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Remediation of liquefaction effects for an embankment using soil-cement walls: Centrifuge and numerical modeling

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ABSTRACT

Numerical simulations of a centrifuge model test of an embankment on a liquefiable foundation layer treated with soil-cement walls are presented. The centrifuge model was tested on a 9-m radius centrifuge and corresponded to a 28 m tall embankment underlain by a 9 m thick saturated loose sand layer. Soil-cement walls were constructed through the loose sand layer over a 30 m long section near the toe of the embankment and covered with a 7.5 m tall berm. The model was shaken with a scaled earthquake motion having peak horizontal base accelerations of 0.05 g, 0.26 g, and 0.54 g in the first, second, and third events, respectively. The latter two shaking events caused liquefaction in the loose sand layer. Crack detectors embedded in the soil-cement walls showed that they developed only minor cracks in the second shaking event, but sheared through their full length in the last shaking event. The results of the centrifuge model test and two-dimensional nonlinear dynamic simulations are compared for the two stronger shaking events using procedures common in engineering practice. The effects of various input parameters and approximations on simulation results are examined. Capabilities and limitations in the two-dimensional simulations of soil-cement wall reinforcement systems, with both liquefaction and soil-cement cracking effects, are discussed. Implications for practice are discussed.

1. Introduction

Soil-cement grid and wall systems have been used to remediate embankment dams and other civil infrastructure against the effects of earthquake-induced liquefaction in their foundations. Soil-cement treatments have the advantage that they can be constructed in a wide range of soils, including silty soils that can be difficult to treat by densification techniques. A soil-cement grid or wall system is often constructed near the toe of an embankment and covered with an overlying berm to increase confinement and reduce deformations that bypass the treatment zone. An example of this type of configuration is the remediation at the 24-m tall Clemson Upper and Lower Diversion Dams (Wooten and Foreman [23]) as shown in Fig. 1. Other embankment dam remediation projects using soil-cement grid or wall systems in the US include: Sunset North Basin Dam, CA (about 23 m high; Barron et al. [2]); San Pablo Dam, CA (about 44 m high; Kirby et al. [14]); Perris Dam, CA (about 39 m high; Friesen and Balakrishnan [9]), and Chabot Dam, CA (about 30 m high; EBMUD).

The seismic performance of soil-cement grids and walls have been studied using three-dimensional (3D) analysis methods (e.g., Fukutake and Ohtsuki [10], Namikawa et al. [16]), but design practices generally rely on two-dimensional (2D) approximations with equivalent composite strengths for the treatment zones (e.g. Wooten and Foreman [23], Barron et al. [2], Kirby et al. [14], Friesen and Balakrishnan [9]). Some common concerns in the design of soil-cement grids for liquefaction remediation include the potential for cracking and brittle failure in the soil-cement elements, the ability of 2D analysis procedures to approximate the 3D response, and the lack of experimental or case history data to validate 2D or 3D numerical analysis methods.

This paper presents results of centrifuge model tests and numerical simulations of an embankment on a liquefiable foundation layer treated with soil-cement walls, expanding on results presented in Boulanger et al. [4]. The centrifuge model was tested on a 9-m radius centrifuge and corresponded to a 28 m tall embankment underlain by a 9 m thick saturated loose sand layer (prototype units). Soil-cement grids were positioned through the loose sand layer near the toe of the embankment and covered with a berm. The model was shaken three times with a scaled earthquake motion; the peak horizontal base accelerations (PBA) were 0.05 g, 0.26 g and 0.54 g, respectively. The latter two events liquefied the loose sand layer. The soil-cement walls developed limited cracking in the 0.26 g shaking event and sheared through their full length in the 0.54 g event. Two-dimensional nonlinear dynamic analyses were performed using the finite difference program FLAC (Itasca [11]) and the user-defined constitutive model PM4Sand (Boulanger and Ziotopoulou [6]) for the sands. The treatment zone was represented with area-averaged composite properties as is common in design practice. The centrifuge model test and numerical simulation procedures are described, followed by comparisons of the measured and

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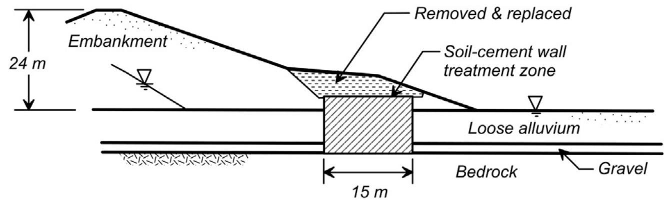

Fig. 1. Soil-cement shear walls at Clemson Diversion Dams, SC (after Wooten and Foreman [23]).

simulated responses. The effects of various input parameters and approximations on simulation results are examined. Capabilities and limitations in the two-dimensional simulations of soil-cement wall reinforcement systems, with both liquefaction and soil-cement cracking effects, are discussed. Implications of the centrifuge and numerical analysis results for practice are discussed.

2. Centrifuge model tests

The centrifuge model was tested in a flexible shear beam container at a centrifugal acceleration of 65 g on the UC Davis 9-m radius centrifuge. Standard scaling laws are followed and results are presented in prototype units unless otherwise specified. The experiment and data are documented for distribution in Khosravi et al. [12] and summarized in Khosravi et al. [13].

The centrifuge model configuration (Fig. 2) consisted of a foundation

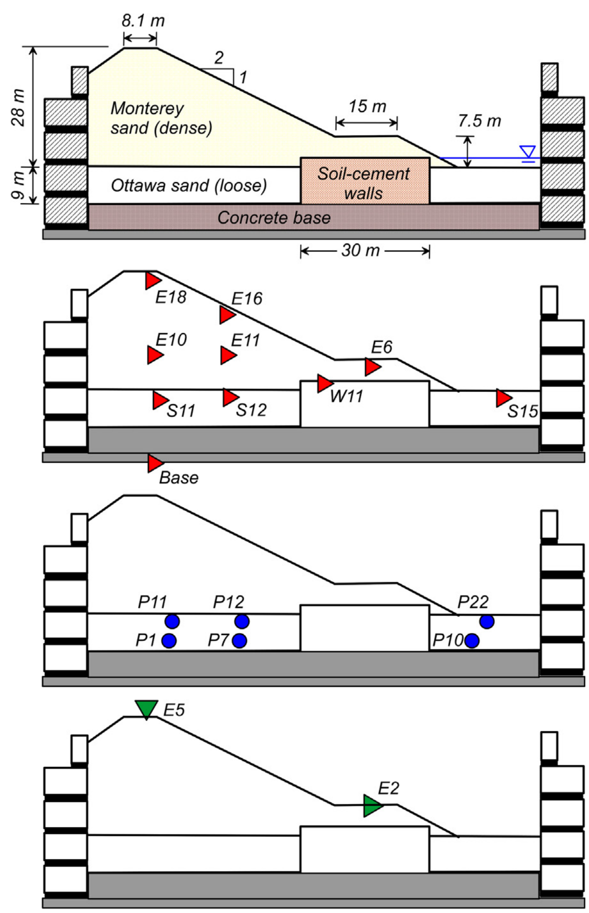

Fig. 2. Cross-sections showing model dimensions (prototype scale) and the locations of accelerometers (red triangles), pore pressure transducers (blue circles), and displacement transducers (green triangles) that are later compared with simulation results. (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.)

layer of loose Ottawa F-65 sand (relative density, D_r = 42%; D10 = 13 mm), an embankment and berm of dry, dense Monterey #0/30 sand (D_r = 85%; D10 = 0.4 mm), and a set of nine parallel soil-cement panels over a 30 m long section near the toe of the embankment. The pore fluid was a methylcellulose solution with a viscosity about 15 times that of water. The water table was above the top of the foundation layer and slightly above the tops of the walls. A thin layer of Monterey medium aquarium sand (D_{10} = 1.7 mm) was placed at the water surface elevation to provide a capillary break during model construction.

The soil-cement walls were formed and cured in molds and then arranged in the flexible shear beam container prior to pluviation of the foundation sand layer. The walls were 1.4 m thick and spaced 5.8 m apart (center to center), for an area replacement ratio of A_r = 24%. The soil-cement had an average unconfined compressive strength (q_{ucs}) of 2.06 MPa at the time of centrifuge testing. The walls were set into preformed slots in a concrete base layer and grouted into position.

Seventeen crack detectors were embedded in four of the soil-cement walls at the time they were formed. The crack detectors were 2-mm diameter pencil leads connected to a circuit by wires at each end (Tamura et al. [20]). These brittle conductors provide a binary indication of if, and when, cracking occurs. The pencil leads were oriented vertically at different locations along the walls, with their lower end below the top of the concrete base.

The model was also extensively instrumented with accelerometers, pore pressure transducers, and displacement transducers as described in Khosravi et al. [13]. The locations of the transducers whose recordings are later compared with simulation results are shown on the cross-sections in Fig. 2.

The model was shaken three times with a scaled version of a recording from Port Island in the 1995 Kobe earthquake. The first shaking event had a PBA = 0.05 g, for which the response was essentially elastic with no excess pore pressure generated. The second shaking event had a PBA = 0.26 g, which triggered liquefaction in the saturated sand layer but caused only minor cracking in the soil-cement walls. The third shaking event was applied long after full dissipation of the excess pore pressures from the second event; it had a PBA = 0.54 g, triggered liquefaction throughout the saturated sand layer, and caused the soil-cement walls to develop shears/cracks through their full lengths. The crest settled about 0.7 m and the toe berm displaced laterally about 1.3 m in the PBA = 0.54 g event, whereas movements in the PBA = 0.26 g event were only a quarter to half these amounts.

A photograph of the soil-cement walls when the foundation soils had been excavated to the elevation of a blue paper marker is shown in Fig. 3. The blue markers were placed flush against the faces of the soil-cement walls on the upstream and downstream faces during construction. The blue markers have been pushed forward between the soil-cement walls on the upstream side (by approximately 0.1 m), and pushed away from the walls by up to 0.8 m on the downstream side. These photos illustrate how the loose sand between the walls, which liquefied during strong shaking, displaced downslope relative to the walls during the course of imposed shaking.

Photographs of the soil-cement walls during model dissection after testing are shown in Fig. 4. Crack detectors indicate that only portions of the panels were cracked during the PBA = 0.26 g event, such that the majority of damage and the offsets along the cracks occurred during the larger PBA = 0.54 g shaking event.

3. Numerical simulation model

Two-dimensional (2D) nonlinear dynamic analyses were performed using the finite difference program FLAC (Itasca [11]). The mesh and material zones are shown in Fig. 5. Analyses were performed in large-strain mode with coupled pore water flow. Analyses used 0.5% Rayleigh damping at a frequency of 1 Hz.

The sands were modeled using the user-defined constitutive model PM4Sand version 3, which is a stress-ratio controlled, critical state