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A model study on earthquake behaviour of breakwaters

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ABSTRACT: Earthquakes can cause great life and property loss depending on settlement conditions. Modern urban areas should be equipped with structures that can resist earthquake effects. Port structures should be earthquake resistant in order to sustain their economic functions and also for post-earthquake aid to reach disaster areas. To achieve expected performance, breakwaters which are designed to limit the effects of waves on port structures must also be built to withstand earthquakes.
Earthquake behavior of a rubble mound breakwater on a liquefiable sand layer is investigated by means of model experiments executed on a one dimensional shaking table. The shaking table has 4.0 m length, 1.0 m height and 1.0 m depth. The breakwater model has a 40 cm high cross section with ½ side slopes and built on a 40 cm thick sand layer with relative density of Dr=40%. One dimensional cyclic loading is applied at the base of sand layer and pore water pressures increases which observed to start from the first load cycle are recorded. The magnitude of excess pore water pressures increased with the level of applied cyclic loading and reached vertical effective stress level under strong base motions, leading to liquefaction in sand layer. This has caused large deformations and failure at the toe to occur in the breakwater constructed on sand layer.

1. INTRODUCTION

Marine structures such as breakwaters, quay walls, piers, dolphins are important economical facilities of modern cities. These structures are expected to withstand devastating effects of the earthquakes. Not only that these structures can be demolished but also secondary effects such as large economical losses due to disruption of port post earthquake functions can have very important impacts.

Earthquakes can cause structural integrity loss due to loss of underlying soil strength. When the seismic waves arrive to loose cohesionless soil deposits, porewater pressure increases up to confining stress levels can develop which may lead to liquefaction. As a result of liquefaction soil can behaves as viscous liquid and catastrophic failures of structures such as sinking of structures, lateral spreading etc. can be observed.

This kind of failures can be catastrophic, as it is observed in the recent earthquakes in Japan and Turkey. Liquefaction-induced damage to marine structures has been reported by Wyllie et al. (1986); Iai and Kameoka (1993), Iai et al.(1994), Hall (1995), Sugano et al.(1999), Boulanger et al. (2000), Sumer et al(2002), Yuksel et al.(2004), Cihan (2009).

This study is focused on model experiments conducted to investigate seismic-induced liquefaction failure and its implications for breakwaters. Following a brief review of liquefaction and related breakwater behaviors, details of the experimental set up and some results are presented.

2. LIQUEFACTION

Saturated granular soils subjected to cyclic shear stresses show tendency for volume decrease and subsequent pore water pressure increase, resulting in effective stress losses and soil material can transform from solid to liquid state. These changes generally occur in loose and moderately dense cohesionless soils such as sands and silty sands. Ishihara(1996) defined liquefaction as the state of large deformations occurring with complete strength loss due to pore pressure build up in loose sands. For medium to dense sands, deformations are limited and do not reach large levels and do not cause complete strength loss. In silty sands, plasticity has an effect on liquefaction behavior, whereas in cohesive soils cyclic resistance is higher compared to granular materials because of the cohesion of between grains.

Liquefaction attracted wide interest of geotechnical engineers firstly after the Niagata and Alaska Earthquakes in 1964, during which catastrophic results were observed in the areas subjected to these earthquakes. Since then, intense studies had been conducted to clarify liquefaction behavior.

In order to determine the liquefaction potential, cyclic resistance of soils and seismic demand on the soil layer are needed. Liquefaction potential is usually represented by the cyclic stress ratio (CSR) which causes 5% double amplitude axial strain reached at a certain number of cycles. Liquefaction resistance is also correlated by empirical relations from observations and in-situ tests. The three most common in situ tests used in liquefaction resistance assessment are, Standard Penetration Test (SPT), Cone Penetration Test (CPT) and shear wave velocity (v_s).

2.1 Seismic Behavior of Breakwaters

Dams are the structures to resist hydrodynamic forces and there seismic behaviors are well studied. However studies on seismic behavior of marine structures are rare. Some liquefaction induced deformations on marine structures have been reported in past earthquakes. These include damages observed in marine structures during 1995 Hyogoken-Nanbu(Kobe) Earthquake and 1999 Kocaeli Earthquake. In Kobe 1995 Earthquake great economical loss occurred in marine facilities. Iai and Sugano(1999) studied the deformations occurred in marine structures in Kobe. In order to understand the liquefaction induced behavior of caisson type breakwaters, 1g scale model tests were conducted.

There are also limited number of numerical studies modeling liquefaction and related breakwater behavior in the literature. Memos et al (2000) and Memos et al (2003) presented results of numerical and model experimental studies conducted on rubble mound breakwaters built in coast of Greece. The calculated and measured acceleration time histories at the cohesive layers under a breakwater are compared. Yuksel et al (2004) studied seismic performance of breakwaters at Eregli Harbour in Turkey which was subjected to 1999 Kocaeli Earthquake. Lateral and vertical displacements of up to 1.0m were calculated on the deformed section of breakwater, which were in agreement with the site observations.

3. MODEL EXPERIMENTS OF BREAKWATER

In this study, dynamic behaviour of a breakwater seated on liquefiable soil layer is investigated by means of model experiments. The properties of sand which is used for foundation layer of breakwater are obtained by sieve analysis, specific gravity, direct shear and simple shear tests Sand material used in model experiments can be categorized as poorly graded sand according to Unified Soil Classification System. Gravel used to constitute breakwater is classified as coarse gravel (Figure 1).

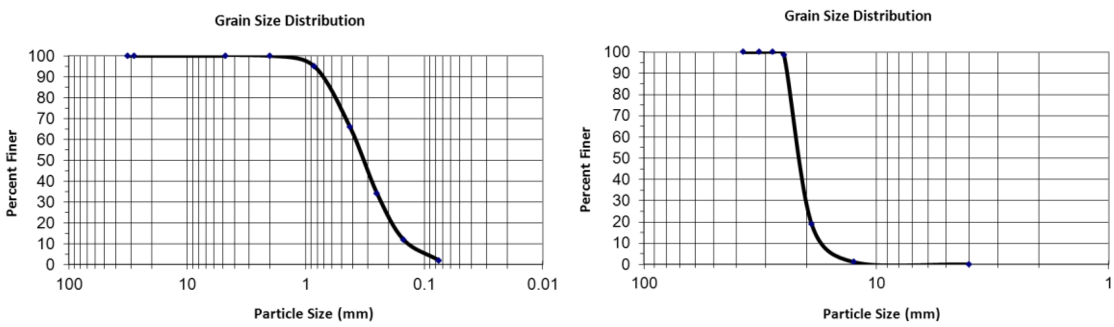


Figure 1 Grain size distribution of sand and gravel used in model experiments

Internal friction angle of sand used in experiments are obtained as 35.8^0 for the relative density of D_r= 50% and 36.4^0 for D_r=70% from direct shear tests. From simple shear tests the internal friction angle of sand is obtained as 33.0^0 and 35.8^0 for D_r=50% and D_r=70%, respectively.

3.1 Model Test Experimental System

A special experimental system is designed for the model tests (Figure2). It consists of a tank with 1.0m height, 1.0m width and 4.5m length, and a particularly designed granular material pluviation system. One side of the tank is covered with a 2cm thick glass outer face of which is grid marked to observe the dynamic behavior of breakwater and measure deformations under imposed cyclic motion.

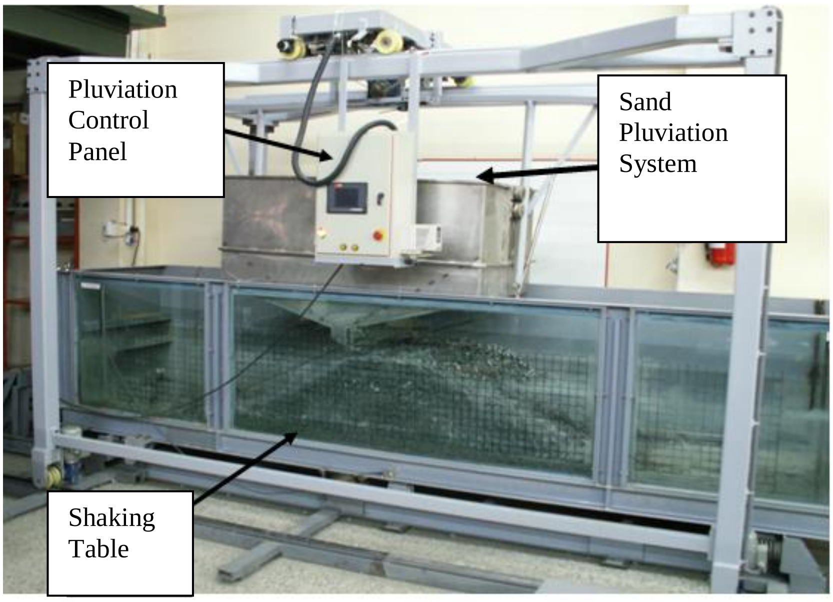
The section of breakwater and its foundation layer is deposited into water by the pluviation system which can move in two directions and form a homogenous layer at desired relative density. A 40 cm thick foundation layer with relative density of 40% is constituted by pouring sand into 30 cm high water from 20 cm above the surface. Then, 40cm high breakwater section with side slopes of ½ is also constituted by the pluviation system. The breakwater section had a 30 cm high core and 10 cm thick cover layer which are made up of gravel and stones, respectively.

Dynamic motion is applied at the bottom of tank at frequencies ranged between 1-15 Hz and amplitudes of 1-10 mm by a power transmission system exerting a uniform sinusoidal motion. The cyclic motion is applied in one direction with 3mm amplitude and at 4 Hz frequency, for the model test results presented in this paper. Time histories of acceleration at different locations are recorded by four accelerometers. One is installed at the base of sand foundation and others are located at the bottom, at the top and on the slope of breakwater cross section. Also, pore water pressures are measured by six transducers. Three of the transducers are installed under the base of breakwater in sand layer which are embedded at the bottom, at mid height and top of the sand layer. Two of the transducers are located at the bottom and at the middle of sand layer under breakwater toe, and one is located at half thickness of the sand layer at the free field (Figure 2). Displacements induced during shaking are measured thru recorded images processed by software coded in Labview.

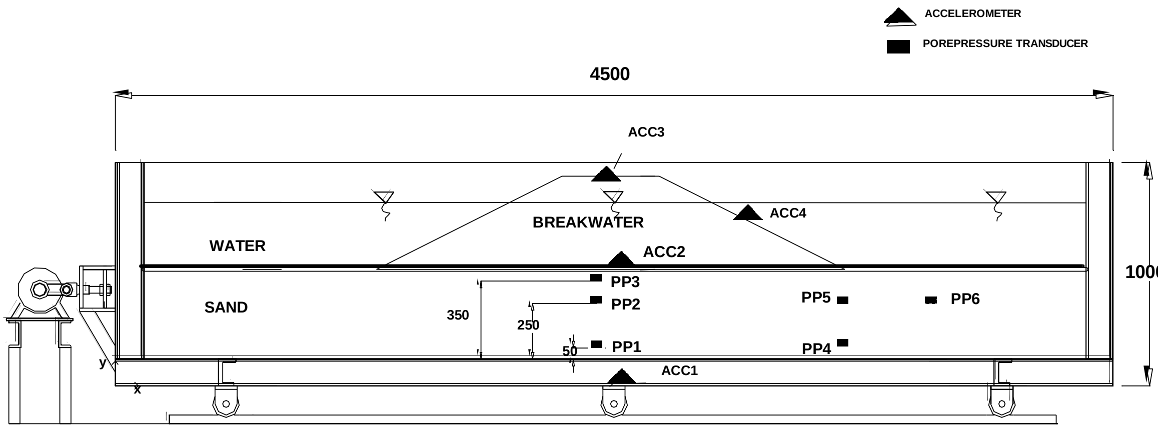
4. RESULTS OF MODEL EXPERIMENTS

In order to observe the dynamic behavior, a 40 cm high model breakwater is constituted on a 40 cm thick liquefiable sand layer of 40% relative density. The amplitude of the cyclic input motion applied at the bottom of the sand foundation layer is measured as 0.17g (Figure 3). Generated accelerations and pore water pressure variations at different locations are recorded with electronic instrumentation. The time history recordings of accelerations on the breakwater had some local peaks which are due to local pounding effect of stones and due to some water fluctuations. These noises are filtered by Butterworth filters to observe real behavior.

The recorded acceleration data show that the input motion does not amplify significantly in the thin sand (foundation) layer. The accelerometer installed at the base of breakwater recorded 0.17g which is at the same level with the input motion. However, amplification occurred at the top and on the slope of breakwater section.



(a)



(b)

Figure 2 (a) Sand pluviation system (b) Model breakwater section and instrumentation locations.

Maximum amplification is measured at the top of breakwater which is considered to be from the focusing effect due to the shape of section tested. Amplitude of accelerations measured at the top and on the slope of breakwater are 0.25 and 0.20 g, respectively (Figure 3).

Excess pore water pressures due to cyclic motion are measured under the breakwater centerline, under the toe and at free field in sand layer. It is observed that maximum excess pore pressure developed under the breakwater centerline is about 3 kPa while it is around 5 kPa under the toe. The pore pressure ratio (r_u) values under the breakwater have reached 0.40 at the bottom and mid height of foundation sand layer, and 0.60 at the base of breakwater. However, exceptionally high r_u values are generated under the toe and at free field due to low in-situ effective stress levels and some possible measurement errors (Figure4).

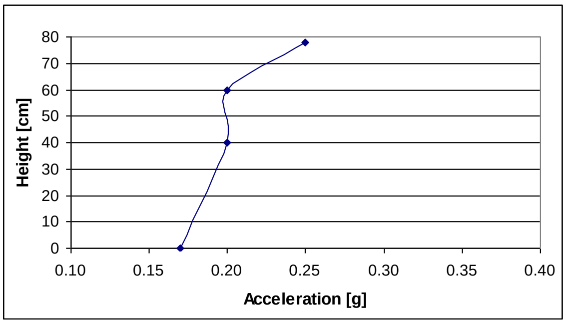


Figure 3 Maximum acceleration variation with height

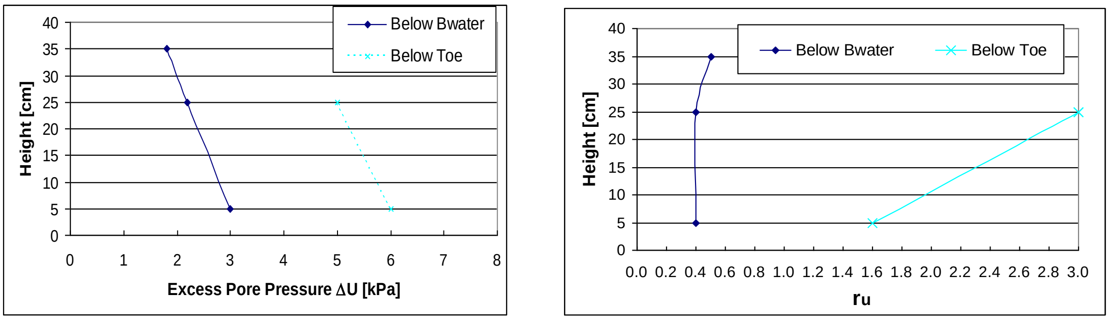


Figure 4 (a) Excess pore water pressure variation with height, and (b) ru distribution

The deformations are also recorded by a camera which takes photos at every one second. The image analyses are done by a program coded in Labview. The total vertical displacement at crest is observed to be about 30% of the breakwater height, half of which took place in first 5 seconds (Figure 5).

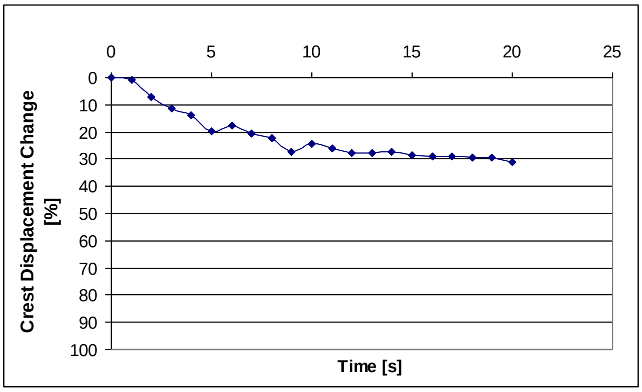


Figure 5 Vertical displacement at crest with time

5. CONCLUSIONS

In order to investigate dynamic behaviour of breakwaters on liquefiable sand layers, model experiment are conducted. The model experiments are performed on one dimensional shaking table. 40 cm high breakwater model is built on 40 cm thick sand layer which is constituted at 40% relative density by a sand pluviation system. A 20 seconds duration cyclic input motion is applied at the base of sand layer and generated accelerations no the breakwater are measured by four accelerometers located at the bottom of sand layer, at the base, at the crest, on the side slope and under the toe of breakwater. The amplitude of input motion was 0.17g. It is observed that amplification occurs at the crest and on slope of the breakwater. Maximum acceleration is measured as 0.20 g on the slope while it was 0.25g at the crest.

Excess pore water pressures developed under cyclic loading are also measured with the aid of six transducers located under the centreline of breakwater, under the toe of breakwater and at free field in the sand layer. Pore pressure ratios are calculated to be around 0.50 under the centreline. Lower excess pore water pressures are measured under the toe and at free field, however due to low in-situ effective stress levels, pore pressure ratios exceeded unity. This is considered to be the main reason for the observed large deformations around the toe due to bearing capacity loss. The vertical displacement at the crest of the breakwater reached 30% of the height, half of which occurred after 5 seconds of cyclic loading.

The observations made in this experimental study are believed to shed some light on the dynamic behaviour breakwaters seated on liquefiable sand layers. Due to strength loss under the toe stability of breakwater is threatened. Observed loss of height in breakwater after initial cycles of loading can be detrimental on their post seismic activity and possible tsunami effects.

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