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Performance based design in geotechnical earthquake engineering

W.D. Liam Finn

Department of Civil Engineering – University of British Columbia, 6250 Applied Science Lane, Vancouver, BC, Canada V6R 1Z4

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ABSTRACT

The key elements of performance-based design will be illustrated and discussed in the context of designing cost effective remedial measures for embankment dams with liquefiable materials in the foundation. This situation is considered one of the more challenging areas of performance based design. Some of the key elements that will be considered will be the selection of performance criteria, selection of an appropriately validated analysis program and calibrating the constitutive model to represent material properties in the field. Major elements of performance based seismic design will be explored using typical case histories from practice such as Sardis Dam in Mississippi, Mormon Island Auxiliary Dam in California, and Flood Protection Dikes in Hokkaido, Japan. A primary source of concern about performance based design based on the results of finite element or finite difference methods of analysis is the reliability of the analyses. Reliability is enhanced by due diligence in the selection of a well-validated program and an appropriately calibrated constituted constitutive model. These issues are discussed in the paper, but there remains a residual concern because there is no field response data on large dams by which our real capability can be assessed.

1. Introduction

Performance based design (PBD) is based on tolerable displacement criteria and has become part of practice in geotechnical earthquake engineering. It has been widely used for developing remediation strategies for embankment dams with foundations susceptible to liquefaction under design seismic loadings. There are two crucial requirements for implementing PBD: acceptable performance criteria and a reliable method of analysis. For embankment dams, the criterion of acceptable performance is usually specified by tolerable displacements of the crest, although additional criteria may also be imposed as will be shown later in the case of Mormon Island auxiliary Dam. A nonlinear analysis is essential for checking performance because soil behaves as a nonlinear solid under strong shaking. If significant seismic pore water pressures are developed during shaking, the analysis must be based on effective stresses. Nonlinear dynamic effective stress analysis has many forms and its safe use requires a good technical understanding of the mechanics of the constitutive model selected for use and knowledge of its validation history based on element test data, centrifuge test data and any evidence from case histories. It also requires an adequate understanding of how the computation procedure works. The elements of performance based design are explored by the following examples from practice; Sardis Dam in Mississippi, flood protection dikes in Hokkaido Japan and the Mormon Island Auxiliary Dam, Folsom, California and screening the seismic stability of slopes for residential development.

2. Sardis dam

Sardis Dam is a hydraulic fill structure located in northwestern Mississippi within the zone of influence of the New Madrid seismic zone. A cross-section of the dam is shown in Fig. 1.

The U.S. Army Corps of Engineers (USACE), Vicksburg District, undertook several studies to evaluate the probable behavior of the dam during and after an earthquake. The earthquake hazard was defined by the seismic design parameters:

- Peak ground acceleration – 0.20 *g*.
- Maximum velocity – 35–45 cm/s.
- Duration – 15 s.
- Two records of the 1952 Kern County Earthquake in California were modified to provide suitable input motions for seismic response analyses. The magnitude and epicentral distances were somewhat similar to those of the selected design earthquake for Sardis.

In situ investigations revealed zones with the potential for liquefaction or significant strength loss that could threaten the upstream stability of the dam. The hydraulically-placed silt core could liquefy, and a discontinuous layer of weak silty clay or clayey silt, ranging in thickness from 1.5 m to 4.5 m, located in the top stratum clay beneath the upstream slope could experience a significant strength loss. The location of this layer is indicated by the thin dark stripe in Fig. 2 and, to

E-mail address: finn@civil.ubc.ca.

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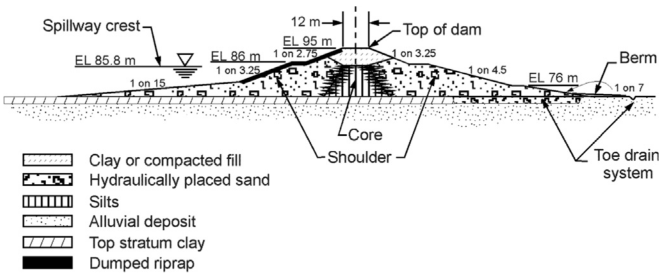

Fig. 1. Cross-section of Sardis dam.

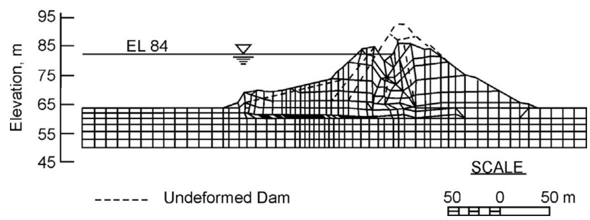

Fig. 2. Post-liquefaction deformed shape of Sardis dam. Note the different vertical and horizontal scales.

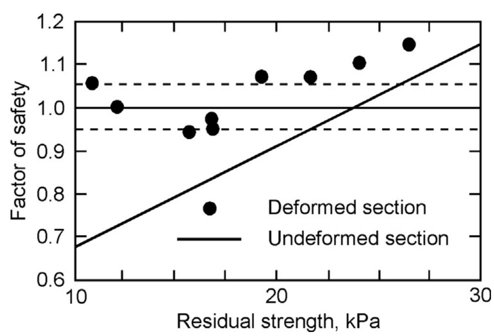

Fig. 3. Factors of safety of Sardis dam as a function of residual strength in weak foundation layer.

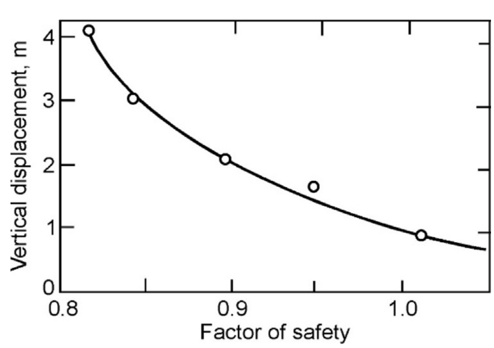

Fig. 4. Variation of loss of freeboard with factor of safety of undeformed dam.

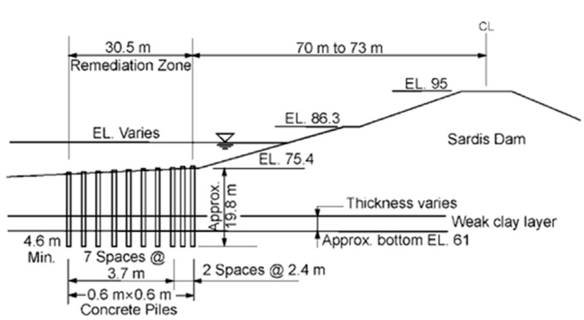

Fig. 5. Elevation of pile remediation of Sardis dam [1].

a larger scale, in Fig. 5. Stability analyses showed that, although the silt core might liquefy along the entire length of dam, the factor of safety with respect to upstream stability of the dam would still be adequate except in areas where the weak layer of silty clay occurred beneath the upstream slope within 75 m of the centerline. The results of these investigations indicated that remedial measures were necessary to improve the stability of the upstream slope of Sardis Dam during seismic loading.

Dynamic effective stress analysis of Sardis Dam was conducted using the program TARA-3 [1-3] and the potential post-liquefaction deformations before and after remediation were estimated using the large strain based version of TARA-3 called TARA-3FL [4]. The large differences between the initial and post-liquefaction strengths in Sardis Dam resulted in major load shedding from liquefied and softened elements. This put heavy demands on the ability of the program to track accurately what was happening and on the stability of the algorithms. Therefore, it was imperative to have an independent check that the computed final deformed positions were indeed equilibrium positions.

To check the performance of TARA-3FL, the post-earthquake deformed shape of Sardis Dam was computed using design specified residual strengths in the weak layer. The initial and final deformed shapes of the dam for this case are shown in Fig. 2. Very substantial vertical and horizontal deformations may be noted, together with intense shear straining in the weak thin layer. The static stability of the deformed shape was analyzed using the program UTEXAS2 [5] which satisfies both moment and force equilibrium. The factor of safety was found to be close to 1.0. The critical slip surface exited the slope near the location suggested by the finite element analysis.

To check the reliability of TARA-3FL more widely, a series of analyses were conducted of Sardis Dam assuming different levels of residual strength in the weak foundation layer in each analysis. The conventional factor of safety of the undeformed dam section varies over the range 1.15–0.68 as the constant residual strength varies from 30 kPa to 10 kPa (Fig. 3). The deformed sections corresponding to the various residual strengths in this range were calculated using TARA-3FL. The factors of safety of these deformed sections were determined using UTEXAS2. In the region defined by a factor of safety less than one for the undeformed section, the computed factors of safety for the deformed sections were in the range of 1 $\pm$ 0.05. This is the theoretical error band associated with UTEXAS2.

Analyses of this type also give the loss in freeboard associated with various conventional factors of safety based on the original configuration of the dam. For Sardis Dam, the variation of vertical crest displacement with such factors of safety, corresponding to various values of residual strength, are shown in Fig. 4. For the first time, a designer could see the consequences of selecting a particular factor of safety for a dam. This information was helpful to engineers in making the difficult transition from a factor of safety based design to displacement based design.

2.1. Remediation studies

The selection of remedial measures for Sardis Dam focused on ways of strengthening the weak foundation layer. The general idea was to develop a plug of much stronger material across the weak layer which would restrain post-liquefaction deformations. The important properties of this remediation plug were its length, strength and location. Two kinds of analyses were conducted to define the properties of the remediation plug; conventional stability analyses and deformation analyses using the program TARA-3FL.

Various methods of creating the plug were investigated. Because of limitations on lowering the reservoir level, any remediation would have to be done under water. In these circumstances, nailing the upstream slope to a stable foundation layer by driven piles was found to be the most cost-effective and reliable method of remediation. The layout of the piles is shown in elevation and in plan in Figs. 5 and 6 respectively.

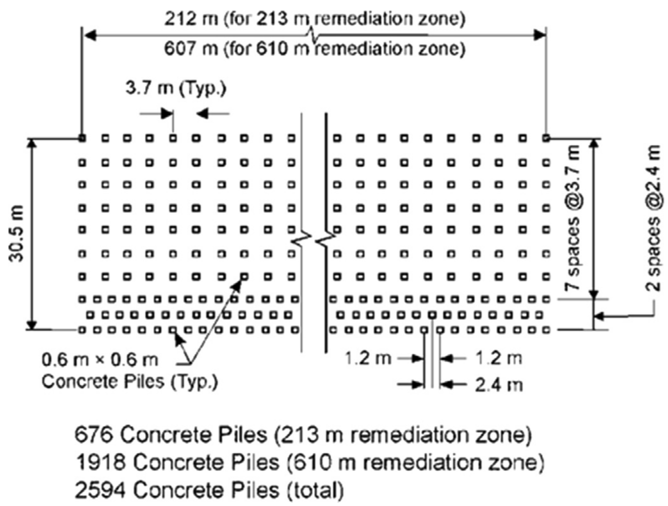

Fig. 6. Plan view of pile remediation of Sardis dam [1].

2.2. Load transfer in pile remediated section

The displacement analysis of the remediated section giving the moments, shears and displacements of the piles are described in [1].

The distributions of post-liquefaction shears and moments controlled the placement of the piles. Maximum design moment and shear occurred in the leading row of upstream piles. The design moment was the sum of 67% of the peak dynamic moment and the moments due to the static thrust of the embankment along the weak plane.

Since the piles were not capped, the only load transfer mechanism between the pile rows was the pressures exerted by the soil between them in response to the deflection of the pile rows. The designers were concerned by the possibility of progressive failure if the pile rows were too far apart. Therefore the spacing of the piles in the first 3 rows was kept small. The spacing was only about one third of the spacing between the other pile rows. The use of displacement criteria for evaluating the post-liquefaction stability of Sardis dam and determining the design and placement of the remedial measures resulted in huge savings over estimated costs for other procedures. This type of displacement analysis has become very much a part of practice today. The process of probing the performance and reliability of the then new method of analysis for Sardis dam is a process that should be followed whenever a new constitutive model is used either in a stand-alone program or inserted in a platform such as FLAC [6].

3. Development of screening criteria for seismic stability of slopes

Parametric studies of the response of a particular type of soil structure such as a flood protection dike using nonlinear dynamic analysis can provide a database for the potential development of simple screening methods for preliminary assessment of the behavior of a similar type of structure under strong shaking and criteria for prioritizing remediation. Two examples are given: (1) preliminary screening of the seismic performance of slopes for residential development that has been approved recently by the Professional Engineers and Geologists of British Columbia (APEGBC) as good engineering practice and has been incorporated in the British Columbia Building Code [7] and (2) a criterion for prioritizing remediation intervention for flood protection dikes in Hokkaido, Japan.

3.1. Screening seismic performance of slopes for residential development

The British Columbia Building Code [7] adopted the ground motions for seismic design proposed in the 2005 National Building Code for Canada [8]. These ground motions have a probability of exceedance of 2% in 50 years (annual rate of exceedance of 1/2475), whereas the previously adopted ground motions for seismic design (NBCC [8], BCBC [7]) had a probability of exceedance of 10% in 50 years (annual rate of exceedance of 1/475). The effect of this change was to double the peak ground acceleration and, as a consequence, double the seismic demand on slopes when their seismic safety is assessed using the seismic coefficient method. This resulted in a sharp increase in the number of slopes considered unstable during an earthquake, and therefore potentially not suitable for residential development. The political fallout from municipalities and developers caused the Provincial Government to ask the Association of Professional Engineers and Geophysicists of British Columbia (APEGBC) to establish a Task Force on Seismic Slope Stability (TFSSS) to study this issue and to make recommendations for dealing with it. During its deliberations, the TFSSS reviewed current engineering practice and recent developments in seismic analysis of slopes and recommended two new methods of analysis based on the concept of acceptable earthquake-induced slope displacements. These methods are easy to use (an essential requirement) and achieve the BCBC 2006 code objective of life safety (Fig. 7).

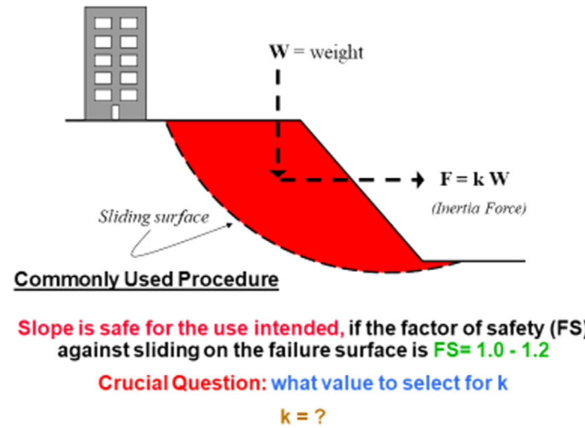

Fig. 7. Mechanism for pseudo-static slope analysis.

In BC, the most common method of seismic safety assessment of slopes slated for residential development was to carry out seismic slope stability analysis using the pseudo-static limit equilibrium method. In this method, earthquake loading is represented by a constant horizontal force, F = kW, applied to the center of gravity of the potential sliding mass, where W is the weight of the sliding mass and k is a seismic coefficient. There was, however, no generally accepted method in BC practice for selecting seismic coefficients for slopes. From a limited survey of BC practice, the TFSSS found seismic coefficients in the range $0.5(PGA) \le k \le 1.0(PGA)$, where PGA is the peak ground acceleration. The TFSSS reviewed recent developments in methods for assessing the potential performance of slopes during earthquakes and selected a new approach based on the work of Bray and Travasarou [9].

Bray and Travasarou [9] conducted approximately 55,000 Newmark-type slope displacement analyses involving eight different soil slope configurations, ten different yield accelerations for each slope configuration, and 688 different recorded ground motions from a database compiled by the Pacific Earthquake Engineering Research Center in Berkeley, California. From a regression analysis of the resulting slope displacements, they developed an equation to estimate the magnitude of slope displacement due to shearing of the soil along a slip surface.

Bray and Travasarou's equation for estimating slope displacements, D, greater than 1 cm, is expressed as:

$$\begin{aligned}\ln(D) = &-1.10 - 2.83\ln(k_y) - 0.333(\ln(k_y))^2 + 0.566\ln(k_y)\ln S(T)\\ &+ 3.04\ln S(T) - 0.244(\ln(S(T)))^2 + 1.5T_s + 0.278(M-7)\end{aligned} \tag{1}$$

where k_y is the yield acceleration, T_s is the period of the potential sliding mass before the earthquake, S(T) is the spectral period of the

Table 1
Calculated slope displacements for 4 problem slopes.

	SAMPLE ANALYSES OF PROBLEM SLOPES								
					NBCC Sa (s)				DISPL
LOCATION	HEIGHT	M	Ts	PGA	0.2	0.5	1	Ky	(cm)
DUNCAN	22	7	0.35	0.54	1.1	0.74	0.37	0.49	1
VICTORIA	13	7	0.26	0.61	1.2	0.82	0.38	0.52	2
NANAIMO	30	7	0.4	0.5	1	0.69	0.35	0.17	10
VICTORIA	15	7	0.15	0.61	1.2	0.82	0.38	0.43	5

Note: The applicable values of S were obtained by interpolation between the values of $S_a(1.5T_s)$ listed in NBCC 2005 [8] as shown above.

potential sliding mass at the degraded period of the sliding mass as a result of shaking and is taken as $T = 1.5T_s$ and M is the moment magnitude of the design earthquake.

This equation is valid for periods, T_s, in the range $0.05\,s < T_s < 2.0\,s$, for values of yield coefficient, k_y, in the range $0.01 < k_y < 0.5$ and for $4.5 < M < 9.0$. TFSSS recommended a limiting displacement of 15 cm be used in conjunction with Eq. (1). This displacement was based on experience with wood frame construction.

As examples of the use of Eq. (1), slope displacements were estimated for four soil slopes considered for residential development in Nanaimo, Duncan, and Victoria, BC. All four slopes had been considered unsafe based on the k value used in the original pseudo-static analyses. Table 1 shows that the estimated median slope displacements, D, were relatively small (1–10 cm). Using the tolerable slope displacement criterion of 15 cm, these slopes were considered suitable for residential development.

The Bray and Travasarou method has three features that made it attractive to practitioner's in British Columbia as a screening tool for use with an acceptable displacement based performance criterion: it rests on an extensive data base, it is simple to use and it precludes the necessity of conducting non-linear displacement analysis except in special cases such as when liquefaction may be a problem.

To make the analysis even simpler, TFSSS asked Bray to develop a seismic coefficient based on an allowable displacement of 15 cm, k_{15}. He developed the equation

$$k_{15} = (0.006 + 0.038M_w) \times S_a(0.5) \tag{2}$$

This equation gives a correct seismic coefficient for slope periods $T_s = 0.33\,s$ and is increasingly conservative for longer periods. The mechanism for pseudo-static analysis is shown in Fig. 8. The slope is considered “safe for the use intended” if the factor of safety, FS, against exceeding 15 cm of displacement is in the range FS = 1.0–1.2.

The TFSS approach is adaptable to reliability analysis and has been applied in this way to slopes with known displacements during an earthquake. The Lexington Dam in California had a slope displacement

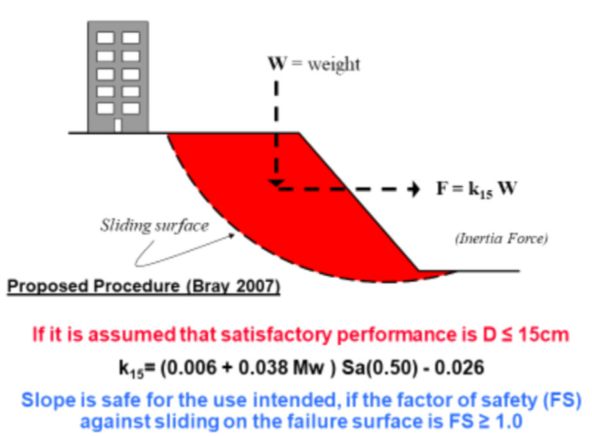

Fig. 8. Mechanism for pseudo-static analysis using k_{15}.

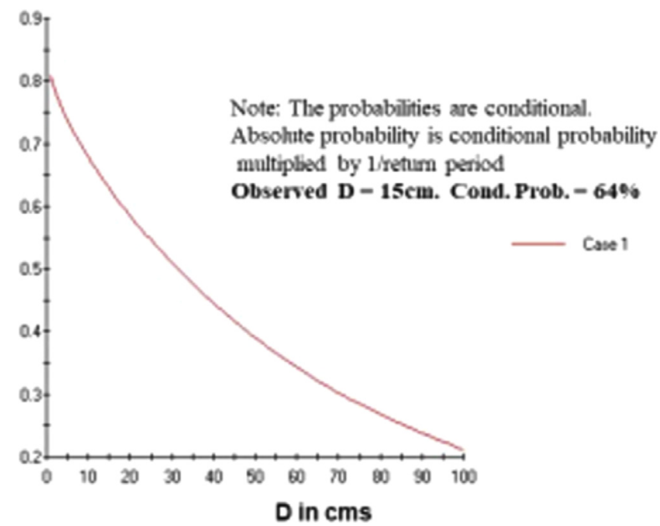

Fig. 9. Conditional probabilities of displacements in Lexington Dam slopes during Loma Prieta Earthquake.

of 15 cm during the 1989 Loma Prieta earthquake in California. The probabilities of various displacements during the Loma Prieta earthquake are shown in Fig. 9.

3.2. Prioritizing locations for remediation

The remediation of long-line structures such as flood protection dikes against the consequences of liquefaction is a very long and expensive process. Therefore, it is necessary to identify the locations that are most at risk and remediate these first. A simple predictive relation based on easily measured characteristics of the dikes was developed for prioritizing the selection of locations for remediation of flood protection dikes along major rivers in Hokkaido, Japan. This project demonstrates clearly another potential role for performance based design and associated nonlinear analysis.

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 6–8 m in height, and were constructed of compacted sand fill resting on a comparatively thick peat layer over liquefiable sand The dikes were severely damaged at 18 locations for a total length of about 10 km along the Kushiro River. The severest damage occurred in Kushiro Marsh [10–12]. Dike sections which failed were reconstructed, after the foundation soils had been improved by the installation of sand compaction piles.

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 considered was to use potential crest settlements as a criterion. The strategy to meet this objective was to conduct parametric studies on many dikes and develop an equation that linked dike characteristics to crest settlements for repeats of the 1993–1994 earthquakes. This equation would then be used to identify the dikes at greatest risk of failure and to prioritize interventions for remediation. The parametric studies were commissioned by the Hokkaido Development Bureau through the Advanced Construction Technology Center (ACTEC) in Tokyo. The analyses were conducted at the University of British Columbia, Vancouver, Canada. and the resulting equation was validated by ACTEC using data from many damaged dikes not included in the parametric studies.

Simulations of some dike failures during the 1993 Kushiro-oki earthquake were conducted based on soil data and input motions provided by ACTEC [13] and JKK [14]. After these simulations proved satisfactory, a program of parametric studies were authorized to investigate how the crest settlements of dikes correlated with slope angle, dike height, and the thicknesses of non-liquefiable and liquefiable layers under the earthquake loading specified by JKK [14]. A typical dike cross-section is shown in Fig. 10.

On the basis of the parametric studies, the following criterion for damaging settlements was developed by Finn, and Wu [15], for symmetrical dikes with 1:2.5 side slopes, where the normalized settlement S is the crest settlement divided by H_D, the height of the dike and H_L and H_{NL} are the thicknesses of the liquefiable and non-liquefiable layers, respectively. This relationship is shown by the black curve in Fig. 11 in terms of the non-dimensional variables S and β where $\beta = H_D \cdot H_{NL}/H_{NL}^2$. The black data points are the settlements computed from the parametric studies. The data is described very accurately by Eq. (3).

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

After the prediction equation was submitted to ACTEC, their engineers conducted an independent evaluation of the prediction equation. They plotted dike deformation data from the western Hokkaido 1994 Nansei-oki earthquake as shown in Fig. 12.

The 8 white square data points close to the curve in Fig. 12 are for symmetrical dikes with side slopes of 1:2.5. The agreement is very good for these dikes. The remaining points significantly to the right the curve describe the response of dikes with uniform side slopes of 1:5, and unequal slopes, of 1:5 and 1:10. These slopes require different correlations.

The predictive equation for dike damage, strongly validated by data from case histories, provides a convenient method for prioritizing dikes with side slopes 1:2.5 in the Hokkaido river basins for remediation. This is another example of the way in which an appropriate analysis in conjunction with a relevant performance criterion can help the engineer formulate a cost effective response to a challenging design problem.

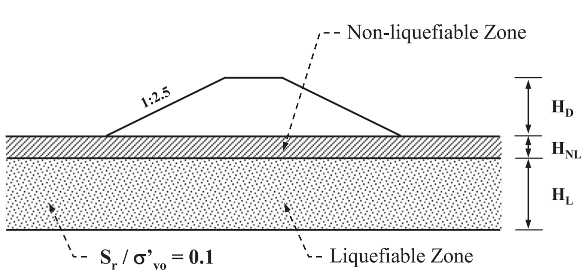

Fig. 10. Typical cross-section of dike used in parametric studies.

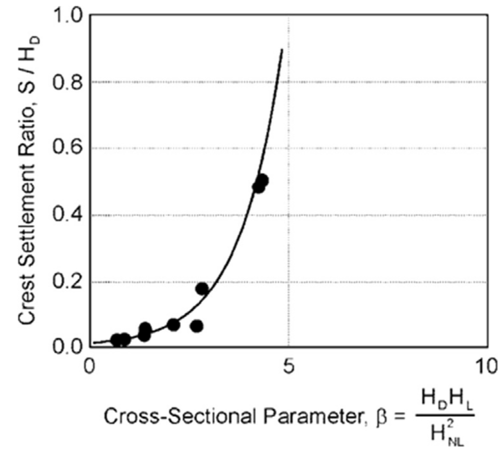

Fig. 11. Comparison of computed settlements with the black prediction curve.

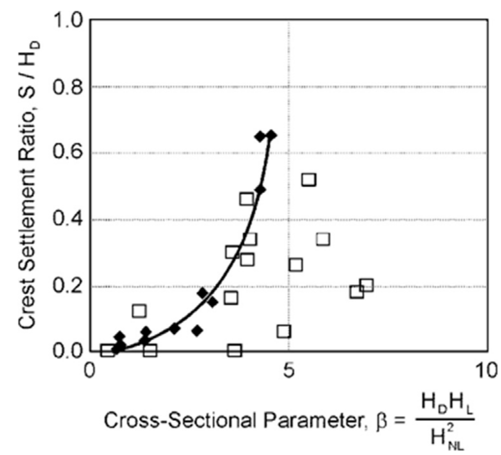

Fig. 12. Comparison of post-earthquake observed settlements for all slopes with predicted settlements.

4. Mormon island auxiliary dam (miad)

MIAD is a 30 m high earth embankment constructed over dredged tailings from gold mining in Folsom, California. It is part of a system of dams that creates the Folsom reservoir that supplies water for municipal, industrial and agricultural uses as well as flood protection for Sacramento. The foundation tailing are in a very loose state and the stability of both upstream and downstream slopes was of concern because of the threat of liquefaction during an earthquake. To achieve stability defined as maximum crest settlement of 1.0 m, it was decided to create densified plugs both upstream and downstream as shown in Fig. 13.

The reservoir level was very low due to prolonged drought so it was possible to densify the required plug upstream by dynamic compaction. Because of the proximity of the downstream slope to the highway, dynamic compaction was not allowed because of potential danger to traffic. Instead a plug of 0.8 m diameter stone columns was created at a

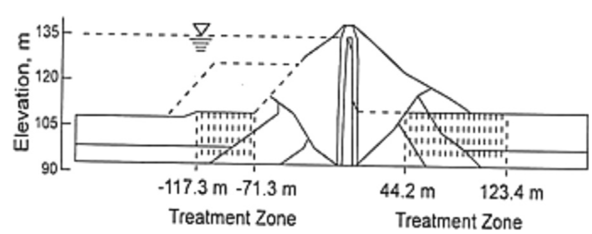

Fig. 13. Cross-section of MIAD showing treatment zones.

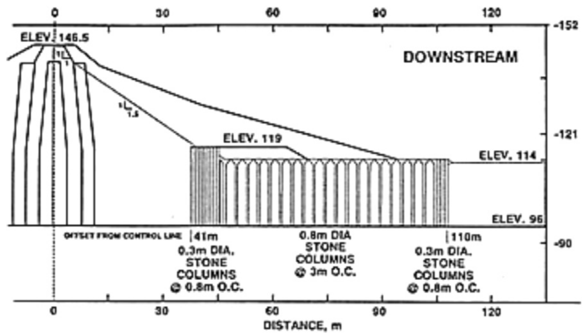

Fig. 14. Details of the downstream remediation plug.

spacing of 3.0 m center to center. The plug was protected upstream and downstream by rows of 0.3 m stone columns at 0.8 m spacing. The plug was also capped by a gravel drainage layer. A performance criterion for the plug was adopted that limited the excess pore water pressure ratio to less than 20%. The details of the remediation plug are shown in Fig. 14.

Dynamic effective stress analysis allowing migration of pore water pressure during and after shaking was conducted using conservative estimates of permeability and gave the pore pressures shown in Fig. 15. The pore pressures in the zone are significantly below the tolerable performance level of 20%.

5. Reliability of nonlinear analysis

The response of earth structures to seismic loading is determined by dynamic analysis using computer programs that incorporate appropriate constitutive models. The reliability of the computer program used for dynamic analysis of critical structures is a crucial issue for a designer. The term reliability is not used in this context in its formal mathematical sense. Rather it describes the confidence of the designer or analyst that the program can help to bracket the likely range of behavior of the structure under consideration and provide him with the data on which he can exercise his professional judgment, to achieve a safe and cost effective design. How does the analyst develop this confidence when using a program for the first time? He requires a knowledge and understanding of the constitutive model, the computational procedures to implement the model, how a program was tested and validated and whether any verification is available from case histories in the field or from centrifuge tests.

Any constitutive model operates in an ideal environment, typically a continuum. Within this continuum with well–defined properties, all models can perform perfectly. One of the main problems in achieving a reliable response from analysis of an actual structure for a specified

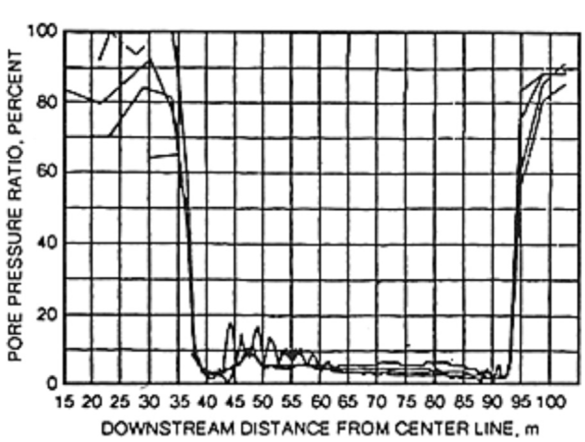

Fig. 15. Pore water pressures around the treatment zone.

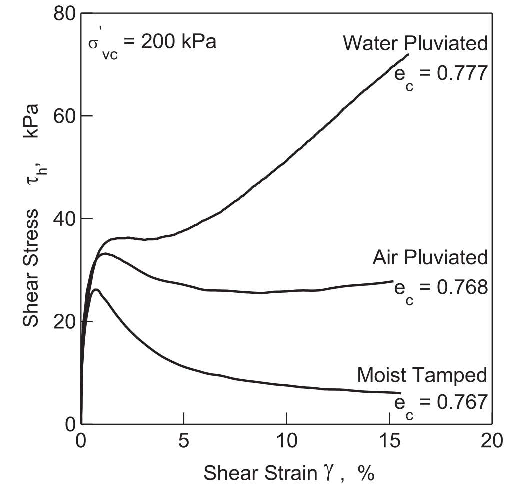

Fig. 16. Effect of stress path on stress-strain response [16].

seismic input is the challenge of calibrating the model so that the continuum in which the model operates adequately represents the conditions in the actual structure. The difficulty in achieving satisfactory calibration is model dependent. Some programs can operate directly using properties determined by in situ tests during site investigation but most models, especially the advanced plasticity models require laboratory test data. These tests are frequently conducted on reconstituted samples. Comprehensive research studies have defined the conditions to be satisfied for the reconstituted test specimens to considered representative of field conditions. The main conditions are: the specimens should be formed by a process that mimics natural deposition in the field and have field density; The stress path in the calibration tests should be comparable to the dominant anticipated stress path in the field.

The process of sample formation has a very significant effect on the stress strain behavior of test specimens. The radically different stress-strain curves for specimens prepared by moist tamping, pluviation in air and pluviation in water are shown in Fig. 16 [16]. Stress-strain curves on undisturbed test specimens of Holocene sand retrieved from frozen ground and curves from reconstituted test specimens of the same sand formed by pluviation in water are shown in Fig. 17. For both types of specimens the results are almost identical. Evidence of this kind clearly indicates the care that should be taken to ensure that as far as possible test specimens should be formed to mimic formation in the field.

The US Bureau of Reclamation have expressed concern about the reliability of dynamic analysis for embankment dams because there are no seismic performance case histories of dams remediated or designed on the basis of performance based criteria. An exception may be the

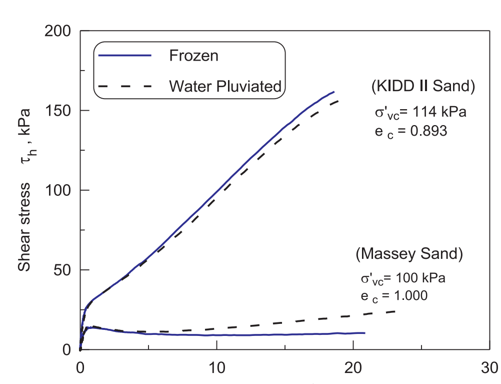

Fig. 17. Comparison of stress-strain curves from frozen specimens and from water pluviated specimens [16].

flood protection dikes in Hokkaido Japan described above but these were relatively small structures compared to dams. A performance criterion for dikes developed on the basis of performance data from dikes subjected to the 1993 Kushiro-oki earthquake predicted very accurately the performance of dikes in western Hokkaido during the 1994 Nansei-oki earthquake. In August 2016, the Bureau held an informal workshop in Sacramento on the reliability issue attended by leading consultants and academic researchers. There was a wide ranging discussion of what could be done to promote reliability. The comments above are probably a reasonably accurate picture of the consensus of the workshop but a niggling concern remains over the lack of case histories.

6. Concluding remarks

Four examples from the writer's practice are presented in the paper: Sardis Dam in Mississippi, the Flood Protection Dikes in Japan, the Mormon Island Auxiliary dam in California and the stability study of slopes for residential development in British Columbia, Canada. The examples were presented to demonstrate the great potential of a performance based design approach to the remediation of embankments to resist liquefaction induced failure. Each example employed a different performance criterion that was dictated by the constraints and demands of the local system.

The Sardis example is of particular interest because it is the first remediation based wholly on performance based design principles. Engineers were reluctant to switch from the traditional Factor of Safety approach to design based on displacement criteria. Many ancillary analyses, some described above, were conducted to build confidence in the new approach. For the Mormon Island Auxiliary Dam an additional performance criterion had to be adopted, limiting the excess pore water pressure ratio in the downstream remediation area to 20%. The Hokkaido dikes are a unique example because the model developed for prioritizing remediation intervention was calibrated for damage during a 1993 earthquake in Eastern Japan and was validated for damage during a 1994 earthquake in western Japan. It very accurately modelled what happened in 1994.

Non-linear dynamic effective stress analysis is an essential tool in seismic design in geotechnical earthquake engineering. It is crucial to success in performance based design especially when extensive remediation is involved. But it is a complex process and requires considerable analytical skills and a very thorough knowledge of soil behavior.

The selection of an appropriate constitutive model for the job at hand requires knowledge of the past history of the model; how it fared in blind predictions tests, how it was validated and what its track record is in practice. The calibration of the model for the job at hand, if it cannot be done on the basis of in situ data, needs to be conducted on test specimens that are representative of field conditions and for stress paths that reflect the dominant stress paths in the field application.

It is advisable for high consequence projects to have on the review board an expert on the important aspects of dynamic response analysis and to use this expert in the early stages of the analysis. Non-linear analysis of critical, high consequence projects is a high risk endeavor to be conducted with the greatest vigilance.

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