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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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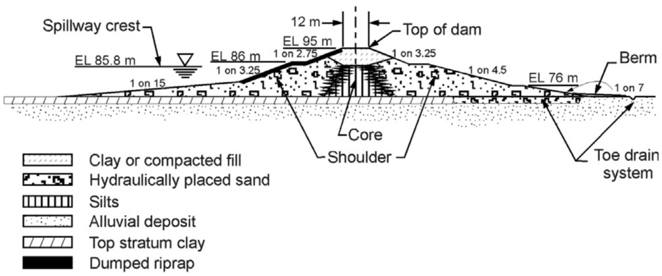

Fig. 1. Cross-section of Sardis dam.

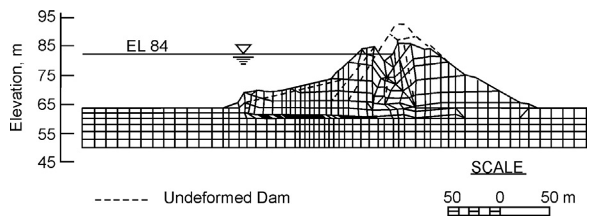

Fig. 2. Post-liquefaction deformed shape of Sardis dam. Note the different vertical and horizontal scales.

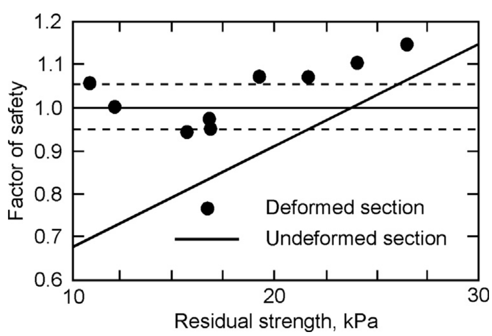

Fig. 3. Factors of safety of Sardis dam as a function of residual strength in weak foundation layer.

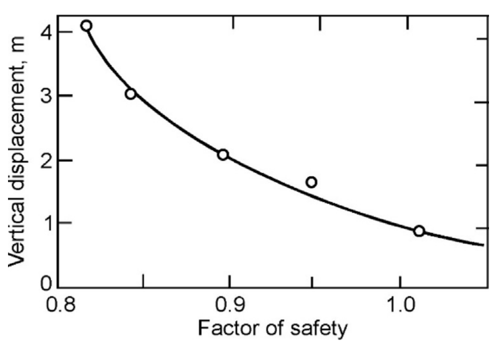

Fig. 4. Variation of loss of freeboard with factor of safety of undeformed dam.

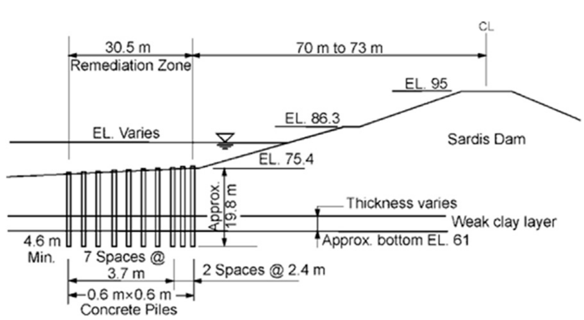

Fig. 5. Elevation of pile remediation of Sardis dam [1].

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 ± 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.