**13th World Conference on Earthquake Engineering**
**Vancouver, B.C., Canada**
**August 1-6, 2004**
**Paper No. 642**

# EMBANKMENT REINFORCEMENT BY GEOGRID TO REDUCE ITS SETTLEMENT DURING EARTHQUAKES

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## SUMMARY

A dike section 3 m high suffered from strong shaking during the Tottori-ken Seibu Earthquake ($M_j$=7.3) on October 6, 2000. This particular section, 850 m long, had been reinforced by geogrid at its bottom of the dike to mitigate the liquefaction inducing damage. The earthquake caused damage to the dike section for 120 m within the treated length. Although this 120 m long section subsided by about 1 m, however 80 % of the treated length of the dike survived without apparent deformation. Strong motion of 160-200 gal acceleration was recorded at this site, and traces of sand boiling were observed. The survey results showed that the sound length of the section suffered from settlement of about 20 cm, but no visible deformation was observed on the dike or on the concrete facing which had been laid on the outer slope. This paper describes the performance of the geogrid reinforcement against soil liquefaction on the basis of the observed results and the results of shaking table tests conducted after the earthquake.

## INTRODUCTION

Embankments have often been damaged in the past earthquakes due to soil liquefaction. However it is rare that an embankment which had been remediated has been subjected to an actual damaging earthquake. Remediation of embankments against soil liquefaction is carried out for embankments which have important roles such as protecting the urbanized area from flooding. Remediation is accomplished by a ground improvement method such as SCP or DMM. Not only for large embankments, but also for small embankments, preventive measures are desirable. However, effective and reasonable measures for small embankments have not yet been fully developed.

The authors have experience with a treated 3 m high dike subjected to a damaging earthquake [1]. This dike section, called the Arashima dike, had to be situated on a liquefiable ground, so when it was constructed in 1996, a geogrid was laid at the bottom of the dike to reduce liquefaction induced deformations during possible future earthquakes, as a pilot project. Four years later, the Tottori-ken Seibu Earthquake hit around this area on October 6, 2000, and yielded valuable data on the effectiveness of the geogrid reinforcement. To establish a design method for reasonable remedial measures for embankments against soil liquefaction, these records should be fully studied.

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## ARASHIMA DIKE SECTION

Arashima dike section is located on the southern shore of the Naka-umi Lake near the mouth of the Ii-nashi River as illustrated in Fig. 1. The height of the section was designed to be 3 m, and the length of the section was 850 m as illustrated in this figure.

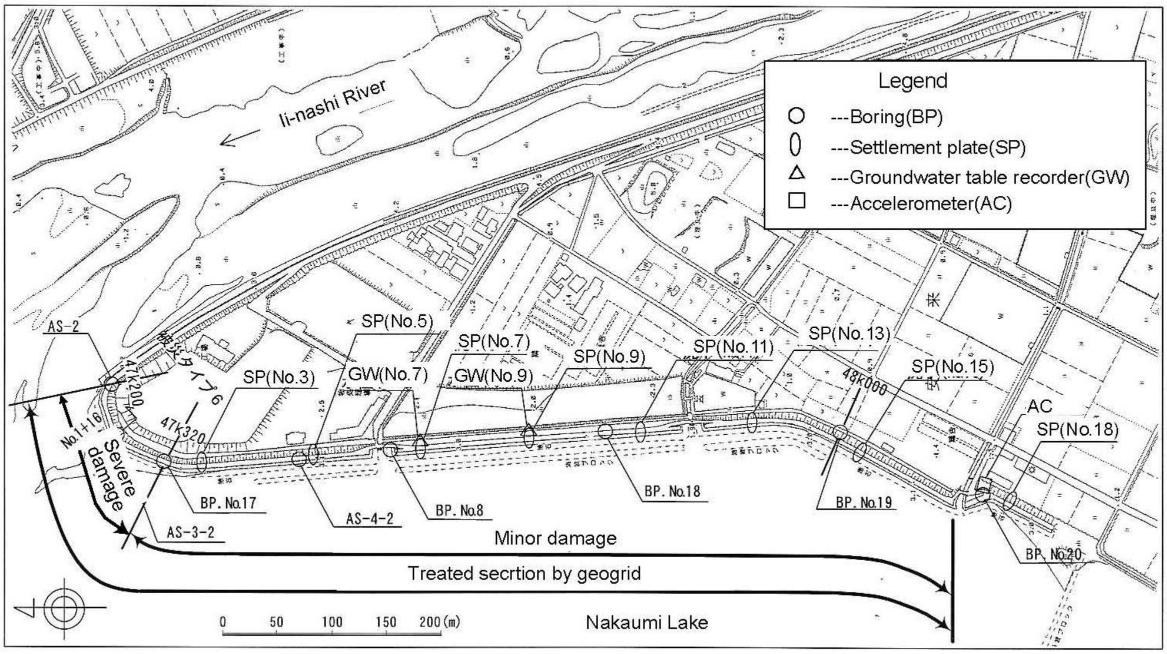



**Fig.1 Plan view of the Arashima dike section**

The soil profile along the dike axis is shown in Fig. 2. The uppermost layer is a loose sand deposit underlain by a clay layer and a lower sandy layer. The thickness of the uppermost sand layer is about 8-10 m except at section No.1 near the river mouth and at No.5.

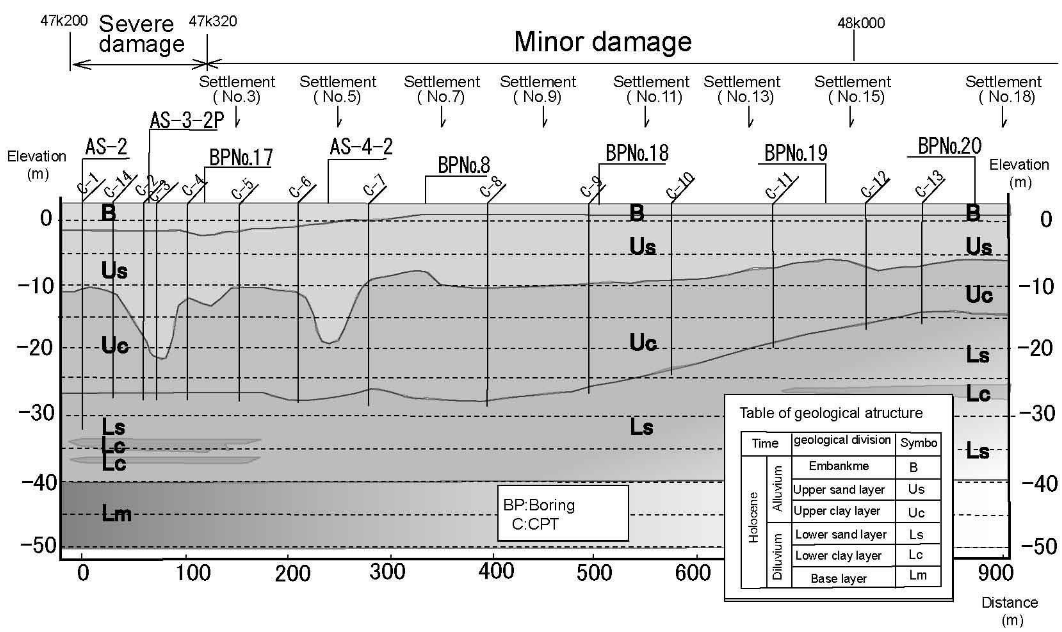



**Fig. 2 Soil profile**

At the design stage in 1996, liquefaction susceptibility was evaluated by the JRA (Japan Road Association) method [2] using measured soil properties. The sand has a median $D_{50}$=0.2-1.7 mm, a uniformity coefficient Uc=3-15, and SPT N values 1-11, with an average value of $N_{ave}$=6. Judging from the thickness where the factor of safety against liquefaction is $F_L$<1.0 for a design acceleration $A_{max}$=0.15 g, the dike had to be remediate against soil liquefaction.

However, the hinterland of this section is mainly used for farming. Therefore it was decided on grounds not to take remedial measures against the liquefaction of the 8-10 m thick sand deposit by soil improvement measures for 3 m high embankment. Instead, the design concept of allowing limited amount dike settlement but avoiding catastrophic deformation of the dike was chosen.

Past cases of embankment failure tells us that embankment body was split by stretching [3], caused by the reduction of shearing resistance between the embankment bottom and the foundation ground due to raised pore water pressure when liquefaction took place at shallow depth as at this site. If this type of failure takes place, more settlement occurs than in the case without stretching.

As a result of these experiences, the primary objective of the remedial treatment at this particular site was to avoid catastrophic deformation caused by the soil liquefaction beneath the dike, though settlement of the dike might take place to a limited extent. For this purpose, it was decided to put geogrid sheets at its bottom to reduce the stretching effect.

Following this concept, three sheets of geogrid were laid as shown in Fig. 3; each sheet of geogrid has a tensile strength of 27 kN/m [4]. Decomposed granite soil was used as construction material for the embankment.

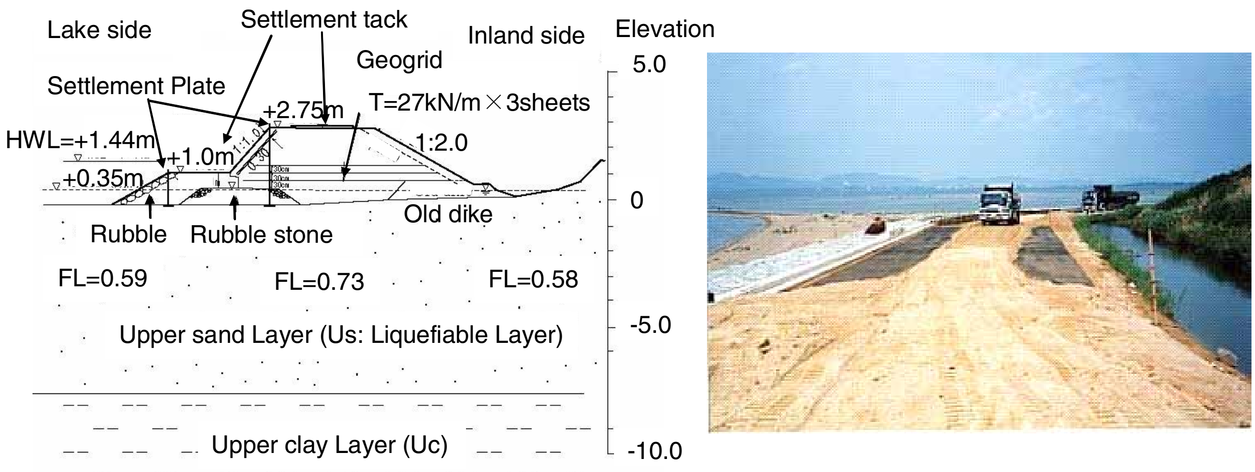



**Fig. 3 Cross section of the Arashima dike section and a view of its construction**

Since there was no other experience of using geogrid as a remedial measure for an embankment against liquefaction, this section was recognized as a test site for geogrid. Instruments were installed to monitor the strong ground motion during an earthquake and to monitor groundwater table changes (not the pore water pressure). For addition, it was decided to check the subsidence of the dike by conducting a survey four times a year. Table 1 shows the instrumentation at this section. The location of the accelerometer and the groundwater table recorders are shown in Fig. 1.

In October, 2000, a survey was conducted on October 5 just before the day of the Tottori-ken Seibu Earthquake and an additional survey was conducted as soon as possible on the day after the event, on October 7. Therefore a valuable record of embankment settlement during an actual earthquake was obtained as described later.

**Table 1 Instrumentations**

| Kind of instrumentation | Location | Amount |
|---|---|---|
| Settlement plate | No. 3,5,7,11,13,15,18 (Shoulder of rubble mound, Outer shoulder) | 16 points (2 points / 1 line) |
| Settlement tack | No. 3,5,7,11,13,15,18 (Outer toe of embankment, Center of crest) | 16 points (2 points / 1 line) |
| Groundwater table gage SDL WL-10 | No.7, No.9 | 2 points |
| Accelerometer | Near No.18 | 1 point |

## PERFORMANCE OF THE ARASHIMA DIKE DURING THE EARTHQUAKE

A damaging earthquake named the Tottori-ken Seibu Earthquake ($M_j$=7.3) hit the area near the site at around 13:30 hours on October 6, 2000, and caused damage to dwelling houses, slope failures, and failures of dikes along the shore of the Naka-umi Lake. Figure 4 shows the maximum horizontal acceleration distribution by this earthquake [5].

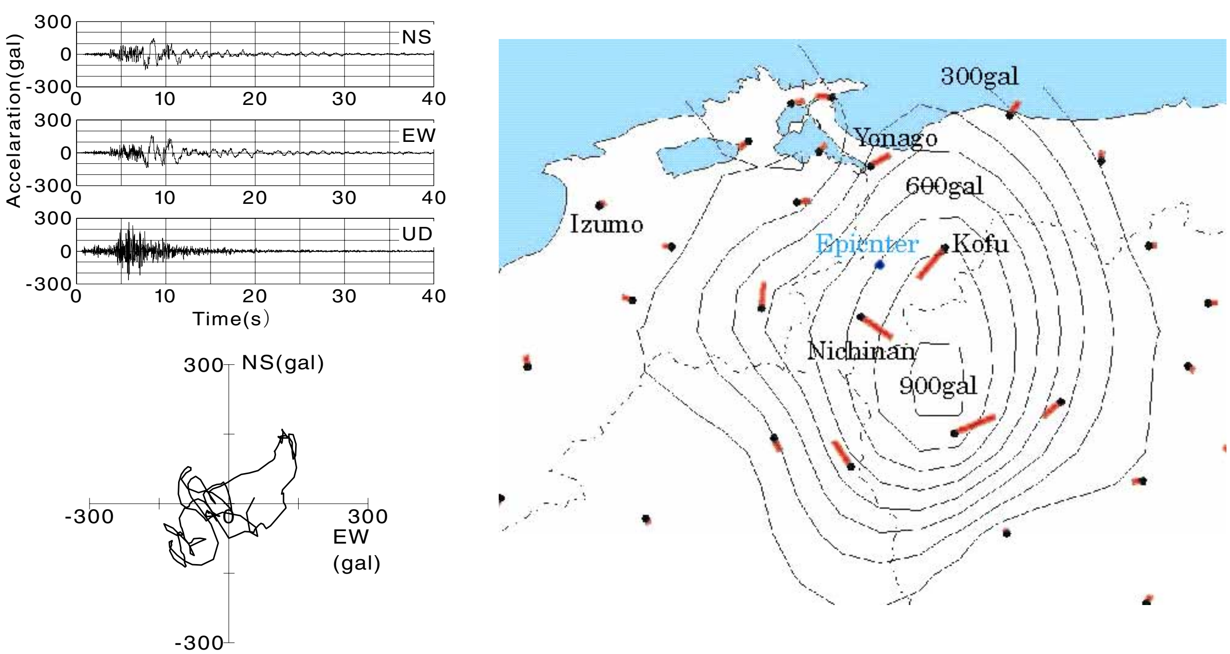



**Fig.4 Observed acceleration and the distribution of the PGA near the epicenter**

Observed maximum acceleration at the Arashima dike was 146 gal in north-south direction, 159 gal in east-west direction, and 267 gal in vertical direction. Synthesized maximum horizontal acceleration was 203 gal. It was recorded that the groundwater table was raised by 20 cm at 14:00 hours at section No. 7 and 9. However, since the groundwater table was recorded only hourly, an accurate account of the groundwater table rise at the time of occurrence of the earthquake was not known. It is known that the rise of the groundwater table remained until about 18:00 hour, four and half hour after the earthquake from the comparison of the recorded values with those in the Lake [1].

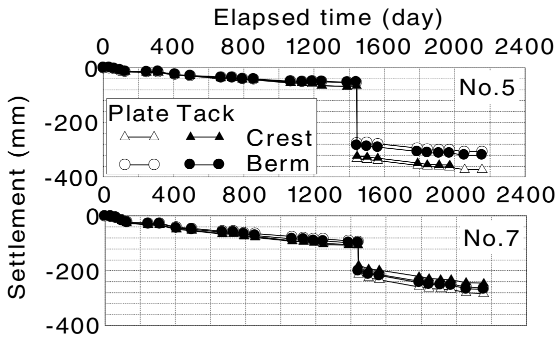
Just after the occurrence of the earthquake an urgent inspection was conducted. Except that the subsidence of about 1 m was detected near the mouth of the Ii-nashi river for about 120 m long, no deformation signals like cracks or subsidence of the embankment, no cracks on the concrete facing on outer slope of the dike, and no sign of damage to rubble mound around the toe of the outer slope was observed for about 80 % of the whole length of the Arashima dike section. Thus it was found that about 80 % of the whole length of the section remained sound during this earthquake.
The length of the severely damaged section was about 120 m, located on the curved part of the dike axis continuing to the river dike of the Ii-nashi River as illustrated in Fig. 1. Many transverse cracks as shown in Fig. 5, and the discrepancy at the joint of concrete facing were observed at this section as shown in Fig. 6, resulting 1 m subsidence of the dike crest. Beside these deformations of the embankment, up-heaving of the bottom of the side ditch occurred as shown in Fig. 7 and traces of sand boiling were observed at the surrounding ground surface near the toe of the embankment. It is considered that the curvatures of this part was a possible cause of the transverse cracks judging by an experimental results described elsewhere [6].

**Fig. 5 Transverse cracks observed at the crest**

**Fig.6 Discrepancy of concrete facing joint**

**Fig.7 Up-heave of the ditch bottom**

Figure 8 shows examples of the time histories of the settlement observed at crest, shoulder, and berm of the dike at 8 cross sections since the completion of the embankment construction. Figure 9 shows the comparison of the crest elevations along the dike axis detected before and after the day of the earthquake.



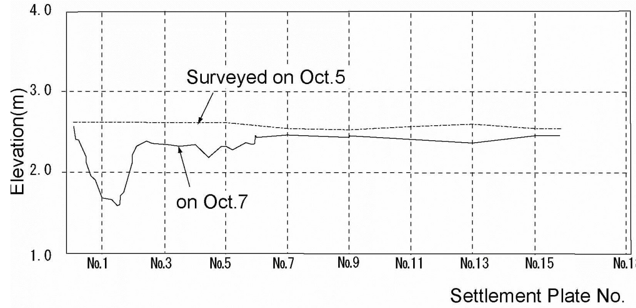




**Fig. 8 Time histories of the settlement after the construction of the dike**

**Fig.9 Comparison of the crest elevations**

From these figures, it is known that the section which seemed to be sound experienced settlement by about 5-20 cm even though no cracks on the embankment crest nor on concrete facing were observed by eye inspection. This is thought to be due to the geogrid preventing the stretching of the embankment. Therefore no emergency treatment of the dike was necessary even though the settlement were about 20 cm.

Figure 10 shows the comparison of the cross section at Section No.1+16 (1 m subsidence point) and the Section No.7 (minor settlement point) surveyed just after the earthquake. It should be noted that there was no longitudinal crack, although the settlement of the embankment crest reached about 1 m at the Section No.1+16.

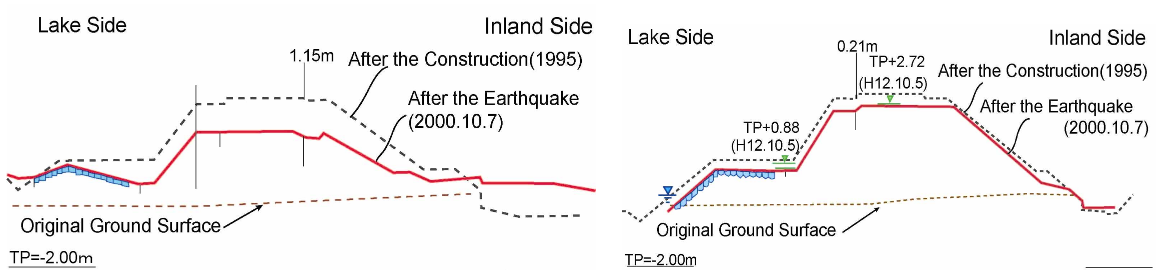
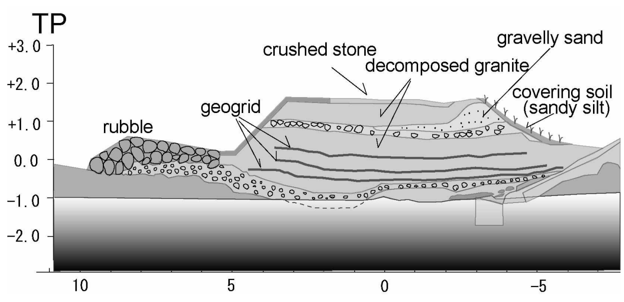



**Fig. 10 Comparison of the cross section at No.1+16 and No.7**

The deformed layout of the geogrid was examined at Section No.1+16 and at Section No.3 during the occasion of repairing works conducted in 2001 as shown in Figs.11 and 12.



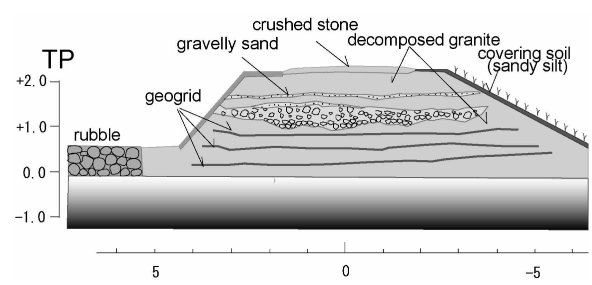




**Fig. 11 Geogrid after the event at No.1+16**

**Fig. 12 Geogrid after the event at No.3**

It was found that the geogrid at the Section No.3 was still horizontal as shown in Fig. 12 even though the crest settlement of about 20 cm was recorded at this section. This fact was in harmony with the aforementioned observation that no deformation was detected on the embankment at this section except for the uniform settlement. Contrary to this, the geogrid at the Section No.1+16 was found to have been bent downwards as shown in Fig. 11 and was also found that the interval distance between the lowest sheet of geogrid and the middle one was slightly decreased. It is considered that not only the foundation ground but the bottom part of the embankment at this section became also to be liquefied so that the sand between these sheets of geogrid could be squeezed out.

## SHAKING TABLE TEST OF THE REINFORCED MODEL EMBANKMENT

A series of shaking table tests on 10 models was conducted in 2003 and on 4 models in 2002.The results of the 2002 experiment was reported elsewhere [7]. Figure 13 shows a model embankment resting on a liquefiable model ground used in the 2003 experiments. Two sets of model embankments, one reinforced by a sheet of model geogrid and the other not reinforced, were set at once on a shaking table and were excited simultaneously by 3 Hz sinusoidal input motion of 200 gal. Model grounds ($D_r$=30 %) were constructed by pluviation in water to make loose saturated layer in a 6 m long, 1 m wide, and 1.5 m high container shown in Fig. 14. Thirty percent glycerin solution was used as pore water so that the dissipation of pore water pressure was delayed. The size of the model embankment was the same for all cases. The thickness, the depth of liquefiable layer and the numbers of cycles of excitation were varied for each case as shown in Table 2. Acceleration, pore water pressure, displacement of the embankment and deformation of the ground were monitored during the shaking. Monitoring points in a model are illustrated in Fig. 15.

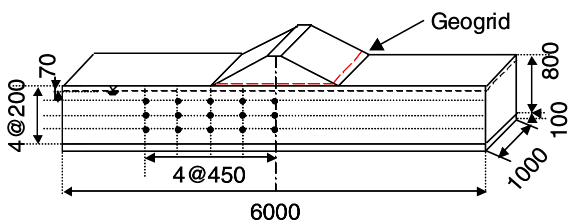



Fig. 13 Model embankment on model ground

Fig. 14 Container

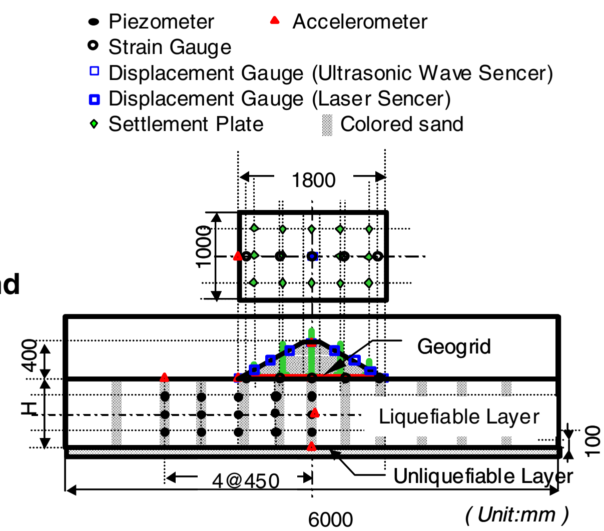
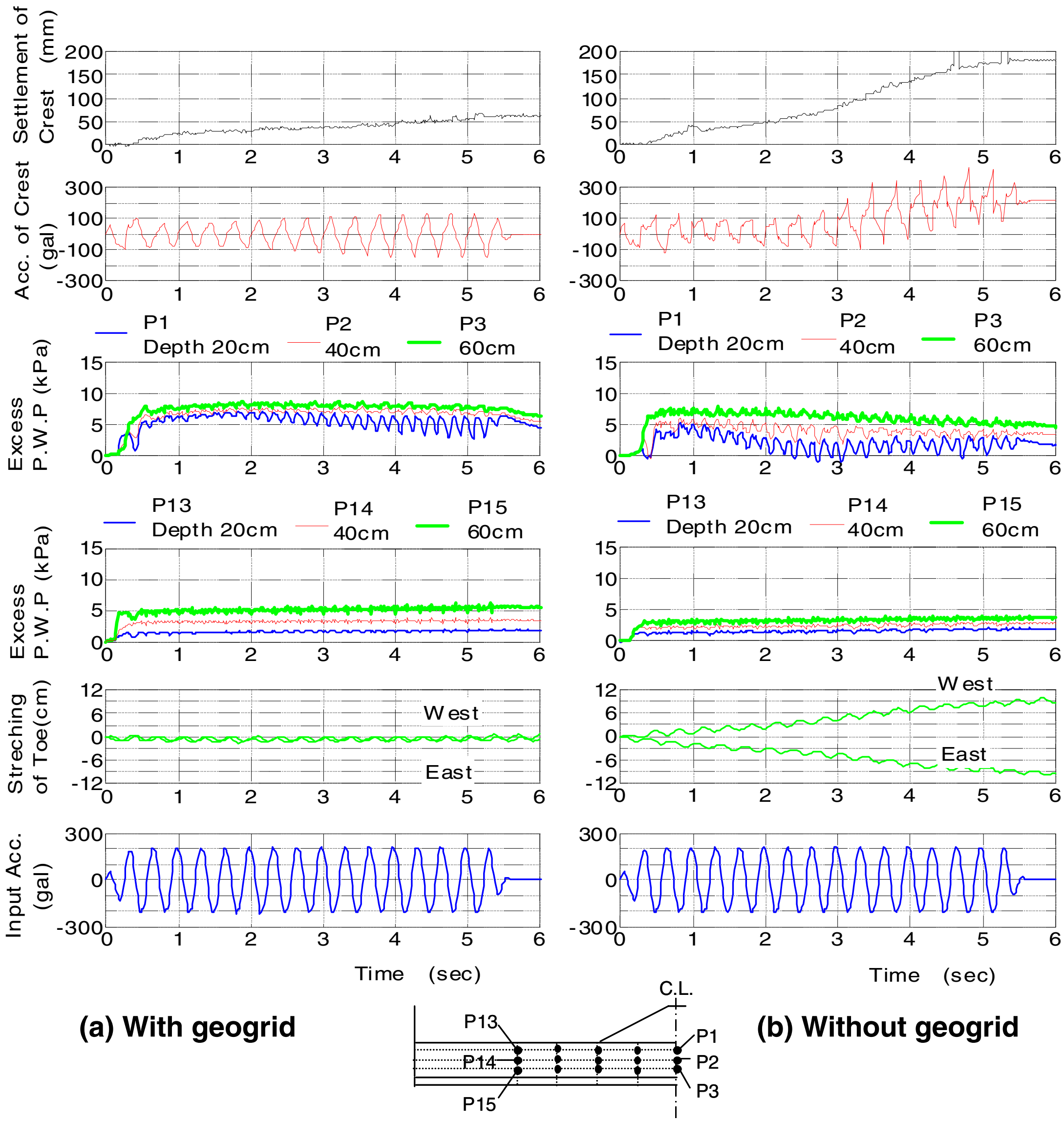



Fig. 15 Monitoring points

Table 2 Models used for shaking tests

| Case No. | H (cm) | Cycles of excitation | Depth of GWT (cm) |
|---|---|---|---|
| 1, 2 | 80 | 5, 5, 10 | -7, -7, -7 |
| 3, 4 | 60 | 5, 3, 10 | -7, -7, -7 |
| 5, 6 | 80 | 5, 3, 10 | -20, -20, -20 |
| 7, 8 | 60 | 5, 3, 10 | -20, -20, -20 |
| 9,10 | 40 | 15 | -7 |

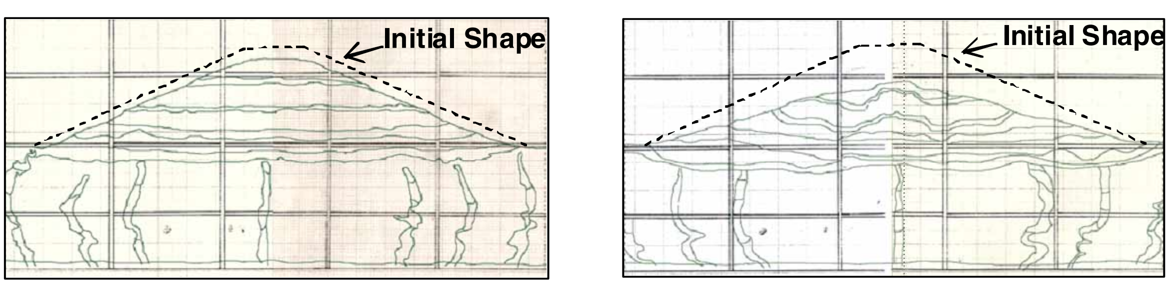
In the second cycle of input motion as shown in Fig. 16, measured pore water pressures in the ground were raised to the initial overburden effective pressure in the ground far from the toe of embankment, signaling full liquefaction except just beneath the embankment. From this moment, the embankment started to settle and continued to subside until the end of excitation. Figure 17 shows a comparison of the deformed shapes of the embankments with and without geogrid.




(a) With geogrid

(b) Without geogrid

**Fig.16 Behavior of a model**




(a) with geogrid

(b) without geogrid

**Fig. 17 Comparison of deformed embankments with and without geogrid**

It is known from Fig. 17 that the settlement of the embankment crest was caused by a decrease of the embankment height and the reduction of the thickness of the foundation ground. These reductions increased almost linearly during the shaking. It should be noted that in case No. 10, where no geogrid was laid at the bottom of the embankment, slippage in the embankment took place around 3 seconds after the beginning of shaking causing farther increase of the crest settlement. However a colored sand layer inside the embankment remained almost horizontal in the case of an embankment with geogrid as illustrated in Fig. 17 (a). This means that a sheet of geogrid prevented the deformation of the embankment although the embankment subsided due to the liquefaction in the foundation ground. Figure 18 shows the total settlement of the embankment crest after 18 cycles of shaking, together with the decrease of the embankment height within it. Figure 19 shows the relationship between the decrease of the embankment height and the stretching amount of the toe.

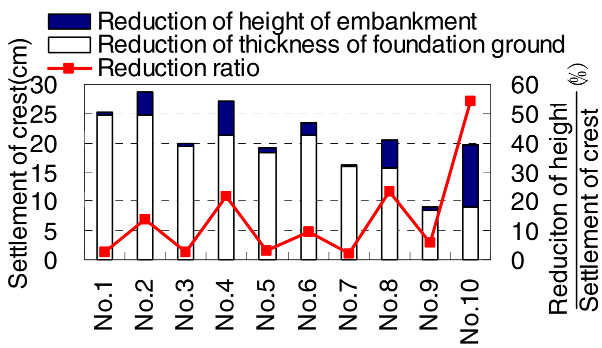



Fig. 18 Total settlement and the reduction of embankment heights

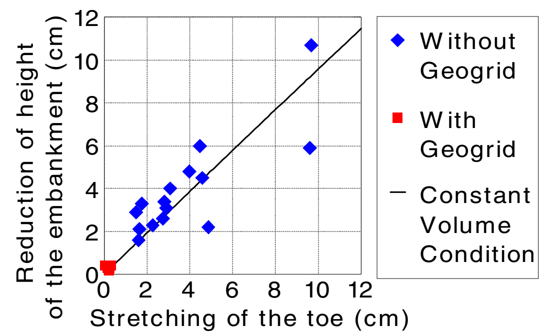



Fig.19 Reduction of the embankment height and the stretching of the toe

From these figures, it is clear that the reduction of the embankment height is well correlated with the amount of stretching in the cases without geogrid. And also it is clear that the installation of the geogrid effectively reduces the deformation of the embankment.

As the results of the embankment deformation, there appeared many longitudinal cracks on the crest surface as shown in Fig. 20. From the photographs at the end of shaking, the numbers of cracks per unit area on the crest surface, crack density, were read off and were plotted against the pavement serviceability index for a road [8] as shown in Fig. 21. It is known from this figure that the seismic deformation of an embankment could be evaluated by crack density and could be expressed by the pavement serviceability index.


Fig. 20 An example of cracks appeared on the surface of the embankment

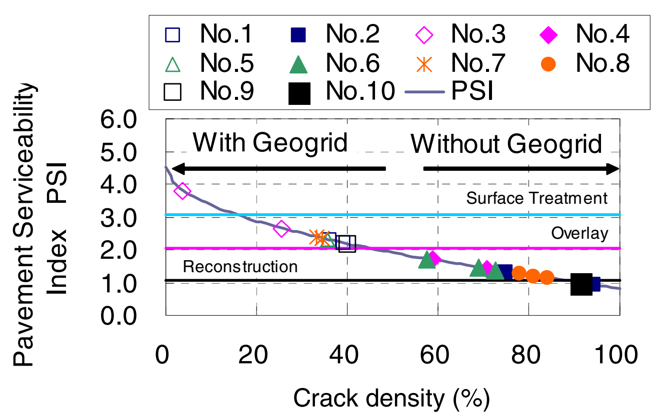



Fig.21 Crack density

As mentioned before the settlement of the embankment crest occurred mainly during the shaking. Therefore assuming that the decrease of embankment height also varied linearly with time, the unit rate of the reduction of the embankment height against the amplitude of the acceleration acting to the embankment was examined. As in the cases without geogrid, the accelerometer was inclined during shaking, so the acceleration records from the cases with geogrid shown in Fig. 22 were used in place of the acceleration for the cases without geogrid. Figure 23 shows the reduction rate of the embankment height in the cases without geogrid. From this figure, it is known that the seismic decrease of the embankment height per one cycle of 100 gal acceleration was governed by the ratio of the thickness of the non-liquefiable layer to the thickness of the liquefiable layer. It was found that this relationship was valid until around 10 – 12 cycles of input motion.

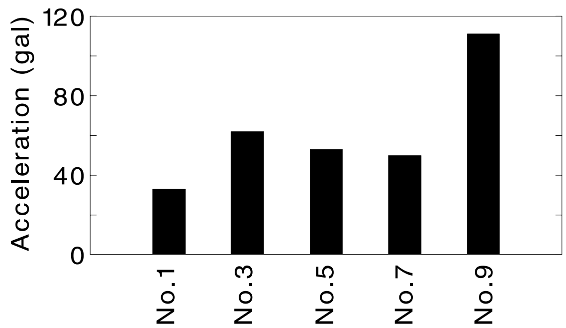



**Fig. 22 Acceleration at the crest**

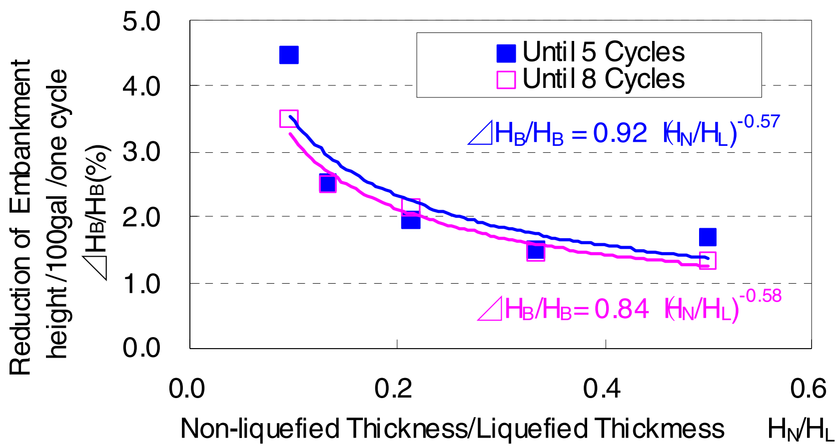



**Fig.23 Rate of the reduction of the embankment height**

Coming back to the total settlement of the embankment, it is mainly composed of the deformation of the liquefied layer beneath the embankment. Figure 24 shows the deformed shape of the colored sand columns beneath the slope of the embankments. The maximum displacement of the colored sand column was observed at the 40-60 % depth of the liquefiable layer. It is also known that the horizontal displacement of the column in the case with geogrid was fairly confined at its top due to the tensile resistance of the geogrid, and this confinement looked effective until 40 % depth of the liquefied thickness as shown by arrows in the figure. In accord with this confinement, the maximum horizontal displacement of the colored column took place at a slightly shallower depth in the case of 80 cm thick ground.

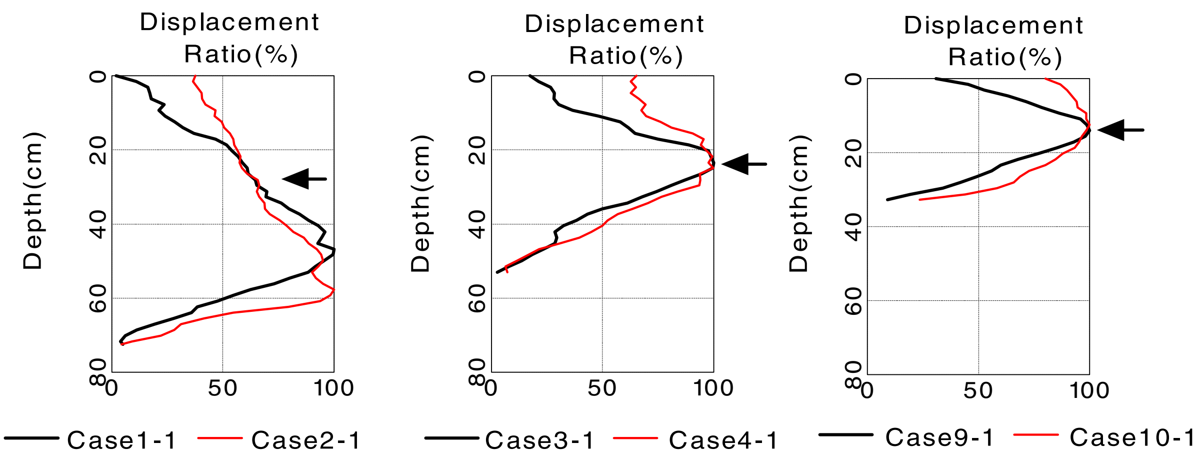



**Fig. 24 Deformed shape of colored sand column**

It is thought that as this deformation of the ground was caused under almost undrained conditions, though not completely undrained, so the deformation in the ground was brought by the shearing strain due to the initial stress induced by the embankment.

Although the deformation of the ground beneath the embankment is not uniform, however to make it simple, a rectangular element of the ground beneath the embankment was taken as illustrated in Fig. 25 (a) for the initial condition. And the final shape of this element was converted to a deformed rectangle as shown in the figure (b), though the deformed shape of the ground was actually barrel like in shape. As the horizontal displacement of the side boundary of this deformed rectangular element was given from no change of volume condition by assuming the undrained condition, then the vertical strain, horizontal strain, and the maximum shear strain of the rectangular element were simply calculated. Figure 26 shows the relation between calculated vertical strain and the thickness of the layer. In this figure, the vertical strain of the liquefied layer is divided by cycles of shaking and the overburden pressure considering the stress dispersion inside the non-liquefiable layer [9].

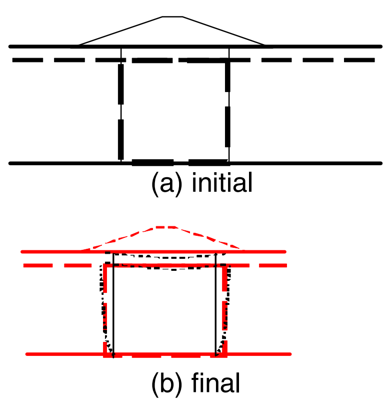



**Fig. 25 Rectangular element of the ground beneath the embankment**

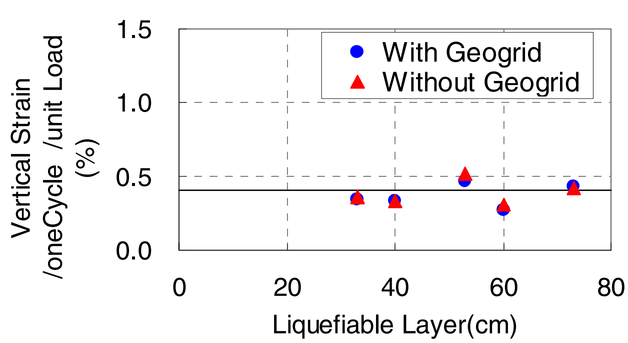



**Fig. 26 Vertical strain of the liquefied layer**

It is known that the vertical strain per cycle of the shaking was independent from the thickness of the liquefied layer as shown in Fig. 26, and took the unique value of 0.4 %/100 gal/cycle/1 kPa in this experiment, regardless the installation of the geogrid. And the decrease of embankment heights for the case without geogrid is already shown in Fig. 23. So the settlement of the embankment crest can be estimated from the results of the experiment in 2003 as follows.

$$\left.\begin{aligned} &S = \Delta H_L + \Delta H_B \\ &\Delta H_L = \varepsilon_v \times N \times \sigma_v \times H_L \\ &\Delta H_B = 0.84 \times \left( {H_N}/{H_L} \right)^{-0.58} \times H_B \text{ (Until 8 wave cycles)} \end{aligned}\right\} \text{-------------- (1)}$$

where $S$: the crest settlement, $\Delta H_L$: the vertical deformation of the ground, $\Delta H_B$: the decrease of embankment height, $\varepsilon_v$: unit vertical strain in the liquefied layer, $N$: number of cycles, $\sigma_v$: the initial overburden pressure to the liquefied layer, $H_L$: the thickness of the liquefied layer, $H_N$: the thickness of the non-liquefied layer and $H_B$: the height of embankment.

The crest settlements observed during the experiment in 2002 are shown against thus estimated settlement in Fig. 27. From this figure, it is clear that the total settlement of the top crest in the 2002 experiment is effectively estimated.

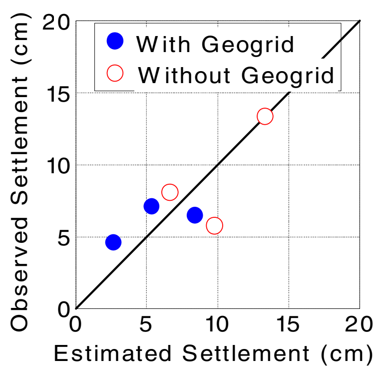



**Fig. 27 Observed and estimated crest settlement**

Next, the dependency of the above vertical strain on the initial relative density of the liquefied layer is needed to compare the test results with the actual settlement of the Arashima dike. The settlement of a model embankment into the liquefied foundation layer after an instantaneous liquefaction was given by the authors [10] as follows. In those model tests, embankment model did not deform, so $\Delta H_B$=0.

$S/H_L=10^{2.096}$ x $D_r^{-1.932}$ -------------- (2)

This empirical equation was derived from results on small scale model tests under a level of 1 kPa of the overburden pressure on the liquefiable layer.

Knowing that the dependency on the relative density is proportional to $D_r^{-2}$, and that $D_r$ is around 60 % at field, the unit vertical strain is known from Fig. 26 and equation (2) as 0.4 x $(60/30)^{-2}$ %/100 gal/ cycle/1 kPa, namely the unit vertical strain at site is 0.1 %/100 gal/ cycle/1 kPa.

Since it is known from the acceleration record at site shown in Fig.4, the numbers of cycles of acceleration exceeding 100 gal was 2.5, and knowing that the overburden pressure at site was 37 kPa, the estimated vertical strain for the foundation ground beneath the Arashima dike induced by the Tottori-ken Seibu Earthquake is shown to be 9-10 %.

The observed settlement at the sound section was 20 cm, and 1 m at the severely damaged section. These settlement amounts and the above mentioned vertical strain implies that the liquefied thickness was about

2-3 m at the sound section, and about 10-11 m at the severely damaged section. These thicknesses of the liquefied layer were close to the thickness estimated by a numerical analysis [11].

The deformation of the ground beneath the embankment caused a decrease of pore water pressure due to the dilatancy effect, and then the strength of soil beneath the embankment was not fully lost even in the stages of shaking. This is considered to be the reason why the deformation of liquefied soil is limited and does not flow infinitely.

Figure 28 shows the distribution of the pore water pressure ratio in the ground at the end of shaking.

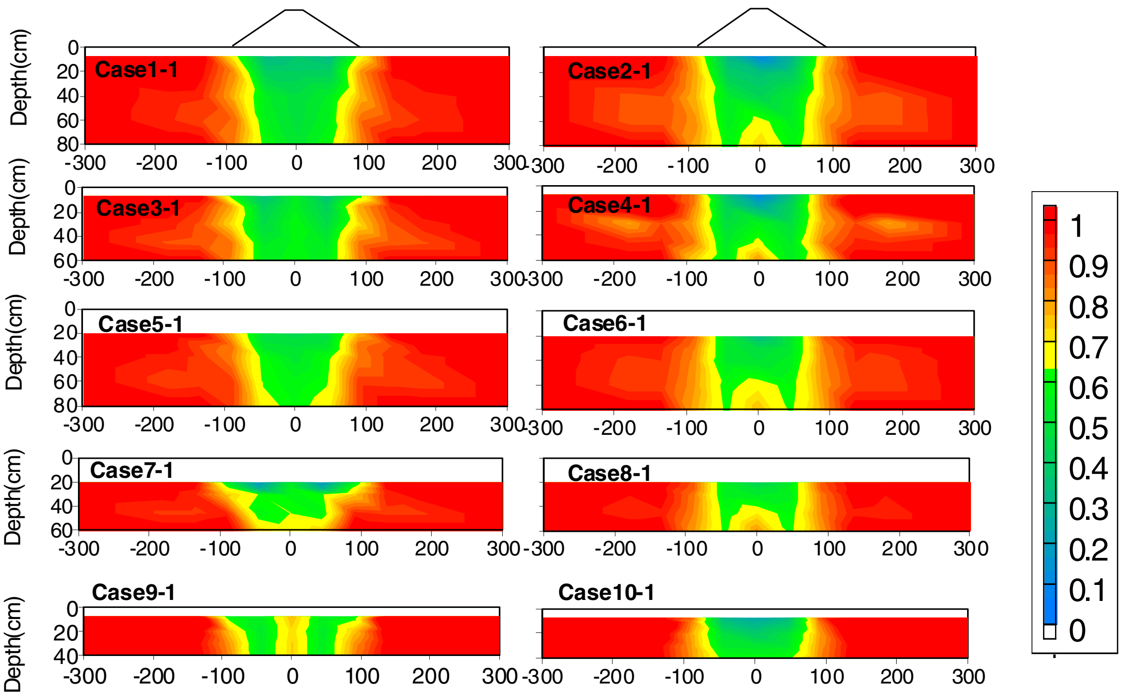



**Fig. 28 Distribution of the pore water pressure ratio at the end of shaking**

Effective stress condition could be read from this figure as (1- P.W.P ratio).

Effective stress conditions of the aforementioned rectangular element of the ground beneath the embankment are plotted against the shear strain in the rectangular element in Fig. 29. In this figure is also shown the test results on the effective stress recovery of the liquefied specimen during shear in the hollow cylinder torsion test by Yasuda et al. [12].

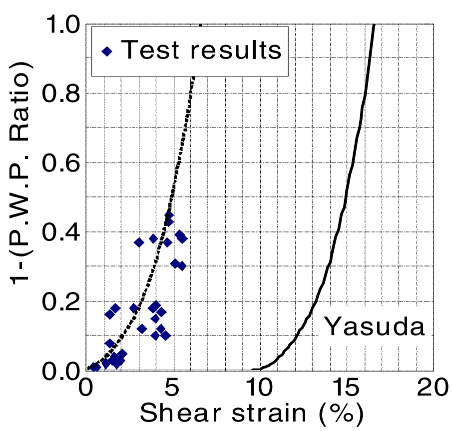



**Fig. 29 Recovery of the effective stress due to shear strain**

It is known from this figure that the effective stress recovery becomes larger when the shear strain becomes larger both in the shaking table test and in the hollow cylindrical specimen. However, it is noticed that the effective stress recovery was induced in the smaller range of the shear strain in the shaking table test than the ones in the hollow cylindrical specimen. The reason of this difference is not known. It should be pointed out that the knowledge on the post liquefaction strength of soils should be accumulated.

## CONCLUSION

The performance of the Arashima dike reinforced with geogrid during the Tottori-ken Seibu Earthquake was described. Since this type of remediation was a first attempt, the newly constructed section of the dike had been monitored for subsidence, variation of the underground water table and strong motion during earthquakes.

The Tottori-ken Seibu Earthquake caused damage on a 120 m long section within the 850 m long dike which subsided by about 1 m. However 80 % of the treated length of the section survived without damage. Although the survey results showed that the sound length of the section suffered from settlement of 20 cm, no visible deformation was observed on the dike or on the concrete facing which had been laid on the outer slope.

The results from a series of shaking table tests revealed that the reinforcement by geogrid does not reduce the settlement due to the deformation in the liquefied layer; however it does change the deformation amount of an embankment remarkably. It was shown that the preventive effect against deformation of embankment by the reinforcement can be evaluated by crack density.

Thus it is concluded that the reinforcement by geogrid is effective for an embankment against soil liquefaction.

It was also attempted to derive a simplified method to estimate the settlement of an embankment due to the large deformation of the underneath liquefiable layer. The proposed simplified method could effectively estimate the actual settlement of the Arashima dike section. This attempt was made based upon the assumption that the liquefied soil recovers its strength in accordance with its deformation, and compared the effective stress recovery in the model ground obtained from the shaking table test with the stress recovery in a hollow cylindrical specimen. It was pointed out that the knowledge on the post liquefaction strength recovery during the deformation should be accumulated.

## ACKNOWLEGEMENT

Authors are grateful for the assistance of Mr. T. Murakawa and Mr. T. Ikeoka in conducting the shaking table tests.

## REFERENCES

1. Sasaki Y, Fukuwatari T, Tsuji T, Kano S, Sawada S, and Uekuma H. “A study on the performance of a reinforced dike section with geogrid during the Tottoriken-Seibu Earthquake.” Proceedings of the Fifth International Conference on Case Histories in Geotechnical Engineering, New York, U.S.A., 2004 (in printing).
2. Japan Road Association, “Specification for Highway Bridge”, 1996. (in Japanese)

3. Sasaki Y, Moriwaki T, and Ohbayashi J. “Deformation process of an embankment resting on a liquefiable soil layer.” Deformation and progressive failure in geomechanics, Proceedings of the IS-Nagoya’97, Nagoya, Japan, 1997: 553-558.
4. Kanayama Y, Abe T, Arase Y. and Kamatsu H. “The remedial measure by geotextiles for a river dike against soil liquefaction and field observation for its effectiveness during an earthquake.” Ground Engineering, 1997: 15(1): 47-52. (in Japanese)
5. Investigation committee of Tottori-ken Seibu Earthquake, “Report on the immediate investigation of the damage by the Tottori-ken Seibu Earthquake.” Tsuchi-to-kiso, 2001: 49(1): 32-35. (in Japanese)
6. Kano S, Sasaki Y, Hata Y, and Eno H. “Experimental study on a seismic response of a dike at a curved section.” Journal of Earthquake Engineering, JSCE, Paper no.248, 2003: 27 (CD-Rom). (in Japanese)
7. Sasaki Y, Kano S, Murakawa T, Tsuji T, “Shaking table tests on effectiveness of the reinforcement for an embankment by geogrid against soil liquefaction.” Geosynthetics Engineering Journal, 2003: 18: 267-274 (in Japanese)
8. Fuse Y, Editor. “Road (V) -maintenance-, New Series of Civil Engineering”, Gihodo Shuppan, 1984: 21-36. (in Japanese)
9. Sasaki Y, Finn WDL, Shibano A, and Nobumoto M. “Settlement of embankment above a liquefied ground which is covered by non-liquefiable surface layer.” Proceedings of the Sixth Japan - U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Soil Liquefaction, 1996: 361-390.
10. Sasaki Y, Ohbayashi J, Shigeyama A, and Ogata Y. “Model tests on a seismic failure of an embankment due to soil liquefaction.” Proceedings of the Second International Conference on Earthquake Geotechnical Engineering, Lisbon, Portugal, Rotterdam: AA Balkema, 1999: 2: 691-696.
11. Sasaki Y, Fukuwatari T, Sakamoto Y, Kano S, Sawada S, and Uekuma Y. “Study on the performance of a reinforced dike section with geogrid during the Tottoriken-seibu Earthquake Geosynthetics Engineering Journal, 2002: 17: 207-214. (in Japanese)
12. Yasuda S, Yoshida N, Adachi K, Kiku H, Gose S, and Masuda T. “A simplified practical method for evaluating liquefaction-induced flow.” Journal of Geotechnical Engineering, JSCE, 1999: 49(4): 71-89. (in Japanese)