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W. D. Liam Finn

University of British Columbia, Vancouver, B.C., Canada

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ESTIMATING POST-LIQUEFACTION DISPLACEMENTS IN EMBANKMENT DAMS AND PRIORITIZING REMEDIATION MEASURES

W.D. Liam Finn
University of British Columbia
Vancouver, B.C., Canada V6T 1Z4

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ABSTRACT

Large displacement analysis is used to simulate the failures of flood protection dikes in Hokkaido, Japan, which occurred during the 1993 Kushiro-oki earthquake. These studies served to validate the method of analysis. The analysis was then used to predict displacements in dikes with potential for liquefaction by relating displacements to geometric characteristics of the dikes: height, slopes, and thickness of potentially liquefiable layer. The predictions were verified using dike displacement data from the Nansei-oki earthquake of 1994. The basis for reliable post-liquefaction analysis is a good estimate of the residual strength of the liquefied soils. For this reason, recent developments in evaluating residual strength are reviewed.

KEYWORDS

Embankments, liquefaction, residual strength, large strain analysis, post-liquefaction deformations, deformation criteria.

INTRODUCTION

In the context of this review, liquefaction is synonymous with strain softening of sand in undrained shear as illustrated by curve 1 in Fig. 1. When the sand is strained beyond the point of peak strength, the undrained strength drops to a value that is maintained constant over a large range in strain. This is conventionally called the undrained steady state or residual strength. Castro (1969) was the first to investigate this phenomenon systematically. He used dead load tests in which the steady state strength developed at constant acceleration. Such tests are now commonly run at constant rates of strain. Additional conditions, such as zero dilation (Schofield and Wroth, 1968), are often stated to define and delimit more precisely the steady state.

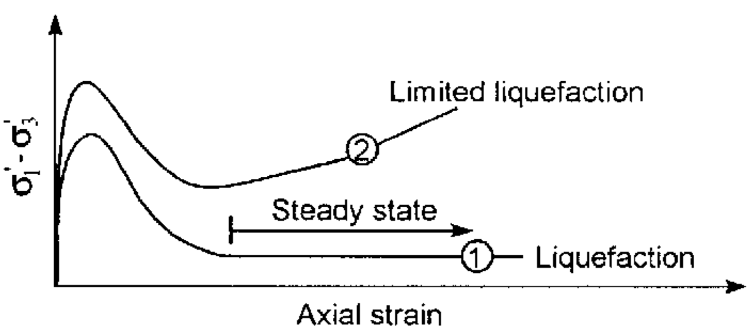


Fig. 1. Types of contractive deformation (Vaid et al., 1989).

If the strength increases after passing through a minimum value, the phenomenon is called limited or quasi-liquefaction and is illustrated by curve 2 in Fig. 1. Limited liquefaction may result in significant deformations because of the strains necessary to develop the strength to restore stability.

If the driving shear stresses due to gravity on a potential slip surface through liquefied materials in an embankment are greater than the undrained steady state strength, deformations will occur, until the driving stresses are reduced to values compatible with static equilibrium. The more the driving stresses exceed the steady state strength, the greater the deformations to achieve equilibrium. Clearly, residual or steady state strength is the major factor controlling post-liquefaction stability. The main steps for evaluating the seismic response of embankment dams containing potentially liquefiable soils in state-of-the-art engineering practice are as follows:

- Determine in which soils liquefaction will be triggered during the design earthquake.
- Determine the residual or steady state strengths of the liquefied soils.
- Conduct a stability analysis incorporating residual strengths to determine the factor of safety of the soil structure in its original configuration.

- Depending on the factor of safety, decide whether to conduct a deformation analysis.
- If deformations are unacceptable, plan remedial measures.
- For cost-effective remediation, use a tolerable deformation criterion to constrain the extent of remediation. This requires a displacement analysis of the remediated embankment.

The assessment of post-liquefaction performance is based on deformation analysis. The stability analysis is not essential. However, it can be useful; high and low factors of safety can rule out the need for deformation analysis of the embankment before remediation. The factor of safety also remains a good general indicator of what to expect in the way of performance from the embankment.

Therefore, the fundamental tasks for evaluating the post-liquefaction behaviour of embankment dams are to determine;

- Will liquefaction be triggered?
- What residual strength should be used in post-liquefaction stability and deformation analyses?

CRITERIA FOR TRIGGERING LIQUEFACTION

Triggering criteria were based originally on Seed's liquefaction assessment chart only (Seed et al., 1985). The critical resistance ratio, CRR, against liquefaction, was determined following the guidelines in Seed et al. (1985). The equivalent cyclic stress ratios, CSR, caused by the earthquake were determined from the results of equivalent linear finite element analysis of the embankment dam following the procedures outlined for dams by Seed et al. (1975).

The triggering of liquefaction can be evaluated directly using dynamic effective stress analysis (Finn 1988, 1993). Triggering can be determined by the critical stress presented by Vaid et al. (1989) or the critical strain criterion of Castro (1969, 1997).

Revised Criteria for Triggering Liquefaction

In 1996, the National Centre for Earthquake Engineering Research (NCEER), established a committee to review the criteria for evaluating the triggering of liquefaction. This kind of review was last done by a committee appointed by the National Research Council (NRC, 1985). After reviewing developments in research and practice since 1985, the committee issued a preliminary report NCEER (1997) suggesting new criteria for engineering practice. The main differences between the proposed criteria and the previous criteria are:

- The Seed liquefaction resistance chart for M = 7.5 (Seed et al., 1985), was modified slightly for $(N_1)_{60}$ less than 10. Instead of passing through the origin, the curve would now show a cyclic resistance ratio of CRR = 0.05 at $(N_1)_{60} = 0$.
- A new chart is presented for determining K_σ, the scaling factor for the effect of effective overburden pressure on the critical resistance ratio. The reduction in liquefaction resistance now has a minimum value of 0.6 at high pressures which is significantly higher than the values previously used.
- The scaling factors for earthquake magnitudes were revised. The principal effect of this change is to reduce the liquefaction potential for earthquakes less than M = 7.5. The reduction is considerable in the cases of magnitudes below M = 6.5. This has a big impact on liquefaction potential in many parts of the United States.
- Updated liquefaction charts are also recommended based on Cone Penetration Test (CPT) data, shear wave velocity, V_s and Becker Hammer Test (BHT) data.

The many changes proposed by the NCEER Committee are not presented in detail because the report has not yet been finalized. Further discussions are planned and some changes are likely.

RESIDUAL STRENGTH

Current Practice

Originally, the residual strength was considered to be a function of the void ratio only and was evaluated using undrained triaxial compression tests on undisturbed samples from the field. Potentially liquefiable soils are very difficult to sample without disturbance. They are likely to densify during sampling, transportation, and the process of setting up the samples for testing. Therefore, tests cannot be conducted at the field void ratio. Poulos et al. (1985) developed a procedure for correcting the residual strength measured at the laboratory void ratio to the void ratio in the field. There are some practical difficulties with this approach. The steady state strength is very sensitive to relatively minor changes in the composition of the sample and, therefore, many tests are required to establish a reliable basis for selecting the residual strength for design. In addition, the corrections for disturbance can lead to order of magnitude changes in the measured residual strength. Such large corrections are a matter of concern to engineers.

Seed (1987), in an effort to overcome the perceived difficulties with direct measurement of the residual strength, back-analyzed a number of case histories in which significant displacements had occurred in saturated embankments during earthquakes. He published a chart linking the residual strength to the $(N_1)_{60c\text{-}s}$, the normalized penetration resistance for clean sands (Seed, 1987). A revised version of this chart by Seed and Harder (1990) is shown in Fig. 2. These residual

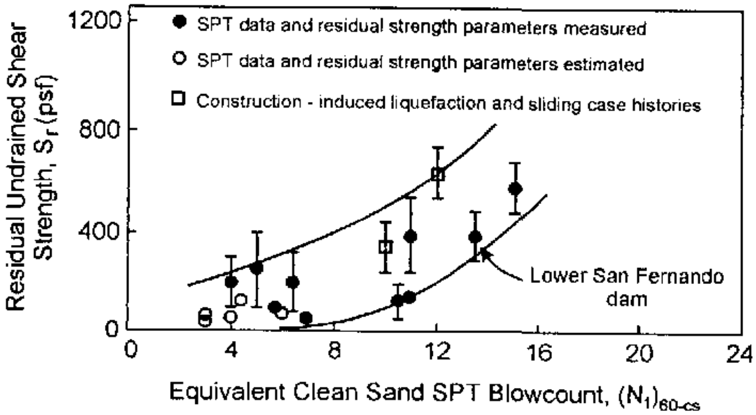


Fig. 2. Relationship between corrected "clean sand" blowcount $(N_1)_{60c-s}$ and undrained residual strength, S_r, from case studies (Seed and Harder, 1990).

strengths are generally considerably less than the average residual strengths obtained using the Poulos et al. (1985) procedure.

A major study of the failure of the San Fernando dam which occurred during the 1971 San Fernando earthquake was undertaken by both Castro and Seed in 1986-1987 with the objective of resolving the uncertainties surrounding the determination of residual strength. Seed et al. (1989) reported that the average steady state strength of all samples tested in undrained compression was 5250 psf before correction for disturbance, and 800 psf after correction, a correction factor of about 6.5. The corrected average value did not allow the dam to fail by sliding instability in a static equilibrium analysis. The average residual strength obtained from back-analysis of the failed dam in the final configuration was 400 ± 100 psf. The 35 percentile residual strength based on laboratory data would predict failure of the San Fernando dam. This suggests that, on the average, laboratory compression tests overestimate the residual strength. Castro (1997) commented that if the accelerations and velocities of the sliding mass were taken into account, as in Davis et al. (1988), the back-figured steady state strength would be approximately equal to the mean of the statically determined strengths based on the initial and final configurations of the dam (520 psf). This latter stress would allow the dam to fail in a static analysis.

The San Fernando study did not resolve the difficulties surrounding the determination of residual strength. However, use of the Seed (1987) chart for estimating reduced strength became widespread in engineering practice. Initially, engineering practice began to use the lower bound of the Seed chart to estimate residual strengths in-situ. This procedure gave very low strengths at $(N_1)_{60c-s} \leq 12$. These penetration resistances are typical of many easily liquefiable sands. The US Bureau of Reclamation later adopted a position at about the 35th percentile of the Seed values in the low $(N_1)_{60c-s}$ region. However, they were more conservative at the higher $(N_1)_{60c-s}$ values. The lower bound values in the Seed correlation remain a concern for engineers because they almost invariably lead to a need for remediation.

Two ways of dealing with this problem have emerged in practice:

- taking frozen samples to ensure undisturbed samples for the determination of residual strength in laboratory tests.
- re-examining the case histories to see whether adjustments need to be made to the data on the basis of the present state of knowledge.

The frozen sample approach proved very effective in determining the post-liquefaction behaviour of Duncan Dam (Byrne et al., 1994). Taking and testing frozen samples is an expensive process, but in the case of Duncan Dam proved very cost-effective. The measured residual strengths were considerably higher than the Seed lower bound in the potentially liquefiable areas and ensured that no remedial measures had to be carried out (Byrne et al., 1994). This resulted in savings of about \$15 to \$20 million Canadian. Unfortunately, for many soils with substantial fines content, the freezing method cannot be used.

The Duncan Dam case history shows the potentially huge financial cost of not being able to get a reliable estimate of the residual strength for a particular job. It is a powerful incentive to develop a reliable, widely applicable procedure for estimating residual strength.

The other approach to obtaining more reliable estimates of residual strength has been to re-assess the data in the Seed residual strength chart to determine the case histories that may be relevant to a job at hand and to re-evaluate the basis for the correlation between $(N_1)_{60c-s}$ and residual strength for relevant cases. Some concerns have been expressed that, in some case histories, displacements may not be sufficient to have mobilized residual strength. In others, because of lack of direct data, either assumptions or data from adjacent locations had to be used to generate the appropriate $(N_1)_{60c-s}$. There seem to be very few case histories for clean sand in the correlation. The correction for fines content has also been queried. Recently, the National Science Foundation held a workshop on residual strength at which these questions were discussed (NSF, 1997). A committee has been appointed to re-examine these case histories and to report in due course. This report should be of great interest and may result in proposed changes to the correlation. It should also provide data to facilitate independent interpretation of the case histories.

Factors Controlling Residual Strength

Extensive research has been conducted in the laboratory on the factors controlling the residual strength. Originally, it was thought to be a function only of the void ratio for a given sand (Castro, 1969; Poulos et al., 1985; Poulos, 1997). Research studies, since 1988, some of which will be described below suggest that the residual strength measured within the strain capacity of laboratory equipment is a function of:

- sample preparation technique
- stress path followed during loading
- effective confining pressure

Effect of Sample Preparation

Many laboratory studies in liquefaction use samples prepared by moist tamping because it is the easiest way to form relatively loose samples. However, it frequently results in void ratios which are not accessible to the same sand under deposition conditions in the field. Other methods in use are air pluviation and pluviation under water. Moist tamped samples are less uniform than pluviated samples. Typical results comparing moist tamped (- - -) and water pluviated samples (-----) are shown in Fig. 3. Both Vaid et al. (1998,a) and Yoshimine (1998) have demonstrated that the residual strength measured on samples prepared in different ways are quite different (Fig. 4). As part of the CANLEX program, Vaid et al. (1998,a) tested frozen samples of two different sands to determine the residual strength. He then reconstituted the same samples to the same void ratio using pluviation in water. The reconstituted samples gave residual strengths very similar to the frozen samples (Fig. 5). In this case, since the same sand sample was tested in the undisturbed and reconstituted states, there was no variation in gradation or fines content. This, of course, ensures that the only difference between the samples was the procedure for creating the sample.

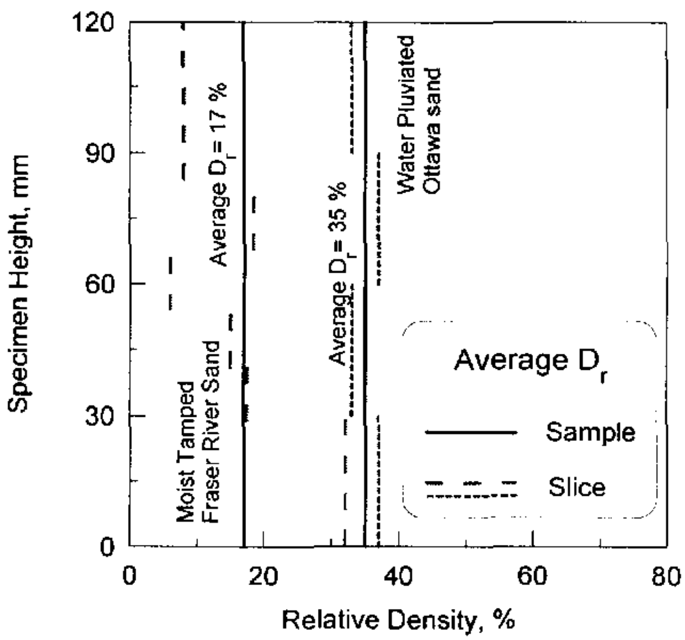


Fig. 3. Uniformity of reconstituted specimens of water pluviated Ottawa sand and moist tamped Fraser River sand (Vaid and Negussey, 1986).

This data is a strong argument for using pluviation under water to form representative samples of soils which were originally deposited under water or were placed by hydraulic fill construction. The moist tamping method would seem to be more appropriate for unsaturated compacted soils.

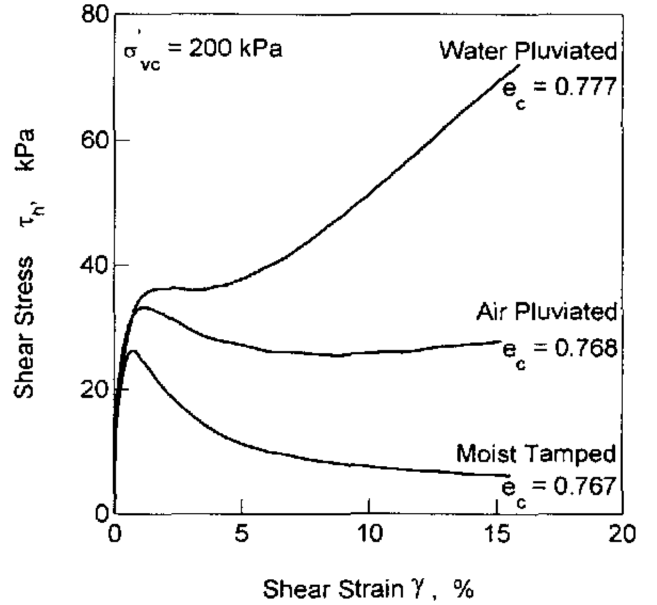


Fig. 4. The effect of sample preparation on undrained simple shear response of Syncrude sand (Vaid et al, 1995).

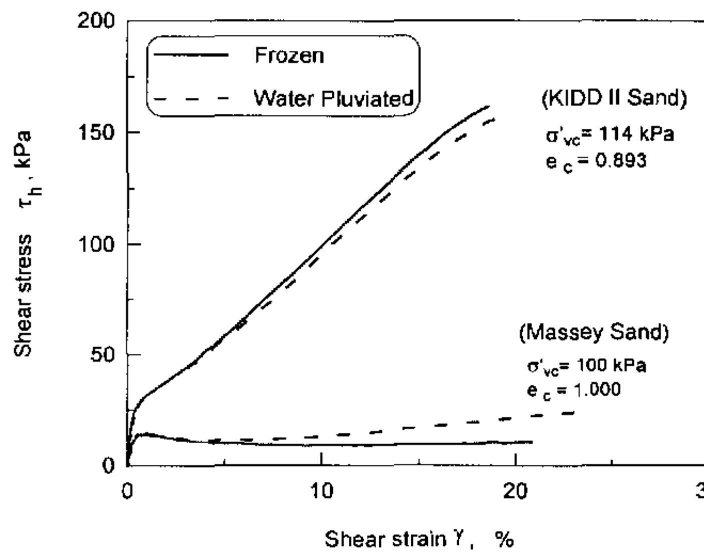


Fig. 5. Comparison of undrained simple shear response of undisturbed in-situ frozen and equivalent water pluviated sand specimens from Massey and Kidd Sites (Vaid et al, 1998,a).

Stress Path

Vaid and Chern (1985) were among the first to draw attention to the fact that the residual strength measured in extension was much smaller than the strength in compression and that sands in a given state were much more contractive in extension than in compression. These differences are often dismissed as being due to nonuniformity of the test specimens and the development of necking at large strains. In 1971, Reades (Bishop, 1971) conducted undrained extension tests on loose sands in which collapse occurred at strains at low as 0.3%, and the residual strength was reached at strains of 1.5%. Little nonuniformity would be introduced at these low strains. In the study by Vaid and Chern (1985), samples were frozen both before and after undrained extension tests. The samples were frozen by injecting gelatin following the procedures described by Emery et al. (1973). The samples were then sliced to determine the distributions of void ratio before and after testing. In these samples, collapse occurred at 5% strain and loading was continued to 9% strain. The samples showed remarkable uniformity, both before and after testing.

Tests by Vaid and Sivathalayan (1996) and Yoshimine et al. (1998), have confirmed these results for triaxial tests and have also shown that the strength is different also in simple shear. Uthayakumar and Vaid (1998) and Yoshimine et al. (1998) have explored the effects of stress path on residual strength over a wide range of stress paths defined by α, the inclination of the principle stress to the vertical axes of the sample, and the parameter $b = (\sigma_2-\sigma_3)/(\sigma_1-\sigma_3)$ which is a measure of the intermediate principle stress. The samples were tested using the hollow cylinder torsional shear device. Typical examples of this kind of data (Yoshimine et al., 1988) are shown in Fig. 6. These results suggest that different residual strengths should be assigned to different parts of the liquefied region depending on the predominant stress conditions. This selective use of shear strength for design is not new. Bearing capacity of clays under offshore structures in the North Sea is evaluated using compression, simple shear and extension strength data to suit stress conditions at different locations along potential sliding surfaces.

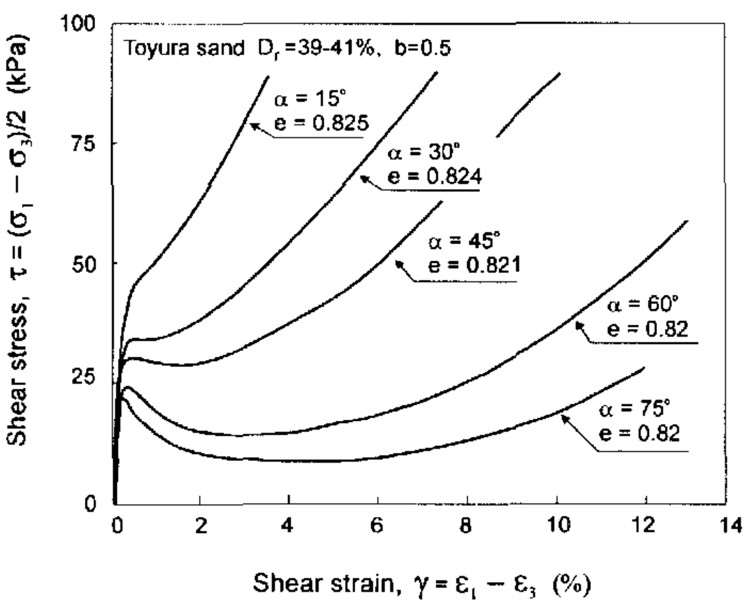


Fig. 6. Effect of stress path on undrained behaviour of Toyoura sand (Yoshimine et al., 1998).

Finn (1990) drew attention to the important practical implications of the dependence on stress path. Commenting on the difference between the average steady state strength in the liquefied zones of the San Fernando dam from back-analyses and the values measured in laboratory compression tests, he wrote,

> "The quantitative effect of stress path on steady state strength suggested by the work of Vaid et al. (1989), may account for a substantial part of the difference noticed in the San Fernando studies. This effect is also crucial to a reliable stability analysis.
>
> There are clearly sharp differences between recent research findings and current practice in the determination of steady state strength from laboratory tests. The key assumption underlying current practice that the steady state strength is a function of void ratio only needs further investigation. More studies on the effects of stress path are needed to establish a generally acceptable position on this very important problem."

Major research studies since 1990 have confirmed the importance of stress path. It is now essential for those in practice to come to an acceptable understanding of the implication of these studies.

At some loose densities, sands may not be contractive in compression. This is especially true for angular sands such as the Fraser River sand and the Brenda Mine Tailings sand which have been tested extensively at the University of British Columbia. Therefore, it appears that design decisions based on compressive undrained tests on loose sands may be potentially unconservative. Tests conducted by Vaid suggest that these sands may not become compressive until very high confining pressures are used, approaching 1000 kPa (Vaid et al., 1998,b).

Residual Strength as a Function of Effective Confining Pressure

The practice of expressing the residual strength as a fraction of the effective confining pressure has been used in practice on several water-retaining and tailings dams since first used by the US Army Corps of Engineers on Sardis Dam in 1989 (Finn et al., 1991). For the most part the ratio selected has been between 0.06 and 0.1. Similar results have been reported by Baziar and Dobry (1995) and by Ishihara (1993). A value of $S_r/p' = 0.23$ was used for Duncan Dam (Byrne et al., 1994), based on the extensive testing of frozen samples.

Vaid and Thomas (1994) using extension tests with frictional end platens, determined the residual strength of Fraser River sand over a range of void ratios and confining pressures. The results clearly demonstrated the dependence of residual strength of overburden pressure. Their results are replotted in Fig. 7; normalized with respect to the effective confining stress. Despite some scatter, the variation of S_r/p' is well represented by a straight line. The ratio varies from about 0.05 at a void ratio of about 0.96, which is the loosest void ratio obtainable by water pluviation, to a value of about 0.2 at a void ratio close to 0.8. Vaid (1998) repeated some of these tests using frictionless end platens without any significant change in the results. Vaid and Sivathalayan (1996) tested Fraser River sands at a number of different confining pressures and over a wide range of void ratios in simple shear. The test data are shown in Fig. 8a. The residual strength is clearly a function of effective confining pressure. However, the S_r/p' ratio at any particular void ratio was not a constant

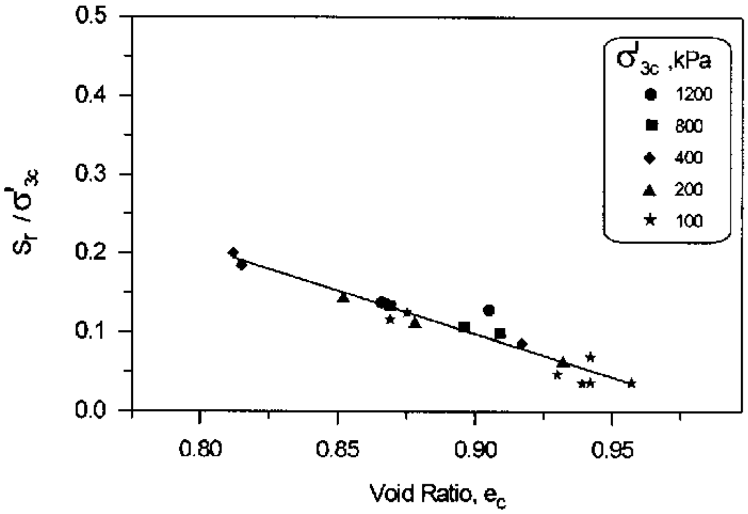


Fig. 7. Relationship between the residual strength normalized by the effective confining stress and void ratio in extension tests on Fraser River Sand in extension tests (derived from data by Vaid and Thomas, 1994)

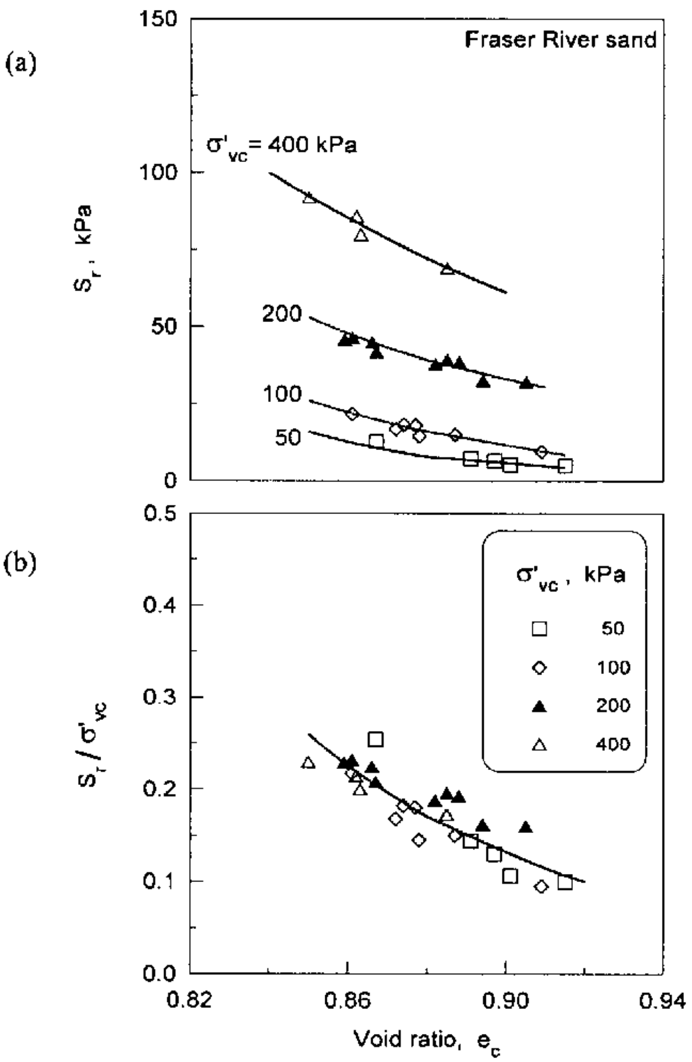


Fig. 8. (a) Variation of steady-state strength in simple shear with void ratio and confining stress, (b) Variation of normalized residual strength with void ratio (after Vaid and Sivathalayan, 1996).

over the range of pressures ranging from 50 kPa to 400 kPa. For S_r/p' to be a constant for a given soil, it is necessary for the steady-state line to be parallel to the isotropic consolidation line.

This is not generally the case, although data from Yoshimine et al. (1998) suggest that it is approximately true for Toyoura sand. The residual strength normalized with respect to the effective confining pressures is plotted against void ratio in Fig. 8b. Although there is some scatter in the data, the curve seems a reasonable interpretation for these sands of the ratio of residual strength to confining pressure in simple shear. In these tests, S_r/σ'_{vc} varies from 0.1-0.2 as the void ratio changes from 0.85-0.92.

Residual Strength for Analysis and Design

The following procedure for obtaining the residual strength in practice is tentatively proposed. It is based on the S_r/p' ratio.

1. Determine the dependence of the residual strength ratio, S_r/p', on void ratio by testing reconstituted samples using appropriate stress paths. Generally this will involve compression, simple shear and probably some extension tests.
2. The reconstituted samples should be formed by pluviation in water, if the soils in-situ were deposited under water or by hydraulic fill construction.
3. Conduct the tests over the pressure range of interest in the field.
4. Select the strength ratio at void ratios, relative densities or densities representative of field conditions.

ANALYSIS OF POST-LIQUEFACTION DISPLACEMENTS

Early Developments

In 1989, during the seismic safety evaluation of Sardis Dam by the US Army Corps of Engineers, a tolerable deformation criteria was adopted for evaluating post-liquefaction behaviour and selecting remediation measures. The potential instability in the dam was due to the liquefaction of a thin layer of clayey silt near the top of the foundation layer under the upstream slope. Conventional methods of remediation were considered to be very difficult to implement and expensive because of the nature and extent of the soil to be treated and severe restrictions on reservoir levels.

The consequences of liquefaction being triggered were determined by a large strain-large displacement analysis which tracked the stresses and displacements from the initiation of triggering to final stability. The displacement analyses were carried out using the program TARA-3FL (Finn and Yogendrakumar, 1989). The program was first developed to study the post-liquefaction ground displacements in Niigata

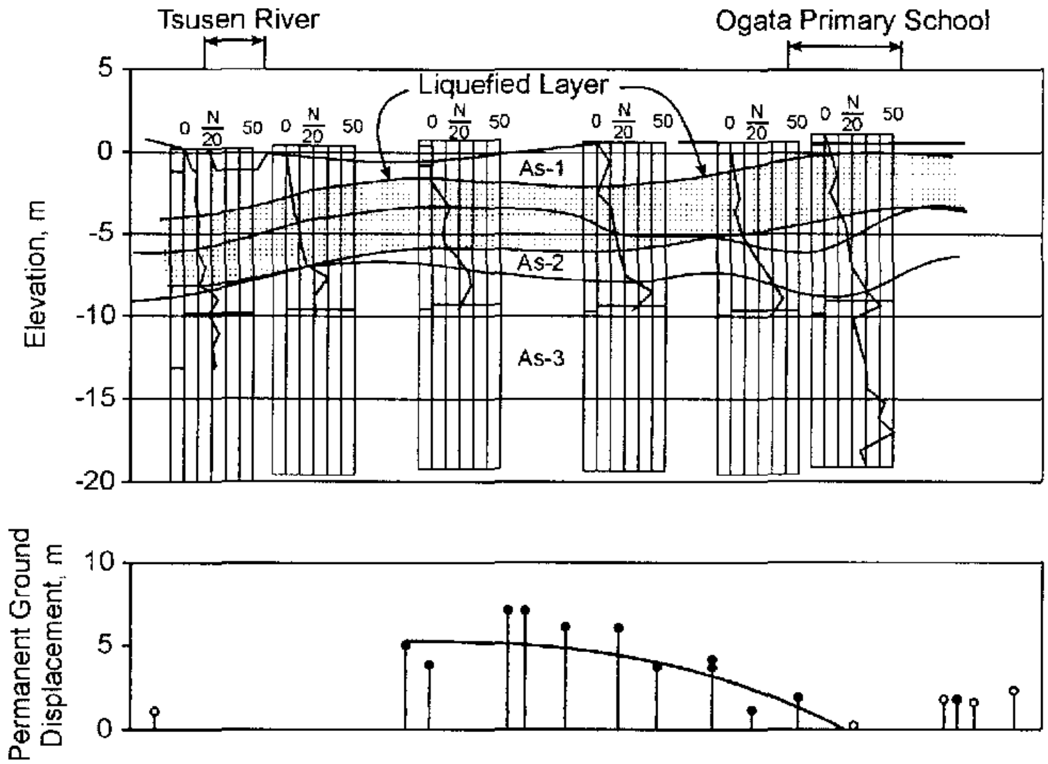


Fig. 9. Comparison of computed and measured deformations in Niigata on a line from Ogata School to the Tsusen River (Finn, 1990).

after the 1964 earthquake. The displacements had been mapped by Hamada et al. (1988) on the basis of air photographs. Studies were conducted over a distance of about half a kilometer on a line between Ogata school and the Tsusen River. The liquefiable layer and the measured and computed displacements are shown in Fig. 9. The residual strengths were selected on the basis of the 1987 Seed lower bound correlation. The predicted displacements are good except at the location of maximum displacements. The discrepancy might be partly due to the fact that in the analyses an average slope was assigned to the liquefied layer rather than the actual slightly undulating form.

The pattern of displacements in Sardis Dam is shown in Fig. 10 from one of the parametric studies to determine the dependence of deformation on residual strength. Crest settlement (loss of freeboard) was adopted as the criterion of performance. The crest settlements associated with different factors of safety are shown in Fig. 11. The freeboard at Sardis was about 30 ft. The predicted crest settlements at a factor of safety of 0.95 was about 6 ft. The deformation steadily increased as the factor of safety dropped, but because of the flat slopes of the dam the incidence of catastrophic movement did not become apparent until factors of safety of about 0.8 were reached.

These preliminary studies on the dam were reported by Finn et al. (1989), Finn (1990) and Finn et al. (1991). The final remediation involved nailing the upstream slope to the foundation using prestressed reinforced concrete piles and was completed in 1996. The final design is described in Stacy et al. (1994). The cross-section of the remediated upstream

Fig. 10. Post-liquefaction deformations in Sardis Dam (Finn, 1990).

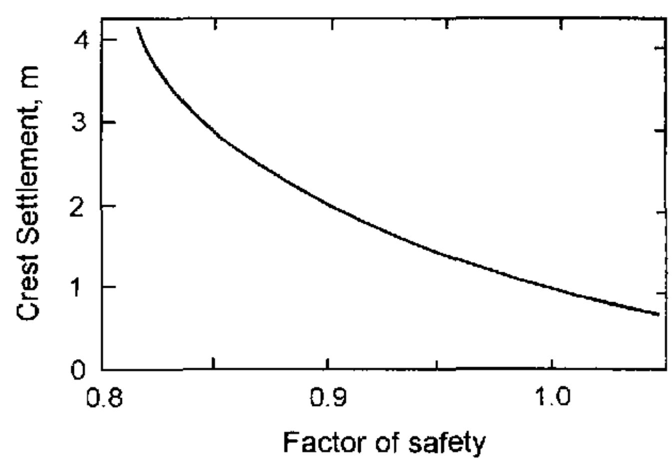
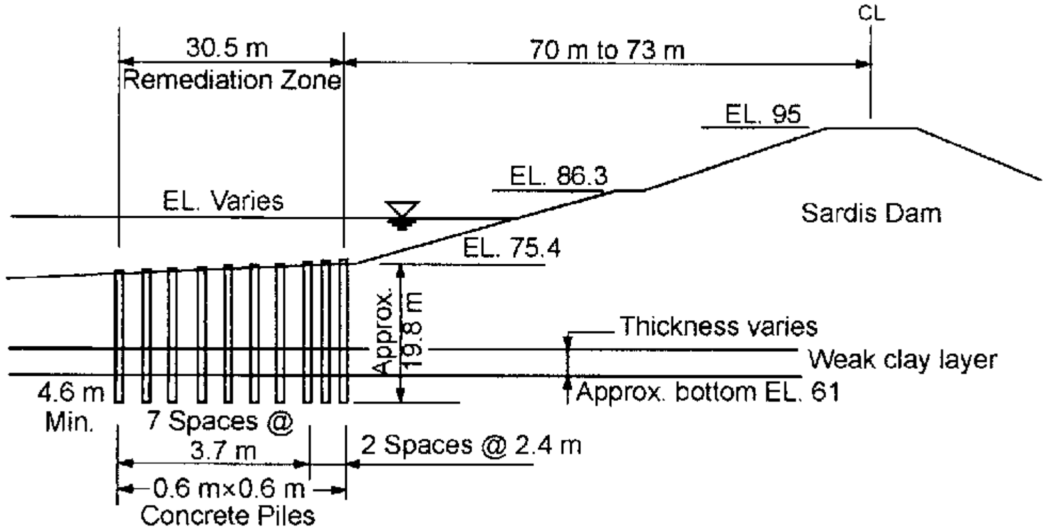
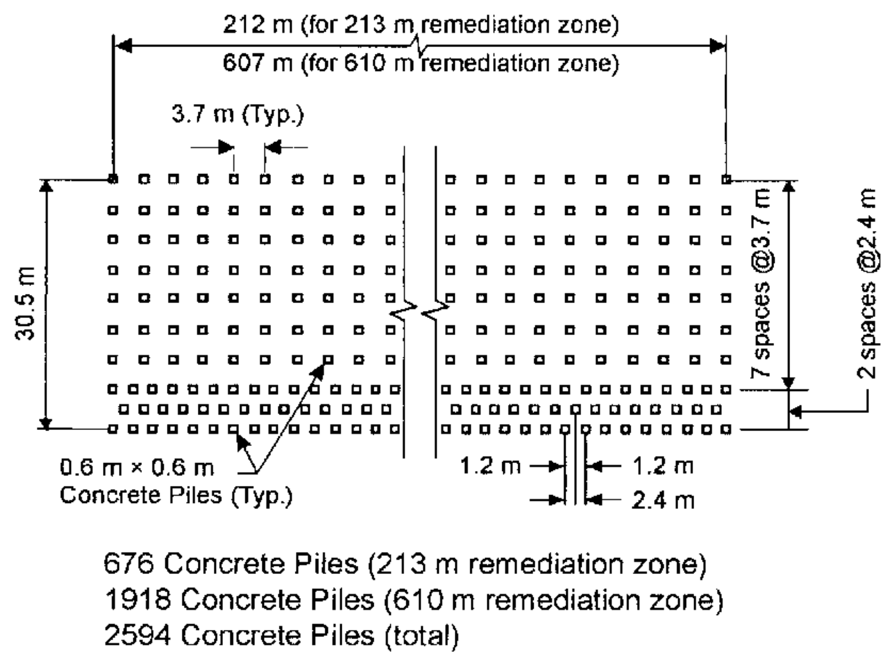


Fig. 11. Crest settlements as a function of factors of safety (after Finn, 1990).

slope showing the pile layout is given in Fig. 12. The displacement analysis of the remediation section is described by Finn et al. (1997). The general procedures adopted for Sardis Dam in 1989 are very much a part of practice today.



(a)



(b)

Fig. 12. Final form of pile reinforced section of Sardis Dam: (a) elevation, (b) plan (after Stacy et al., 1994).

ANALYSIS OF CASE HISTORY

The 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 in height, and were constructed of compacted sand fill resting on a comparatively thick peat layer. The dikes were damaged at 18 locations for a total length of about 10 km along the Kushiro river. The severest damage occurred in Kushiro Marsh (Sasaki et al., 1993; Sasaki, 1994,a,b). Dike sections which failed were reconstructed, after the foundation soils had been improved by the installation of sand compaction piles.

This paper describes a simulation of the failure of a dike during the 1993 Kushiro-oki earthquake, and the response of the reconstructed dike to the 1994 Hokkaido Toho-oki (Toho-oki) earthquake. These simulations were conducted as part of a more extensive study of dike behaviour commissioned by the Hokkaido Development Bureau through the Advanced Construction Technology Center (ACTEC) in Tokyo.

Outline of Damage to Section 9K850

The failure mode at a location on the left bank of the Kushiro river is shown in Fig. 13. 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

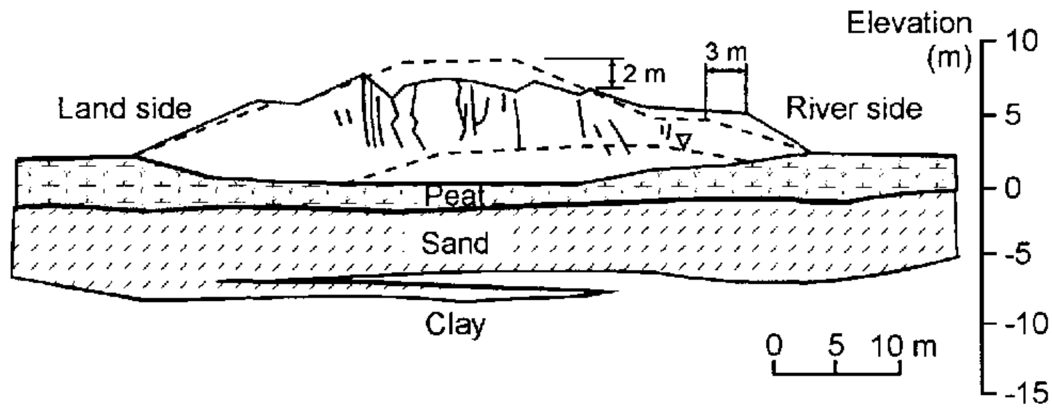


Fig. 13. Mode of dike failure at a location along left bank of Kushiro River (Sasaki et al., 1995).

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.

Sasaki (1994,a,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. 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. 13. 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. These N values indicate a low resistance to liquefaction.

Sasaki (1994,a,b) established a relationship between the height of the stress relaxation zone and the settlement of a dike during earthquake shaking.

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.

Soil Properties

Soil properties for both static and dynamic analyses were provided by ACTEC (1995).

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) shows that the residual strength, S_{ur}, can vary between $0.05\,\sigma'_{vo}$ and $0.15\,\sigma'_{vo}$, 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 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.

$$\frac{S_{ur}}{\sigma'_{vo}} = 0.06 + 0.025\left[(N_1)_{60} - 6\right] \quad 6 \le (N_1)_{60} \le 30 \qquad (1)$$

The residual strength $S_{ur} = 0.1$ used in this analysis would correspond to an $(N_1)_{60} = 7.6$ according to Eqn. 1. This compares favorably with the $(N_1)_{60} = 7$ used in the analysis.

Seismic Analysis of Dike

Appropriate input motions for seismic analysis, shown in Fig. 14, were specified by Jishin Kogaku Kenkyusho (JKK, 1995). 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

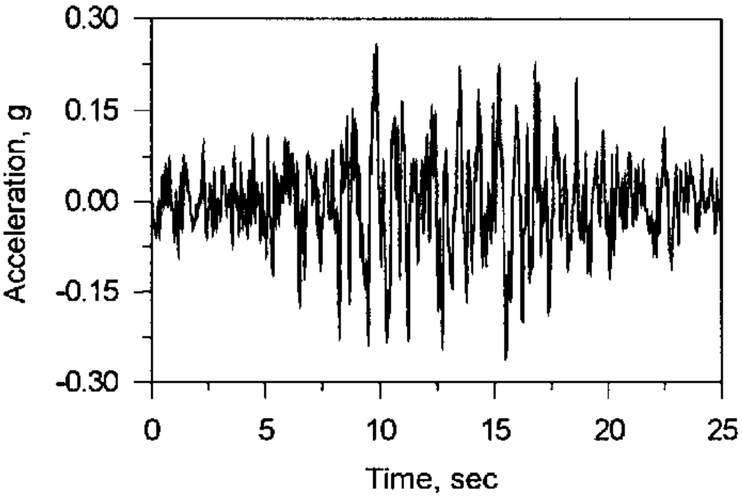


Fig. 14. Input motions for Kushiro-oki earthquake (JKK, 1995).

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. 15. 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.

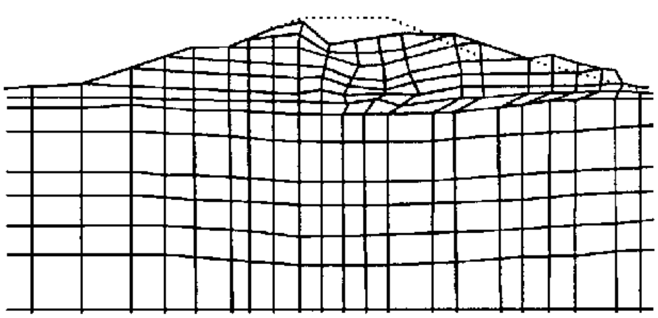


Fig. 15. Computed post-liquefaction shape of the dike.

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. 16. The peak value is 10 kPa corresponding to a porewater pressure ratio, u/σ'_{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 made is not known, nor is it known how much dissipation had occurred before the measurement was made.

As anticipated by Sasaki (1994,a,b), the analysis showed that all the saturated sand fill liquefied during the earthquake. The development of the porewater pressure shown in Fig. 17 is typical of locations at the base of the dike. Liquefaction was reached after 10 s.

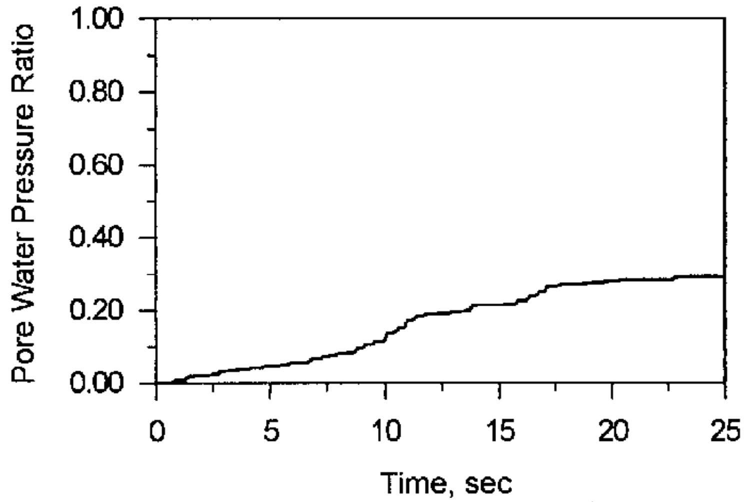


Fig. 16. Computed porewater pressure in the upstream alluvial sand.

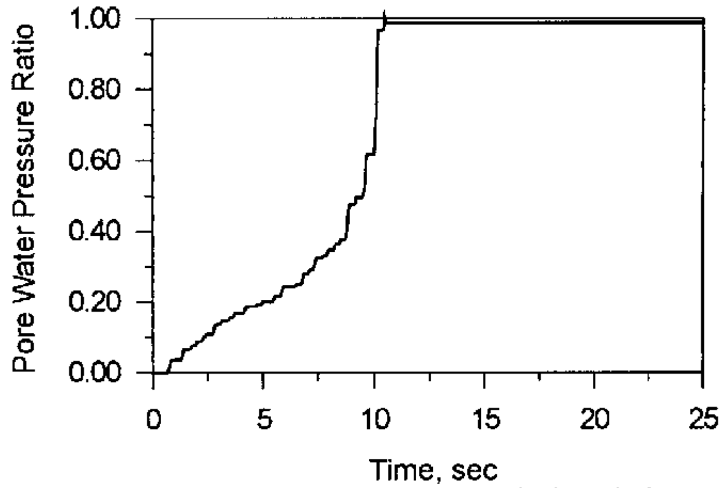


Fig. 17. Computed porewater pressure in the liquefied zone at the base of the dike.

<u>Response of Reconstructed Dike to 1994 Toho-Oki Earthquake</u>

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. 18. 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 (JKK, 1995), from recorded 1994 Toho-oki earthquake motions, are shown in Fig. 19.

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.

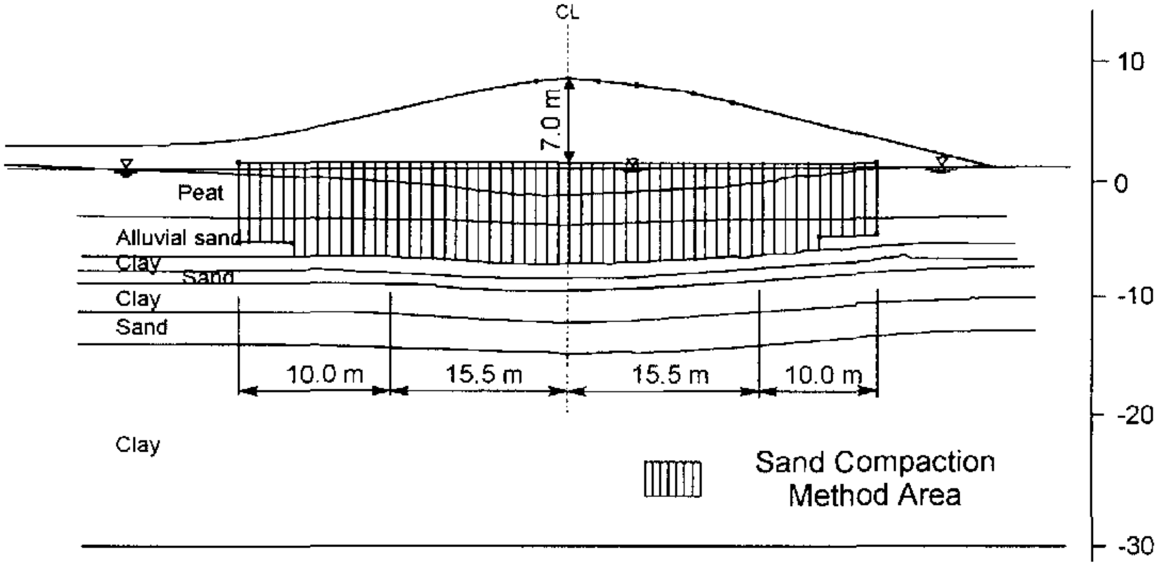


Fig 18. Cross-section of remediated Kushiro dike at location 9K850 (ACTEC, 1995).

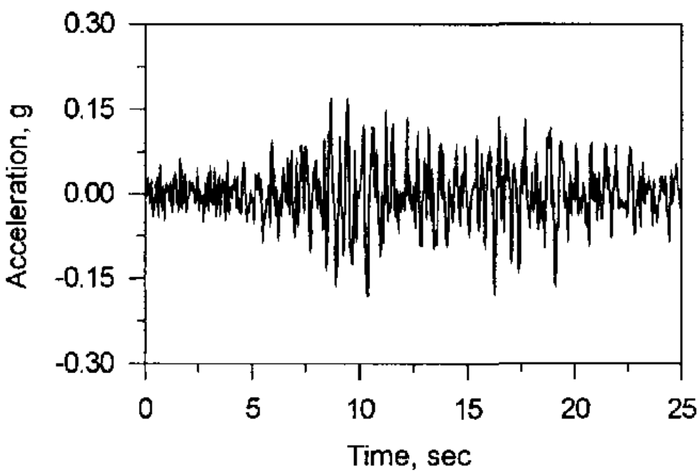


Fig. 19. Input motions for Toho-oki earthquake (JKK, 1995).

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 (1995) 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. 20). 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.

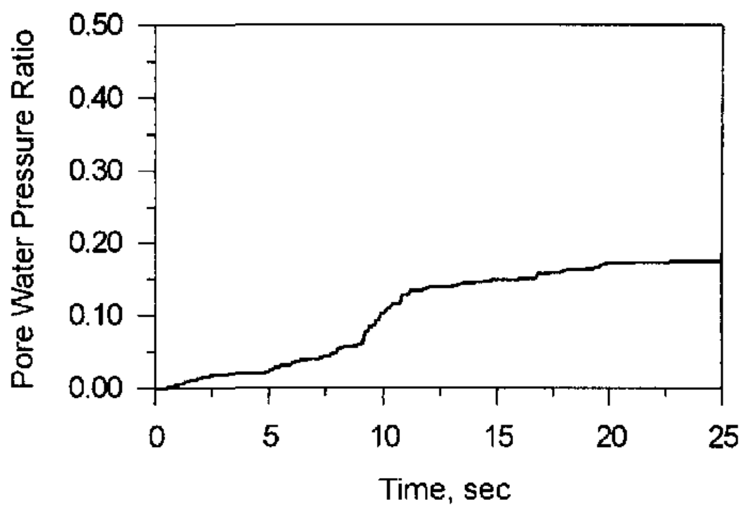


Fig. 20. Computed porewater pressure in the saturated zone at the base of the dike under the Toho-oki earthquake.

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 no consequence. Since the post-liquefaction deformation are primarily generated by gravity forces, the computed deformations were 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.

Conclusions from Simulation Study

The seismic analysis of the Kushiro dike simulated the important features of the failure during the 1993 Kushiro-oki earthquake. The computed settlement of the crest of the order of 2 m, agrees with the measured settlement. The movement of the dike towards the river was estimated to reach up to 2.7 m which is of the same order as the observed movements.

The displacement analysis provided a comprehensive picture of how the different parts of the dike cross-section responded during the earthquake including time histories of porewater pressures. The results confirm the conclusions of Sasaki (1994,a,b) that the failure was caused by liquefaction of the saturated sand fill.

A very interesting feature of the dike failure is the fact that the conditions for the failure were created by consolidation of the peat which lowered the base of the embankment below the water table. This is a possibility which should be kept in mind in similar situations.

The reconstructed dike on an improved foundation survived the 1994 Toho-oki earthquake with no sign of damage or high porewater pressures. Analysis of the dike showed that the porewater pressures in the saturated base of the sandfill dike were only about 20% of the effective overburden pressure. The Toho-oki ground motions were about 30% less than the Kushiro-oki motions. However, when applied to the original dike they caused liquefaction in the saturated sandfill. These results confirmed the crucial importance of improving the saturated sandfill to prevent future liquefaction and large deformations.

Studies of this type were used to determine whether analyses by TARA-3 and TARA-3FL could simulate reasonably well the response of the flood protection dikes. Simulations such as that described above were considered adequate for engineering purposes. Therefore the analyses were approved for establishing priorities for dike improvement by identifying those most at risk. This work is described in the next section.

DEVELOPMENT OF SCREENING CRITERIA FOR REMEDIATION

Parametric studies were conducted to establish the displacement patterns for dikes with side slopes 1:2.5 as shown in Fig. 21. The thicknesses of the liquefiable and nonliquefiable layers were varied in the studies.

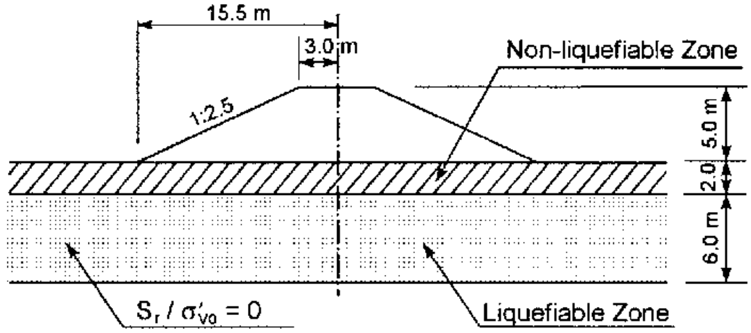


Fig. 21. Typical cross-section of dike for analysis.

The computed crest settlements for the dikes are represented in terms of the nondimensional variables S and β in Fig. 22. The equation of the curve in Fig. 22 is given by,

$$S = 0.01 \exp\left(0.922 \frac{H_D}{H_{NL}} \frac{H_L}{H_{NL}}\right) \tag{2}$$

where S is the crest settlement divided by H_D, the height of the dike; H_L and H_{NL} are the thicknesses of the liquefiable layer and the overlying nonliquefiable layer, if any, respectively.

In general form, the nondimensional settlement is expressed as $S = f(\beta)$, where,

$$\beta = \frac{H_D}{H_{NL}} \frac{H_L}{H_{NL}} \tag{3}$$

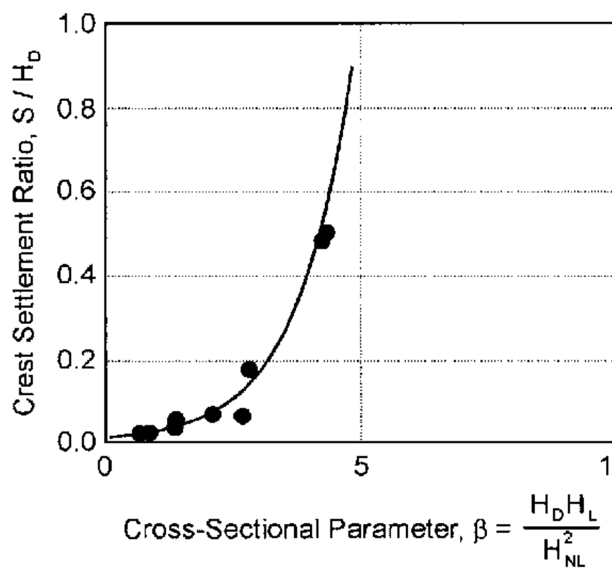


Fig. 22. Comparison of nondimensional computed settlements with nondimensional settlement prediction curve given by the equation, $S/H_S = 0.051 \exp[0.922 H_D * H_L / H_{NL}^2]$.

The curve in Fig. 22 can be used to prioritize the remediation of dikes with side slopes 1:2.5. The first step is to determine the β parameter. The thickness of potentially liquefiable and nonliquefiable layers can be determined quickly by a limited number of penetration tests, either cone or CPT, conducted along the line of the dike, either along the upstream or downstream toe. Then the dikes with the highest potential deformation would be remediated first, other things being equal. However, priorities based on displacements may be set aside on the basis of potential consequences of dike failure.

Before using this procedure, ACTEC wished to validate it by comparing the displacements predicted by the curve, $S = f(\beta)$, with the measured crest displacements during the 1994 Nansei-oki earthquake. This field data is plotted in Fig. 23, together with the curve $S = f(\beta)$. The Nansei-oki data is for a wide variety of dikes with different side slopes. For side slopes of 1:2.5, the data corresponds fairly well with the curve $S = f(\beta)$. However, it was clear that similar curves should be developed for the other predominant slopes in the diking systems. Therefore, criteria were also developed for dikes with side slopes of 1:5 and with unequal side slopes of 1:10 and 1:5.

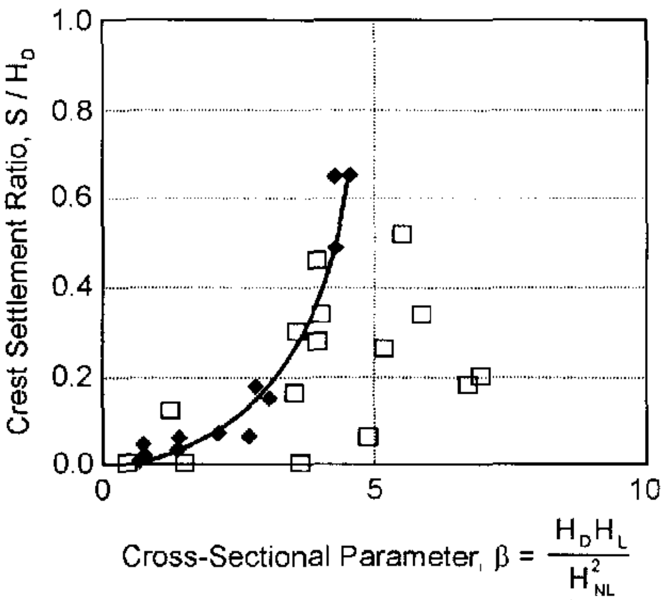


Fig. 23. Comparison of computed dike settlements with settlements during the 1994 Nansei-oki earthquake.

CONCLUSIONS

One of the major challenges facing geotechnical earthquake engineers is the seismic safety evaluation of embankment dams when potentially liquefiable soils are in the dam itself or in the foundation. The evaluation procedure has to provide answers to three important questions:

- Will liquefaction be triggered?
- What will be the consequences?
- How can the consequences be mitigated?

The state of the art for evaluating the triggering of liquefaction in potentially liquefiable soils is defined by the preliminary report by NCEER (1997). This report is not yet in final form, but the official document should be available in 1998.

The consequences of liquefaction depend primarily on the residual strength of liquefied soils, but the strain required to reach that strength may be an important factor in some cases. How to determine residual strength and what factors control it are still controversial matters.

The most feasible and reliable general procedure seems to be to base the residual strength, S_r, on the effective overburden pressure, p', specifying the ratio S_r/p'. This ratio can be determined over a range of void ratios by tests with stress paths (triaxial compression, simple shear and triaxial extension) appropriate for the failure stress conditions in different locations in potentially liquefiable material. The S_r/p' ratio associated with field void ratio is the selected for design. The samples should be pluviated in water for soils deposited under water or placed by hydraulic fill construction. Pluviation in water is shown to reproduce the fabric of field deposition.

The consequences of liquefaction are best evaluated using large-strain large-displacement analysis. These kinds of analysis have been used to evaluate the extent and optimum location of remediation measures in about 12 dams since 1989, and have also been used to establish criteria for prioritizing remediation measures for long linear structures such as flood protection dikes. However, the experience in conducting this kind of analysis is still concentrated in a few hands. Further development and refinement of the methods are under way, and more widespread distribution of user-friendly computer packages should make the method more accessible.

Cost effective remediation of embankments with potentially liquefiable soils should be based on deformation criteria. Experience with a large number of dams in recent years has clearly demonstrated that remediation costs are significantly reduced by adopting a displacement criterion rather than a factor of safety criterion.

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