SEISMIC EFFECTIVE-STRESS ANALYSIS OF CAISSON QUAY WALL WITH LIQUEFIABLE BACKFILL

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Gravity caisson quay walls are one of the most common types used for docks and harbours. These quay walls are designed for three main geotechnical criteria: sliding, overturning and allowable bearing stress under the base of wall.

In this research, a series of two-dimensional finite difference effective-stress based analyses are performed in the prototype scale to investigate the seismic performance of caisson-type quay walls. In all models, the foundation soil beneath the caisson is assumed to be dense and non-liquefiable. However, the backfill soil is constructed with a relative density of 25% to reproduce a loose liquefiable backfill. The schematic cross section of the model is shown in Figure 1. For the gravity quay wall, the concrete caisson is modelled by linear elastic elements. The granular soil beneath the caisson is modelled elastic-plastic soil with a Mohr Coulomb failure criterion. Maximum shear modulus for sandy soil are determined from formula developed by Kanatani (1991). During the dynamic solutions excess pore water pressures are allowed to generate and also the dissipation of these pore pressures in backfill soil. Therefore, UBCSAND nonlinear elastic-plastic constitutive model is employed to simulate the seismic liquefaction behaviour of backfill materials.

In the non-linear dynamic analyses, element size is selected small enough (smaller than approximately one-tenth to one-eighth of the wavelength associated with the highest frequency component of the input wave) to allow the seismic wave propagation throughout the numerical model. The sea water is simulated through the hydrostatic pressures applied to the front side of the wall and top of the seabed soil (Figure 1). The input seismic excitation is shown in Figure 2. The acceleration wave with maximum acceleration of 0.2 g and constant frequency motion of 3 Hz is applied to the base of numerical model.

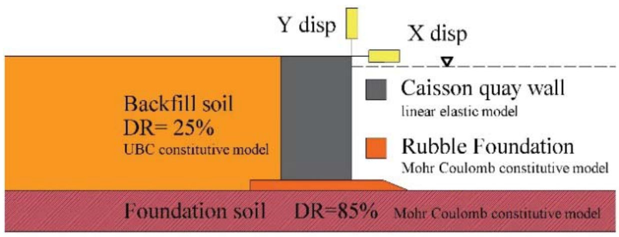

Figure 1. Schematic cross section of quay wall.

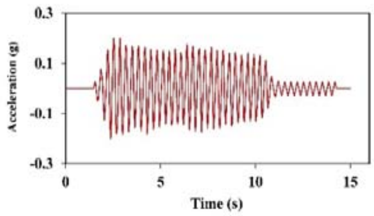

Figure 2. Horizontal acceleration time histories.

The free-field condition is exerted to the lateral boundaries eliminating the wave reflection into the model (Itasca, 2014). Around 0.2% of Rayleigh damping, centered at a frequency of around 3.2 Hz, is considered in the dynamic analysis. Contact conditions between wall and adjacent soil are modelled via special interface elements allowing for slipping and gapping through the Coulomb frictional law.

The current numerical results are validated against the corresponding observations obtained from 1g shaking table tests (Mostafavi et al., 2009). It is seen in Figure 3 that the obtained numerical results agree reasonably well with actual observation in the shaking table tests. The displacement of quay wall head increases quickly during seismic loading. In numerical model, the caisson quay wall movement toward the sea and its settlement are respectively about 92 and 22 cm.

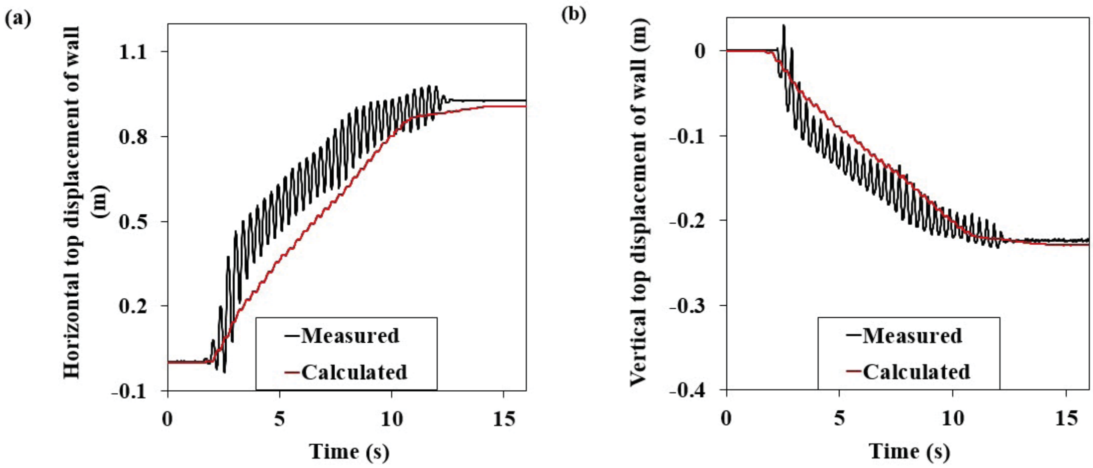

Figure 3. Deformation time histories: (a) horizontal and (b) vertical displacements at top of quay wall.

The deformed configuration of wall–soil system at the end of shaking in the numerical model is shown in Figure 4. The lateral spreading of backfill soil is clearly observed near the areas affected by the quay wall. Moreover, the failure mode of caisson quay wall can be observed in Figure 4. Significant translation and tilting of wall are the consequences of the combined effects of shaking and pore water pressure development within the backfill sand behind the wall.

As may be expected, more ground surface settlement is observed in the backfill near the wall than at the far field. Since the wall is rigid, its movement is due to the deformation of the foundation rubble, which creates a significant heave at the toe of the wall.

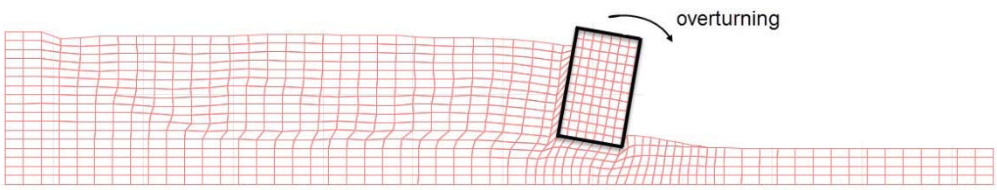

Figure 4. Deformed conf guration of wall–soil system at the end of shaking.

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