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RESEARCH AND PRACTICES ON REMEDIAL MEASURES FOR RIVER DIKES AGAINST SOIL LIQUEFACTION

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ABSTRACT: Research and practices on remedial measures for river dikes against soil liquefaction during earthquakes are presented. Though the river dikes have been constructed without rational seismic design but with empirical rules in their long history, intense urbanization to lowland behind dikes is demanding rationalization of the seismic design of river dikes. Geotechnical features of the foundation condition for dikes to fail, and the mechanism of dike failure are being revealed by what happened during the recent earthquakes in Japan. Advanced methods of estimating the earthquake-induced settlement of river dikes are also being developed in recent years. This paper attempts to compile the current trend of research direction and practices in Japan for strengthening the river dikes against earthquakes.

Key Words: River dikes, Soil liquefaction, Remedial measures, Deformation, Stretching,

INTRODUCTION

Past earthquakes in Japan caused damage to river dikes many times worse than to other structures. **Figure 1** shows the Torishima section of the Yodo River dike just after the Kobe earthquake in 1995 (Matsuo 1996, Sasaki and Shimada 1997b). As shown in this figure, pavement at the crest was intensely inclined, the parapet wall and the concrete facing of the revetment also inclined, and traces of intense sand boiling were found near the dike.

Fig. 1 Torishima section of the Yodo River dike damaged by the Kobe earthquake

The dike with an original height of 6.5 m settled down about 4 m at the most. The section was located near the river mouth and the hinterland was highly urbanized. There was concern about an inundation because the residual height of the dike was only 2 m higher than the maximum tidal water level.

Damaging earthquakes have occurred in Japan about 50 times since the Kanto earthquake in 1923 to date (2003), if such earthquakes are selected as those given their names by Japan Meteorological Agency (JMA) (National Astronomical Observatory 2003). Therefore only the comparatively large earthquakes are counted. The magnitudes of those earthquakes are illustrated in **Fig.2**.

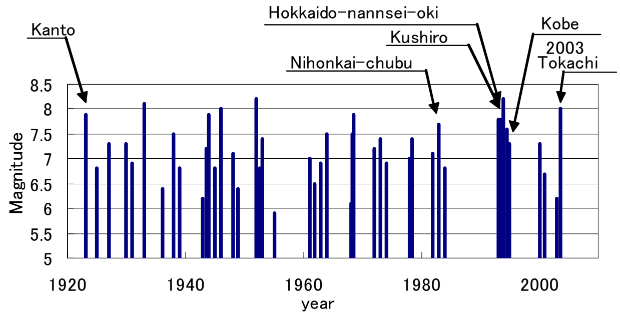


Fig. 2 Magnitude of earthquakes since the Kanto earthquake

The dikes in Japan have their long history of construction and are maintained by central or local government according to their importance, and most of the river dikes are controlled by MLIT (Ministry of Land, Infrastructure and Transportation), while some are controlled by MFAF (Ministry of Forest, Agriculture, and Fishing) according to their roles.

Dikes have been constructed without rational seismic design but with empirical rules in their long history. This is because dikes constructed using soils as construction materials on the basis of the empirical rules have been considered to have seismic stability to some extent and also been considered to be easy to repair in short period even if they are damaged by earthquakes. So dikes were not rehabilitated considering seismic effects for a long time even when they were damaged.

However, during the Nihonkai-chubu earthquake in 1983, the Hachirogata dike was seriously damaged for more than 80 km, half of it had an important role to prevent flood for the Ohgata village inside the reclaimed former Hachirogata Lake where the average altitude of ground surface is 4 m lower than the water level outside. So the rehabilitation works were conducted on the basis of avoiding future possible seismic damage, though in Japan the budgeting system for rehabilitation of damage by natural hazard was on the basis of restoring the damaged structure to its previous state without additional strengthening (Hashimoto et al.1985).

Therefore it is considered that the damage of the Hachirogata dike marks the beginning of a change in the philosophy of treating damaged dikes. Later in 1993, the river dike sections controlled by the central government were seriously damaged at the Tokachi River and the Kushiro River in Hokkaido due to the strong shaking of the Kushiro-oki earthquake. The MOB (Ministry of Budgeting) at that time accepted including the cost for additional strengthening against future earthquakes in the restoration budget for these damaged dike sections for the first place in the history of restoration works in Japan (Oshiki et al. 2001). The authors consider that this experience together with the seriousness of the damage to the Yodo River dike helped to change the attitude of decision makers in the MOB. The MOB accepted proposals for rehabilitation of damage by the Kobe earthquake in 1995 that included additional strengthening against future earthquakes. Therefore the history of research and practice in mitigating the seismic damage of the river dikes is divided into two periods. In the earlier period, from the bitter experience of the damage by the Niigata earthquake in 1964 to the Kushiro-oki earthquake in 1993, the geotechnical features of the subsoil condition for dikes to fail and the mechanisms of dike failure were studied on the occasions of occurrences of damaging earthquakes.

Compilation of the damage records revealed that the river dikes during earthquakes did not exceed 3/4 of its original height. It has also been revealed that serious damage to river dikes had often been observed at the location of particular subsoil conditions such as an old river bed, and that the main cause of the large settlement of dikes during earthquake is soil liquefaction.

Although the damage cause is being thus identified, it is hard to reduce the liquefaction induced damage to river dikes. Length of existing dikes is too long and the concentration of private properties just adjacent behind dikes makes it difficult to treat the problem by improving the subsoil condition against earthquakes.

In the period after the Kushiro-oki earthquake, the Kobe earthquake accelerated the demand to establish a rational method to predict and mitigate the dike damage against earthquakes as the urbanizations behind the waterfront near river mouths are intensely progressing.

Accordingly it is stated in the report on the countermeasures for river facilities against earthquakes compiled after the Kobe earthquake (Technical Committee on Seismic Countermeasures for River Facilities 1996), that the dike sections where the high tide water level plus 1 m of altitude exceeds the hinterland ground level should avoid a secondary disaster due to inundation even if the dike settles during earthquakes. It is also said in this report that the dike sections in such situations should keep the function to protect the hinterland from flood by the residual height of the dike even if they are damaged.

In order to give this function to the dike sections, it is necessary to establish a reliable method to estimate the magnitude of dike deformation during earthquakes. For this purpose, research on the damage mechanism of river dikes during earthquake and the effort to develop a better method to estimate the subsidence of top crest of dikes are currently being carried out in Japan. This paper attempts to summarize these research and efforts.

FINDINGS FROM PAST DIKE DAMAGE BEFORE THE KUSHIRO-OKI EARTHQUAKE

River dikes in Miyagi prefecture were damaged along four rivers, the Kitakami, the Naruse, the Natori, and the Abukuma during the 1978 Miyagiken-oki earthquake. The total length of damaged dikes was 29.8 km, that is 7.1 % of the total length of 422.3 km controlled by the regional Bureau of the MLIT; formerly MOC (Ministry of Construction).

One of the most severely damaged sections was found at the Yoshida River, a branch of the Naruse River as shown in **Fig.3**. As seen in this figure, seismic damage to river dikes was generally composed of the settlement of the top crest accompanied by longitudinal fissures. But it is not so easy to get the accurate estimate of seismic settlement due to the lack of the data on measured elevations of the dike crests before the earthquake.

Fig.3 Damage of the Yoshida River dike due to the Miyagiken-oki earthquake

Several features are seen on the dike damage about their location in view of the geomorphologic condition, epicentral distance, and the settlement of the crest and so on. Technical findings from the past studies on these features are briefly reviewed below before mentioning lessons learned from the recent earthquakes.

Geomorphologic condition

It has been found that the locations of the dike damage during earthquake are well correlated with the subsoil conditions. **Figure 4** shows the histograms of the seismic settlement of dike crests during the past earthquakes such as the Nobi (1891), the Kanto (1923), the Fukui (1948), the Niigata (1964), and the 1968 Tokachi-oki earthquakes (Sasaki 1980).

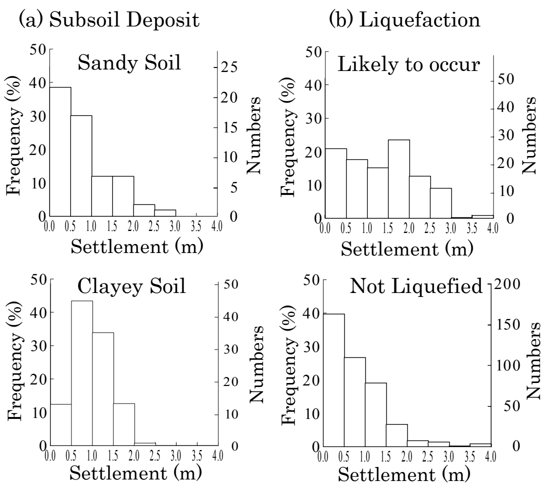


Fig.4 Histogram of the seismic settlement of dike crest

It is known from this figure that the dikes on the sandy soil tend to settle more than on cohesive soil. And that the settlement where liquefaction was supposed to occur is larger than that where liquefaction was not supposed to occur. It should be also noted that not so few damaged sections were located where the subsoil was a cohesive soil deposit.

Kikkawa et al. (1965) pointed out that dike damage had often been seen at locations where the dikes were situated on abandoned river channels or former ponds. These kinds of facts offer valuable knowledge for the purpose of filtering out potentially susceptible sections. **Figure 5** shows an example of the comparison of the damage rate to river dikes with the geomorphologic condition. This figure shows that damage along the Eai River, branch of the Naruse River, during the Miyagiken-oki earthquake, is concentrated to a section on the old river bed (Sasaki 1980).

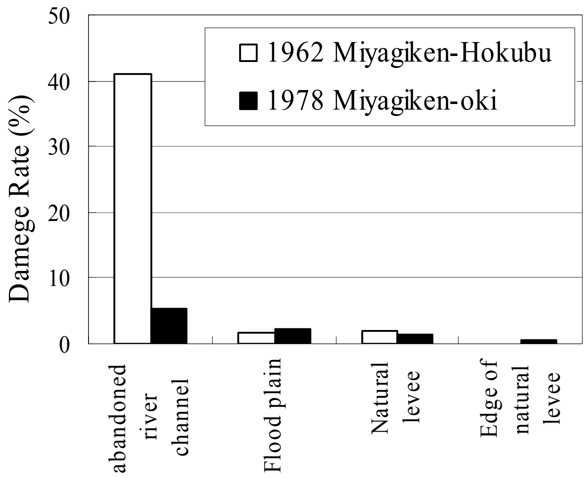


Fig.5 Damage rates of dike in relation to the geomorphologic condition

It is known that the subsoil condition could be estimated from geomorphologic information, and it plays useful role in compiling a hazard map (TC4 1999), or in diagnosing existing dike sections in cases where sufficient information on the subsoil condition of dikes is lacking.

Maximum epicentral distance

As mentioned before, about seven percent of the whole length of dike was damaged during the Miyagiken-oki earthquake. But it is known that the ratio of the damaged length to whole length was not uniform. Sasaki (1983) reported that the ratio of the length of the damaged section to total length decreased with the epicentral distance during the earthquakes of the Niigata, the Miyagiken-oki, and the Nihonkai-chubu as shown in **Fig. 6**.

It is known from this figure that the envelopes of the damage ratios tend to diminish at certain distances from the epicenter for each earthquake occasion.

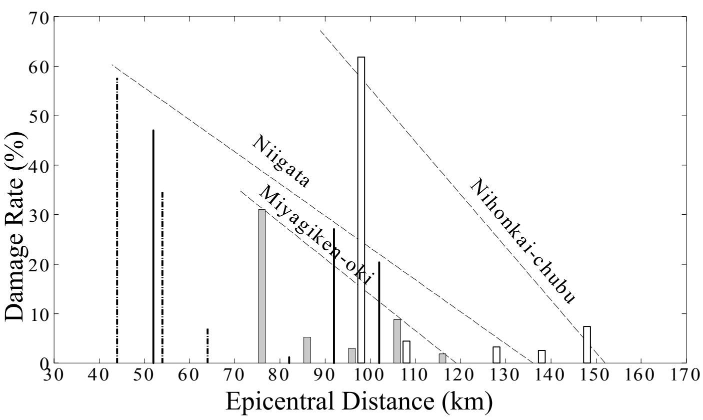


Fig.6 Damage rates of dike in relation to the distance from the epicenters

By utilizing the following equation proposed by Iwasaki et al. (Iwasaki et al. 1978),

$A_{max}=562\text{x}10^{-0.00453D}$ (60 km<D<170 km for the Miyagiken-oki earthquake)

Sasaki estimated 160 gal of the peak ground acceleration for triggering level to cause dike failure during the Miyagiken-oki earthquake.

Maximum settlement

Figure 7 shows the observed settlement of river dikes during past earthquakes (Sasaki et al. 1999). The distance from the original elevation of the top to a certain level of the damaged dike where the original width is assured at this height was taken as the settlements in this figure.

As shown in this figure, settlement of river dikes during earthquakes did not exceed 3/4 of its original height.

The reason for this is that the maximum possible amount of an embankment subsidence is governed by the balance between the buoyant force acting to the embankment from the liquefied layer and the weight of the embankment (Towhata et al. 1999). This implies that the liquefied layer behaves like a liquid in the worst case.

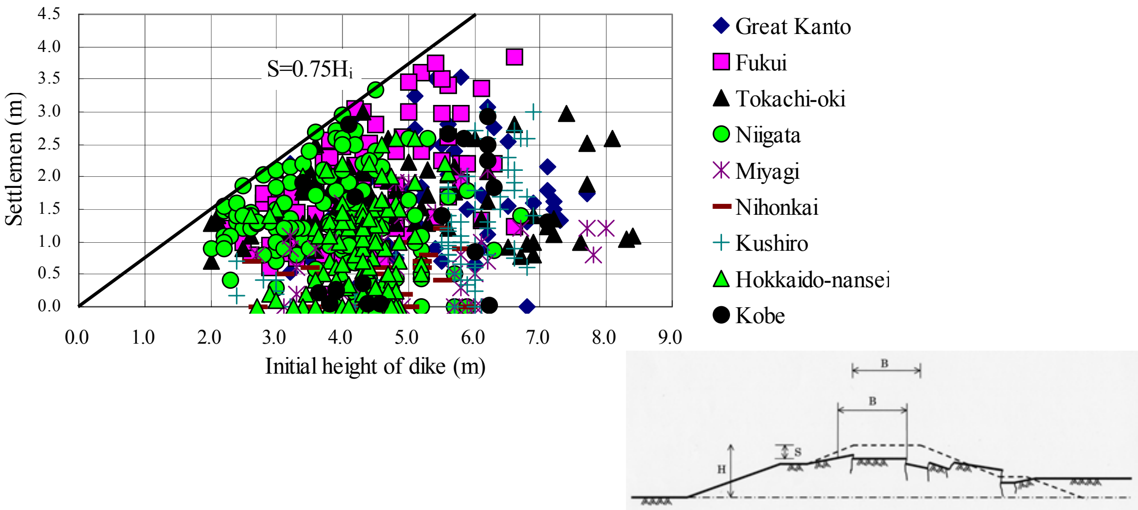


Fig. 7 Settlement of river dikes during past earthquakes

Depth and thickness of liquefiable layer

Past cases of dike damage indicated that the crest settlement and the degree of deformation of dikes are governed by the depth and the thickness of liquefied layer beneath the embankment.

Ishihara showed that if the thickness of the non-liquefiable layer is larger than that of underlying liquefiable layer, the damage to wooden houses on the ground surface may be insignificant (Ishihara 1985).

Because of this experience, the relation between the crest settlement and the depth of the liquefiable layer was examined after the Hokkaido-nansei-oki earthquake (Hakodate Construction Office 1996).

When the liquefiable layer exist shallow portion of the foundation ground and the thickness of the liquefiable layer is large, then the settlement of the dike tends to appear as shown in **Fig.8**.

This finding was explained by Sasaki et al. (1996) as stress dispersion in the non-liquefiable layer using a bearing capacity formula for a two layers system.

The fact that if the liquefiable layer is covered by thick non-liquefiable deposit then it leads to insignificant effect to structures on the surface implies the efficiency of the partial improvement to a certain depth of thick deposit of liquefiable layer. This will reduce the cost of remedial treatment for the liquefiable subsoil condition.

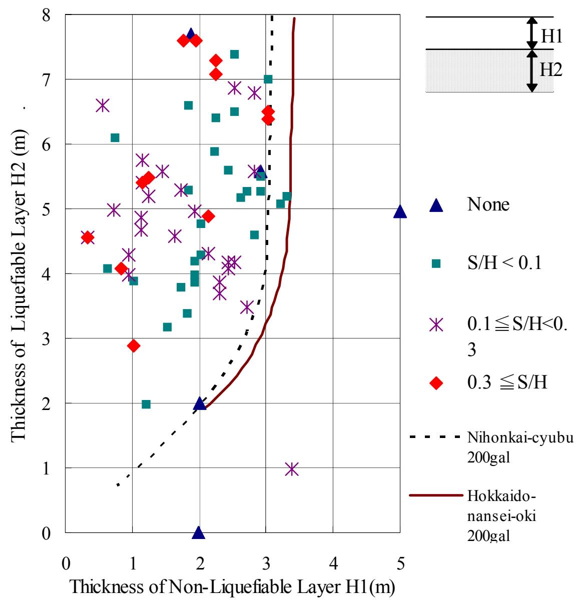


Fig.8 Depth and thickness of liquefiable layer

Technical guidelines

After the Kushiro-oki earthquake in 1993, the Japan Construction Engineers' Association (JCEA) published a technical guideline for treating a seismic damage to river dikes under the supervision of the River Bureau of the MOC in 1994 (River Bureau 1994).

It was aimed in this booklet to disseminate the experiences gained during the earthquake to the practical engineers of the construction offices in the field in an easily understandable manner. Because not so many engineers in the field experienced a damaging earthquake, therefore it was considered necessary to give guidelines for necessary action in the confused situation. Therefore only the important steps for avoiding secondary disaster and doing restoration work are summarized in the 20 pages booklet. Most of the steps such as how to get information on potential aftershocks, to locate damaged spots, or the sections to be reported are explained by using actual pictures or schematic drawings. One example is the special vehicle, shown in **Fig.9,** having a gondola on the end of a boom for taking birds-eye view pictures of large dikes. Another example shown in **Fig.10** is an illustration of a typical damage report.

Fig.9 Gondola vehicle

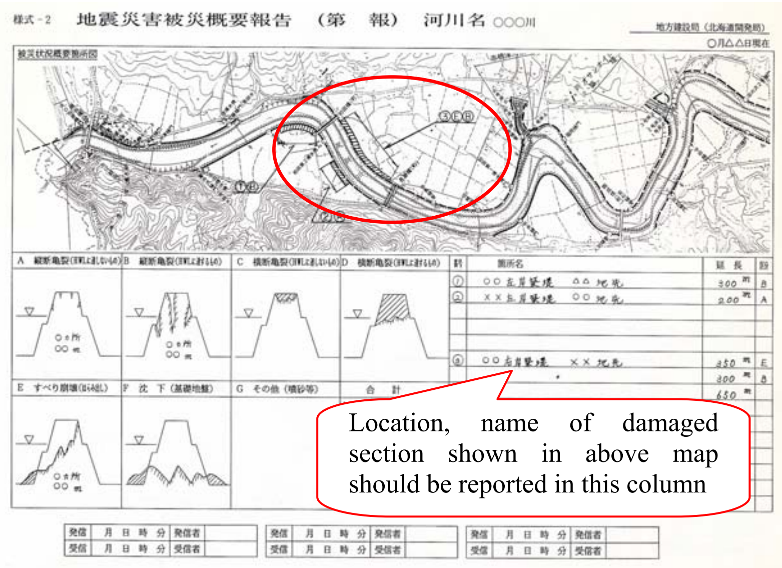


Fig.10 Example of damage report format

Schematic drawings of the different forms of damage to dikes, illustrated in **Fig.10**, are actually the failure modes used in Japan in diagnosing the stability of existing dikes. Failure modes of observed seismic damage to river dikes were classified into the four types in **Fig. 11** (Sasaki 1980). Type 1 is a shallow sliding failure on the slope of an embankment, Type 2 is a sliding failure deep inside the embankment, Type 3 is a failure involving both the embankment itself and the foundation, and Type 4 is a subsidence of the crest without apparent cracks. It was reported that one third of the past cases cited in **Fig.4** were Type 2. The rest of the past cases were fairly evenly divided between Types 3 and 4 (Sasaki 1980). However, it should be noted that the actual locations of the slip surfaces in past failures were not determined by investigation or observation but were estimated from external evidence only. A similar classification is used for railway embankments (Takei 1971).

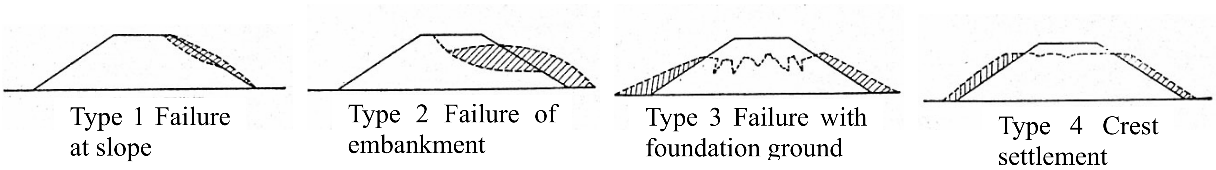


Fig.11 Classification of damage modes of failed dikes

The ISSMGE (International Society for Soil Mechanics and Geotechnical Engineering) Technical Committee for Geotechnical Earthquake Engineering (TC 4) published a technical manual named "Manual for Zonation on Seismic Geotechnical Hazard" in 1999 (TC4 1999) and "Case histories of Post-Liquefaction Remediation" in 2001 (TC4 2001). Some of the aforementioned findings from the past earthquake damage are reflected in these publications.

DAMAGE TO RIVER DIKES DURING RECENT EARTHQUAKES

Recent damaging earthquakes in Japan

Table 1 shows the recent damaging earthquakes in Japan which caused serious damage to river dikes.

Table 1 Recent damaging earthquakes in Japan

Name of the earthquake	Magnitude	Date	Damaged dikes
Nihonkai-chubu	7.7	May 26, 1983	Hachirogata
Kushiro-oki	7.8	January 15, 1993	Tokachi River, Kushiro River
Hokkaido-nansei-oki	7.8	July 12, 1993	Shiribeshi-Toshibetsu River
Hokkaido-toho-oki	8.2	October, 4, 1994	Kushiro River
Hyogoken-nanbu (Kobe)	7.3	January 17, 1995	Yodo River
Tottoriken-seibu	7.3	October 6, 2000	Naka-umi Lake
Miyagiken	6.2	July 26, 2003	Naruse River
Tokachi-oki	8.0	September 26, 2003	Tokachi River

Eight damaging earthquakes caused severe damage to river dikes in Japan in the last twenty years. The locations of epicenters of these earthquakes in **Table 1** are shown in **Fig.12**.

After the Nihonkai-chubu earthquake, SPT energy measurements were conducted by an US-Japan collaborative team under the auspice of the UJNR (Kovacs et al. 1983) in the field at the Hachirogata dike.

The Kushiro-oki earthquake took place in the mid-winter season so the ground surface was covered by snow and frozen to a depth of about 70-80 cm. The potential existence of sand boils, where the surface snow was found to be curiously mounded, was investigated by the use of gas burners to melt away the snow. The Hokkaido-nansei-oki earthquake hit the western Hokkaido in 1994 while the damaged dikes in eastern Hokkaido were still being repaired. The experiences gained in the east were rapidly transferred to west for treating the earthquake damage.

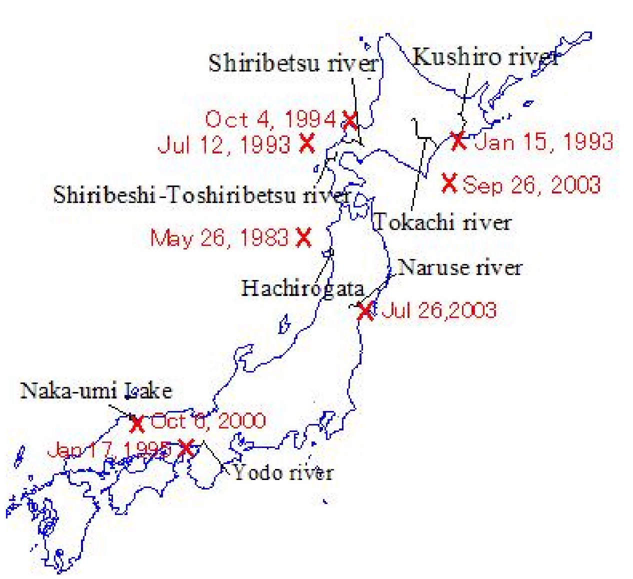


Fig.12 Locations of epicenters of recent damaging earthquakes

The Kobe earthquake caused serious damage to the Yodo River dike for a length of more than 8 km in a highly urbanized area.

Accurate measurement of the crest of a dike was successfully obtained during the Tottori-ken-seibu earthquake as mentioned later.

The Miyagiken earthquake in 2003 hit the Naruse River dikes when the water level in the channel was fairly high due to the rain which had begun 5 days before the main shock. The foreshock of Mj=5.5 and an aftershock of Mj=5.3 hit the area within a short period. The foreshock occurred 7 hours before the main-shock and the aftershock hit 9 hours after the main-shock.

The 2003 Tokachi-oki earthquake was followed by a tsunami which went back up the Tokachi River channel. Strong motion records were obtained by the Hokkaido Development Bureau during the earthquake (Civil Engineering Research Institute of Hokkaido, 2003). This earthquake caused damage to the Tokachi River dikes which had been enlarged by flattening the side slopes. The dike on the Kiyomappu River, a branch of the Ishikari River, was damaged at the section where the dike had been constructed using the pile-net method (Noto 1991).

Findings from these recent earthquakes are briefly presented below.

New type of liquefaction induced damage

During the Kushiro-oki earthquake in 1993, new type of liquefaction induced damage to embankments was recognized. That is the liquefaction of the bottom part of the embankment (Sasaki et al. 1993, Kaneko et al. 1995).

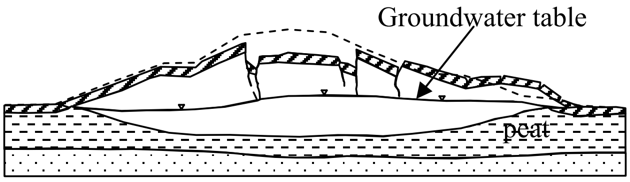


Fig. 13 Cross section of the Kushiro River dike

Four points below were commonly found at severely damaged sections from the field reconnaissance.

1) The top layer of foundation ground beneath the dike was a peat layer, which had a comparatively large thickness.
2) The height of the damaged dike was comparatively high.
3) Construction materials for dike were sandy soil.
4) Traces of the sand boils were found at the berm or near the toe of the slope; however the number of sand boil spots was not great.

Kohno and Sasaki (1968) also pointed out these four points in their report on damage to river dikes during the 1968 Tokachi-oki earthquake.

Figure 13 shows the typical cross section of severely damaged sections of the Kushiro river dike. It should be noted that the boundary between the dike bottom and the surface of peat layer after construction had settled due to the consolidation of the peat layer as illustrated in **Fig. 13**. The peat in this area usually has 800-1000 % water content and has a large compressibility coefficient.

Therefore the bottom part of the dike settled down below groundwater table. The groundwater table inside the dike is usually higher than the elevation of the groundwater table in adjacent horizontal ground. Furthermore, during the compression of peat layer, the dike slope easily moves towards outside, which brings about loosening the lower part of the embankment. Measured SPT N values in this region of the dike were quite low. Therefore it was clear that the bottom part of the embankment had become susceptible to liquefaction before the earthquake (Finn et al. 1997).

During the same earthquake, periodical occurrences of localized failure of embankments were seen in the Kushiro dike in spite of the homogeneous ground condition as shown in **Figs. 14** and **15**. **Figure 15** shows the subsidence and stretching distribution along the dike

Fig. 14 Periodical appearance of failures on the Kushiro River dike

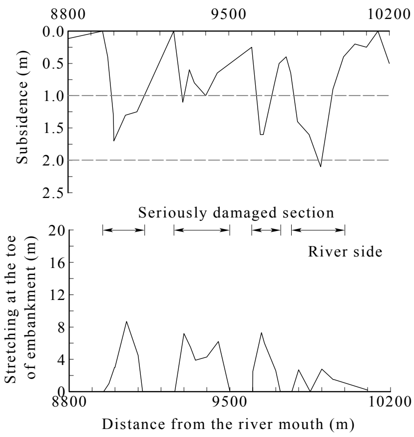


Fig. 15 Distribution of subsidence and stretching

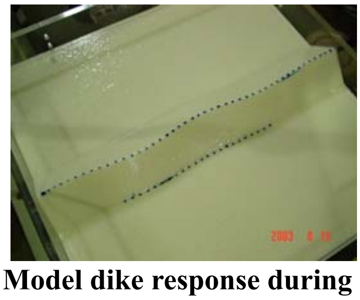


Fig. 16 Model dike response during shaking

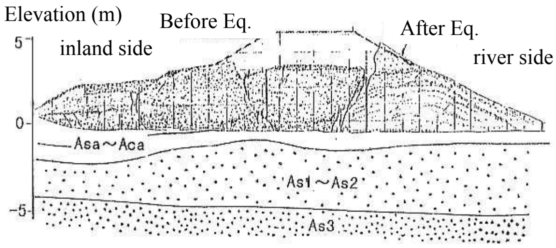
axis. The reason of this is discussed elsewhere (Hata et al., 2003). Shake table tests on model dikes show that the model responds largely intermittently at a certain input frequency as shown in **Fig. 16** due to three dimensional effects. This experimental result shows that the localized failure in **Fig, 14** is due to the three dimensional response of the embankment during shaking.

Stretching type of deformation

During the Hokkaido-nansei-oki earthquake in 1994, a stretching type of failure was clearly detected as shown in **Fig. 17** at a damaged section of the Shiribeshi-Toshibetsu River dike (Sasaki et al. 1997a).

As Terzaghi et al. (1996) mentioned this type of failure is caused by the decrease of shear strength at the bottom boundary of the embankment due to the raised pore water pressure. When the shear resistance at the embankment bottom is lost, slippage takes place in the embankment as shown in the model dike in **Fig. 18** (Sasaki and Ohbayashi 1997c). If this type of failure happens, longitudinal cracks and gaps may appear on top of the dike or on the surface of the side slope as shown in **Fig. 19**. It was shown elsewhere that this type of fissures may appear on the side slope near the shoulder at first (Sasaki et al. 1997a).

(a) External appearance



(b) Sketch from trench-cut surface

Fig. 17 Damage to the Shiribeshi-Toshibetsu dike

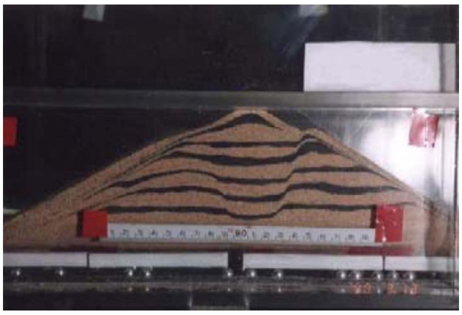


Fig. 18 Slip planes appeared inside embankment

Fig. 19 Longitudinal fissures seen at the Hachirogata dike in 1983

During the Kobe earthquake, a dike section of the Yodo River, named the Torishima section, was severely damaged (Matsuo 1996, Sasaki and Shimada 1997b) due to the soil liquefaction in the foundation ground as shown in **Fig. 20** (Sasaki and Shimada 1997b). At this site, stretching of the embankment was also seen as shown in **Fig. 20**. Boundaries of the subsided embankment were surveyed by Swedish sounding and it was detected that the broken block of the dike body sank into the loose foundation layer as illustrated in this figure.

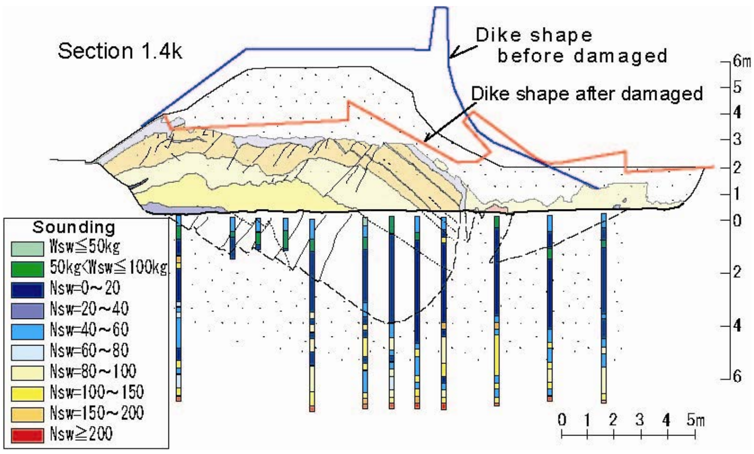


Fig. 20 Torishima section of the Yodo River dike

Measurement of the dike settlement during earthquakes

Knowing that stretching causes an increase in the seismic deformations of embankments, reinforcement against stretching was employed for the construction of the Arashima section of the Naka-umi Lake shore dike. The Arashima section was subjected to shaking by the Tottoriken-seibu earthquake four years after its construction. The reinforced dike behaved perfectly, demonstrating the effectiveness of this method for stabilizing dikes (Sasaki et al. 2004b).

As there was no previous experience in using geogrid as a remedial measure for an embankment against liquefaction failure, it was decided to monitor the strong earthquake ground motions and groundwater table changes (not pore water pressures) at the Arashima section of the Naka-umi Lake shore dike. For addition, it was decided to check the subsidence of the dike by conducting a survey four times a year (Sasaki et al. 2004a).

Figure 21 shows the time history of crest settlement due to consolidation of the clay layer from the completion of construction in 1996 until 2002. It is clear that the dike continued to settle since its construction.

In October, 2000, a survey was conducted on October 5 just the day before the Tottoriken-seibu earthquake and an additional survey was conducted as soon as possible on the day after the event, on October 7. Therefore a record of embankment settlement during an actual earthquake was obtained as shown in **Fig. 22**.

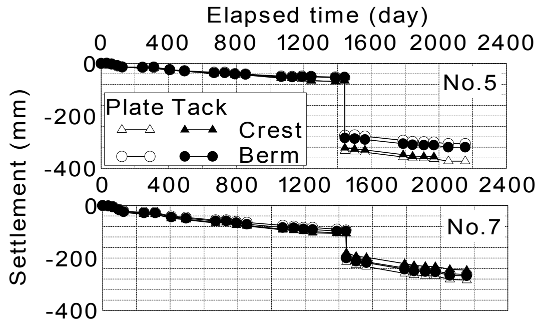


Fig. 21 Time histories of the settlement after the construction of the dike

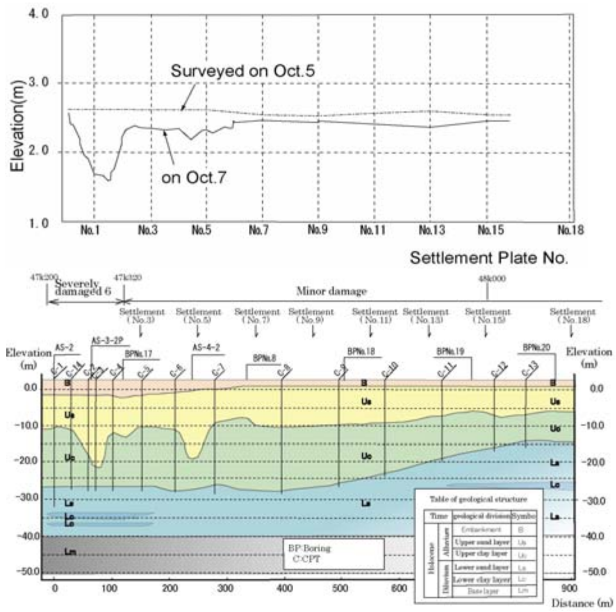


Fig.22 Comparison of the crest elevations

From the survey results, it was revealed that the section which seemed to be sound subsided by about 5-20 cm even though no cracks were observed on the embankment crest or on the side slope. No emergency treatment of the dike was necessary even though the settlement were about 20 cm.

The cross-section of the Arashima dike in **Fig. 23** shows how the dike was strengthened. The construction stage of the dike is shown in **Fig. 24** (Sasaki et al. 2004a).

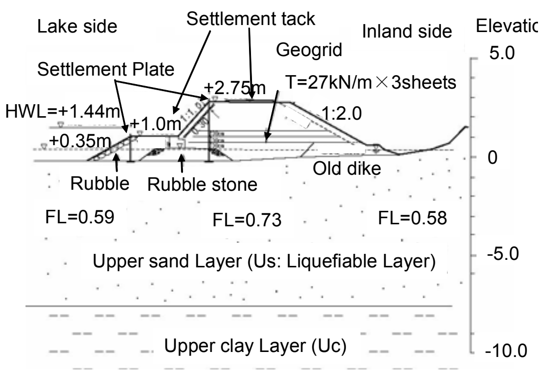


Fig.23 Cross section of the Arashima dike

Fig.24 Construction stage of the Arashima dike

Most recent experience

During the Miyagiken earthquake in 2003, the Naruse River dike was damaged at 66 locations. In general, the subsoil at heavily damaged dike sections is composed of alternating layers of soft clay and loose sand. **Figure 25** shows the failure at the Kimazuka section of the Naruse River.

Apparent sand boiling was not frequently observed near the damaged site and it was apparent that the embankment had high water content due to the preceding rainfall. It was detected that a sand vein was left in the embankment at the seriously damaged sites, while the dike were repaired. The deformed shape of the embankment, shown in the aerial photograph in **Fig. 26** (Kitakami River Office 2003), implies that the bulging type deformation of the dike was accompanied by slip failure.

Fig. 25 Large deformation of the Naruse River dike during the2003 Miyagiken earthquake

This case history poses a new problem: to what degree does the material strength decrease due to unsaturated but high water content condition during earthquake shaking should be considered. This means that not only the deformation of the foundation ground due to liquefaction but the deformation due to strength loss caused by an increase in water content must be carefully considered when estimating crest settlement.

Fig. 26 Large deformation of embankment at the Kimazuka section of the Naruse River

A new tool called the Geoslicer (Nakata and Shimazaki 1997) was used to get the information on subsoil conditions at the damaged sites along the Naruse River.

When looking at the recent earthquake damage during the 2003 Tokachi-oki earthquake (Civil Engineering Research Institute of Hokkaido 2003), the following issues may draw technical interest. As the foundation ground is widely covered by a peat layer in Hokkaido, the side slopes of river dikes there are being made flatter to assure static stability for raising the crest elevations. These types of dikes with gentle slopes, such as 1:5 - 1:10 have not experienced earthquake shaking before the Tokachi-oki earthquake. Therefore the damage of the Tokachi River dikes by the 2003 Tokachi-oki earthquake is expected to be studied more deeply.

A section of the Kiyomappu River dike, located about 250 km distant from the epicenter, was damaged during the 2003 Tokachi-oki earthquake. As this section of the dike was situated on very soft subsoil, the foundation of the dike had been treated by the pile-net method. This is the first example of damage to a dike on a pile-net treated foundation, therefore this experience is also expected to investigate deeply.

Table 2 shows the size and length of the damaged dike sections caused by the earthquakes in **Table 1**

and the treatment method used for restoration works for each site.

Table 2 Recent damage to river dikes

Earthquake	M	Damaged river	H (m)	W (m)	L (km)	Restoration Method
Nihonkai-chubu	7.7	Hachirogata, Iwaki River	3.4~17.5 3.6~5.2	2.0 5.0	69,080 5,750	Cut off wall, Drain
Kushiro-oki	7.8	Kushiro River, Tokachi River	7.8~9.5 9.3~10.0	6.0~15.0 4.7~6.0	11,204 7,105	SCP SCP
Hokkaido-nansei-oki	7.8	Shiribeshi-Toshibetsu, Shiribetsu River	5.3~8.4 5.4~7.1	3.6~6.3 5.0~5.4	18,008 3,185	SCP, DC
Hokkaido-toho-oki	8.2	Kushiro River				
Hyogoken-nanbu (Kobe)	7.3	Yodo River	6.8~8.3	7.0~8.3	6,590	DMM, SSP
Tottoriken-seibu	7.3	Naka-umi Lake				SSP
Miyagiken	6.2	Naruse River	6.3~8.2	2.8~6.0	11,507	SCP
Tokachi-oki	8.0	Tokachi River, Kiyomappu River				SCP Jet Grouting

H: height of dike, W: width of top crest, L: total length of damaged sections, SCP: sand compaction pile, DC: dynamic compaction, DMM: deep mixing method, SSP: steel sheet pile

REMDIAL MEASURES USED IN RESTORATION WORKS FOR RECENT DAMGE TO RIVER DIKES

Statistics on currently used remedial measures in Japan against liquefaction

The remedial treatment of a liquefiable layer for engineering purposes are classified into three categories: the first is to improve the ground so that the liquefaction will not take place during the design earthquake, the second is to strengthen the structure so that the structure maintains stability or its function at least even though the foundation ground may liquefy, and the third is to prepare replacement of the function of the structure (Japanese Geotechnical Society 1998).

The remedial treatments to improve foundation ground against liquefaction can be categorized by the principle of the treatment as follows (Japanese Geotechnical Society 1998).

1) Improvement of soil properties such as the density.
2) Improvement of conditions for stress, deformation, and pore water pressure.

The first category methods include densification, solidification, change of particle size distribution, and replacement of the soil. The second category methods include increase of confining pressure, increase of the pore water pressure dissipation, and restraint of the shear strain during shaking.

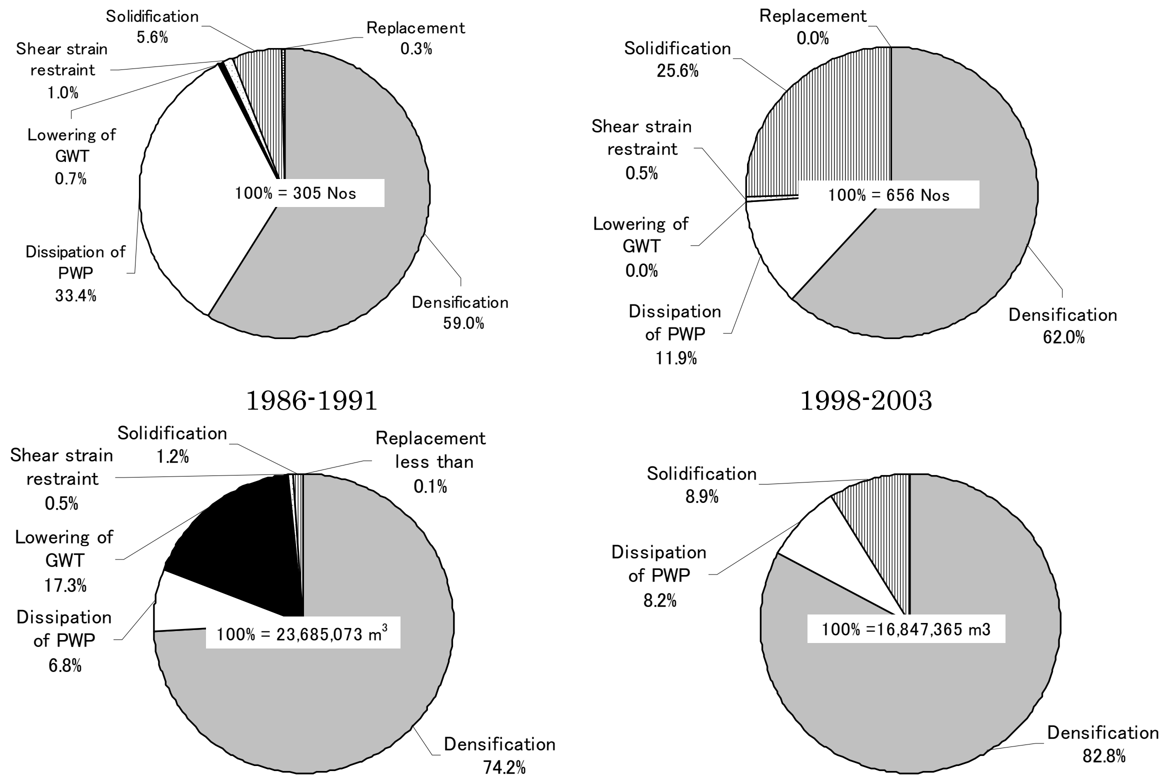
JGS has surveyed the actual trend of the ground improvement methods used as remedial measures during the periods 1986-1990 and 1998-2003. Some results from the survey are shown in **Fig.27** (Japanese Geotechnical Society 2004).

The subsoil improvement was used twice as often during 1998-2003 compared with 1986-1990. This is interpreted as due to growing recognition of the effectiveness of ground improvement. The most frequently used type of the remedial measure is densification (62 % of all cases and 83 % in treated volume). Between the same two periods, the use of drainage methods decreased by more than 50% but methods based on solidification increased. It was found that solidification achieved by the deep mixing method (DMM) was used mainly for river dikes.

The vibration and noise associated with ground improvement became unacceptable in urban areas and a quieter type of remediation became necessary. This led to the development of a new type of sand compaction pile (SCP) method (Tsuboi et al. 1998).

Subsoil improvement can not generally be used for existing structures. Therefore a partial treatment of the ground at both toes of embankments is being sometimes executed.

Several examples of remedial measures against liquefaction in Japan in the last twenty years (TC4



1986-1991 1998-2003

Fig. 27 Statistical tendency of the remediation practice in Japan

2001) are presented below. These examples are shown in an order from inexpensive measure to costly measures.

Counter-weight fill at the Naka-umi Lake shore dike

The Tottoriken-seibu earthquake damaged the dikes along the Naka-umi Lake shore for more than 20 km long as shown in **Fig. 28** (Izumo Construction Office 2003). **Figure 29** shows an example of dike damage due to this earthquake.

Most of these damaged sections were found on newly reclaimed land where the subsoil liquefied. Since the hinterland of these damaged sections is not so urbanized, an inexpensive remedial method was favored for the restoration work for the economical reasons. Most of the damaged sections were repaired by constructing a riprap mound counterweight at the waterfront

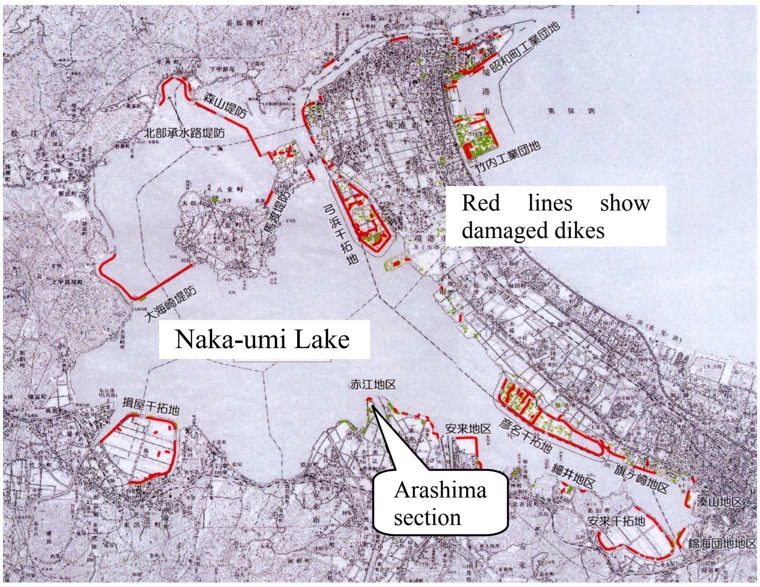


Fig.28 Damage to Naka-umi Lake shore dikes during the Tottoriken-seibu earthquake in 2000

toes. As shown in **Fig. 30**. The effectiveness of the counterweight approach was checked using the Towhata analysis, described later (Towhata et al., 1995).

At some sections, soil improvement was conducted, and steel sheet piles or steel pipe piles were installed to increase the stability of the damaged dikes.

Fig.29 Kyuhin dike damaged during the Tottoriken-seibu earthquake in 2000

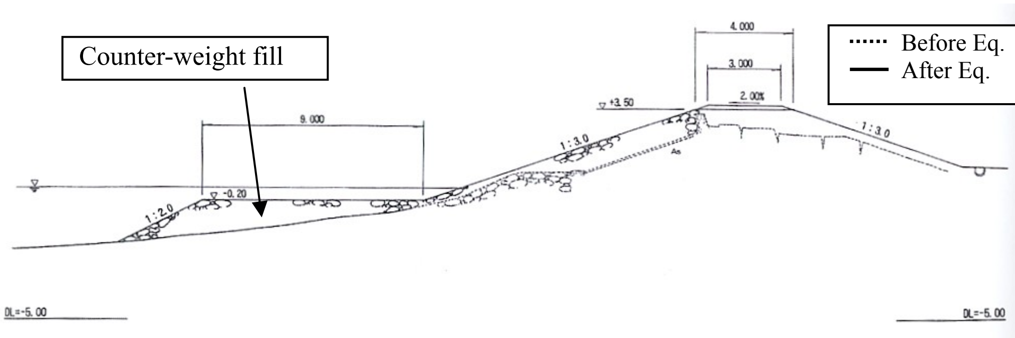


Fig.30 Rehabilitation of the Naka-umi Lake shore dike by counter-weight fill

Lowering the ground water table at the Hachirogata dike

It was found that the distribution of the damaged sections of the Hachirogata dike due to the 1983 Nihonkai-chubu earthquake was well correlated with the thickness of the loose sandy layer beneath the dike. From a study of various site conditions at the damaged sections such as particle size distributions of sands, SPT N values, and laboratory test results of liquefaction resistance of the replaced sand layer in the foundation ground and the observed ground motions together with evidence of sand boils, it was concluded that the dike failures were caused by the liquefaction of the loose sand layer beneath the dike.

Since the total length of the damaged sections of the dike was so long, it was necessary to seek a very economical solution. After a lot of study lowering the ground water level beneath the dike was taken as the basic measure for improving the stability of the damaged dike sections against possible future earthquakes.

The solution was to place a sheet pile cutoff wall at front toe of the dike, and also to install drainage pipe to lower the groundwater table in the dike as illustrated in **Fig. 31** (Hashimoto et al. 1985).

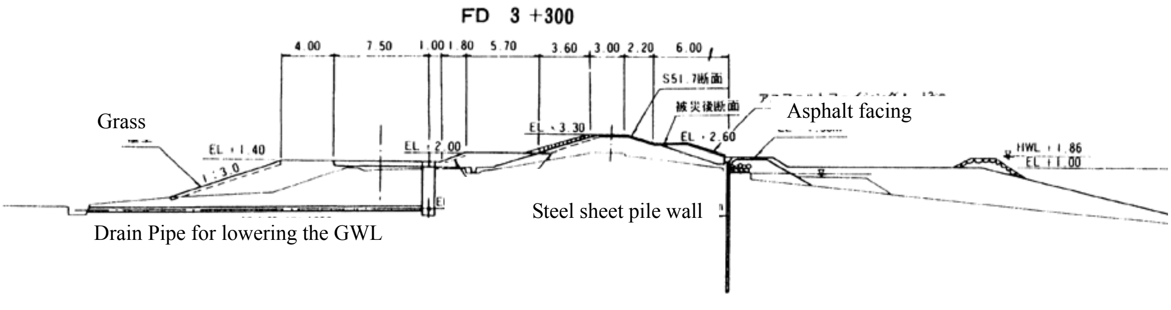


Fig. 31 Lowering the ground water table in the Front dike at Hachirogata

The framework of restoration work in this case can be summarized as follows.

1) New berm is placed at front slope.
2) At some sections where the original cross section was small, new berm is added at rear slope.
3) Cut off sheet wall is placed at front toe of the dike.

4) Riprap mound is placed in front of the cut off sheet wall.
5) The surface of the dike is covered by asphalt facing.
6) Drainage pipe is installed at 100m interval to reduce the ground water height.
7) Width of the crest is enlarged to 3 m from originally 2 m.

Subsoil improvement by SCP method at the Tokachi River and the Kushiro River dikes
As pointed out earlier, the main cause of dike failure at severely damaged sections of the Tokachi and the Kushiro River dikes during the 1993 Tokachi-oki earthquake was the liquefaction of the loose saturated lower part of the embankment. It was decided to densify the loose layer at severely damaged sections by the SCP method as illustrated in **Fig. 32** (Oshiki and Sasaki 2001).

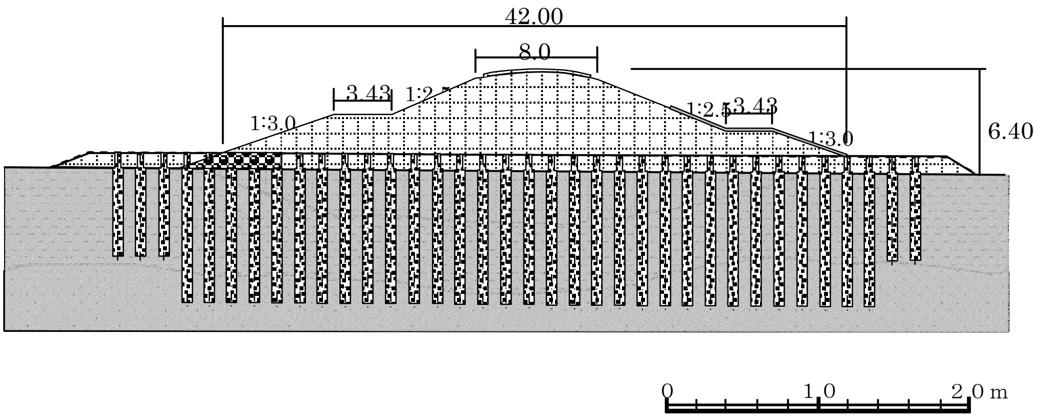


Fig. 32 Restoration work for the Tokachi River and the Kushiro River dikes damaged by the Kushiro-oki earthquake in 1993

There was concern about that the SCP method could effectively densify the saturated loose sand layer at shallow depth because of low overburden pressure. Therefore a thicker than usual working mat was placed above the foundation ground based on the results of pilot tests. Since this working mat became a part of the dike embankment, it had to have a low enough permeability, therefore the construction material for the mat was selected to match this demand and was compacted by bulldozer.

Since the damaged embankment had to be removed and the ground surface be made flat for the SCP work, temporary protection to prevent flooding during the restoration work was needed. Therefore a steel sheet pile cofferdam was constructed outside the damaged section, the damaged dike was removed, ground treatment by SCP method was completed and the new embankment section was constructed.

As it was clear that the drainage function of existing gabions as shown in **Fig. 33** was effective in lowering the groundwater level inside the embankment, a drainage system at toe was used as the standard method for the repaired section regardless of the use of subsoil improvement, in order to prevent any liquefaction of the bottom part of the embankment.

Fig. 33 Gabion at the toe of the Tokachi River dike

The framework of this remediation is as follows；
1) Strengthen the sections (limited to seriously damaged sections only) to prevent future liquefaction.
2) To accomplish this, liquefiable layer was compacted by SCP.
3) Coarse grain materials were laid for transporting embankment materials and left as a rear toe drain.

4) Lightly damaged sections were repaired as they were before the earthquake without additional strengthening.
5) To accomplish this, only the cracked parts of the embankment were re-compacted.

Repaired sections of the Kushiro River dike were subjected to the same order of shaking intensity during the Hokkaido-toho-oki earthquake in 1994, but the sections repaired by SCP survived without damage (Finn et al. 1997, Oshiki and Sasaki 2001). This showed the effectiveness of the restoration works.

Subsoil improvement by DMM work and restraint by steel structures at the Yodo River dike

The Kobe earthquake in 1995 caused severe damage to the Yodo River dike. Heavily damaged sections were seen at three sections, namely the Torishima, Takami, and the Nishijima sections as described elsewhere (Matsuo 1996, Sasaki and Shimada 1997b).

It was apparent that the liquefaction of the foundation ground, with SPT N values in the range 8-10, caused the damage to the Yodo River dike. The deformation process at the Torishima section was thought to be as illustrated schematically in **Fig. 34** (Sasaki and Shimada 1997b).

Because of the liquefaction of an 8-10 m thick sand layer beneath the dike, the embankment was stretched by the reduction in shearing resistance of the foundation. This split the embankment and the separated sections subsided into the liquefied deposit due to the weight of the embankment, resulting in large deformations.

This part of the dike section was armored by a concrete facing on outer slope and parapet wall was constructed on its shoulder as shown in **Fig. 34**. This prevented access to the waterfront for public enjoyment, so the cross section of the rehabilitated dike was changed to have a moderate slope at front side to the river channel. **Figure 35** shows the typical cross section of the restored section.

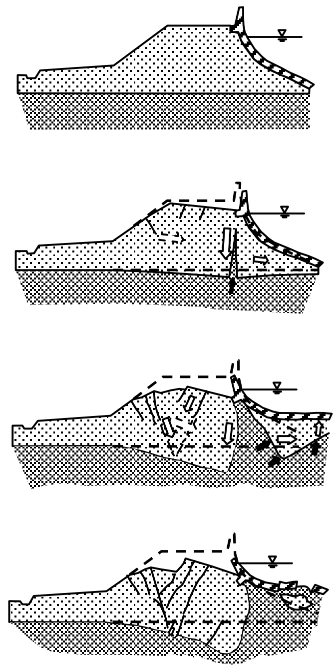


Fig.34 Deformation process of the Yodo River dike

As the foundation ground was expected to liquefy again during a future earthquake, it was decided remediate the liquefiable layer by a subsoil improvement. DMM was chosen as the treatment method at the Torishima section because of the small disturbance by noise and vibration during the construction period, since the hinterland of this section is highly populated as shown in **Fig. 36**.

Before construction of the ground improvement, it was necessary to construct the temporary cofferdam to protect the hinterland against the possible flooding during construction. Therefore a steel sheet pile cofferdam was installed. However, because of concern that the soil between the sheet piles might liquefy during aftershocks

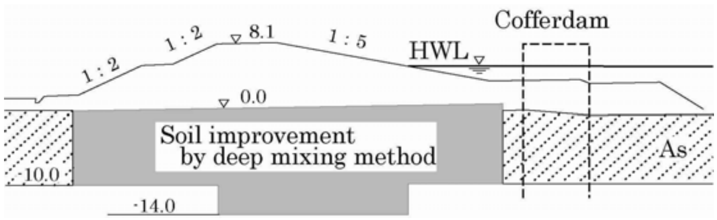


Fig. 35 Remediation of the Torishima dike by DMM

Fig.36 Aerial view of the restoration work at Torishima

drainage pipes were installed along the sheet piles.

Figures 37 and **38** show the cross sections of restoration works for the Nishijima and Takami dikes. At these sections, the river side slope of the dike did not fail. Therefore only half of the cross section was rebuilt to improve stability. Because houses were close to the damaged section of the dike at Takami section steel sheet piles were installed to resist dike failure. The damaged Nishijima section was repaired by installing double rows of steel sheet piles with gravel packed in between them to dissipate the pore water pressures in any future earthquake through the holes in the sheet piles.

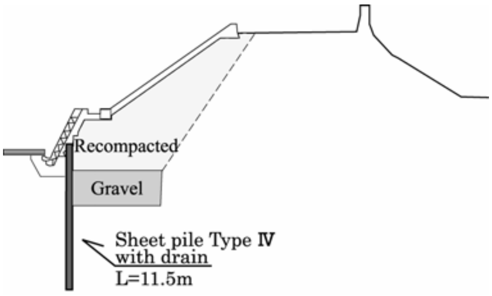


Fig. 37 Remediation at the Takami section

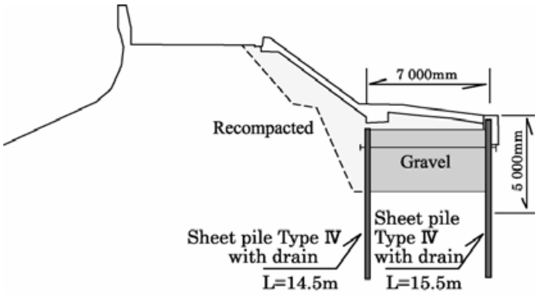


Fig.38 Remediation at the Nishijima section

The framework of the rehabilitation of the damaged section of the Yodo River dike is as follows:

1) Severely damaged Torishima section was rehabilitated by changing its cross sectional shape, so that people can easily access the waterfront by a gentle slope.
2) Parapet wall to assure the flood protection was replaced by raising the crest elevation.
3) At this section, the liquefiable subsoil layer was improved by DMM, in order to keep noise and vibration to a minimum.
4) At partially damaged sections at Takami and Nishijima, only the failed parts of the dike were restored.
5) At these sites, retaining walls were embedded at their toe by using steel piles and providing drainage capacity to prevent high pore water pressures in future earthquakes.

METODS TO ESTIMATE RIVER DIKE SETTLEMENT

Classification of the estimating methods of river dike settlement due to soil liquefaction

The settlement of river dikes is composed of the subsidence of the bottom boundary and the deformation of the dike. The subsidence of the bottom boundary is due to the large deformation of the subsoil layer. The subsoil deformation includes the deformation due to the liquefaction during the shaking in an undrained condition followed by the deformation due to the dissipation of the raised pore water pressure, namely by consolidation after the shaking.

Though it is clear that the most of the deformation of the subsoil layer is generally caused by the decrease of its shear strength due to the liquefaction, however, it is not yet fully clear how the deformation progresses during the actual ground shaking. Several research investigations are being conducted to clarify the mechanical properties of soil in the post-liquefaction state, but a consensus on how to treat liquefied soil for analyzing its deformation has not yet been reached.

Two different constitutive model of liquefied soil are used in practice. One model treats the liquefied soil as a softened solid, and the other model as a viscous liquid. There are several numerical methods to estimate the river dike settlement in practice used in Japan based on these models.

A technical committee organized by the Japan Institute of Construction Engineering (JICE) examined the efficiency of these analytical approaches (Japan Institute of Construction Engineering 2002). Those numerical techniques can be divided into empirical approach and analytical approaches for the most part.

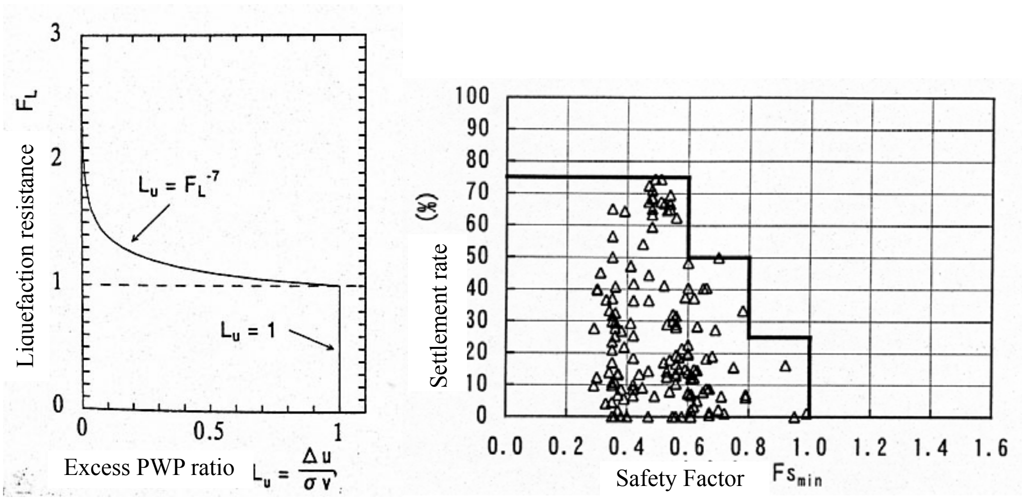
The committee picked an empirical approach and four analytical approaches; The analytical approaches are: a computer code named ALID for static analysis using softened soil concept: the Towhata method, a static approach using the viscous liquid concept: a computer code named LIQCA using coupled effective analysis, and a computer code named FLIP using an uncoupled analysis. Computed deformations were compared with the dike settlements observed during the Hokkaido-nansei-oki earthquake and the Kobe earthquake.

Empirical approach

Safety factor calculated by the stability analysis using the slip arc method has long been used for estimating the dike settlement in practice in Japan. Though the slip arc method is based on the limit equilibrium concept and deformation is not obtained, however, it is experienced that the deformation of an embankment becomes larger as the safety factor becomes smaller. Therefore the safety factor has been used for estimating the dike settlement based on this experience because there was no other simple and reliable method to estimate the dike settlements.

Subsoil conditions inferred from geomorphologic conditions and the damage history in the past are used as a basis for selecting a dike section which should be treated against seismic effects.

Then the safety factor is examined for selected sections by considering the inertia force due to earthquake shaking by seismic coefficient, or the safety factor is calculated by considering the decrease of shear strength in liquefiable layers separately. And the smaller one of the obtained safety factor is selected as the safety factor for this embankment. The decrease of the shear strength is usually calculated from the susceptibility to liquefaction of the subsoil layer, since it is known that the F_L value obtained from the evaluation of the susceptibility of liquefaction is correlated to the increase of the pore water pressure (Japan Road Association 1986). **Figure 39a** shows the chart to estimate the excess pore water pressure ratio and **Fig. 39b** shows the empirical correlation between dike settlement and safety factor (River Bureau 1995).



(a) r_u-F_L **(b) S/H-F_s**

Fig.39 Charts used for estimating the dike settlement

The reason why the safety factor against the inertia force and the safety factor against the liquefaction are calculated separately is that the instability of dike due to the liquefaction is considered to be brought at or after the end of shaking.

This method is still being used in practice to prioritize the dike sections of long dikes in need of strengthening.

Analytical approach

Analytical approaches used in Japan for estimating the dike settlement due to subsoil liquefaction can be subdivided into the static approach and the dynamic approach (Japan Institute of Construction Engineering 2002). Both approaches are accomplished by using Finite Element Method. There are two types of analysis in the static approach. One is to treat the liquefied soil as a softened solid, and the other is to treat the liquefied soil as a viscous liquid. The committee selected ALID for the former and Towhata method for the latter method.

There are two types of analysis in the dynamic approach, one is a total stress analysis and the other is an effective stress analysis. And the effective stress analysis is further divided into two types depending on whether the generation and dissipation of pore water pressure is coupled with the equations of motion or not. The committee selected LIQCA for the former and FLIP for the latter method.

ALID:

Yasuda (1999) proposed a simplified method for estimating residual deformation of an embankment using FEM. In this computation named ALID, the residual deformation is due to the action of the gravity force on the embankment after the stiffness of the soil has been softened by raised pore water pressure. Therefore the susceptibility of the subsoil layers to liquefaction has to be evaluated independently beforehand.

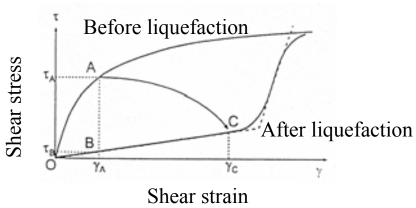


Fig.40 Schematic explanation on degradation of G

The deformations of the embankment and foundation are given the difference in the coordinates of the finite element nodes between the pre- and post earthquake states.

Figure 40 shows a schematic explanation of the decrease of the shear modulus by the softening (Yasuda et al. 1999). Inclination between OA, namely G_N, in this figure is used as the shear modulus for the pre-earthquake situation and the inclination between OC, namely G_1, in this figure is used for the post-liquefaction state.

Degradation of the shear modulus in this method, G_1/ G_N, is considered to be governed by the fines content and the liquefaction susceptibility, F_L.

Figure 41 shows the chart to estimate the decrease of the shear modulus from the F_L value.

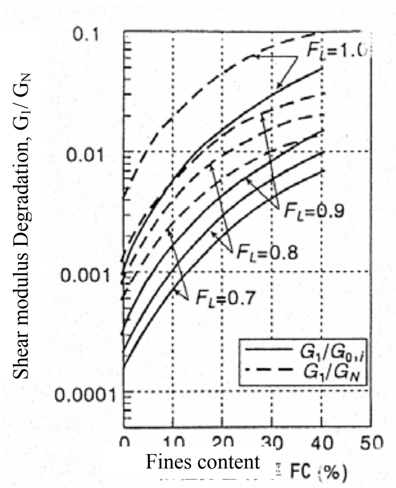


Fig.41 G/G_0- F_c-F_L

Towhata method:

The Towhata et al. (1992) proposed a method to predict the maximum possible displacement of liquefied ground using a large-deformation formulation based on the minimum energy principle. Later in 1995 a revised version of a method to predict liquefaction induced subsoil deformation during an earthquake excitation was presented (Towhata et al. 1995). In these methods of analysis, the liquefied soil is treated as a viscous liquid and the deformation is induced by gravity force.

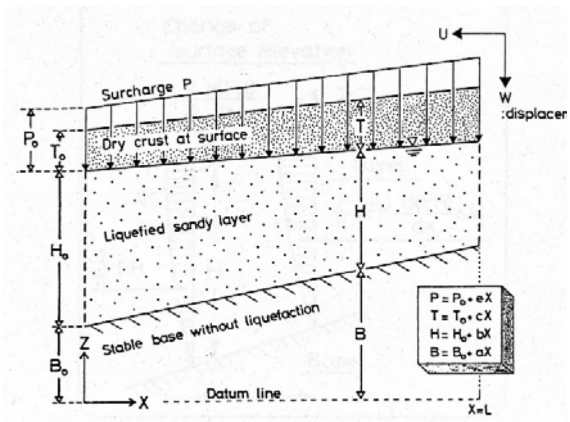


Fig.42 Basic concept of the Towhata method

In this method, foundation ground is divided into two layers, one is a non-liquefiable surface layer and the other is the liquefied layer. Firstly the deformation of the liquefied layer is calculated from the state of force balance where the potential energy satisfies the minimum state condition. This deformation is the maximum possible amount of deformation. The vertical displacement of the liquefied layer is assumed to follow a sinusoidal distribution as illustrated in **Fig. 43**.

Next, the time history of deformation during the earthquake excitation is calculated using the maximum possible amount of deformation and virtual damping ratio of the liquefied layer. The virtual damping ratio was introduced in this analysis in order to express the retarding effect of the viscous liquid for the motion of the model ground.

Model ground for the calculation of an embankment displacement is separated into segments for modeling the irregular conditions of the ground and embankment weight as shown in **Fig. 43**. Then the solution for the total model is calculated considering the interface conditions between segments (continuity condition and minimum potential energy condition for overall system).

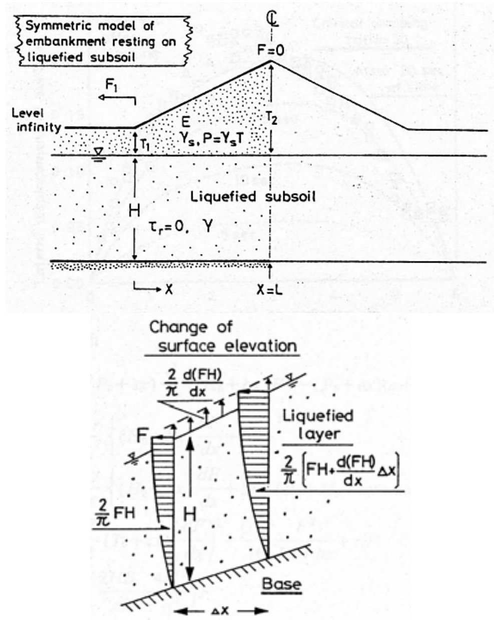


Fig.43 Separation to segments

LIQCA:

Oka et al (1999) developed a two dimensional effective stress analysis computer code to analyze seismic response of ground, named LIQCA. In this analysis, a cyclic elasto-plastic constitutive model based on a non-linear kinetic hardening rule is used. As this computation code is coupled with the pore water flow, consolidation due to the dissipation of the pore water pressure, the time history of effective stresses in the ground, liquefaction process in the ground, possible failure zones in an embankment due to stress re-distribution inside the embankment, and the deformation of the liquefied layer can all be obtained.

FLIP:

Iai et al (1992) developed a two dimensional effective stress analysis computer code named FLIP for analyzing a seismic response of a ground. In this method, generation of pore water pressure in a saturated layer is calculated by using a strain space plasticity model. As this code is not coupled with the pore water flow, the deformation of the liquefied soil due to consolidation can not be obtained. However the raised pore water pressure in the foundation ground during earthquake shaking, deformation of the liquefied ground, and the embankment deformation can be analyzed.

Comparison of analyzed settlements to the actual settlement of dikes during past earthquakes

Estimated dike settlements by the analytical approaches mentioned above were compared to the observed settlement during the Hokkaido-nansei-oki earthquake and the Kobe earthquake.

Table 3 shows the dike sections used for this comparison.

Table 3 Analyzed dike sections

Earthquake	River	No.	Dike section	Estimated PGA	Damage	Observed settlement
1993 Hokkaido-nansei-oki Earthquake	Shiribeshi-Toshibetsu River	No.1	Left Bank, 4 k 440	260 gal	Serious	2.6 m
		No.2	Left Bank, 5 k 000	320 gal	none	0 m
		No.3	Left Bank, 1 k 710	225 gal	large	1.3 m
		No.4	Right Bank, 4 k 440	280 gal	none	0 m
		No.5	Left Bank, 4 k 440	290 gal	minor	0.6
1995 Kobe Earthquake	Yodo River	No.1	Left Bank, 4 k 440	265 gal	serious	2.7 m
		No.2	Left Bank, 4 k 440	260 gal	minor	0.3 m

Figure 44 shows the comparison between the calculated dike settlements and the observed settlements.

As seen in this figure, the predicted settlements by analytical approach agree fairly well, but the empirical approach by the slip arc method predicts always larger settlements than the observed ones. This too conservative prediction by the sliding arc method arises from the use of the step-wise relationship between the calculated safety factor and the settlement (see **Fig. 36**), which envelopes the maximum.

It is also noticed that the static approach by Towhata method can estimate the dike settlement well as the dynamic approaches (LIQCA and FLIP) do.

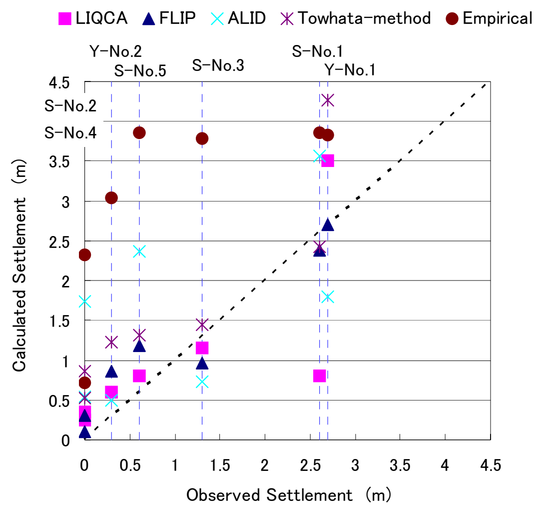


Fig.44 Comparison between calculated and observed settlements

Test results on the effectiveness of remedial treatment of a liquefiable layer for an embankment using 50-G centrifuge models were also analyzed by the aforementioned analytical approaches. Tests were conducted for the cases of partial solidification of the foundation ground near the embankment toe, partial densification of the foundation ground near the toe, and a restraint of the dike deformation by sheet pile wall at the embankment toe. Due to the limited space of this paper a quantitative description is not presented here. However, it should be noted that the estimated settlements agreed well with the test results for each case of treatment.

In assessing the seismic safety of river dikes, it is considered essential to predict the earthquake induced permanent displacements. Such analytical techniques as presented above are expected to be used in practice. Past case studies have, however, indicated that the numerical results show wide scattering depending on the numerical code and the user. It is especially the case when the numerical predictions are made for the field case in which available geotechnical data are very limited. In order for such numerical procedures to get wide acceptance in practice, it is recommended that their capabilities, limitations and prediction accuracy are studied and published.

CONCLUSIONS

Findings from past earthquakes in Japan and numerical methods for predicting dike deformations were reviewed. They are summarized as follows.

1) In general seismic damage to river dikes in Japan were triggered by 160 gal of PGA or higher.
2) Most of the damage to dikes was due to subsoil conditions. Subsoil liquefaction was the main cause of serious damage to river dikes. Geomorphologic information aids in estimating the susceptibility to damage.
3) A non-liquefiable layer over the liquefiable layer in the subsoil deposit beneath dikes may reduce crest settlement.
4) A stretching type of failure is apparently induced when the liquefaction takes place at shallow depth in the foundation ground.
5) Embankments on peat layers may fail due to liquefaction of the embankment materials, if the material subsides beneath the ground water level.
6) Periodical appearance of localized failure of a dike was seen during the Kushiro-oki earthquake.

7) Evidence of effectiveness of remedial measure to river dikes is being accumulated, including cases of improving the subsoil layer and the use of geogrid.
8) Measurement of the dike settlements were successfully accomplished during the Tottoriken-seibu earthquake.
9) Deformations of dikes might be increased by wetting during the rainfall before the earthquake as shown in the case during the 2003 Miyagiken earthquake.
10) Continuous shaking of pre-event, main-shock, and after-shock in 17 hours was experienced during the 2003 Miyagiken earthquake.
11) A design method for river dike against the strong ground motion, called Level-2, needs to be established.

Finally, it was shown that the currently used numerical analyses give predictions of crest settlements that agree reasonably well with observed dike settlement during earthquakes.

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