Coastal Engineering 123 (2017) 1–10

Contents lists available at ScienceDirect

Coastal Engineering

journal homepage: www.elsevier.com/locate/coastaleng

An experimental method to verify the failure of coastal structures by wave induced liquefaction of clayey soils

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ARTICLE INFO

Keywords:
Pore pressure
Liquefaction
Muddy soils
Breakwater sinking

ABSTRACT

Although the dynamics involved in the liquefaction process are understood reasonably well, experimental work to reproduce the sinking of structures due to liquefaction, which is representative and repeatable, has so far not been recorded. In this work, three sets of experiments were performed in an attempt to fill the gaps in the knowledge by modelling a small scale reproduction of the failure. Firstly, an analysis of the role of the proportions of the initial fine sediment and water content is presented; secondly, a group of tests involving a vertical breakwater were performed, and thirdly, an experiment was carried out to reproduce the failure of a submerged structure on a clayey bed in the presence of waves. From these experiments, we were able to set thresholds for bed composition, below which soil liquefaction is likely to occur. It was determined that the potential to liquefy increases with the initial water content and that soils of 40% or more clay content may liquefy. This methodology has proven to be repeatable, allowing the reproduction of the sinking of coastal structures due to liquefaction of the underlying soil.

1. Introduction

The coastal zone is of great importance for transport, tourism and leisure, as well as marine and food industries. Consequently, urban, tourist and industrial infrastructure is often built and this must be protected, usually with structures such as breakwaters and seawalls [11,25]. Breakwaters are being built in deeper waters and with bigger dimensions, which are therefore more vulnerable to geotechnical failure. For example, liquefaction of the seabed can lead to significant degradation of the foundations of a breakwater during or after its construction; sometimes resulting in its total collapse [10]. Other coastal structures, such as pipelines placed on the sea bottom, can also be damaged if the load capacity of the soil is reduced due to the build-up of pore water pressure, causing the pipes to sink or float.

Wave-induced liquefaction has been studied, mostly through experimental work, since the 1970 s by recording and analysing pore pressures, effective stresses and soil displacements [10]. Thus, nowadays it is well understood that when ocean waves propagate over an undrained seabed, the dynamic pressure acting in the soil mass will increase pore pressure and effective stresses on the seabed. This occurs mainly as a result of the reduction of the voids volume, due to the rearrangement of the soil grains [18]. When the pore pressure exceeds a critical value, the seabed may liquefy or fluidize, the load capacity of the soil will vanish and the foundation of a coastal structure will become unstable, causing structure instability or even sinking [9].

Bjerrum [1] was the first to present a methodology to calculate excess pore water pressures developed beneath a gravity-type offshore structure due to cyclic wave loads. The offshore structure was a concrete oil tank placed on a sandy seabed in the Ekofisk field of the North Sea, where the water is 70 m deep. Bjerrum analysed the pore pressure rise generated by individual waves in storm conditions. The results showed that, assuming undrained conditions for the sand during a storm, the pore pressure rises to 31.1% of the average vertical stress under the tank foundation, but that liquefaction would not occur. The procedure adopted by Bjerrum [1] considers only average conditions and ignores the actual distribution of stresses in the soil profile and the effect of pore pressure dissipation. Jeng [7] studied wave-induced liquefaction potential at the tip of a breakwater. He concluded that all the components of the diffracted wave affect the wave-induced pore pressure and liquefaction potential distribution, and that the wave incident angle directly influenced the magnitudes and distribution of the wave induced liquefaction potential. He also observed a scour zone at the tip of the breakwater and later Jeng et al. [8] experimentally studied the interaction between waves, a submerged breakwater and a vertical seawall. The results show that wave-induced pore pressure is greater beneath the submerged breakwater than below the toe. They

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http://dx.doi.org/10.1016/j.coastaleng.2017.02.002
Received 28 June 2016; Received in revised form 17 January 2017; Accepted 5 February 2017
0378-3839/

also observed that the strong interaction between both structures caused a significant change in the wave-induced pore pressure in the seabed. They concluded that an interaction of the incident waves, the reflected waves from the vertical seawall and the submerged breakwater increased the pore pressure in the seabed, and this interaction created a short wave sea state. Ulker et al. [24] investigated standing wave-induced dynamic instability of the seabed around a caisson breakwater. A seabed-rubble-caisson breakwater system was modelled, using finite elements, and the dynamic response of the foundation was analysed by determining the stress and pore pressure distributions below the breakwater. They concluded that seabed saturation increases with pore pressure in the seabed, while the shear stresses decrease. They found that while only some small areas of the seabed liquefied, the breakwater presented significant displacements and rotation, as a result, leading to failure of the whole structure. Zhang et al. [26] investigated the interaction between waves, soil and the geometry of the structure under wave-induced pore pressure, using a numerical model. They concluded that longer wave periods and greater wave heights induced higher magnitudes of pore pressure on the leeside of a breakwater. The magnitude of the pore pressure below the leeside of the breakwater decreased with an increase in structure porosity. Also pore pressure varies over time with the height of the structure. The seabed thickness, the soil permeability and the degree of saturation also affected the dynamic soil behaviour. They also found that the wave period changes the magnitude of the shear stresses around the breakwater.

Even though wave-seabed–structure interaction has been widely studied on sandy seabeds (i.e. [15] and [13]), these interactions are not fully understood in soils with high portions of fine sediment (silt and clay). There is little information about the consolidation (self-weight) and compaction (due to wave loading) processes, the soil liquefaction or the dynamics of scour (i.e. [20,13,14]). For that reason, this study investigates the wave-seabed-structure interaction experimentally, focusing on the role of the initial water content of the clayey soil. The main objective of this work was to reproduce the failure (sinking) of a structure due to liquefaction of a seabed with high clay content, which may lead to design and construction recommendations for structures built on muddy soils.

Section 2 presents a description of the experimental set-up; and tests. Section 3 addresses our findings regarding the soil-wave interaction; describes the tests focused on studying the interactions between a vertical breakwater, a muddy soil and waves; and presents the experiments carried out to reproduce the sinking of a submerged structure into the muddy bed. Recommendations for testing are presented in Section 4 and Section 5 offers the conclusions reached.

2. Experimental set-up and tests

The experiments were conducted at the Laboratorio de Costas y Puertos of the Instituto de Ingeniería, UNAM; there, a wave flume 22 m long, 0.60 m deep, and 0.40 m wide was used. The bottom of the flume features two removable bottom sections 13 m and 15.5 m from the wave paddle. The mud was placed in an acrylic pit, 20 cm deep (d=20 cm) and 84.5 cm long, in the removable section closer to the paddle (for a full description see [4]). At the end of the flume, a gravel dissipative beach was positioned.The spacial and temporal evolution of the pore pressure within the mud was registered with 28 pressure transmitters, distributed as shown in Fig. 1, which also shows the coordinate system used. Eleven wave gauges were set along the flume as shown in Fig. 2.

The mud used for the tests was made “in house”, mixing natural sand from the Mexican Caribbean and commercial kaolinite. The particle size distribution of these materials is shown in Fig. 3; these values were measured using an optical particle analyser for the coarser part and a hydrometer for the finer portion. The mechanical properties of the soil are summarized in Table 1, where ρ_s/ρ is the relative density,

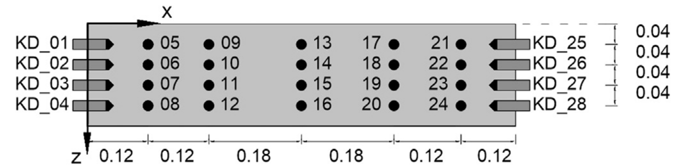

Fig. 1. Distribution of the pressure transmitters in the pit containing the muddy soil (dimensions in metres).

d_{50} the mean diameter and $e_{max,min}$ the maximum and minimum porosity values of the material in dry conditions.

For the kaolinite, the Atterberg limits (liquid, LL, and plastic, LP) and the plasticity index, IP, were determined in the laboratory, obtaining the following results: LL=34.20, LP=26.15 and IP=8.05, thus according to the Unified Soil Classification System (USCS), this material is a clay with low plasticity (CL).

2.1. Case 1: No coastal structure

To investigate the behaviour of the mud, experimental tests were carried out on five soil blends when interacting solely with monochromatic waves, that is: sand only (100 A), kaolinite only (100 C) and three mixtures: 85% sand –15% kaolinite (85A15C), 60% sand –40% kaolinite (60A40C) and 30% sand –70% kaolinite (30A70C), see Table 2. The particle size distributions of the mixtures are shown in Fig. 4.

Two experimental methodologies were developed, based on the experiments by Lindenberg, et al. [16]: Experiments I, focused on analysing the response of the wave-induced pore pressure in soils with different mud contents; and Experiments II, focused on investigating the effect of the initial water content in the soil mass. The procedure followed in these experiments is described below (the specific conditions for each type of experiment are detailed in Table 3):

1. Prepare the soil by mixing the corresponding proportions of sand and kaolinite, then add water and rub until a homogenous, saturated blend is obtained. For Experiments II, three specific solids-water concentrations were determined: 1.2 kg/L (100C-1.2, D_r=0.605), 1.5 kg/L (100C-1.5, D_r=0.797) and 1.8 kg/L (100C-1.8, D_r=0.982).
2. Fill the pit with the mixture, flush with the bottom of the flume.
3. Fill the flume with water to a depth of 0.30 m (h=0.30 m) and allow the soil to consolidate (first consolidation period).
4. Calibrate the water gauges and set the pressure transmitters to zero to directly record the excess pore pressure.
5. Switch on the waves and the measurement system (phase 1); 15 minutes after each wave condition has started, stop the measurement system.
6. Let the soil consolidate again (second consolidation period), repeat steps 4 and 5 (phase 2).

2.2. Case 2: Vertical breakwater

To analyse the interaction between mud and a vertical breakwater in the presence of waves, a small scale structure of 1.37 N was placed over the bed of kaolinite (100 C), exerting a pressure due to dead weight of 7.80 kN/m^2. The dimensions of the model were 44 cm length and 50 cm height. A 15 cm high gravel layer held in place by a rectangular mesh box represented the rubble mound foundation of the breakwater. The superstructure was formed by a 35 cm, impermeable wood case filled with concrete cubes. The leeward side of the structure was aligned with the border of the pit, as shown in Fig. 5.

100 C mud was selected after determining, in the wave-soil interaction tests, that 1.8 kg/L was the threshold solid-water concentration for liquefaction. However, intending to have a mud able to bear the weight of the scaled vertical breakwater, a lower initial water content in the mud had to be tested. The mud was made for the test with 3.1 kg/L