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ABSTRACT

Recent earthquakes have highlighted many seismic hazard concerns for western U.S. ports. Port waterfront structures are commonly constructed utilizing pile-supported wharves in combination with rock dike structures retaining a hydraulically placed backfill. Seismic damage is generally attributed to weak soils that are often prevalent in the marine environment (e.g. liquefiable sands, sensitive cohesive soils). In response to past damage, many ports are instigating soil improvement strategies to eliminate or minimize potential occurrences of liquefaction and to increase the strength of cohesive soils. The design of a seismically resilient wharf requires an understanding of its performance during design level earthquakes. Due to the complex nature of pile-supported wharves, state-of-the-art centrifuge modeling techniques are being used to better understand their seismic performance. The authors used the large-scale centrifuge facility at the University of California at Davis. This paper presents details on the construction, instrumentation, and testing of the models. Results from the tests are also included, such as the seismic pile behavior, effect of soil improvement, and the overall behavior.

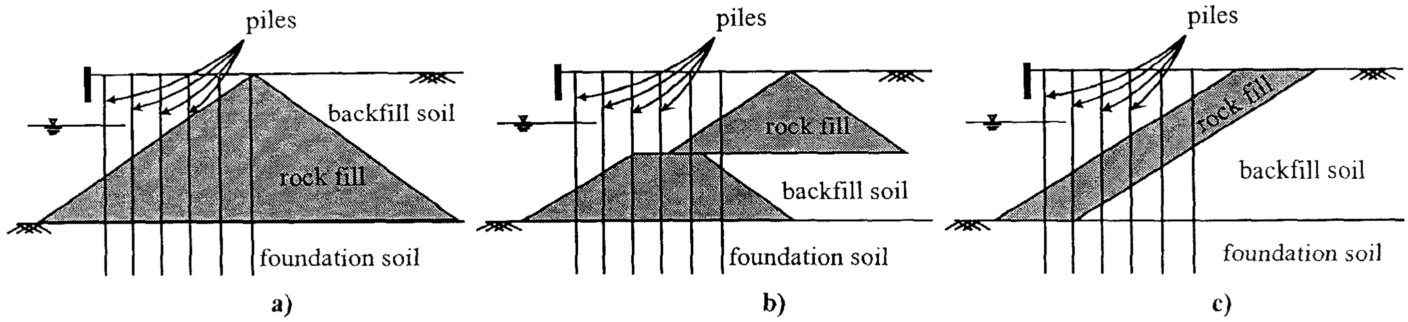
INTRODUCTION

Pile-supported wharves and rock dikes are commonly used in the construction of port facilities, especially in land reclamation projects for new port construction. The construction process consists of building a rock dike (commonly a single lift, multi-lift, or sliver configuration) that is used as a retaining structure for the backfill soils (commonly hydraulically placed sands). After the backfill soils have been placed to grade, construction equipment moves onto the reclaimed land, where the piles are jetted and/or driven to depth, after which the wharf deck and pavement section are constructed. Typical pile-supported wharf geometries are shown in Fig. 1. These structures are economically feasible in regions were land is being reclaimed (since they require less fill than a typical sheet pile or cellular bulkhead), and they have generally performed well during earthquakes. When poor seismic performance has occurred, it has usually been attributed to ground failures associated with the weak soils that are often prevalent in the marine environment (e.g, liquefiable sands, sensitive cohesive soils).

In response to the historical damage caused by weak soils, many port authorities are instigating soil improvement programs for both new construction and in the rehabilitation of existing structures. The soil improvement strategies generally utilize densification techniques (e.g. vibro compaction, stone columns, etc.) for cohesionless soils and cementatious techniques (e.g. cement deep soil mixing) for cohesive soils. Unfortunately, as can be seen in Fig. 1, it may be very difficult, as well as economically unfeasible, to improve the backfill soil directly beneath the rock dike for the multi-lift and sliver geometries.

The current standard-of-practice for the design of port structures (and their remediation) typically utilizes traditional limit-equilibrium methods, whereas more appropriate performance-based design methods are generally not used due to the lack of available guidelines. The deficiencies in the limit-equilibrium methods are compounded by the fact that many port authorities are developing performance criteria based on allowable deformations. However, it is generally acknowledged that limit-equilibrium methods are not well suited for establishing whether seismically induced deformations of waterfront structures will be within the specified limits.

In moving from a limit-equilibrium method of design to a performance-based method, there is a need to better understand the seismic performance of pile-supported wharves. Their performance can be estimated through a comparison to past performance (which there are only a limited number of pile-supported wharf seismic case histories), or through the use of modeling (either physical and/or numerical). The approach of the authors has been to use the limited case history data, in addition to testing physical models, to develop a database of pile-supported wharf performance for use in validating a numerical model. The



a) single-lift rock dike, b) multi-lift rock dike, and c) sliver rock dike.
Fig. 1. Typical pile-supported wharf geometries; a) single-lift rock dike, b) multi-lift rock dike, and c) sliver rock dike.

physical models are advantageous in examining the pile performance and in examining the overall behavior of pile-supported wharves. The numerical model is advantageous for running *what if?* scenarios that compare the performance of different geometries, as well as determine the incremental benefit of soil improvement. Through the use of both physical and numerical modeling, it is possible to better understand the complex behavior of pile-supported wharves. The data presented within this paper is only the physical modeling portion of the authors' research, while the subsequent numerical analyses have yet to be completed.

Some of the issues that the authors are addressing with the physical models include: What is the soil-structure interaction behavior of piles in a sloping rock fill?; What is the soil-structure interaction behavior at soil layer interfaces?; What is the effect of soil improvement?; and What is the overall deformation behavior of pile-supported wharves?

The study discussed within was conducted to address these questions, using the most advanced physical modeling research equipment and methods currently available. Only one centrifuge model is discussed in detail within, while all three models of this test series have been completed. Even though only one model is discussed, all models followed the same general construction and testing procedures. It should also be noted that the presented results are in prototype (full-scale) units, unless noted otherwise.

CENTRIFUGE FACILITY

The authors utilized a centrifuge for the physical modeling portion of the study due the ability of a centrifuge to correctly represent in situ stresses at a model scale. The centrifuge utilized was the large-scale centrifuge facility at the University of California at Davis, which is currently one of the largest centrifuges in the world, having a radius of 9.1 m, payload mass of 4500 kg, and capable of spinning at approximately 40 g centrifugal acceleration. The facility has a shake table which is placed between the centrifuge platform and the model container that is capable of applying dynamic shaking to the model while the centrifuge is spinning at the desired centrifugal acceleration. A complete description of the centrifuge and shake table is given by Kutter et al. (1991) and Kutter et al. (1994).

A flexible model container was used for this study, with the inside dimensions being approximately 1720 mm long by 702 mm deep by 685 mm wide. In order to reduce the boundary effects at the edge of the model container, the container was designed such that the shear modulus in the direction of shaking is approximately equal to that of a liquefied soil deposit. The container consists of six layers of hollow aluminum rings separated by layers of soft rubber, mounted on a solid aluminum base plate (Fig. 2).

SCALING RELATIONSHIPS

The model was scaled using both centrifugal and geometric scaling relationships. The geometric scaling was performed because the centrifugally scaled model was too large to fit within the flexible container, and thus necessitated the additional geometric scaling. The scaling factors are n (centrifugal acceleration of the model) for the centrifugal scaling and λ for the geometric scaling. Both factors refer to the ratio of the prototype (full-scale) dimension to model dimension. Equations (1) and (2) give the approximate scaling factors used for this model.

$$n = \frac{\text{prototype}}{\text{model}} \cong 40 \tag{1}$$

$$\lambda = \frac{\text{prototype}}{\text{model}} \cong \frac{1}{0.7} \tag{2}$$

Table 1 provides the centrifugal and geometric scaling relationships of interest for this study.

It can be noted in Table 1 that the time scaling for dynamic time can be different than that for the fluid flow (diffusion) for both the centrifuge and geometric scalings. In order to achieve an equal time scale for the centrifuge scaling, the coefficient of consolidation (c_v) of the model has to be n times less than that of the prototype For the geometric scaling, the coefficient of consolidation of the model has to be $\lambda^{1.5}$ times less than that of the prototype.

Table 1. Scaling relationships.

Quantity	Centrifuge Scale Relationship	Geometric Scale Relationship
Acceleration	$1/n$	1
Velocity	1	$\lambda^{1/2}$
Length	n	λ
Time (Dynamic)	n	$\lambda^{1/2}$
Time (Diffusion)	$n^2 \cdot (c_v^*)^{-1}$	λ if $c_v^*=\lambda$ $\lambda^{1/2}$ if $c_v^*=\lambda^{1.5}$
Mass Density	1	1
Mass	n^3	λ^3
Force	n^2	λ^3
Stress	1	λ
Pile Stiffness (EI)	n^4	λ^5
Moment	n^3	λ^4

$$c_v^* = \left(\frac{c_{v\text{-prototype}}}{c_{v\text{-model}}} \right)$$

Since the coefficient of permeability and viscosity of the fluid are inversely related, the viscosity of the model fluid was increased in order to decrease the coefficient of permeability (which is directly proportional to c_v) and to provide dynamic and diffusion time scales that are approximately equal. The increased viscosity was accomplished by mixing the organic compound, hydoxy-propyl methylcellulose (HPMC), with benzoic acid and de-ionized water. The benzoic acid was added as a preservative to postpone the decomposition of the HPMC fluid. Stewart et al. (1998) discuss the details on using an HPMC fluid as a viscous pore fluid in centrifuge experiments. Dewoolker et al. (1999) have highlighted the importance of scaling pore fluid viscosity in the dynamic centrifuge modeling of saturated soils.

TEST GEOMETRY

To date, three pile-supported wharf centrifuge models have been tested, with each model having a slightly different geometry. A complete description of each test may be found in the data reports by McCullough et al. (2000) and Schlechter et al. (2000,a; 2000,b). These data reports also include the complete set of recorded and reduced data. Only the most recent model will be discussed herein, which has been designated as SMS01 (Schlechter et al., 2000,b).

The geometry of the model was based on typical geometries of pile-supported wharf structures at western United States ports. The model geometry is shown in Fig. 2. The prototype piles were 24 inch (61 cm) diameter prestressed, reinforced, octagonal, concrete piles, which were modeled using 3/8 inch diameter (9.5 mm) aluminum tubing. There were 21 piles in the model, three sets of seven rows (Fig. 2 and Fig. 3). The model piles spacing was 3.5 inch (89 mm) by 4 inch (102 mm). The stiffness and the diameter of the piles was scaled, but it was not possible to also scale the strength of the piles using readily available aluminum tubing. Therefore the

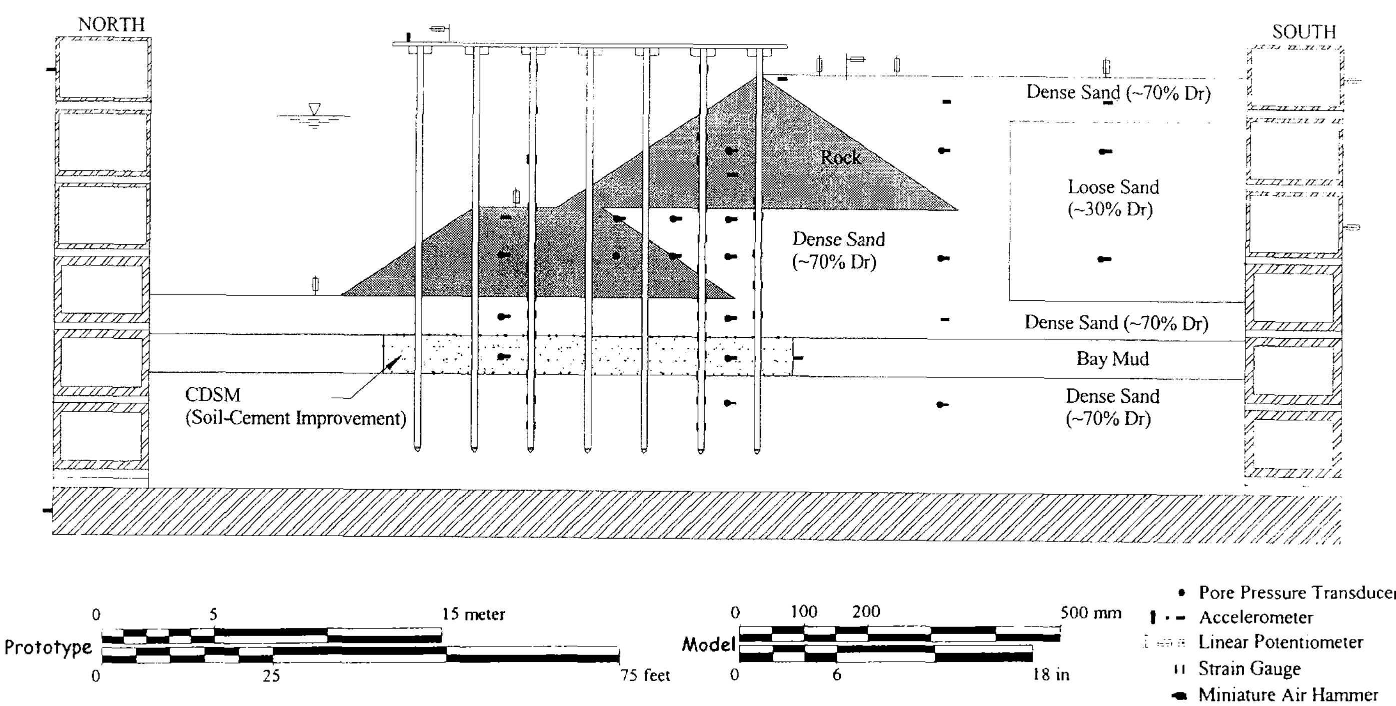


Fig. 2. Cross-section view of the container, model geometry, and instrument locations.

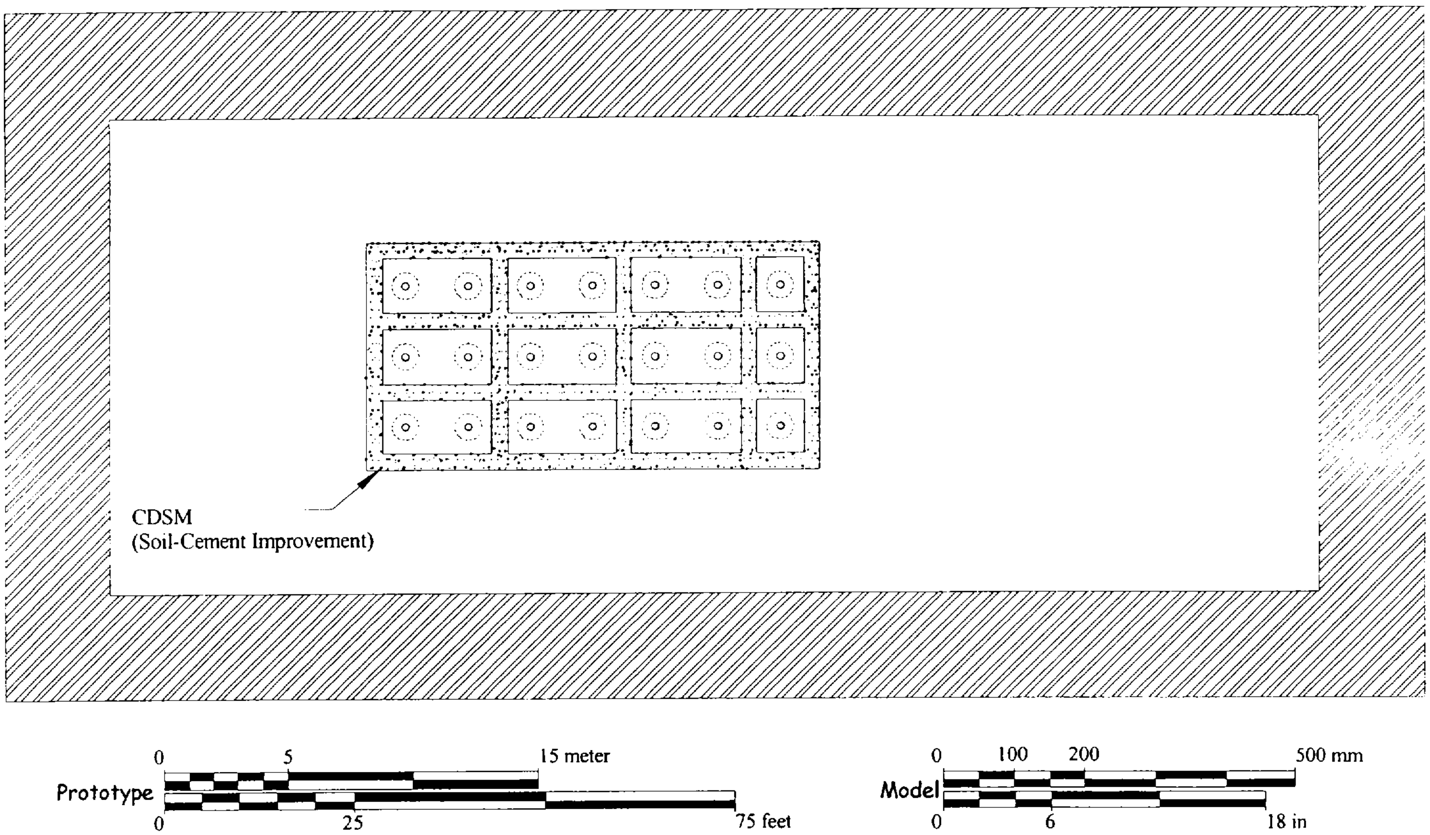


Fig. 3. Plan view of the model showing the container, location of the clay soil improvement, piles and pile caps.

strength of the scaled model piles was much greater than that of the prototype piles. For example, the scaled plastic moment of the aluminum tubing is approximately 7.5 MN-m, while the plastic moment for a 24 inch (61 cm) reinforced prestressed concrete pile typically ranges between 0.6 to 1 MN-m. The wharf deck is commonly a cast-in-place reinforced concrete deck, overlain with aggregate fill and a pavement section. The modeled wharf deck was a ¼ inch (6.4 mm) solid aluminum sheet (622 mm x 306 mm), scaled to match the mass of the prototype wharf deck. The piles were attached to the wharf deck using pile caps (38 mm diameter, 13 mm thick), which were locked onto the top of the piles and then bolted directly to the wharf deck. This was assumed to provide a fixed moment connection.

The aggregate material for the rock dike was obtained from Catalina Island, the same quarry that is used to supply rock to the majority of construction efforts at the Ports of Long Beach and Los Angeles. The diameter of the larger particles (12 inch (30 cm) nominal diameter) was scaled to accurately represent the pile-rock interaction, yet the finer particles were not scaled so that the pore pressure dissipation and fluid flow would be modeled accurately. Nevada sand was used in the model to represent the loose and dense sand material. Both the dense and loose sands were deposited using air pluviation techniques. Reconstituted San Francisco Bay Mud was used as the clay material.

The model also consisted of several regions of soil improvement. The loose sand beneath and adjacent to the upper rock dike was placed at a relative density of 70 percent, whereas the remaining backfill had a relative density closer to 30 percent (Fig. 2). In addition, the clay layer was improved in the region surrounding the piles. The clay improvement consisted of mixing the clay with cement, typical of CDSM (Cement Deep Soil Mixing) soil improvement techniques. The improved clay region consisted of a gridded pattern in plan view, as shown in Fig. 3. The soil-cement mixture consisted of 13.7 percent cement by total weight, mixed with the clay slurry at a water content of 133 percent. The soil-cement mix design was based on what was used during the Berth 55/56 expansion project at the Port of Oakland. Details of the CDSM modeling procedures are given by Schlechter et al. (2000,b)

MODEL CONSTRUCTION

The model construction sequence consisted of twelve stages, with the instrumentation, noted in Fig. 2, being placed throughout the construction process. Nevada sand dyed black with India Ink was placed throughout the model as horizontal layers and vertical columns to provide an indication of the model deformations, which could be determined by comparing the location of the black sand during construction with the location during dissection.

The stages of construction were as follows:

1) Placement of the lower dense sand. This layer was placed at a relative density of approximately 70 percent.

2) Saturation of the lower dense sand with de-ionized water. It was necessary to saturate this layer before it was sealed with the clay. Water was used as a pore fluid due to the disadvantage of HPMC decomposing during the remaining duration of the model construction and testing.

3) Placement and consolidation of the clay, which was placed in two equal sub layers, with each sub layer being separated by filter paper that was used as a horizontal drainage layer to expedite the dissipation of pore pressures. The clay was consolidated to a pressure equal to the prototype thickness of the backfill.

4) The CDSM material was placed in the indicated gridded pattern. The unconfined strength of the CDSM material at 7 days (the approximate day of centrifuge testing) ranged between 90 and 160 psi (0.6 and 1.1 MPa).

5) Placement of the middle dense sand at a relative density of approximately 70 percent.

6) The piles were driven to depth using a driving template.

7) The lower rock dike was placed at a dry density of approximately 100 pcf (15.7 kN/m^3).

8) The lower backfill dense and loose sands were placed at relative densities of approximately 70 and 30 percent, respectively.

9) The upper rock was placed at a dry density of approximately 100 pcf (15.7 kN/m^3).

10) The upper backfill dense and loose sands were placed at relative densities of approximately 70 and 30 percent, respectively.

11) The upper dense sand was placed at a relative density of approximately 70 percent.

12) The model was vacuum saturated with the de-ionized water mixed with 1.9 percent by total weight HPMC and benzoic acid. This HPMC fluid had a viscosity of 47 cSt (4.7×10^{-5} m^2/s), which decreased to approximately 30 cSt (3.0×10^{-5} m^2/s) after it was diluted with the de-ionized water that was introduced in Step 2. The viscosity remained constant at 30 cSt (3.0×10^{-5} m^2/s) throughout the testing.

INSTRUMENTATION

The performance of the model was monitored using 89 instruments. These included: 30 accelerometers to measure the accelerations within the model, on the wharf structure and on the container; 32 complete strain gauge bridges, which were calibrated to measure the bending moments of the piles; 9 linear potentiometers to measure the displacements of the soil, wharf, and container; and 18 pore pressure transducers to measure the pore pressures within the model. In addition, there were three miniature air hammers placed within the model which were used to generate shear waves for use in measuring the shear wave velocity of the soil while the centrifuge was in flight. A description of the miniature air hammers is given by Arulnathan et al. (2000).

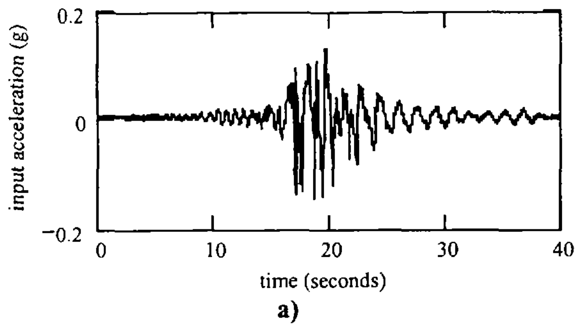
The approximate elevation locations of the instrumentation is shown in Fig. 2. It should be noted that during the testing sequence, the instrumentation may have been shifted from its initial location due to permanent soil displacements.

EARTHQUAKE TESTS

The test program consisted of spinning the model at approximately 40 g centrifugal acceleration, and applying an earthquake motion time history through the shake table. Actual recorded earthquake motion time histories were used for all but one of the small shakes. The frequency content of the recorded motions was adjusted slightly due to limitations of the shake table. The testing sequence consisted of three small motions (peak input accelerations less than 0.05 g) to examine the small strain performance of the structure, and to examine the performance of the data acquisition system. Five large motions were then applied to the model (peak input accelerations greater than 0.4 g). Due to difficulties, the data from two of these tests was not obtained. One small shake motion was a simple step wave, while the other earthquake motions were scaled versions of two different recorded acceleration time histories. One motion was from the 1989 Loma Prieta Earthquake, recorded at the Port of Oakland Outer Harbor. The other motion was from the 1994 Northridge Earthquake, recorded at the Ribaldi station. The testing sequence is shown in Table 2, while the recorded acceleration time histories are shown in Fig. 4 (it should be noted that these acceleration time histories were scaled differently for each test).

Table 2. Test sequence.

Test Number	Earthquake Motion	Maximum Base Input Acceleration (g)
1	Step wave	0.03
2	Loma Prieta	0.04
3	Northridge	0.03
4	Loma Prieta	0.42
5	Northridge	no data
6	Northridge	no data
7	Loma Prieta	0.42
8	Northridge	0.54



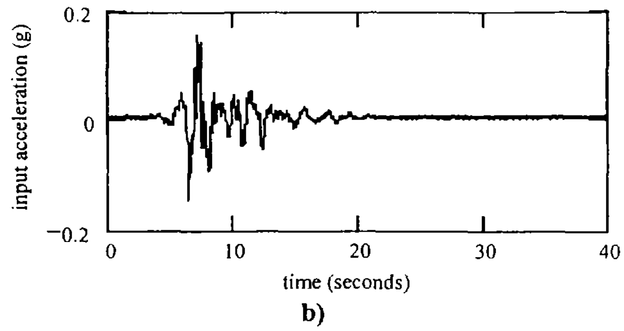


Fig. 4. Input acceleration time histories: a) 1989 Loma Prieta – Oakland Outer Harbor, Port of Oakland, and b) 1994 Northridge Earthquake – Ribaldi station.

TEST RESULTS

After the sequence of tests (Table 2), the raw data for each test was processed and converted to prototype units. There are many items that may be examined from this test sequence, as may be noted by the complexity of this model. Three general types of performance are presented in this paper; 1) performance of the piles, 2) effect of soil improvement, and 3) performance of the pile-supported wharf.

Pile performance

The performance of the piles can be characterized by the moments developed in the piles during the earthquake motion. In the design of pile-supported wharves, it is desirable to keep the pile moments less than the plastic moment to prevent excessive deformations and loss of capacity. This is especially true at depth beneath the soil surface, since it is very difficult to examine, and if necessary, repair piles at depth. The moment data from test number 4 (Loma Prieta with a peak input acceleration of 0.42 g, and a peak ground surface acceleration of 0.40 g) for two of the three instrumented piles has been plotted in Fig. 5 for model SMS01. The moments are plotted at three different snapshots in time; 1) 10 seconds, corresponding to the time before the strong shaking, 2) 20 seconds, corresponding to a time during the strong shaking, and 3) 30 seconds, corresponding to a time after the strong shaking had occurred.

As noted earlier, typical plastic moment capacities of 24 inch (61 cm) reinforced prestressed concrete piles are in the range of 0.6 to 1.0 MN-m. It can be seen in the figure that 0.6 MN-m is reached and exceeded after 20 seconds at several locations, both near the pile top and near the pile toe. The locations of large moments within the soil profile correspond to locations near soft-stiff soil interfaces.

Effect of soil improvement

Soil improvement is utilized in order to increase the soil strength and/or reduce the susceptibility to liquefaction. Figure 6 shows the benefit of soil improvement as related to the decrease in excess pore pressure generation (for the same test and earthquake as discussed in the pile performance section). The unimproved sand reaches a state of full liquefaction, whereas the improved soil, at the same elevation within the model, has much less pore pressure generation. Pore pressure generation and dissipation lead to volumetric strain of the soil, which in turn leads to ground surface settlement. Figure 7

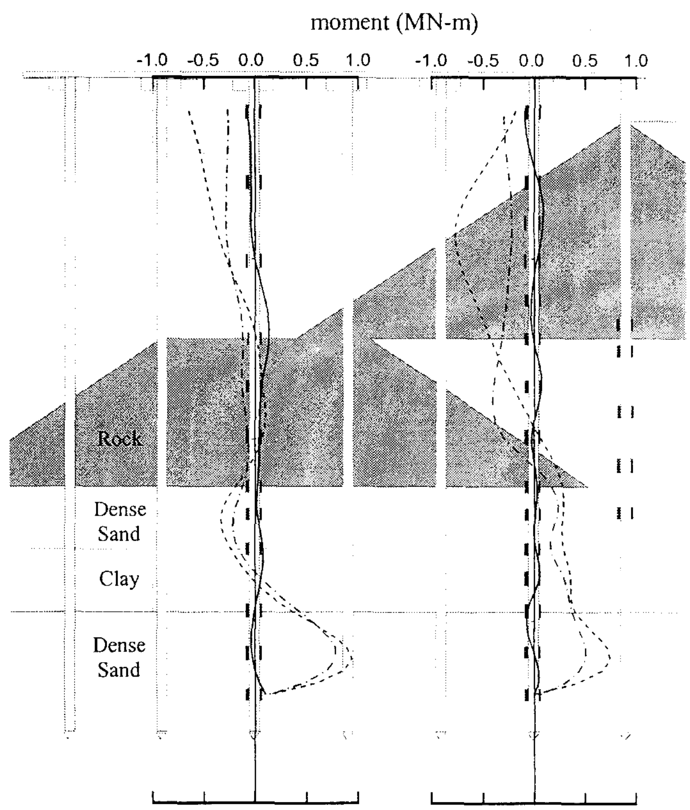


Fig. 5. Pile moments developed at three different times from test number 4. Typical plastic moment capacities range from 0.6 to 1 MN-m for the prototype concrete piles.

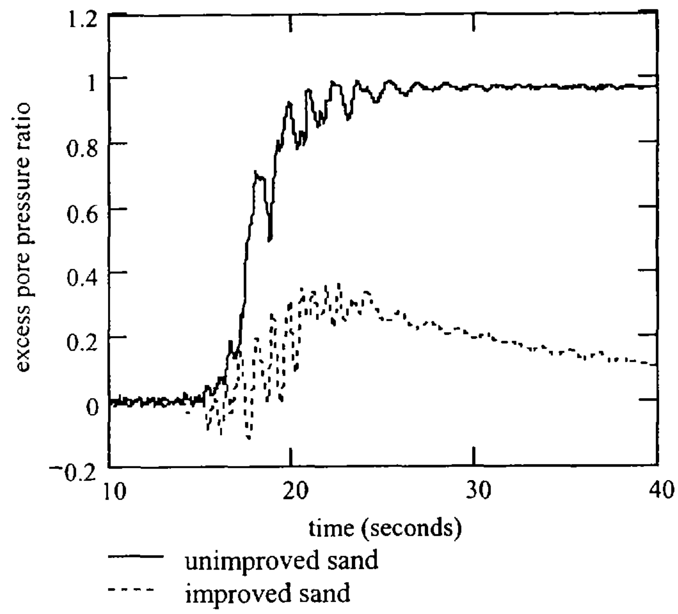


Fig. 6. Excess pore pressure ratio (ratio of the excess pore pressure to the effective overburden stress) comparison between the improved and unimproved sand from test number 4.

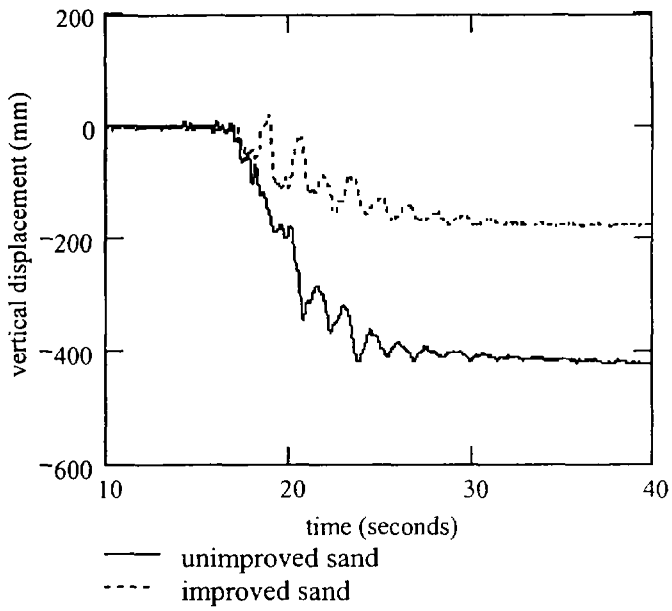


Fig. 7. Vertical displacement at the ground surface comparison between the improved and unimproved sands from test number 4.

shows the subsequent difference in settlement at the ground surface above both improved and unimproved soils. It can be noted from this figure that the ground surface settlements above the improved zone are approximately half those above the unimproved zone.

Performance of pile-supported wharves

Figure 8 shows the performance of pile-supported wharves, characterized by the permanent lateral deformations and peak ground surface accelerations. The data in this figure is from this model (SMS01), with the addition of two previous centrifuge models (NJM01 and NJM02). The geometries between the three studies were generally the same, except for the following; NJM01 did not include the layer of clay, and neither NJM01 nor NJM02 included soil improvement. It should also be noted that the initial relative density of the backfill for each of the three models also varied, as indicated in the figure legend. Although for SMS01, the backfill was composed of approximately half 30 percent and half 70 percent relative density sand (Fig. 2).

The lines connecting the points in Fig. 8 indicate the time sequence of the tests. The points hovering around zero are the initial small shakes, while the larger values are the larger shakes. The decreased lateral deformation, for the same or greater peak ground accelerations, after the first several large shakes indicates the incremental densification of the loose sands within the model and the greater liquefaction resistance.

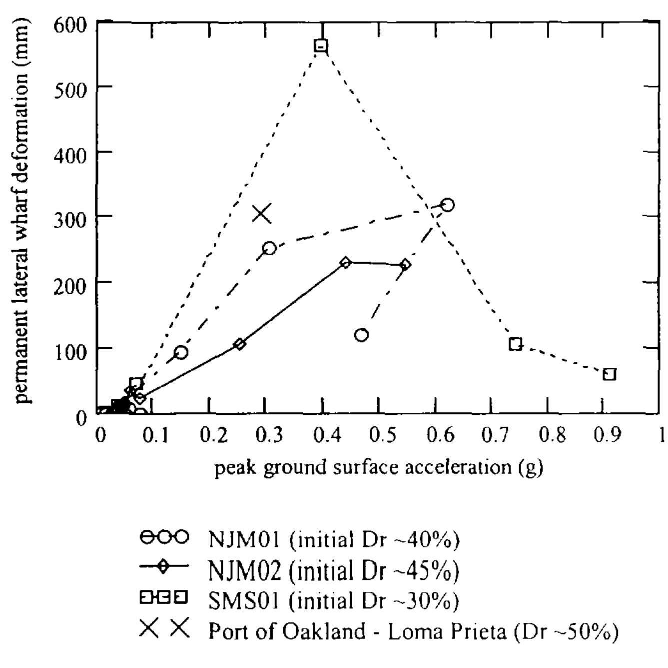


Fig. 8. Relationship between the peak ground surface acceleration and the permanent lateral deformations of the wharf deck for all centrifuge tests, including data from the Port of Oakland 7th Street marine Terminal during the 1989 Loma Prieta Earthquake.

The single point from the Port of Oakland from the 1989 Loma Prieta Earthquake indicates the performance of the 7th Street Marine Terminal (Egan et al., 1992). The 7th Street Marine Terminal consisted of a single-lift rock dike (in comparison to the multi-lift rock dike of these studies) and included a row of battered piles (in comparison to the all vertical pile system that was modeled). However the data point seems to indicate the same relative deformation-acceleration trend as that of the centrifuge models. It should also be noted that the majority of the battered piles failed at the pile-wharf deck connection during the earthquake.

SUMMARY

Pile-supported wharves are very complex structures, involving soil-structure interaction, as well as the independent behavior of the piles, wharf, and soils. Typical limit-equilibrium methods of analysis are often inadequate at obtaining the overall seismic performance of the system. Therefore, state of the art centrifuge modeling has been utilized to more accurately model these structures, and to provide a method for validating numerical models. This paper has presented the design and construction methods that were used during these tests. Results have been provided that show the performance of the piles, the effect of soil improvement, and the overall performance of pile-supported wharves. It is anticipated that these models will contribute to a more accurate assessment of the seismic performance and facilitate the development of performance-based design methods for pile-supported wharves.

The results presented in this paper are only a very small portion of the collected data. The entire data set for all models can be found in McCullough et al. (2000) and Schlechter et al. (2000,a; 2000,b). There are many aspects of these data sets still being examined, including dynamic p-y behavior and numerical validation. The authors are also currently investigating the seismic behavior of pile-supported wharves which incorporate battered piles through the use of additional centrifuge and numerical modeling.

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