Stability of flood protection dikes with potentially liquefiable foundations: analysis and screening criterion

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Abstract: Many flood protection dikes were damaged in both eastern and western Hokkaido, Japan during large earthquakes in 1993 and 1994 due to liquefaction in the foundations. A criterion based on crest settlement was developed for prioritizing remediation of the diking system to resist future earthquakes. The potential crest settlement during earthquakes of M=7.5 to M=8.0 was expressed in terms of geometrical properties of the dike cross-section. The procedure used to estimate the settlements was first used to simulate dike failures in eastern Hokkaido. Then, blind prediction tests were carried out for typical dikes in western Hokkaido. These predictions were then checked against actual field performance. The criterion proved to be very effective in predicting the performance of the damaged dikes.

Introduction

Flood protection dikes along the Kushiro and Tokachi rivers suffered considerable damage during the 1993 Kushiro-oki earthquake off eastern Hokkaido, Japan. Damage included longitudinal and transverse cracks, slope failures and cave-ins. The more severely damaged dike sections were 6 m - 8 m high, and were constructed of compacted sand fill resting on a comparatively thick peat layer. The dikes were damaged at 18 locations over a length of about 10 km along the Kushiro river (Fig. 1). The severest damage occurred in Kushiro Marsh (Sasaki et al., 1993; Sasaki, 1994a,b). Dike sections which failed were reconstructed, after the foundation soils had been improved by the installation of sand compaction piles (SCP) (Fig. 1).

In 1994, a major earthquake occurred off the west coast of Hokkaido, the Nansei-oki earthquake, which caused failures of flood protection dikes along several river basins in western Hokkaido. After these earthquakes, the Hokkaido Development Bureau initiated a program of improving the diking systems. Because of the great length of dikes, they wished to develop a criterion for prioritizing the remediation work. One of the approaches they adopted was to use potential crest settlements as a criterion. As part of the settlement studies, simulation of some dike

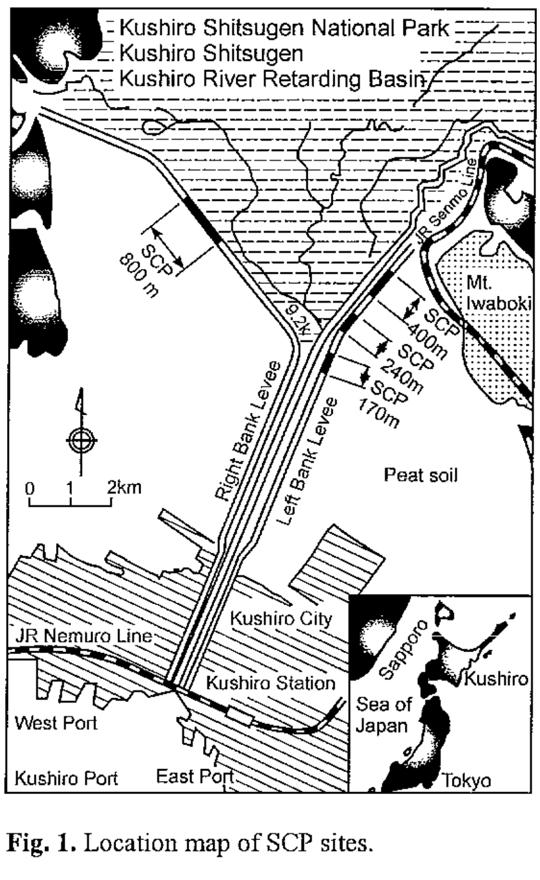

Fig. 1. Location map of SCP sites.

failures during the 1993 Kushiro-oki earthquake were conducted by the writer. After these simulations proved satisfactory, a program of parametric studies on the behaviour of different types of cross-sections in the diking system were authorized with the objective of linking crest settlements of the dikes with easily determined parameters of the diking system. On the basis of the parametric studies, the displacement criterion presented later was developed. The studies were commissioned by the Hokkaido Development Bureau through the Advanced Construction Technology Center (ACTEC) in Tokyo. Some of the major findings of these studies are presented here to show the role of displacement analysis in establishing criteria for remediation.

Outline of earthquake damage and rehabilitation work on the left-bank dike of the Kushiro river

Fig. 2 illustrates a typical failure mode in at a cross-section of the dike on the left bank of the Kushiro River between stations 9K400 and 9K850. The description of the failure mechanism which is appended to Fig. 2 is adapted from Sasaki et al. (1995).

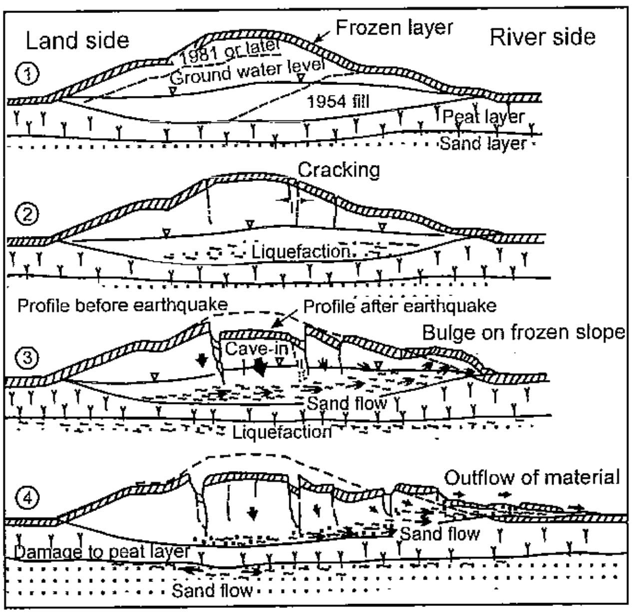

Fig. 2. Mechanism of embankment failure.

①The 6 m high embankment settled 2.5 m. Up to 2.5 m of the embankment was below the ground water level.
②Earthquake motions cracked the crown of the embankment and liquefied the saturated embankment material.
③The embankment slope slid towards the river on the liquefied material, and the crest of the dike settled.
④Cracking of the frozen surface layer allowed an outflow of liquefied materials from the toe. The crest then settled about 2.5 m.

The repair work to the dikes was carried out very quickly after the earthquake. The dike sections which were cracked and suffered only relatively small deformations were repaired by excavating around the cracks and re-filling the excavation.

Dikes which had suffered foundation failures and large displacements were reconstructed. First, a temporary dike was constructed using steel sheet piles. The failed portions of the dike were removed and the foundation sands were remediated using the sand compaction (SCP) method. A new dike was then constructed on the improved foundation.

This procedure was adopted to ensure fast and timely repair. However, full scale remediation of the foundations and dike reconstruction was a very expensive procedure and studies were conducted to determine how much remediation would be necessary to ensure dike stability. Schemes for partial remediation were developed which reduced costs considerably.

Because of the extent of the diking system, remediation would be a continuing activity. Therefore, a key element in the remediation strategy was to develop a relatively simple criterion for determining which dikes were more critical so that the requirements for remediation could be prioritized properly. The Bureau adopted potential crest settlement as a criterion.

The role of displacement analysis in evaluating post-liquefaction response and assessing the adequacy of proposed remediation measures is now becoming a part of engineering practice. Since 1988, large displacement post-liquefaction analysis and its application in design of remediation measures has been the subject of several reviews (Finn, 1993; Ledbetter and Finn, 1993; Finn, 1998). Displacement analysis also has a role in prioritizing the sequence of remediation for long linear structures such as flood protection dikes. An example of this role in developing a strategy for remediating the flood protection dikes in Hokkaido is given below.

Preliminary studies

Deformation analyses of dike cross-sections were conducted as follows. First a dynamic effective stress analysis was carried out using the program TARA-3 (Finn et al., 1986). This analysis gave stresses, seismic porewater pressures and both dynamic and permanent deformations in the dike. However, the program is based on small strain theory. If large displacements were indicated, typically when extensive liquefaction occurred, the large displacement analysis was carried out using the program TARA-3FL (Finn and Yogendrakumar, 1989). This program uses a Lagrangian updating scheme to handle large strains and displacements and allows the liquefied material to deform at constant volume.

It was decided to test these methods of analysis first by simulating the behaviour of some dikes that underwent large post-liquefaction displacements in eastern Hokkaido during the 1993 earthquake. If these simulations were successful, then the plan was to validate the predictive capability of the displacement analyses by making blind predictions about the displacements of selected dikes in western Hokkaido during the 1994 earthquake. One of these simulation studies is described in the next section.

Trial analysis of dike section 9K850

The failure mode at a location on the left bank of the Kushiro river is shown in Fig. 3. The height of the dike before the earthquake was about 7 m and the crest width was about 8 m. As a result of earthquake shaking, the crest of the dike settled about 2 m and movement of the slope of the order of 3 m took place towards the river. The ground was frozen to a depth of about 0.7 m at the time of the earthquake. The brittle nature of the frozen layer is probably responsible for the sharp step feature in the crest near the upstream slope. This frozen layer was taken into account during the simulation.

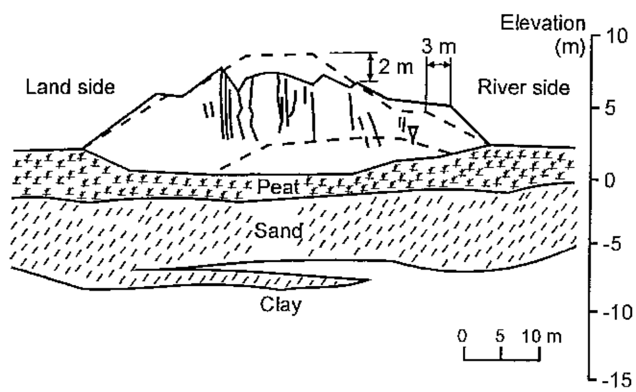

Fig. 3. Mode of failure on left bank of Kushiro River at station 9K850 (after Sasaki et al., 1995).

Sasaki (1994a,b) reported on a detailed critical study of this failure. The dike is a compacted sand fill resting directly on a layer of Hokkaido peat. The peat settled 2 m to 3 m under the weight of the dike (Kaneko et al., 1995). This settlement had two important effects. First it brought the lower part of the sand fill below the water table that existed at the time of the earthquake, as shown in Fig. 3. This created the opportunity for liquefaction in the fill. Secondly, the large settlement caused a redistribution of stresses in the lower part of the fill. The stretching and arching of the fill reduced the confining stresses in the saturated region. The stress relaxation zone has a major impact on the estimation of liquefaction resistance in the bottom of the fill. The low confining stresses in the stress relaxation zone lead to unusually low N values for a compacted fill.

No uplift of the ground was observed near the toe of the failed section. This indicates that the underlying peat did not deform significantly. This is in keeping with the findings of Noto and Kumagai (1986) that Hokkaido peat does not lose strength during cyclic loading. The lack of uplift also suggests that it is unlikely that liquefaction occurred in the alluvial sand layer under the peat.

Excavation of some of the damaged spots revealed signs of sand boil from cracks in the lower part of the dike. This indicates that the damage was caused by liquefaction in the part of the levee that had embedded into the alluvial peat layer, as shown in Fig. 3.

A detailed site investigation was conducted using primarily standard penetration tests with a limited amount of sampling. As shown in Fig. 3 and Fig. 7, the soil profile in this area consists of flat layers deposited near the ground surface. A 3-4 m thick peat layer covers the underlying layers of alluvial sand and clay. The thickness of the upper sand layer varies from 2 m to 10 m at locations along the left-bank dike. Beneath these layers lies a 20-30 m thick clay layer which contains a thin sand layer. The predominant SPT-N values (7-15) of the upper part of the alluvium sand layer indicate that the material is relatively loose, but the lower part of the same layer in some thicker sections had SPT-N values of more than 20.

Soil properties

Soil properties for both static and dynamic analyses were provided by ACTEC.

For liquefaction analysis, Standard Penetration Resistance-N values of 7 and 15 were selected as representative of the saturated sand fill and the alluvial sands, respectively. These N values were then converted to $(N_1)_{60}$ values corresponding to a standard nominal pressure of 100 kPa and an energy level of 60% of the free fall energy of the hammer. The liquefaction resistances for different segments of the dike were then determined using the liquefaction resistance chart developed by Seed et al. (1985).

If the saturated sand fill should liquefy, the residual strength of the fill would be mobilized by the large shear strains generated as the embankment deformed to reach an equilibrium position. In current engineering practice, residual strength is often expressed as a function of the effective overburden pressure. Research by Vaid and Thomas (1994) and Vaid and Sivathayalan (1996) shows that the residual strength, S_r, can vary between 0.05 σ'_{vo} and 0.15 σ'_{vo}, depending on the void ratio of the sand, where σ'_{vo} is the effective overburden pressure. In the present study, a residual strength $S_{ur} = 0.10\ \sigma'_{vo}$ was selected initially based on past experience with potentially liquefiable constructed fills. Davies and Campanella (1994) proposed a formula for estimating residual strength which was subsequently corrected to the form given in Eqn. 1.

$$\frac{S_r}{\sigma'_{vo}} = 0.06 + 0.025\left[(N_1)_{60} - 6\right] \qquad [1]$$

$$6 \le (N_1)_{60} \le 30$$

The residual strength ratio, $S_r/\sigma'_{vo} = 0.1$ used in this study corresponds to an $(N_1)_{60} = 7.6$ according to Eqn. 1. This compares favourably with the $(N_1)_{60} = 7$ used in the analysis. The residual strength ratio also corresponds to median values of residual strength given by the Seed and Harder (1990) correlation between $(N_1)_{60}$ and S_r.

Seismic analysis of dike

Appropriate input motions for seismic analysis were specified by Jishin Kogaku Kenkyusho, Tokyo. Dynamic analysis was conducted in the effective stress nonlinear mode using the program TARA-3 (Finn et al., 1986). The large strain post-liquefaction deformations were calculated using the program TARA-3FL (Finn and Yogendrakumar, 1989). This program allows the liquefied region to deform at constant volume and uses a Lagrangian updating scheme to handle large strains.

The analyses were conducted assuming a uniform embankment, except for the frozen upper layer, because detailed knowledge of the distribution of soil properties was not available. Therefore, purely local details of the failure surface could not be modelled.

The computed deformed shape of the dike is shown in Fig. 4. The sharp break in the surface shows the effect of the frozen ground. The deformed shape and the magnitudes of displacements agree fairly well with the displacements measured after the earthquake. The computed maximum settlement and horizontal displacement are 2.3 m and 2.7 m, respectively, compared to measured displacements of 2 m and 3 m.

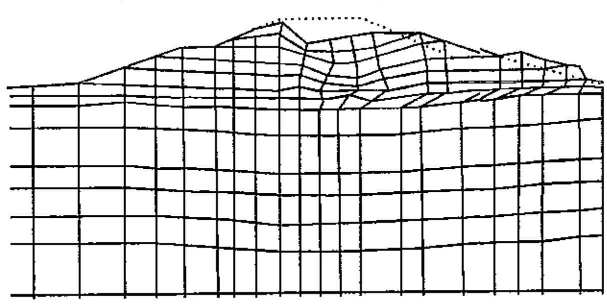

Fig. 4. Computed post-liquefaction shape of the dike.

The porewater pressure in the alluvial sand about 10 m from the toe of the dike on the side away from the river is shown in Fig. 5. The peak value is 10 kPa corresponding to a porewater pressure ratio, $u/\sigma'_{vo} = 30\%$. The computed porewater pressure is 7 kPa at the top of the alluvial sand. A porewater pressure of 5 kPa was measured in this area after the earthquake. The depth at which the recording was

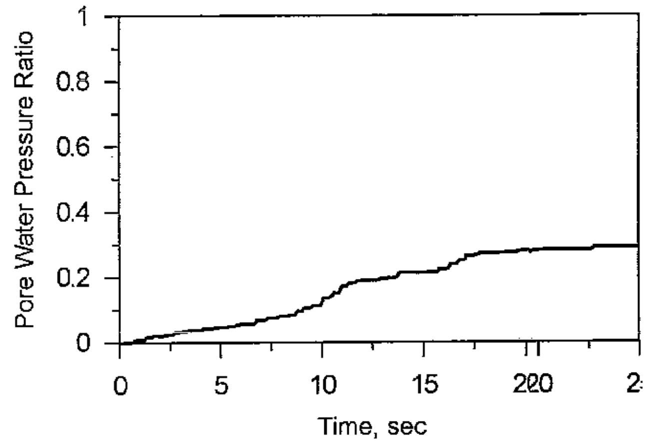

Fig. 5. Computed porewater pressure in the upstream alluvial sand.

made is not certain, nor is it known how much dissipation had occurred before the measurement was made.

As anticipated by Sasaki (1994a,b), the analysis showed that all the saturated sand fill liquefied during the earthquake. The development of the porewater pressure shown in Fig. 6 is typical of locations at the base of the dike. Liquefaction was reached after 10 s.

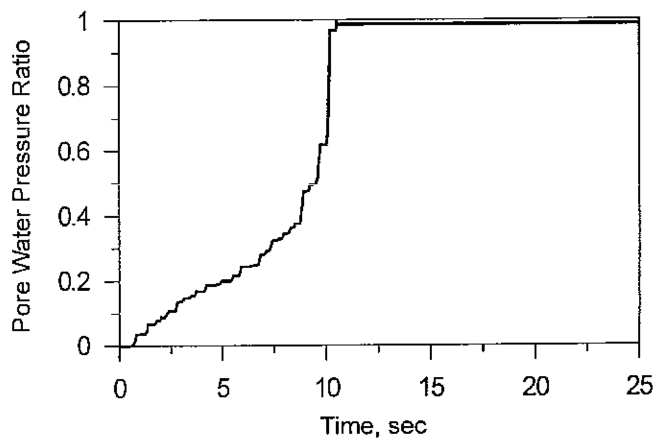

Fig. 6. Computed porewater pressure in the liquefied zone at the base of the dike.

Response of reconstructed dike to 1994 Toho-oki earthquake

After the earthquake the foundation soils were improved by the installation of sand compaction piles and the dike was reconstructed with somewhat different geometry as shown in Fig. 7.

Sand compaction piles were used to get an average N_1 value of 15 across the foundation soils above the peat layer and N_1=20 for the alluvial sand. The compaction piles were spaced to achieve an area ratio of a_s=0.17. The area ratio is referred to as the improvement ratio, in Japanese

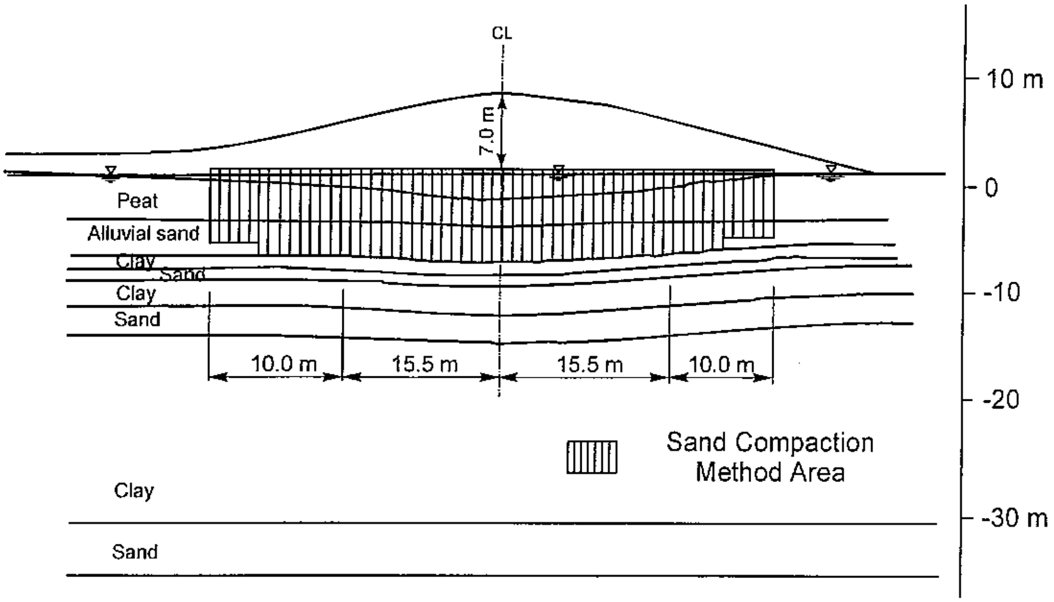

Fig. 7. Cross-section of remediated Kushiro dike at location 9K850 (from ACTEC).

terminology. The chart in Fig. 8 was used to determine the required improvement ratio.

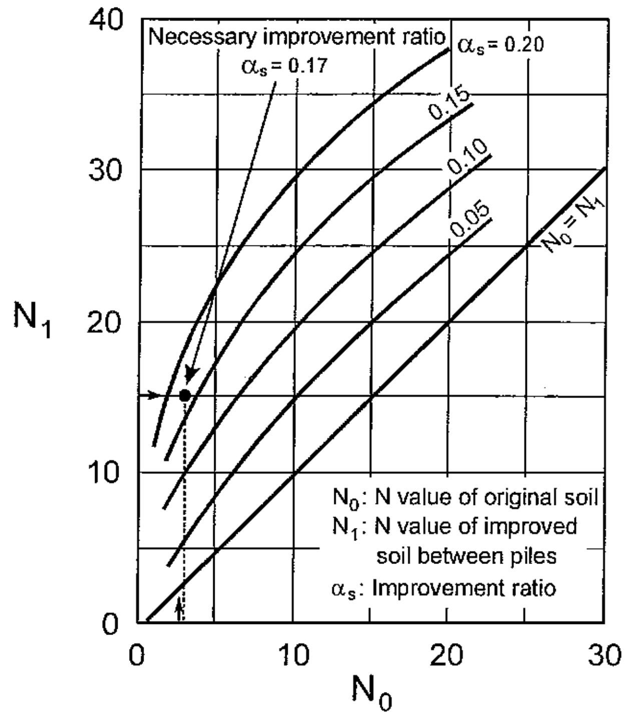

Fig. 8. Chart of SPT N-values before (N_o) and after (N_1) soil improvement (after Sasaki et al., 1995).

Six months after the completion of the reconstruction, the dike experienced strong shaking from the 1994 Toho-oki earthquake and survived with no damage. The seismic response of the reconstructed dike to ground motions from the Toho-oki earthquake was computed using the TARA-3 program in order to make a comparison between the computed response and the observed behaviour of the reconstructed dike. Site specific input ground motions developed by Jishin Kogaku Kenkyusho, from recorded 1994 Toho-oki earthquake motions, are shown in Fig. 9.

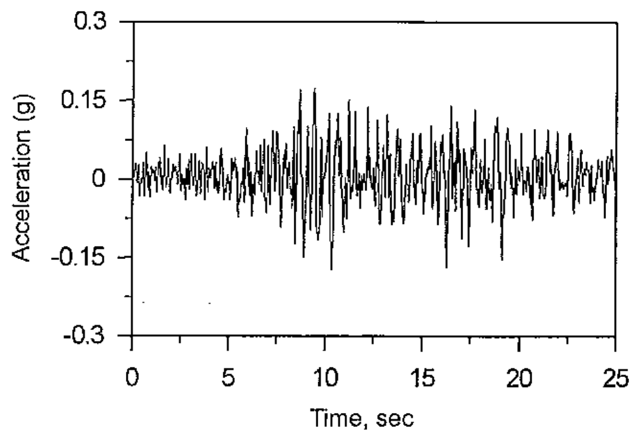

Fig. 9. Input motions for Toho-oki earthquake (JKK, 1995).

The $(N_1)_{60}$ values for the remediated areas of the sand fill embankment and the alluvial sand are now $(N_1)_{60}=15$ and $(N_1)_{60}=20$, respectively. The increases in $(N_1)_{60}$ lead to substantial increases in liquefaction resistance.

The soil properties used for the unremediated parts of the cross-section were those used previously in the simulation of the response of the original dike during the 1993 Kushiro-oki earthquake. All the soil properties provided by ACTEC were corrected where necessary for the effects of effective overburden pressure following normal engineering practice.

The computed porewater pressures in the saturated sandfill elements of the remediated section are computed to be generally less than 20% of the effective overburden pressure, (Fig. 10). At these levels of porewater pressure, the dike is stable and undergoes only very small displacements. During the earthquake, residual horizontal displacements of about 0.025 m were developed towards the river. Additional displacements of about 0.025 m occurred also due to post-earthquake consolidation. These displacements are negligible. The analysis confirms and quantifies the effectiveness of the remediation measures used to stabilize the dike.

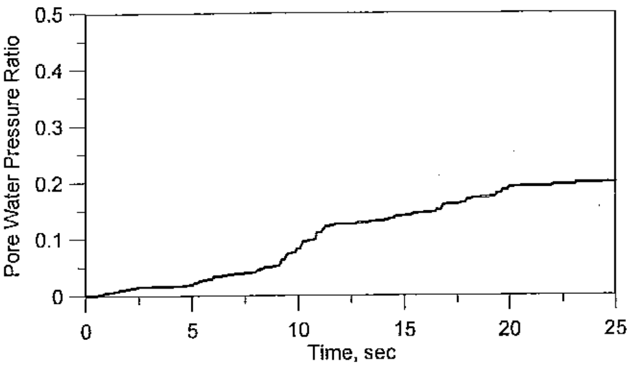

Fig. 10. Computed porewater pressure in the saturated zone at the base of the dike under the Toho-oki earthquake.

The peak ground accelerations at the site during the Toho-oki earthquake were about 30% less than the motions during the Kushiro-oki earthquake. As a consequence, the effectiveness of the soil improvement and reconstruction of this dike can not be properly assessed without knowing how the original dike might have behaved under the Toho-oki ground motions. Therefore the response of the original dike was also computed.

The Toho-oki ground motions were sufficient to liquefy saturated base of the original dike. Clearly the liquefaction susceptibility of the saturated fill was such that the 30% difference in peak ground motions was of little consequence. The computed deformations were fairly similar to those under the Kushiro-oki motions.

The effects of the difference in ground motion intensity were very evident, however, in the response of the alluvial sands. The computed porewater pressures were only about 50% of the pressures computed for the Kushiro-oki motions. These results show clearly that effective soil improvement at the base of the saturated sandfill is crucial to controlling the deformation of the dike to strong shaking.

The simulation of this dike failure and the results of a number of other studies were considered satisfactory and a major parametric study was approved to investigate the effects of some of the important parameters that control the consequences of liquefaction, such as the height of the dike, the slopes of the dike, the thickness of the non-liquefiable layer overlying the liquefied layer and the thickness of the liquefied layer itself. The effects of these parameters were characterized by the settlements of the crests of the dikes after liquefaction.

Development of screening criteria for remediation

The crest settlement criterion was first developed for dikes with side slopes 1:2.5 of the idealized type shown in Fig. 11. The thicknesses of the liquefiable, H_L, and nonliquefiable, H_{NL}, layers were varied and the computed crest settlements, S, after liquefaction of the sand layer are plotted in nondimensional form, S/H_D, in Fig. 12.

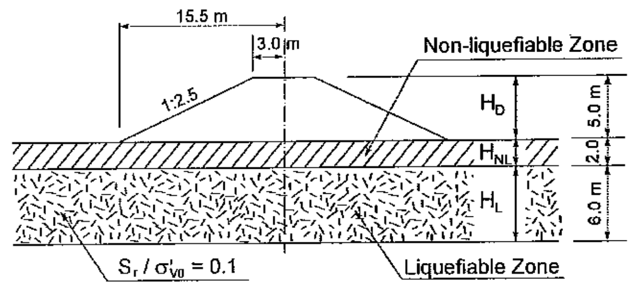

Fig. 11. Typical cross-section of dike for analysis.

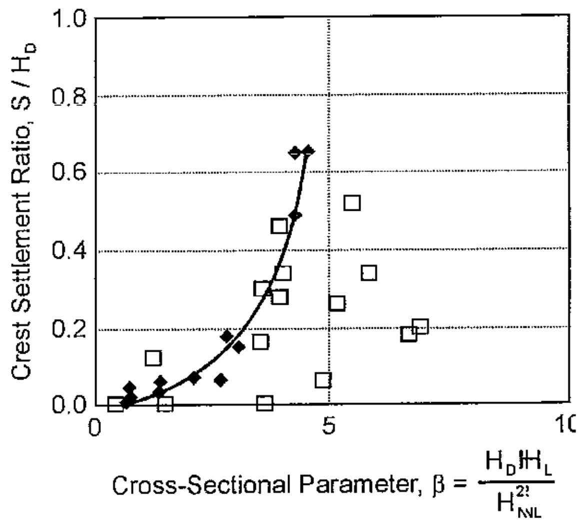

Fig. 12. Comparison of nondimensional computed settlements with nondimensional settlement prediction curve given by Eqn. 2.

The settlements are plotted as a function of the nondimensional variable β which is defined as

$$[2] \quad \beta = \frac{H_D}{H_{NL}} \frac{H_L}{H_{NL}}$$

The nondimensional computed crest settlements are matched by the curve shown in Fig. 12. The equation of this curve is

$$[3] \quad \frac{S}{H_D} = 0.01 \exp\left(0.922 \frac{H_D}{H_{NL}} \frac{H_L}{H_{NL}}\right)$$

ACTEC Engineers compared the measured crest settlements from a wide variety of dikes which underwent noticeable displacements during the Nansei-oki earthquake in 1994 with those predicted by Eqn. 3 as shown in Fig. 12. The black points represent the real cases corresponding to some of the idealised analyses done to develop the curve; the open points represent other dikes. The agreement was very good for dikes with slopes of 1:2.5. However, the field data showed that the side slopes had an important effect on the crest settlements and that separate criteria would be necessary for two other predominant slopes; uniform side slopes of 1:5, and unequal slopes, 1:5 and 1:10.

Parametric studies were conducted also for different values of S_r/σ'_{vo}. Dikes with $S_r/\sigma'_{vo} \geq 0.15$ showed only tolerable displacements.

It can be seen that the criterion based on crest settlement can be useful for deciding which dikes should be remediated first, on the basis of a knowledge of the thickness of a potentially liquefiable layer, the non-liquefiable layer and the height and slopes of the dike only.

Conclusions

Potential crest settlement proved to be a very useful criterion for identifying the dikes most in need of remediation. The criterion is easy to use because it is based only on geometrical parameters: dike height and side slopes and the thicknesses of the liquefiable layer and the overlying foundation layer. The settlement equation based on these parameters was validated by data from several case histories in blind tests.

The settlement criterion worked well for the diking system in Hokkaido because the ratio of residual strength to effective overburden pressure varied little from one site to another. It was possible to use a constant value of 0.1 for all dikes and hence it was possible to define crest settlements by geometrical parameters only. The criterion was developed for the large earthquakes which occur around Hokkaido in the magnitude range M = 7.5 to 8.0.

The Hokkaido dikes study is an important case history because it is the only instance in which post-liquefaction displacement analysis has been validated independently in blind tests using data from a large number of earth structures undergoing different levels of post-liquefaction displacements.

Acknowledgements

The study of the Kushiro dikes was initiated by Masayukii Kaneko, former Head of River Improvement Division, Construction Department, Hokkaido Development Bureau, who maintained an active interest in the project. Takekazu Udaka,, President of Jishin Kogaku Kenkysho, Tokyo, developed the input ground motions for the study. Kenji Satoh, Oyo Corporation, Tokyo, collaborated on the interpretation of soil conditions at the site. Satoshi Maeda and Akio Shibano, Advanced Construction Technology Center, Tokyo, provided technical assistance and cooperation at all stages of the study. In the final year of the project, the managing director was Y. Koga, ACTEC. The authors are grateful to these individuals and organizations for their active interest and support.

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