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A STUDY ON THE PERFORMANCE OF A REINFORCED DIKE SECTION WITH GEOGRID DURING THE TOTTORI-KEN SEIBU EARTHQUAKE

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ABSTRACT

A dike section reinforced with geogrid against soil liquefaction was hit by a damaging earthquake in 2000. As the site condition of this dike section was anticipated to be liquefied by an earthquake, so it was decided to take remedial measures against possible damage. However, since the hinterland of this newly constructed dike section was a rural area, primary objective of the treatment design was set to avoid the stretching effect caused by the liquefaction. For this purpose, three sheets of geogrid were laid at the bottom of the embankment.

It was detected that 85% length of the dike section survived successfully without apparent damage after the earthquake. As this type of remedial treatment was a first attempt, this section was instrumented to monitor strong ground motion, change of groundwater level, and the subsidence of the dike since the completion of the embankment construction in 1996.

Thus the effectiveness of the geogrid was shown together with the instrumented records on the behavior of the dike section during the earthquake.

INTRODUCTION

Embankments, especially river dikes among them have often been damaged in the past earthquakes due to soil liquefaction. However it is rare that an embankment which had been remediated meet an actual damaging earthquake.

Most of the remediation for embankments against soil liquefaction is usually conducted for big or important embankments which must secure the urbanized important area based on the results of seismic diagnosis, and the ordinary remediation is accomplished by ground improvement such as SCP or DMM. Not only for the big dikes, but for small dikes, preventive measures are desired to be established. Such preventive measure must be ones which can be strengthened step-by-step-wise in accord with the development of its hinterland. However, effective and reasonable measures for such dikes have not yet fully established. It is considered much rarer that such a treated small dike meet a damaging earthquake. Authors have never heard about the case that small embankment which had been remediated meet an actual damaging earthquake.

In such a situation, authors have experienced a case that a treated 3 m high dike met a damaging earthquake.

A part of a dike section became to be newly constructed on the shore line of the Naka-umi Lake in 1996. This dike section, named Arashima dike, had to be situated on a liquefiable ground, so when it was constructed in 1996, geogrid was laid at its bottom of the dike to reduce the liquefaction inducing deformation against the possible future earthquakes.

Four years later, the Tottori-ken Seibu Earthquake (M_j=7.3) hit around this area at around 13:30 on October 6, 2000, however this section survived with minor damage although there were detected severely damaged sections to other locations of the dike surrounding the Naka-umi Lake.

This paper describes about the performance of the treated dike sections during the earthquake.

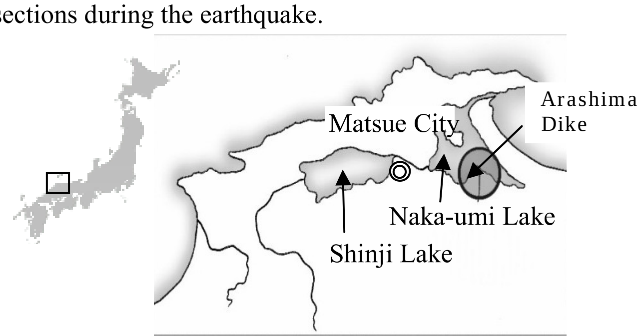

Fig. 1. Location of the Arashima dike section

ARASHIMA DIKE SECTION

Arashima dike is the dike section which is located on the southern shore of the Naka-umi Lake near the mouth of the Ii-nashi River continuing to its left-hand side bank as illustrated in Fig. 1 and Fig. 3. The height of the section was designed to be 3 m as shown in Fig. 2, and the length of the section was 850 m as illustrated in Fig. 3.
Soil profile along the dike axis is shown in Fig. 4. Uppermost layer is loose sand deposit underlain by clay layer and lower sandy layer. The thickness of the uppermost sand layer is about 8-10m except for at section No.1 near the river mouth and at No.5.

Design concept

At the design stage in 1996, liquefaction susceptibility was evaluated by JRA (Japan Road Association) method using obtained soil properties, such as that particle size of the sand D50=0.2-1.7mm, uniformity coefficient Uc=3-15, and SPT N value. SPT N value of the layer varies within 1-11, and average value was N_{ave}=6. Judging from the depth where the factor of safety against liquefaction Fl<1.0 estimated against design acceleration A_{max}=0.15g, liquefiable thickness was thought to be almost the same as the whole thickness of the uppermost sand layer.
The hinterland of this section is mainly used for farming. It was hesitated from the economical reason to take remedial measure against liquefaction of 8-10m thick sand deposit by soil improvement techniques for 3 m high embankment. Instead, strengthening-able method, allowing limited amount of dike settlement but avoiding catastrophic deformation of the embankment, was chosen as the design concept.
Past cases of embankment failure tells us that embankment body was split as shown in Fig. 5 (Sasaki et al., 1997) by stretching effect (Terzaghi, Peck and Mesri, 1996), caused by lost of shearing resistance between the embankment bottom and the foundation ground due to raised pore water pressure when liquefaction took place at shallow depth like this site.

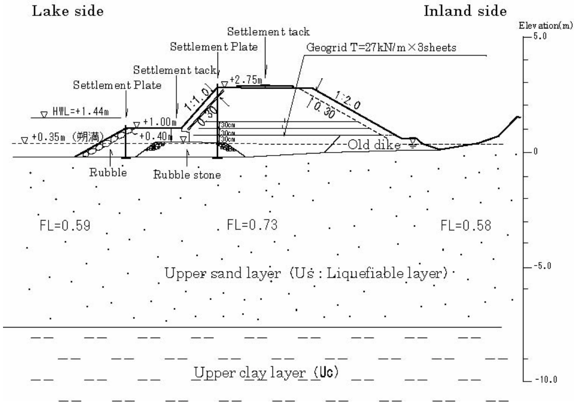

Fig. 2. Cross section of the Arashima dike section

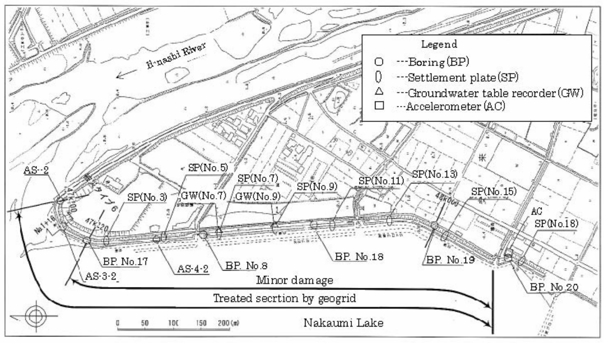

Fig. 3. Plan view of the Arashima dike section

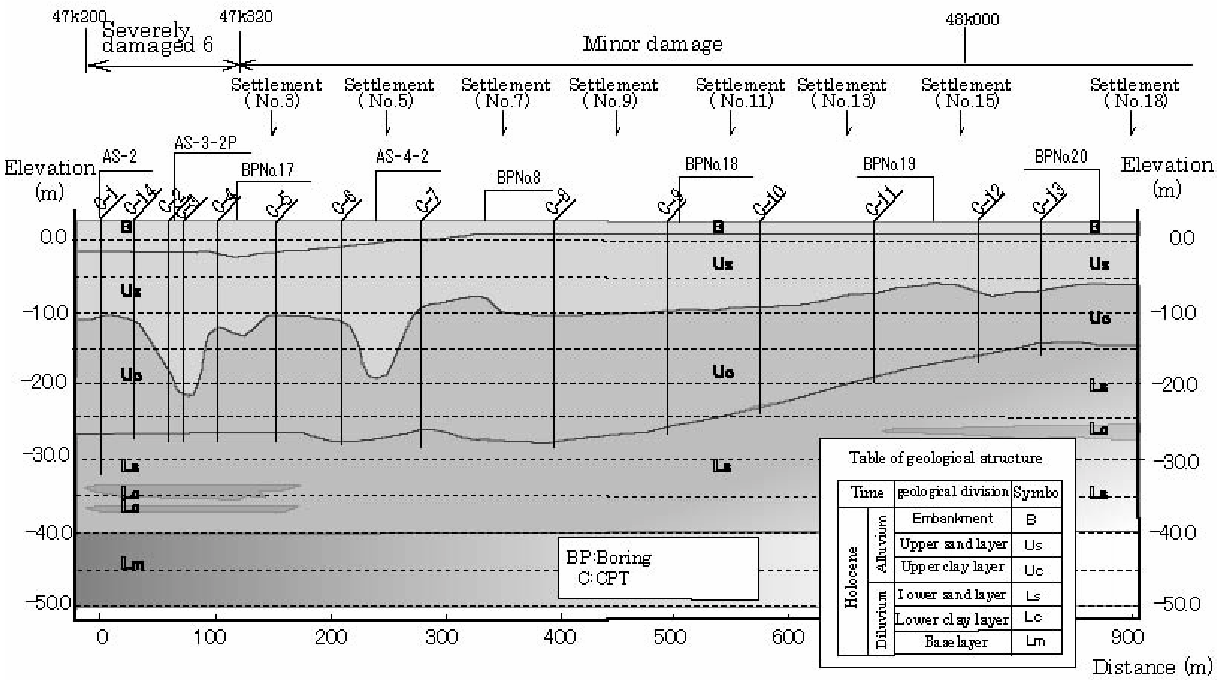

Fig. 4. Soil profile

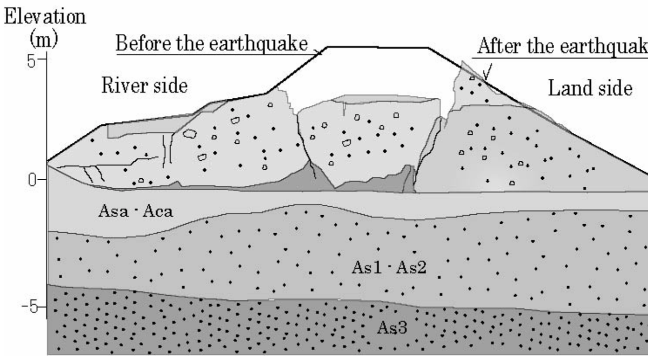

Fig. 5. Shiribeshi-toshibetsu river dike

If this type of failure takes place, the amount of settlement increases than the case without stretching, and the emergent treatment is needed for a long distance soon after the occurrence of earthquake in the confused situation. Past cases also tells us that maximum settlement of embankment did not exceed 75% of the original height of embankments (Sasaki et al. 1999).

From these experiences, primary objective of the remedial treatment at this particular site was set to avoid catastrophic deformation caused by the soil liquefaction beneath the dike, though settlement of the dike might take place to somewhat extent. For this purpose, it was decided to put geogrid sheet at its bottom to reduce the stretching effect. It was judged that the treated dike could be strengthened by additional treatment such as to insert sheet pile which has ability to drain excess pore water pressure around the toe of the embankment slope or to improve the soil next to the toe of the embankment slope when it becomes necessary in accord with the development of hinterland usage.

Fig. 7. Construction of the embankment with geogrid

Standard cross section

Figure 2 shows the standard cross section designed along this concept. As a design method to calculate the necessary tensile strength of the geogrid for this purpose is not established, four type of trial calculation were conducted and decided to select the maximum value as the necessary tensile strength for the used geogrid at this site to be 67kN/m. And three sheets of geogrid were laid (27kN/m x 3 =81kn/m>1.2 x 67kN/m), each sheet of geogrid has 27 kN/m as its tensile strength(Kanayama, et al., 1997).

Decomposed granite soil was used as construction material for

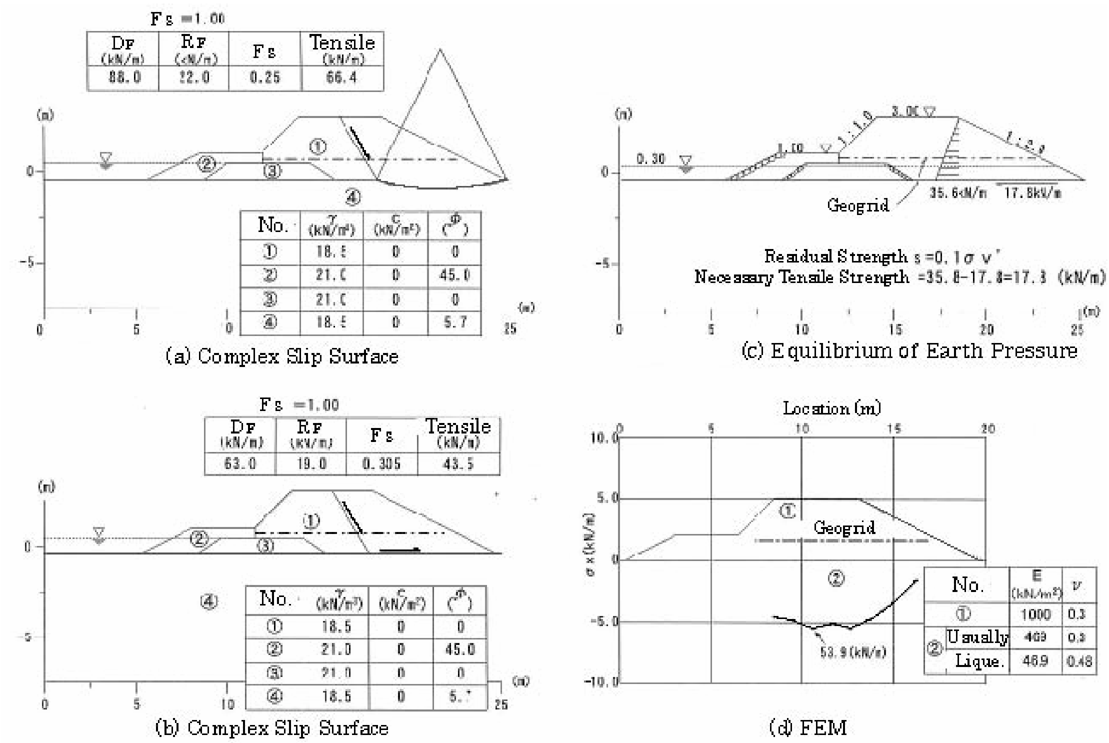

Fig. 6. Calculation of necessary tensile strength of geogrid

the embankment (Fig. 8). Compaction ratio was 89%, the wet density of the embankment was 17.3～17.9 kN/m^3. Outer slope of the dike section was covered by concrete facing as illustrated in Fig. 2.

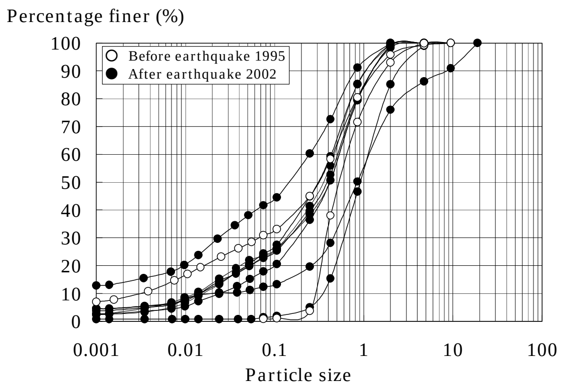

Fig. 8. Particle size distribution curve of the embankment material

Instrumentation at the section

Since there is no other experience of using geogrid as a remedial measure for embankment against liquefaction, this section was recognized as a test site of using geogrid, and then instruments were installed to monitor the strong ground motion during an earthquake and to monitor the groundwater table change (not the pore water pressure). Adding to these instrumentations, it was decided to check the subsidence of the dike by conducting survey four times a year.

Table 1. Instrumentations

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

Table 1 shows the instrumentation at this section, the location of the accelerometer and the groundwater table recorders are shown in Fig. 3.

In October, 2000, survey was conducted on October 5 just before the day of the Earthquake, and off course, additional survey was conducted urgently on the next day of the event, October 7. Thus precious record of embankment settlement during an actual earthquake was obtained as described later.

PERFORMANCE OF THE ARASHIMA DIKE DURING THE TOTTORI-KEN SEIBU EARTHQUAKE

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 on October 6, 2000, four years later after the completion of the Arashima dike. Figure 9 shows the maximum horizontal acceleration distribution by this earthquake (Investigation commission, 2001).

Figure 10 shows the acceleration record observed at the crest of the Arashima dike during the event.

Observed maximum acceleration was 146 gal in north-south

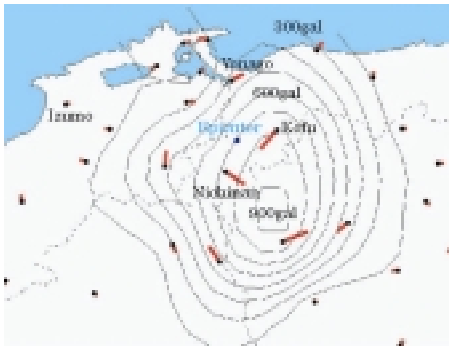

Fig.9. Distribution of the maximum synthesized horizontal acceleration near the epicenter

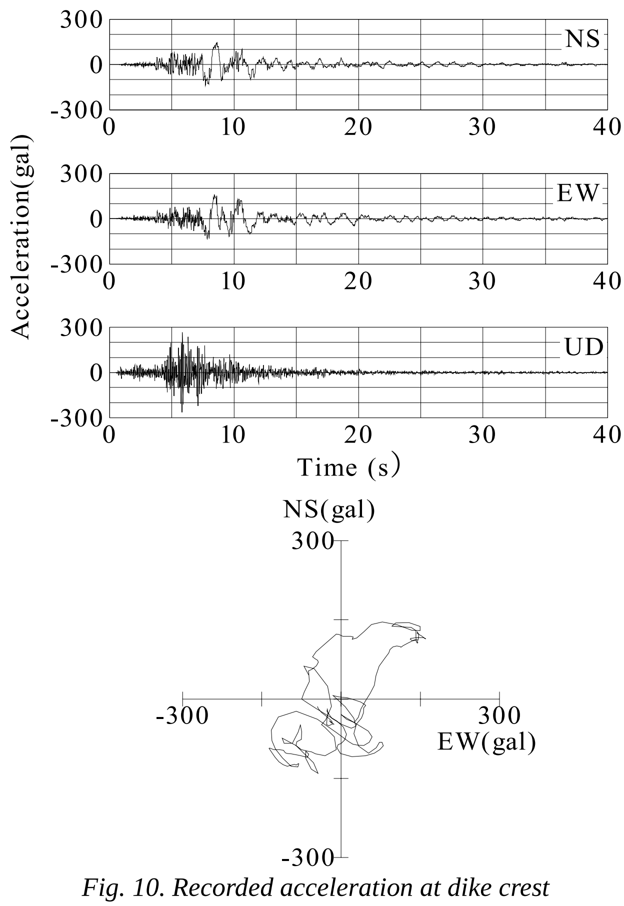

Fig. 10. Recorded acceleration at dike crest

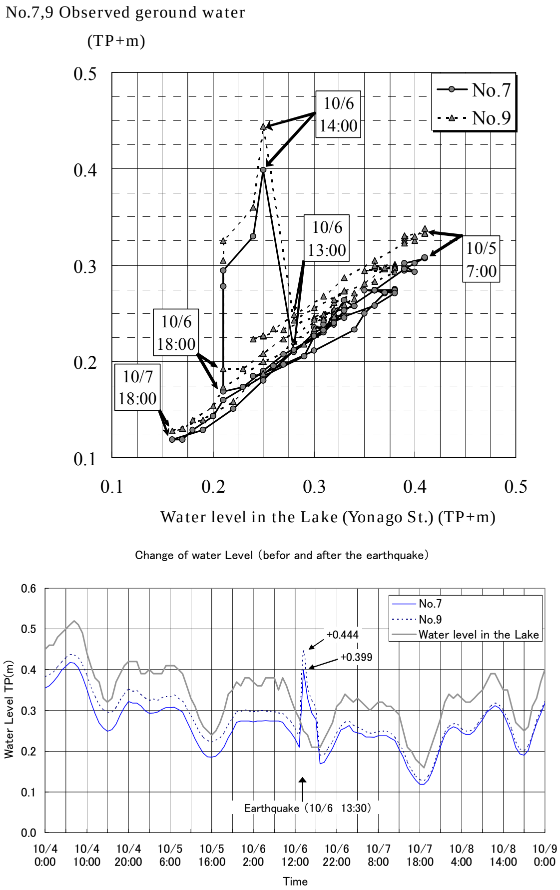

Fig. 11. Recorded groundwater table

direction, 159 gal in east-west direction, and 267 gal in vertical direction. Synthesized maximum horizontal acceleration was 203 gal. This value is slightly smaller than those values read off from the contour map drawn by using the observed records at nearby observatories as illustrated in Fig. 9.

The groundwater level

Figure 11 shows the records of groundwater table observed at section No. 7 and 9. It was recorded that the groundwater table was raised by 20 cm at 14:00. However, since the groundwater table was hourly recorded only, accurate amount of the groundwater table rise at the time of occurrence of the Earthquake was not known. It is known that the effect of the groundwater table rise seemed to remain until about 18:00, four and half hour later after the earthquake from the comparison of the recorded value to those in the Lake.

Performance of the treated dike section during the Earthquake

Just after the occurrence of the Earthquake, 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 signal like cracks or subsidence of the embankment, no cracks on the concrete facing on outer slope


Fig. 12. Transverse cracks observed at the crest


Fig. 13. Discrepancy of concrete facing joint


Fig. 14. Up-heave of the ditch bottom

of the dike, nor no signal 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 seemed 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. 3. Many transverse cracks as shown in Fig. 12, and the discrepancy at the joint of concrete facing as shown in Fig. 13 were observed at this section, resulting 1 m subsidence of the dike crest. Beside these deformation to embankment, up-heaving of the bottom of the side ditch as shown in Fig. 14 and traces of sand boiling at surrounding ground surface near the toe of the embankment were observed. A fisherman told his fear to one of the authors as he watched the sand and water boiling from the lake bed after he felt the shaking when he was catching corbicula in the lake.
Figure 15 shows the time histories of the settlement observed at crest, shoulder, and berm of the embankment at 8 cross sections since the completion of the embankment construction. Figure 16 shows the comparison of the crest elevations along

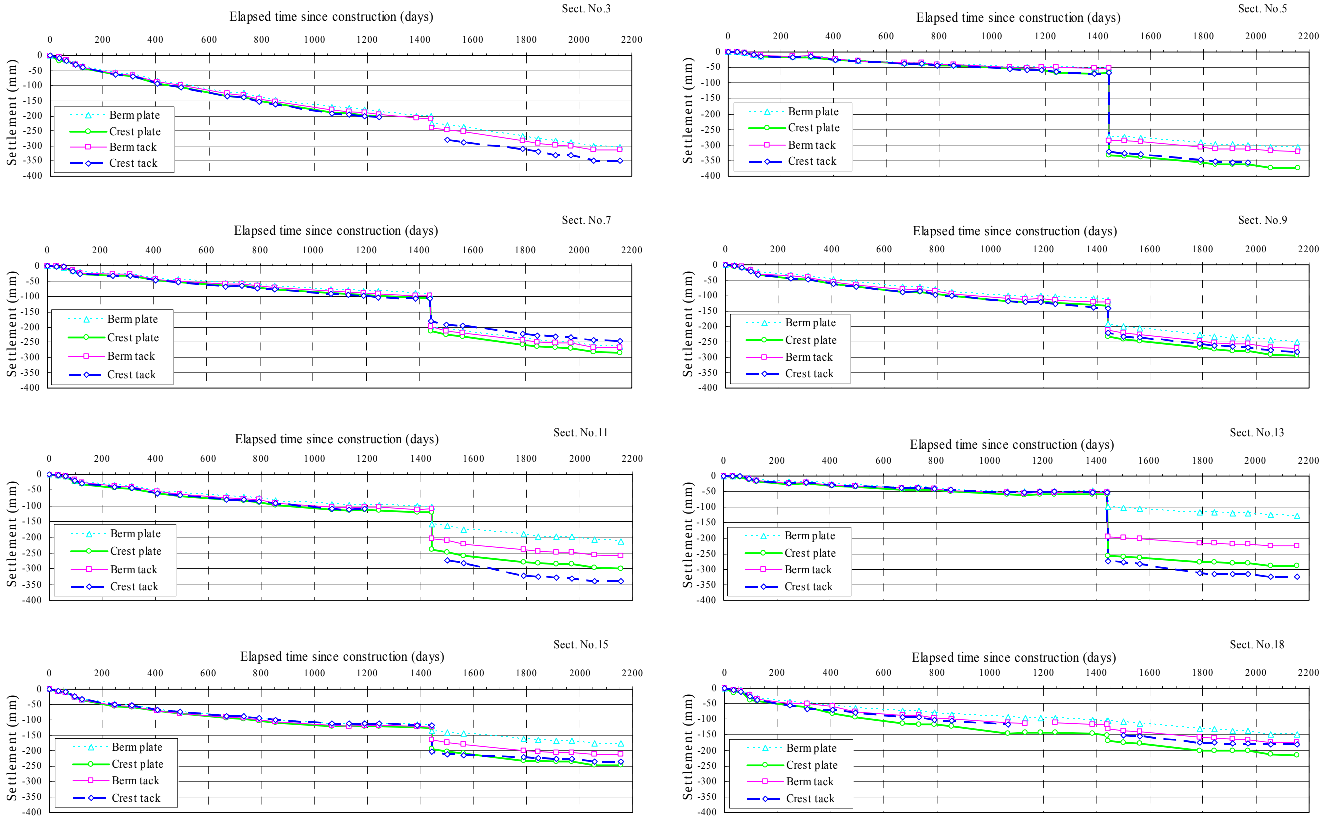

Fig. 15. Time histories of the settlement

the dike axis detected before and after the day of the Earthquake

From these figures, it is known that the section which seemed to be sound experienced settlement by about 5-20cm even though no cracks on the embankment crest nor on concrete facing were observed by eye inspection. This is thought to be owing to an effect of geogrid against stretching of the embankment, and it did help to avoid the emergency treatment to the shaken dike even though the settlement was caused for about 20 cm.

Figure 17 shows the comparison of the cross section at Section No.1+16 (1 m subsidence point) and the Section No.7 (minor

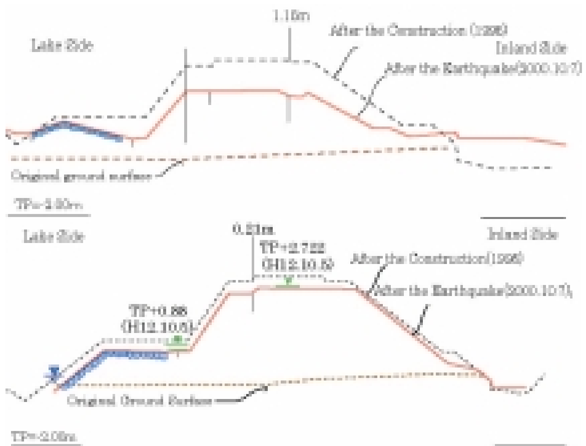

Fig. 17. Comparison of the cross section at No.1+16 and No.7

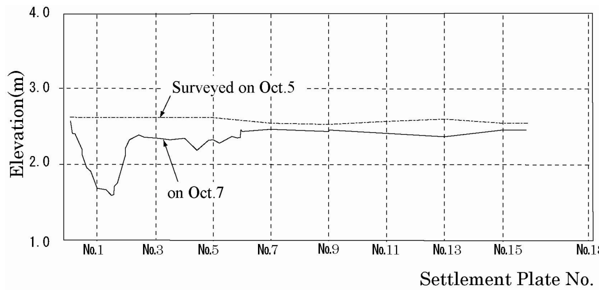

Fig.16. Comparison of the crest elevations

settlement point) surveyed just after the earthquake. It should be noted that there was no longitudinal crack though the settlement amount of the embankment crest reached to about 1 m at the Section No.1+16.

The reason why this portion of the Arashima dike section was caused the severe subsidence is not yet clarified, however it could be pointed out that this portion is situated on the ground where liquefiable sand layer is comparatively thicker, and that

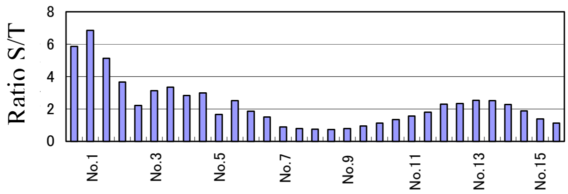

Fig. 18. Settlement/liquefiable layer thickness ratio

this portion is located on the curved part of the dike axis. Curved part of the dike axis may cause comparatively larger cyclic stress to the liquefiable sand layer due to the three dimensional response of the dike. The ratio of the observed settlement to the thickness of liquefiable soil layer is not uniform as shown in Fig. 18. Mean value of this ratio was 2.3%.

<u>Deformation of the geogrid sheet</u>

The deformed shape of the geogrid was examined at Section No.1+16 and at Section No.3 during the occasion of repairing works conducted in 2001.

It was found that the geogrid at the Section No.3 was still horizontal as shown in Fig. 19 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 took place to 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. 20 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.

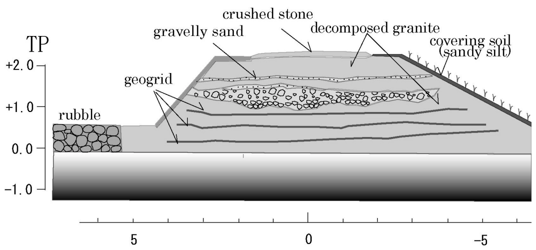

Fig. 19. Geogrid after earthquake at No.3

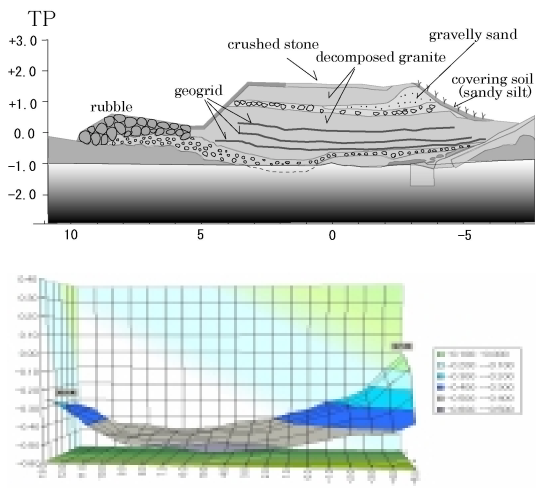

Fig. 20. Geogrid after earthquake at No.1+16

ADDITIONAL SOIL INVESTIGATION AFTER THE EARTHQUAKE

<u>Soil profile</u>

As mentioned before, there were two parts in the treated 850 m long section where the damage was minor and the subsidence was severe. In order to check the soil profile estimated before the construction, additional boring and CPT were conducted after the earthquake. As the results of these investigations, it was found to be short near the river mouth that the length of the section where the uppermost sand layer is thick. It was also found that uppermost sand layer was thinner than that estimated before the construction around the section from Section No.3 to No.5.

SPT N value around the river mouth was slightly larger than

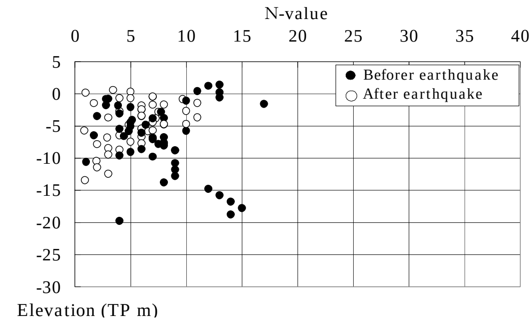

Fig. 21. Comparison of SPT N value

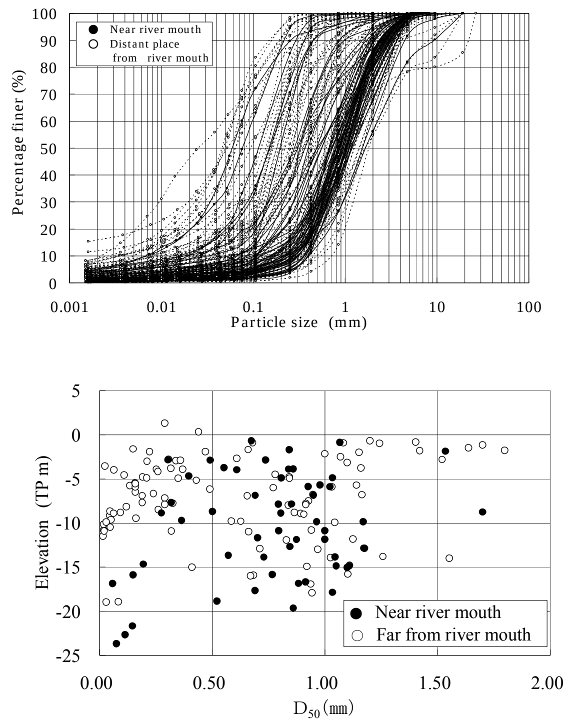

Fig. 22. Particle size distribution curve and D_{50}

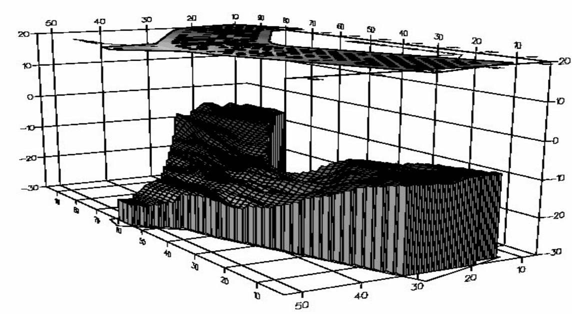

Fig. 23. Trace of Mud Lump estimated from SCP data

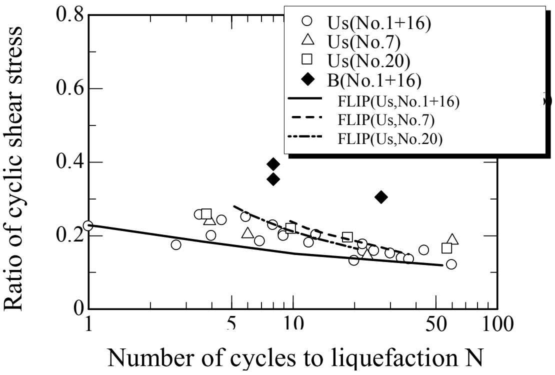

Fig. 24. Cyclic Tri-axial Compression Test

that in minor damaged section as shown in Fig. 21, and that the mean particle size of this layer around the river mouth was slightly large. Figure 22 shows the particle size distribution curve of the uppermost sand layer.

The execution data of the Sand Compaction Piles installed near the river mouth as part of the repair works was analyzed to obtain the thickness of the uppermost sand layer. It was found that the section where sand layer is thick existed very locally near the river mouth as shown in Fig.23, and it was thought that this was caused by a trace of Mad Lump phenomena(Izumo, 1994). Soil profile shown in Fig.3 was slightly modified by reflecting these investigation results from that estimated one before the construction.

Liquefaction strength

Cyclic tri-axial compression test was also conducted after the earthquake for specimens obtained by triple core tube method at Section Nos. 1+16, 7, and 20 together with disturbed sample of embankment material. Figure 24 shows the test results. It seems that the liquefaction strength near the river mouth is slightly smaller than that at Section No. 7 and 20.

BEHAVIOR OF THE ARASHIMA DIKE DURING THE EVENT

Effective stress analysis was tentatively conducted using a code named FLIP (Iai et al., 1992), so that the effectiveness of the geogrid be clarified in mitigating the liquefaction induced deformation of the embankment. Input soil properties and the input motion used for the numerical analysis are described elsewhere (Sasaki et al. 2002).

Liquefied thickness

Variation of liquefied thickness in uppermost sand layer were examined by calculation of pore water pressure (PWP) at three cross sections, at Section Nos. 1+16, 7, and 20 where the thickness of uppermost sand layer are different. Figure 25 shows the calculated PWP and the PWP ratio at these sections. It is known from this figure that the thickness of the zone where PWP ratio is larger than 0.6 ($\Delta u/\sigma' > 0.6$) is almost the same among these three sections, reaching to about several meters. This implies that the heavily softened thickness does not differ between Section No. 1+16, and the Section No. 7.

However, it is known that at Section No. 1+16, thickness of the layer where PWP (absolute value, not the PWP ratio) was raised up reaches to about 20 m deep, much deeper than other sections as shown in Fig.25. This implies that, at Section No. 1+16, upward seepage flow by the dissipation of raised PWP must have continued longer than other sections causing the

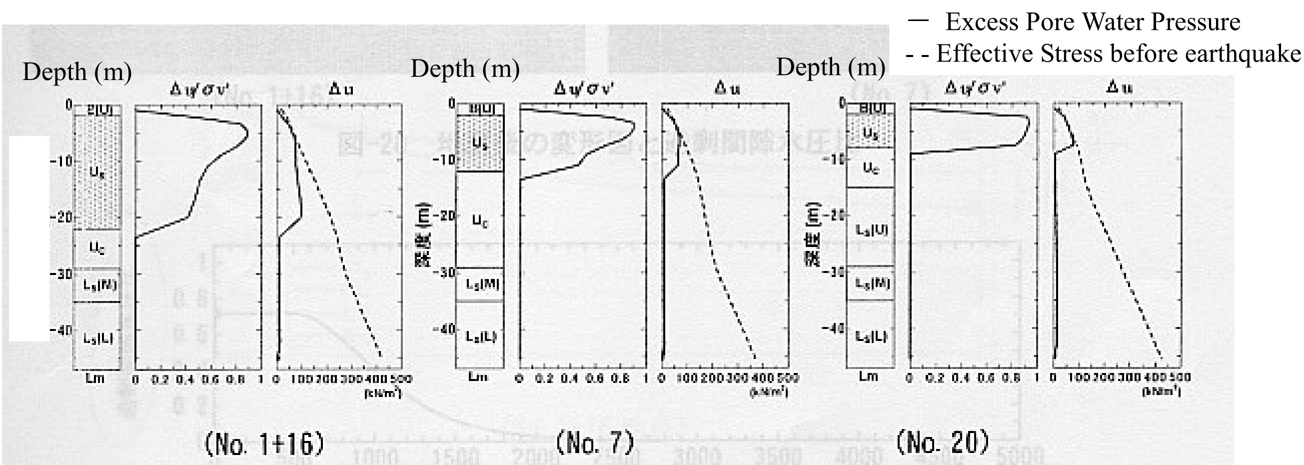

Fig. 25. Comparison of raised pore water pressure

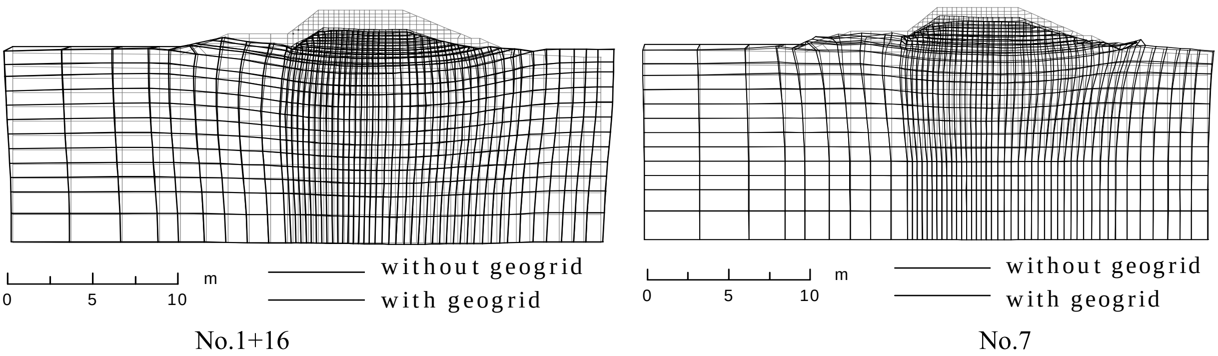

Fig. 26. Calculated deformed shape

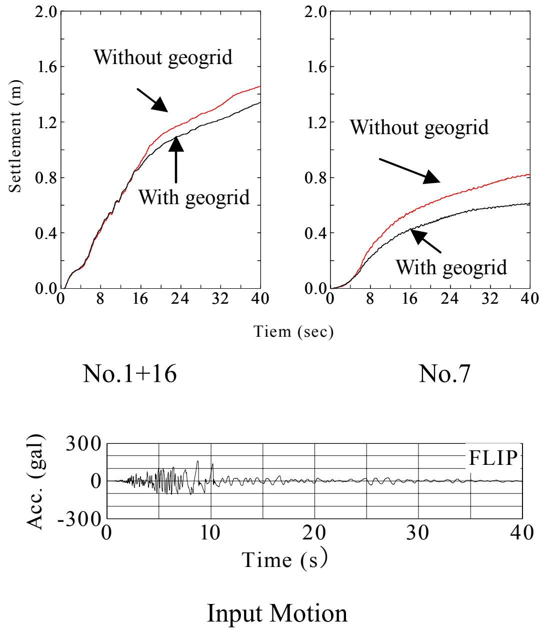

Fig. 27. Time history of calculated settlement

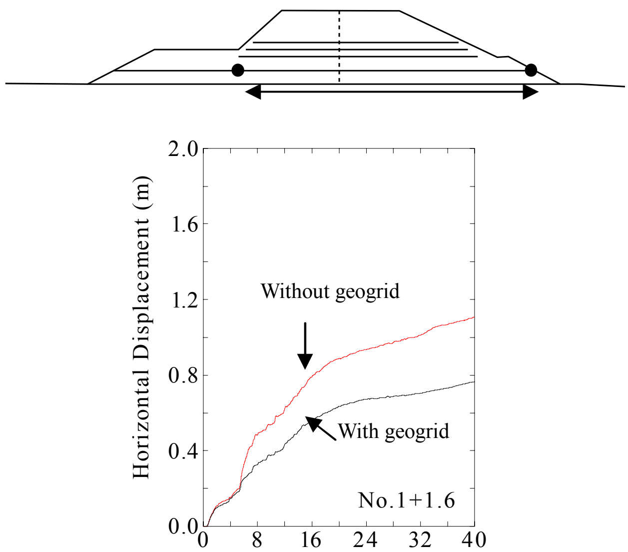

Fig. 28. Time history of calculated stretching

decrease of anti-stretching effect of geogrid due to the lost of friction by immersing water. If this is the case, interval distance between the lowest and middle sheets of geogrid becomes shorter as seen in Fig.20.

Settlement and Deformation of embankments

Crest settlement of the embankment calculated by an effective stress 2D analysis was 1.3 m at the Section No.1+16 (severe damage), and about 0.6 m at the Section No. 7 (minor damage) as shown in Fig.26. Final shape of the embankment and the foundation ground at both sites are also shown in Fig. 26.
These calculated settlements were obtained around the end of ground shaking as shown in Fig. 27. The calculated settlement, under the condition that the groundwater table was set at the same elevation before the earthquake, is very close to the observed one at the Section No.1+16. Contrary to this, the calculated settlement is overestimated at the Section No.7 by this numerical analysis.
Stretching amount in the calculation at the Section No.1+16 for both cases with and without geogrid are illustrated in Fig. 28. According to this tentative analysis, it is known that the stretching amount was reduced to about 70% by deploying the geogrid, although calculated amount of this reducing effect against stretching should be further stufied.

CONCLUSION

A remedial measure against soil liquefaction by geogrid for a comparatively small dike was introduced. This newly constructed 3 m high dike was shaken by the Tottori-ken Seibu Earthquake in 2000.
Since this type of remediation was a first attempt, the newly constructed section had been monitored on its subsidence, variation of underground water table and strong motion during earthquake. Subsidence monitoring was accomplished by surveying four times a year, and a survey was conducted on the day before the earthquake. So, the settlement of the treated section of the dike during the earthquake was successfully obtained quantitatively.
Strong motion record told that 160-200 gal acceleration was input to this area, trace of sand boiling was observed and the

record showed the rise of the underground water table by about 20cm.
This earthquake caused damage for 120 m long section within the 850m long dike where 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 to settlement by about 20cm, no visible deformation was observed on the dike nor on the concrete facing which had been laid on the outer slope.
Thus it is concluded that reinforcement by geogrid is effective for an embankment against soil liquefaction if the liquefiable layer beneath the embankment is not so thick.

REFERENCES

Kanayama, Y., Abe, T., Arase, Y. and Kamatsu, H. [1997]. "The Remedial Measure by Geotextiles for a River Dike against Soil Liquefaction and Field Observation for its Effectiveness during an Earthquake, Ground Engineering, Vol.15, No.1, pp.47-52. (In Japanese)

Iai, S., Matsunaga, Y, and Kameoka, T. [1992] : Analysis of undrained cyclic behavior of sand under anisotropic consolidation, Soils and Foundations vol.32, No.2, pp16-20

Izumo Construction Office [1994], Investigation Report on Mud Lump-Naka-umi Lake and Mouth of the Ii-nashi River (in Japanese).

Investigation commission of Tottoriken-seibu Earthquake [2001], Immediately investigation report of the Tottoriken-seibu Earthquake, Tsuchi-to-kiso, Vol.49, No.1, pp.32-35.

Japan Road Association[1996], Specification for Highway Bridge (in Japanese).

Sasaki, Y., Moriwaki, T. and Ohbayashi, J. [1997], Deformation process of an embankment resting on a liquefiable soil layer, Deformation and progressive failure in geomechanics, Proc. IS-Nagoya'97, pp553-558.

Sasaki, Y. and Fukuwatari, T. [2001], Settlement of an embankment reinforced with geotextile during the Tottoriken-seibu earthquake, Geosynthetics Technical Information, vol.17. No.3, JCIGS, pp15-20 (in Japanese).

Sasaki, Y., Fukuwatari, T., Uekuma, H. and Miyake, T. [2002], Subsurface information gained by controlling data on execution of SCP, Geo-forum (in Japanese).

Terzaghi, K., Peck, R. B. and Mesri, G.[1996], *Soil Mechanics in Engineering practice*., John Wiley & Sons, Inc., p383.