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Seismic response of hunchbacked block type gravity quay walls

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ABSTRACT

Earthquakes near major cities may cause big social and economic impacts. Damages to port facilities may cripple the economy. The past twenty years' experience has proven the high vulnerability of the port facilities. This fact, along with the economic importance of port structures, indicates the need for better seismic design approaches for berth structures and cargo handling facilities. In the recent decades, there have been many incidences of failure of gravity type quay walls. These failures have stimulated research interest in the development of performance-based design methods. In this paper, two different hunchbacked block type quay walls with different back face shape were studied. A series of 1-g shaking tank tests was performed using a 1/10 scaled block type quay wall with gravel backfill materials on firm non-liquefiable sea bed conditions subjected to different harmonic loads. The shaking tank tests provided insight into the wall displacements and the total dynamic pressures by analyzing pressure components at the contact surface between the saturated gravel backfill soil and the wall. It is concluded that the back-face shape of the walls is an important factor and the larger positive slope of the wall improves the overall seismic stability.

1. Introduction

Ports are the main components of maritime transport and they have an important role on commercial transport, hence any damage level is undesirable. Most of the port structures have been located in highly seismic regions and the supporting quay walls are also subjected to earthquake loadings in addition to water wave action and vessels berthing loads. Therefore, the performance of existing port structures should be checked on the basis of earthquake hazard.

Seismic risks at ports have not received sufficient attention and only a limited number of studies has been carried out for the assessment of block type quay walls, which are widely preferred in most of the ports. Yuksel et al. [17] documented and discussed the distribution and the extent of damage and serviceability of marine structures after 1999 Kocaeli Earthquake (M_w = 7.52). The effects of earthquakes, including severity of damage, service losses, and environmental impact at petrochemical facilities, were severe and extensive. Sumer et al. [14] presented a state-of-the-art review of seismic-induced liquefaction with special reference to marine structures.

The seismic response of a port structure is affected by the interaction of the structure with the surrounding and underlying soil, and water. This effect, widely referred to soil-structure-water interaction (SSWI), is a rather complex phenomenon and involves a number of difficult-to-assess problems. One basic problem is the change in amplitude and frequency content of seismic waves when they interact with an inclusion in a propagation medium. This kinematic interaction is initiated when incident seismic propagation away from the causative fault and through the geologic media encounter a structural element or foundation element whose inertial and stiffness characteristics differ from those of the surrounding soils. As these incident ground waves hit the structure-foundation, they are both reflected and refracted. The resulting transmitted waves are the source of inertial interaction and generate inertia forces by exciting the overlying structure, which further alter the motions of the foundation and the surrounding soil (ASCE, 1998).

Researchers have focused on seismic performance of waterfront structures for longer than a decade and a number of research studies have been carried out both experimentally and numerically. It is very important to learn the lessons from past case studies to better understand the vulnerability of waterfront structures exposed to earthquake.

Experimental and/or analytical studies by Miura et al. [9], Fujiwara et al. [3], Mohajeri et al. [10], Mendez et al. [8] and Nakahara et al. [11] were presented to assess the dynamic response of gravity type quay walls. Additionally, Inoue et al. [5], Kim et al. [7], Kim et al. [6], Towhata et al. [16] and Yuksel et al. [18] approached the problem through experimental and/or numerical approaches. There exist also purely numerical studies performed by Alyami et al. [1], Arablouei et al. [2] and Tiznado and Rodriguez-Roa [15]. Most of these studies

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were focused on understanding the response of caisson type quay walls. Hence, in the literature there exists a gap in understanding the seismic performance of block type quay walls. Sadrekarimi et al. [13] investigated the static and dynamic behavior of hunchbacked gravity walls by considering back-face shape of wall. Sadrekarimi [12] also studied the seismic performance of gravity type broken back quay walls through 1 g shaking table model experiments and proposed a simplified sliding block analysis model for estimating lateral displacements, which is calibrated with the experimental results. However, the blocks had shear keys at the top and bottom surfaces to prevent relative sliding.

A contemporary design philosophy for port structures in seismically active regions is expected to suggest assessment methodologies for the estimation of seismically induced foundation, backfill and wall deformations along with the stresses acting on them. Unfortunately, conventional (force-balance) methods are not well suited to fulfill these expectations. While performance-based design procedures attempt to assess the deformation and stress demand and capacity of the systems however appropriate earthquake performance levels needed to be defined along with acceptable block type quay wall damages. Despite their wide use as a quay wall, in the literature there exists a gap on acceptable performance criteria.

This study attempts to assess the static and dynamic performance of the block type quay walls in the form of lateral displacement and tilting as well as settlement of the backfill and is hoped to contribute to close this gap.

In this study, reduced scale models of two different hunchbacked quay walls with different back face shapes were prepared in 1-g shaking tank. The scale ratio of model to prototype was selected as 1/10. Tests were performed on firm bottom conditions and a dense backfill material was used for the purpose of fully concentrating on the response of quay wall and eliminating the effects of soil liquefaction on the overall response.

2. Experimental study

Due to the fact that gravity type water-front structures are usually long, one-dimensional dynamic assessments are widely used to understand the overall response of these structures. 1-g model tests with a 1/10 scale ratio were performed with the similitude of various parameters as recommended by [4] and given in Table 1, for a soil-structure-fluid system.

The experimental study was conducted in a shaking tank at Hydraulic and Coastal Engineering Laboratory of Yildiz Technical University. As shown in Fig. 1, the shaking tank is 4.5 m long, 1 m wide and 1 m high with a glass side to monitor the overall response. The tank was divided by a steel plate in the longitudinal direction and 0.38 m wide models were centered in the tank as shown in Fig. 1. The shaking tank was steered by PLC (Programmable Logic Control). The amplitude and frequency ranges of the tank is 1–5 mm and 1–9 Hz, respectively. Series of calibration tests were performed to develop the optimum amplitude and frequency pair to attain reliable and robust response.

For the purpose of investigating the performance of hunchbacked block type gravity walls, two different models; i) first type hunchbacked wall (FHW), and ii) second type hunchbacked wall (SHW), were prepared as shown in Fig. 2. Both models consist of 6 pieces of concrete blocks with a constant block height (h) of 100 mm, width (w) of 250 mm and varying breadths (B) of different aspect ratios designed without shear keys in between blocks. The properties of the concrete blocks are listed in Table 2. Two different overall geometries were obtained by placing six blocks in a different pattern. For the FHW model the breaking point is lower than that of the SHW as illustrated in Fig. 2. The model blocks were founded on an 80 mm thick, tamped gravel seabed floor and numbered from 1 to 6 starting from the bottom block.

Table 1
Similitude for the 1-g shaking table model [4].

	Prototype/model	Scale factor
Length	λ	10
Time	$\lambda^{0.75}$	5.62
Acceleration	1	1
Displacement	$\lambda^{1.5}$	31.62
Water Pressure	λ	10
Density	1	1
Stress	λ	10

During the test preparation, utmost attention was given to simulate plane strain conditions. For the purpose of eliminating the effects of tank boundaries on the overall response, models were placed in a plexi-glass container, which is fixed to the shaking tank with a gap of 1 mm between the model blocks and the plexi-glass side walls.

With the aim of monitoring the performance of the models, series of transducers were installed as shown in Tables 3, 4, and Fig. 2. One accelerometer was placed on the tank, just at the bottom of the models and six accelerometers were fixed onto each block. Six earth pressure and wire type displacement transducers were installed in the back and front sides of the model blocks, respectively. One displacement transducer (DP 7) was installed on top of the sixth block to measure tilting of the wall. Two pore water pressure transducers were embedded in the backfill behind the models at two different locations. The transducers were mounted on each model block, and subsequently the backfill was filled behind the model walls by using pluviation device as shown in Fig. 3. The total mass of the backfill was 810 kg.

On the basis of the fact that foundation and backfill soil properties govern the overall response along with the properties of the quay wall, special attention was paid to produce models with desired soil properties. Hence, a series of laboratory tests were performed on foundation and backfill soils to identify their geotechnical engineering properties and the results are summarized in Table 5.

The relative density (D_r) of the backfill material was kept constant for each test. To assure a homogeneous relative density, a pluviation device was used. This automated raining crane system is controlled by a digital computer. The device can move both in the vertical and horizontal directions at each pluviation cycle. As shown in Fig. 3, the beam provides the horizontal mobility of the bunker which was connected to the reaction beam by four columns. The backfill material was loaded to the bunker via a conveyor belt as shown in Fig. 4. The relative density of the backfill was selected as 70% for all tests and it is achieved by controlling the discharge cap space, height of the bunker and the speed rate of the horizontal mobility of the bunker.

Following the placement of the model blocks and transducers, the backfill material was poured. Then, water was percolated through the bottom of the tank at a very slow rate to avoid boiling and piping induced problems. The maximum free water surface elevation was selected as 68 cm relative to the tank base and backfill soil was fully saturated. For settlement evaluations, the backfill free surface before and after the shaking was measured and recorded using HR Wallingford Touch-Sensitive Two-Dimensional Profiler.

Before shaking, the static earth pressures and hydrostatic pressures were monitored to assess the static stress state of the model. Then, four regular harmonic input motions with frequencies ranging from 3 to 7 Hz were applied in the longitudinal direction for 20 s. The characteristics of the harmonics were listed in Table 6. The variations of accelerations, dynamic earth pressures, pore water pressures and displacements were recorded throughout shaking. Once the shaking was ceased, the backfill free surfaces were monitored again for settlement assessment purposes.

For the purpose of assessing the performance of the gravity type quay walls, the dimensional parameters are determined as follow:

$$F(\rho_s, d_{50}, H, B, \rho_c, \rho_w, a, g, D, Z_B, \theta) = 0 \quad (1)$$

where; D is the maximum residual displacement of the block, B is the block breadth, H is the wall height, ρ_c, ρ_s and ρ_w are the densities of the