NUMERICAL EVALUATION OF SEISMIC BEHAVIOR OF RUBBLE-MOUND BREAKWATERS RESTED ON A LIQUEFIABLE SEABED SOIL LAYER

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Herein, numerical simulations are conducted by an explicit finite difference model incorporating hysteretic Mohr-Coulomb constitutive model to describe the stress-strain response of soil and the Rayleigh damping to increase the level of hysteretic damping in the model. In addition, the non-linear UBCSAND model is employed in order to capture pore pressure build up and seismic liquefaction behaviour during effective stress based analyses. A new design methodology, named performance-based design, has born from lessons learned from earthquakes in 1990's to overcome the limitations of conventional seismic design (Ebrahimian, 2009). This paper highlights the numerical modeling techniques in accordance with performance-based design approach corresponding to PIANC guidelines.

The rubble mound breakwater consists of three layers including armor, filter and core layers, which are rested on two distinct subsurface soil layers. Upper layer, with the maximum thickness of 12.5 m, was a loose-to-medium dense sandy silt with low plasticity (ML) that is highly susceptible to liquefaction with SPT blow count lower than 10. Lower thick layer was a dense composite gravel layer, silty, clayey gravel (GC-GM). In the numerical model, the rubble mound layers and the lower gravelly seabed materials were simulated by the Mohr-Coulomb model, whereas the liquefiable sandy silt layer was simulated by the UBCSAND model formulation. Schematic of the breakwater and foundation soil layers and the constitutive models that applied in them are shown in Figure 1. The input acceleration wave, shown in Figure 1, was selected as input base motion for the analysis with maximum acceleration of 0.2 g and constant frequency motion of 3 Hz.

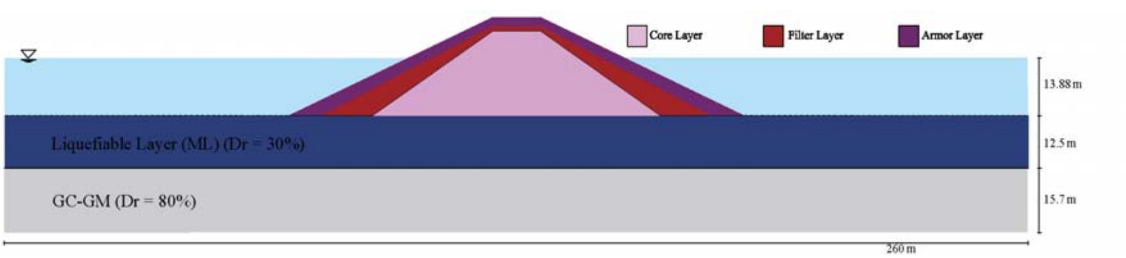

Figure 1. Schematic of breakwater and foundation soil layers and input acceleration wave.

Prior to dynamic analysis, static analysis is carried out to obtain static stress and strain regimes in the numerical model. It is noted that the breakwater and foundation are modeled in several stages corresponding to the stage construction of rubble mound breakwaters. Displacement vectors and slip surfaces achieved from the numerical analysis of the breakwater are illustrated in Figure 2-a at the end of the earthquake. The liquefaction of sandy silt layer resulted in crest-lowering and slumping of the breakwater body. Subsequently, seabed heaves at both sides of the breakwater. Some prominent deformation modes were recognized for the rubble mound breakwater after liquefaction of seabed as presented in Figure 2-b that including, seabed heaving at both sides of breakwater, settlement and volumetric change of rubble mound, infiltration of rubble mound into liquefiable layer and lateral movement of liquefiable layer.

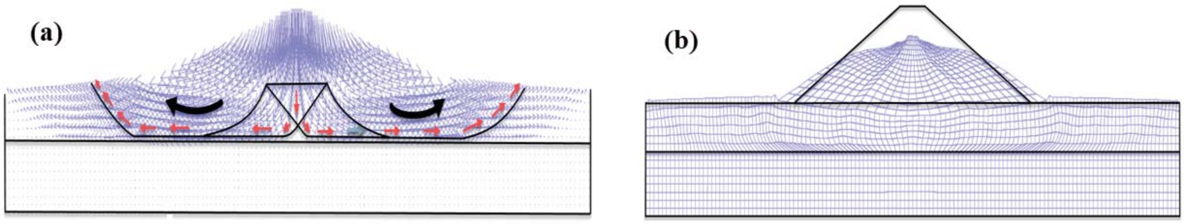

Figure 2. Results of seismic response: (a) displacement vectors, and (b) deformed configuration of rubble mound breakwater after earthquake loading.

To validate the implementation of the Masing rules in the FLAC program, fully coupled nonlinear dynamic analysis on models of embankment dam founded on shallow layers of liquefiable foundation (Adalier and Sharp, 2002) are performed to assess the seismic stability of earth dam. Results of the validation model as shown in Figure 3-a in terms of vertical displacement of the embankment (computed: numerical and measured: centrifuge test results) confirm successful applicability of the numerical model in obtaining seismic performance of breakwater founded on layers of liquefiable foundation.

Time histories of vertical displacement of breakwater crest after the earthquake are plotted in Figure 3-b. It is observed that liquefaction of the soil under the breakwater causes significant settlement of rubble mound due to the lateral movement of ground beneath breakwater.

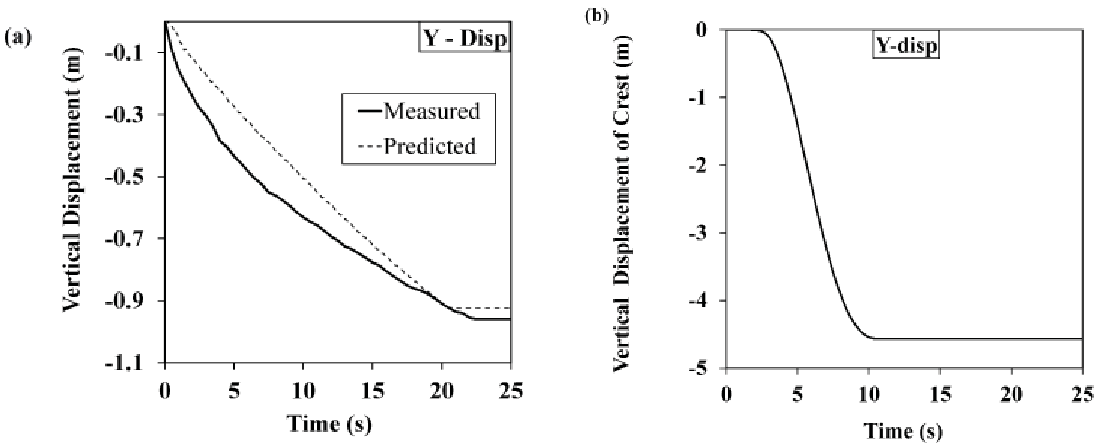

Figure 3. (a) Comparisons between the computed and measured results; (b) vertical displacements of breakwater crest after seismic loading.

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