5, pp.3-10 (in Japanese).

Watanabe,H., Yasuda,S. and Yoshida,N. 1992. A Reconnaissance Report on the 1991 Telire-Limon Earthquake: Association for the Development of Earthquake Prediction (in Japanese).

Yasuda,S., Nagase,H., Kiku,H. and Uchida,Y. 1991.Effect of Permanent Ground Displacement on the Damage to Quay walls : Proc. of the Int. Conf.on Geotechnical Eng.for Coastal Development, pp.525-530.

Yasuda,S., Nagase,H., Kiku,H. and Uchida,Y. 1992. The Mechanism and a Simplified Procedure for the Analysis of Permanent Ground Displacement due to Liquefaction: Soils and Foundations, Vol.32, No.1 (in press).