

CHAPTER 154

Verification of the Analytical Model for Ocean Wave-Soil-Caisson Interaction

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Abstract

An analytical model for wave-soil-caisson interaction is verified by comparison with a finite element model and large scale experimental results. The analytical and finite element model estimates of the stresses and surface displacements of an elastic layer of finite thickness are in good agreement. The physical model experiments were conducted at the O.H. Hinsdale Wave Research Facility at Oregon State University. A 10-ft high, 8-ft long caisson was constructed on a bedding layer overlying 1 to 3 ft of soil. The caisson was exposed to waves with heights of 0.68 to 4.4 ft with periods of 1.77 to 8.80 sec. Experimental and analytical comparisons for porewater pressure were in agreement but the displacements were quite scattered.

1. Introduction

Caissons on permeable bases have been designed and constructed for a variety of needs in coastal and offshore engineering. An evaluation of the adequacy of the foundation beneath the structure is required for an economic and safe design. Recently, an analytical solution (Tsai et al., 1986) has been developed to model the wave-induced displacements, stresses, and porewater pressure under caissons. Before this solution can be used in engineering practice, it must be verified. Therefore, both a finite element model and a physical model are used to verify this analytical model.

2. Finite Element Model

The finite element model developed by Milovic et al. (1970) predicts the stresses and displacements in an elastic soil of finite thickness. The applied load is inclined and eccentric over a rigid strip. This configuration is shown in Fig. 1, in which F is the applied load, e is the eccentricity, θ is the inclined angle of the load, $2c$ is the width of the strip, and d is the thickness of the elastic layer. The strip foundation is perfectly rigid and has a rough base. The soil is assumed to be an isotropic, homogeneous, linearly elastic layer bounded by a plane horizontal boundary above and by a rigid base below. This numerical model corresponds to the radiation problem in the analytical model.

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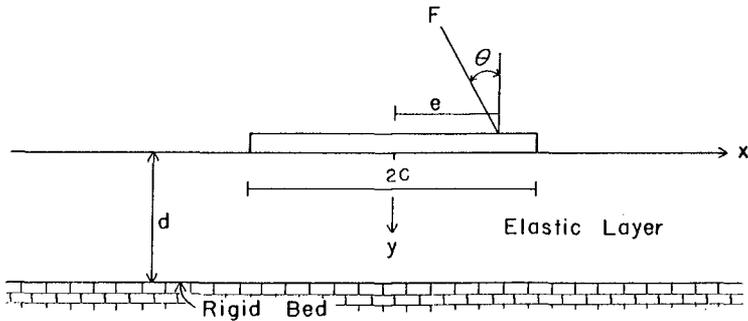


Figure 1. Definition sketch for a loaded infinite long rigid strip overlying an elastic layer

In this comparison the stresses are scaled by the averaged load per unit area and the displacements and thickness of the elastic layer are scaled by the block half-width. The comparison of contact stresses is presented in Fig. 2. The agreement between these two methods is reasonable for dimensionless soil depths of $D = 2.0$ and $D = 6.0$. Figure 3 shows the comparison of the vertical stress profiles along the center line of the strip for $D = 2.0$ and $D = 6.0$, respectively. Again, the analytical model and numerical model are in good agreement. The comparison of surface displacements in Fig. 4 shows that the analytical model predictions are slightly less than the finite element model values for all three degrees of freedom. These comparisons also indicate that the soil responses are not particularly sensitive to the thin soil assumption in the development of the analytical model.

In the development of the analytical model, the solution for a contact problem was modified to provide the displacement boundary condition along the entire mudline for the outer region problem. The contact solution is developed under the assumption of a thin elastic layer so that the solution satisfies negligibly small shear stress and vertical normal stress conditions along the exposed upper surface for the radiation problem. The influence of the soil depth on these stresses is examined for various soil depths and locations along the exposed mudline. For this examination, the conditions in Table 1 are assumed.

Figure 5 reveals that at a dimensionless distance of 0.1 from the toe, the normal stress is less than one percent of the peak stress at the caisson toe, even though the dimensionless soil depth is up to 6. At a distance of 0.001, the stress ratio is small (0.15%) only for dimensionless soil depths less than 0.25. A similar result is also obtained for shear stress.

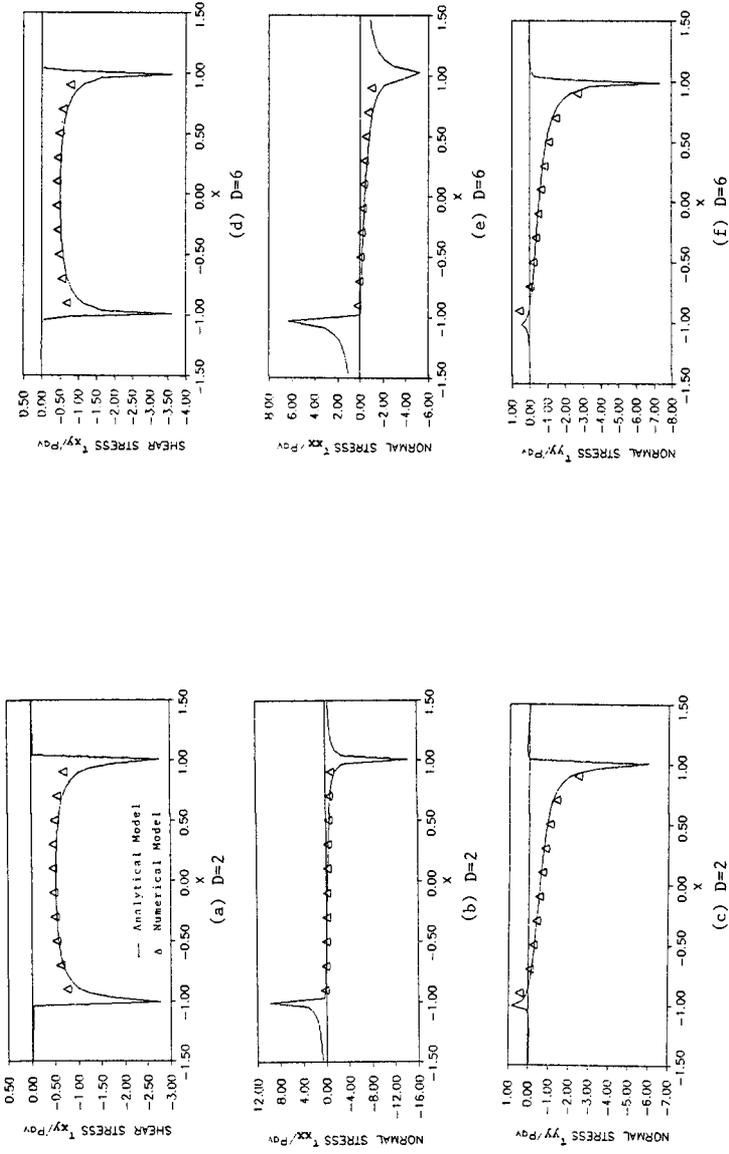


Figure 2. Comparison of the analytical and finite element models for the contact stresses on an elastic layer for Poisson's ratio = 0.3 and eccentricity = 0.5

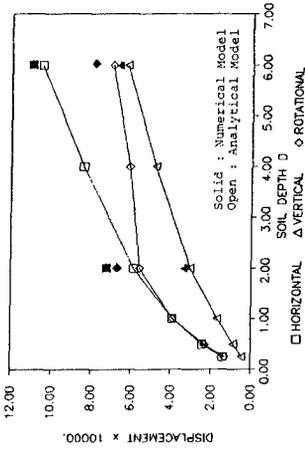


Figure 4. Comparison of the analytical model and finite element model for the surface displacements

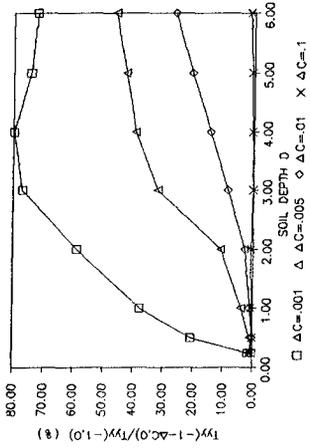


Figure 5. Vertical normal stress ratio along the exposed mudline as a function of soil depth

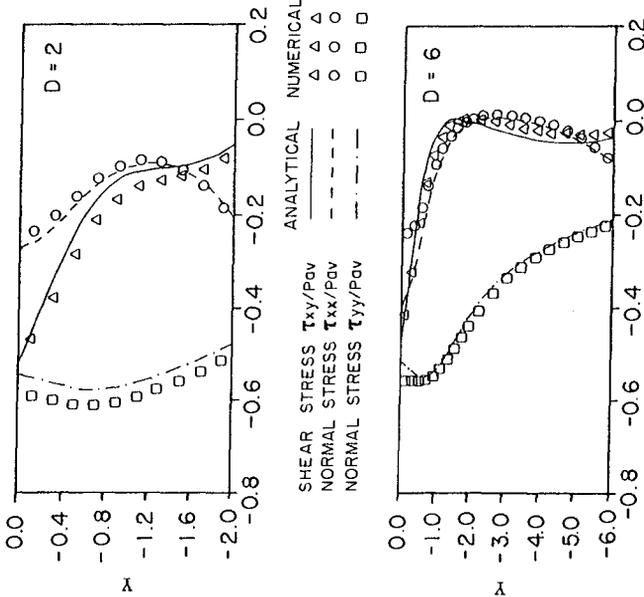


Figure 3. Comparison of the analytical model and finite element model for the vertical stress profiles at the strip center for Poisson's ratio = 0.3 and eccentricity = 0.5

Table 1. Conditions used in the examination of the thin layer assumption

Wave Period	= 4 sec
Wave Height	= 4 ft
Water Depth	= 8 ft
Thickness of the Rubble Bedding Layer	= 1 ft
Caisson Half-Width	= 4 ft
Mass of the Caisson	= 83 slugs
Mass Moment of Inertia of the Caisson	= 3,660 slugs-ft ²
Poisson's Ratio	= 0.3
Shear Modulus	= 80,000 psf

From the above and comparison with the numerical model, the influence of the thin layer assumption on the analytical model may be summarized as:

- a) For the soil regions under the caisson and under the exposed mudline greater than $0.1c$ from the caisson, the analytical model is applicable for all soil depths.
- b) For the response in the region $(0 \sim 0.1)c$ from the caisson, the model is applicable only for soil depths $d/c < 0.25$.

These limits on thin layer assumption are illustrated in Fig. 6.

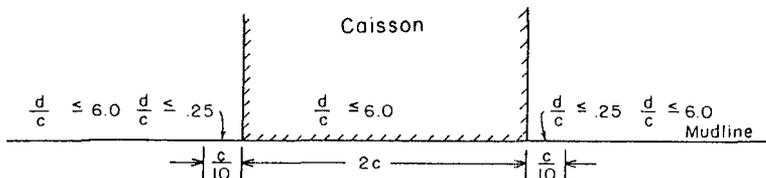


Figure 6. Limits on "thin layer" assumption

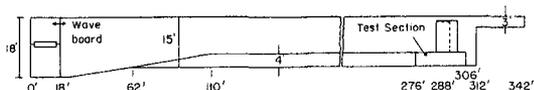
3. Physical Model

Two series of experiments of wave-soil-caisson interaction were conducted at the O.H. Hinsdale Wave Research Facility at Oregon State University during the springs of 1984 and 1985. A variety of wave conditions were examined. Incident waves, reflected waves, and transmitted waves were measured. Porewater pressure was monitored in the soil under the caisson. Three degrees of caisson motion, surge, heave, and pitch were measured with sonic transducers. Wave pressures also measured along the front face and bottom of the caisson and along the upstream portion of the mudline.

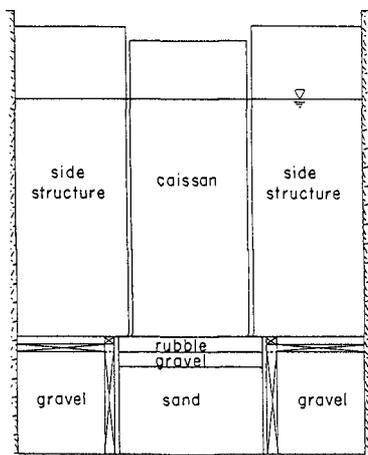
Experimental Conditions

Wave Tank - The OSU wave flume is 342 ft long, 12 ft wide, and 15 ft deep. The hinged-flap-type wave generator is able to produce solitary, periodic, and random waves. Simple periodic waves up to 8 sec in period and 5 ft in height can be generated. A polyurethane seal around the edges of the wave board confines the water to one side of the board. Precast concrete panels are available to form a false bottom with the desired water depth and slope.

Test Section - A test section, 30 ft long, 5 ft wide, and 4 ft deep, was constructed at the downstream end of the wave flume. A false channel bottom was installed to match the test section elevation. The sides and ends of the test section were fabricated with reinforced plyboard. The entire test section was bolted to the channel bottom and side walls. Figure 7 shows the test section. The side chambers of the test section were filled with highly permeable gravel to provide the extra strength and prevent side wall deflection during



(a) Elevation



(b) Cross section

Figure 7. Test section

the test. A perforated pipe was laid on the bottom of the channel to facilitate drainage during dewatering. In the middle chamber, a 3-ft layer for sand was used for the 1984 test. In the 1985 test, a 6-inch thick reinforced concrete slab was constructed as an impermeable hard-bed 1.5 ft above the bottom to provide a 1-ft deep sand layer above the concrete slab. These sand beds were fluidized and then reconsolidated back to a homogeneous condition. The fluidization was accomplished by using an inverted T-shaped manifold to inject a high-pressure water jet into the sand (Nath et al., 1977). This procedure prepared a uniform soil layer to ensure the repeatability of the experiments. Soil conditions are given in Table 2. The reconsolidation was induced through an over-burden of 6 to 12 inches of pea gravel separated from the sand by a geotextile. Rubble then was placed over the lift of the pea gravel to form a rubble bedding layer of approximately 1 ft thickness. The rubble had a mean diameter of 4 inches. The test caisson was then placed on the rubblemound foundation. Toe and heel protection were added.

Table 2. Soil properties for the tests

Year	1984	1985
Poisson's Ratio	0.3	0.28
Porosity	0.49	0.5
Shear Modulus	140,000 psf	110,000 psf
Permeability Coefficient	0.00033 ft/sec	0.00040 ft/sec

To provide a continuous caisson face across the width of the flume, fixed dummy side structures were constructed along each side of the caisson. These dummy sections were rigidly attached to the side walls of the wave flume. To allow caisson motion, a 1-inch gap was left between the caisson and side structures. The front of the gap was covered with a rubber strip to provide a watertight seal. The side structures were 3.9 ft wide, 12.3 ft long, and 10.5 ft high. They were constructed of heavily reinforced plyboard and rigidly bolted to the bottom and sides of the wave flume.

Test Caisson - The test caisson was 10 ft high, 8 ft long, and 4 ft wide. It was also made of heavily reinforced plyboard. To obtain the desired mass, the caisson was filled with concrete cylinders and sand bags. For the 1984 test, only the weight of the cylinders and bags was measured. For the 1985 test, the locations of cylinders and bags were also measured. The weight of the empty caisson in air was 1470 pounds. The total weight of the caisson including the ballast was 5640 pounds in the 1984 test. For the 1985 test, three different weights of the caisson in water were tested, as shown in Table 3.

Instrumentation - The wave profiles and caisson motions were measured with sonic transducers. The dynamic pressures were measured with pressure transducers (Druck model PDCR10). Carborundum filter stones covered the transducer housings to prevent soil from clogging the pressure transducers. A small amount of air in the stone may sig-

Table 3. Weight, mass, and mass moment of inertia of the caisson in water for the 1985 tests

Weight (pounds)	Mass (slugs)	Mass Moment of Inertia (slugs-ft ²)
5,280	164	2,765
7,150	222	5,630
10,690	332	14,631

nificantly affect the dynamic response of the transducers. Therefore, the stones were first boiled to remove air and then always kept underwater. The transducers were calibrated by raising and lowering the still water level in the channel before and after each sequence of runs. The instrument locations for the 1984 and 1985 tests are shown Figs. 8 and 9, respectively.

Wave Conditions - The tests were run at a water depth of 8 ft. The periods and heights of the simple periodic waves were selected to span deep to shallow water conditions based on Dean's stream function wave theory (Dean, 1974). The wave case, height, and period employed in the tests are shown in Table 4. In the 1985 experiments, the wave periods were slightly adjusted to provide pure standing waves in the flume.

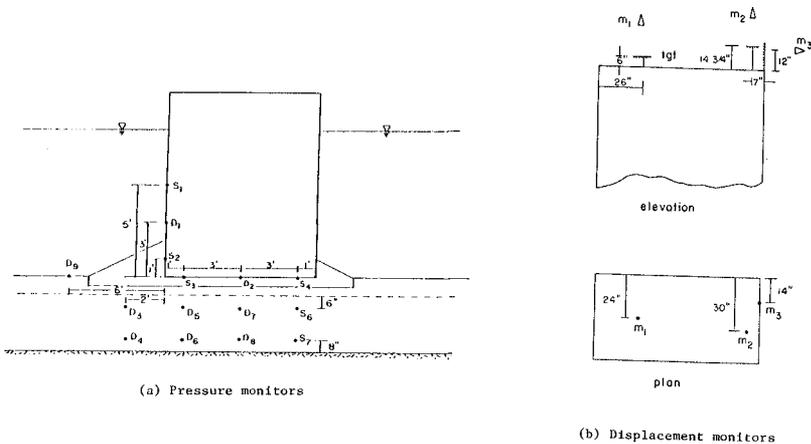


Figure 8. Instrumentation for the 1984 test

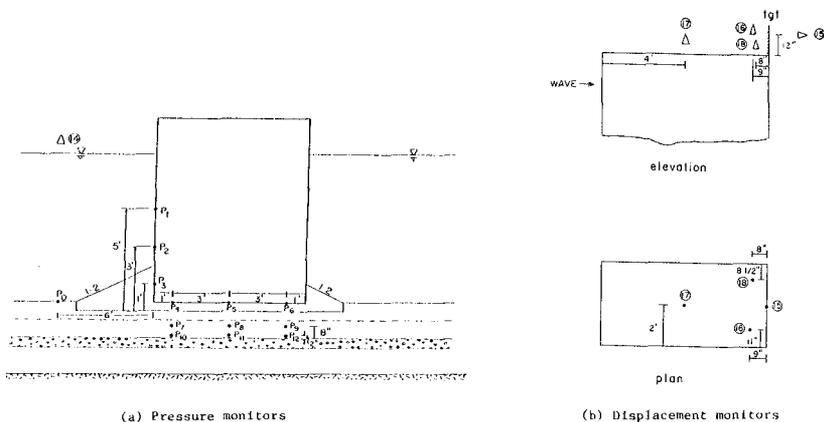


Figure 9. Instrumentation for the 1985 test

Table 4. Wave conditions for the 1984 tests

Wave Case	Wave Period (sec)	Wave Height (ft)
8A	1.77	0.68
8B	1.77	1.36
8C	1.77	2.03
7A	2.80	1.28
7B	2.80	2.52
7C	2.80	3.76
6A	3.95	1.47
6B	3.95	2.92
6C	3.95	4.40
5A	5.59	1.55
5B	5.59	3.07
4A	8.84	1.56

Experimental Results

One of the sonic profilers used to measure the caisson motion malfunctioned in the 1984 experiment. Several of the pressure transducers in the 1985 experiment did not calibrate well. Therefore, only the pore pressure measurements of the 1984 tests and the caisson motion measurements of the 1985 tests were analyzed.

Typical records of pore pressure and displacement are shown in Fig. 10. Significant noise was observed in the displacement measurements. To remove this noise, an eleven-point moving box car (i.e., $\Delta t = 0.076-0.381$ sec) was used. This poor signal-to-noise ratio and heavy filtering reduces the confidence in the displacement data.

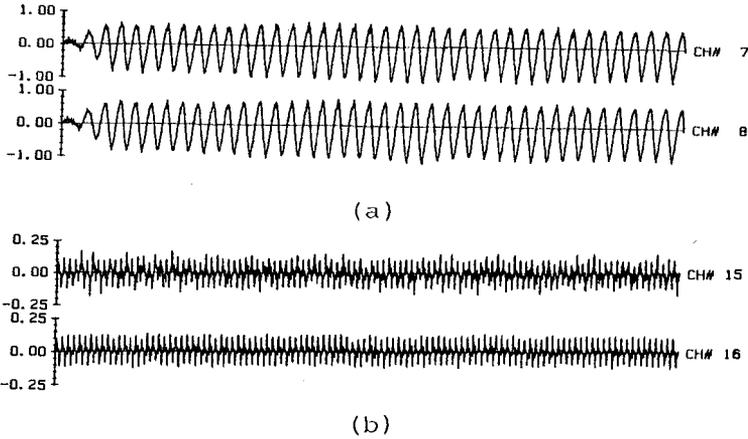


Figure 10. Measurement samples: (a) pore pressure and (b) caisson motion

The dimensionless pore pressure amplitudes are shown as a function of wave period in Fig. 11. The pressure amplitudes were scaled by S_2 ; the pressure on the caisson front face measured one foot above the mudline. A smooth line has been drawn through the data to help identify-

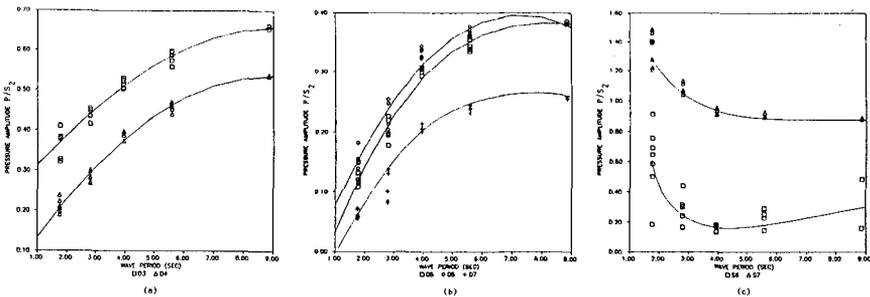


Figure 11. Pore pressure amplitude of measurements: (a) gages D3 and D4, (b) gages D5, D6, and D7, and (c) gages S6 and S7 as a function of wave period

tify trends. The caisson motion had the least influence on gages D3 and D4 since they were not underneath the caisson. The pressure at these locations decayed with the soil depth. The two pairs, D5-D6 and S6-S7, were obviously affected by the caisson motion because the pore pressure increased with the soil depth. For gages D3, D4, D5, D6, and D7 the dimensionless pressure amplitudes increase with increasing wave period. For gages S6 and S7 the opposite is observed.

The dimensionless mudline displacements are plotted against H/h (wave height to water depth) for various values of h/L_0 (depth to deep water wave length) in Fig. 12. Generally, the displacement increases with an increase in water height or wave period. This result is anticipated because the wave force on the caisson is proportional to the wave height and wave period. However, the vertical and rotational displacements are somewhat scattered.

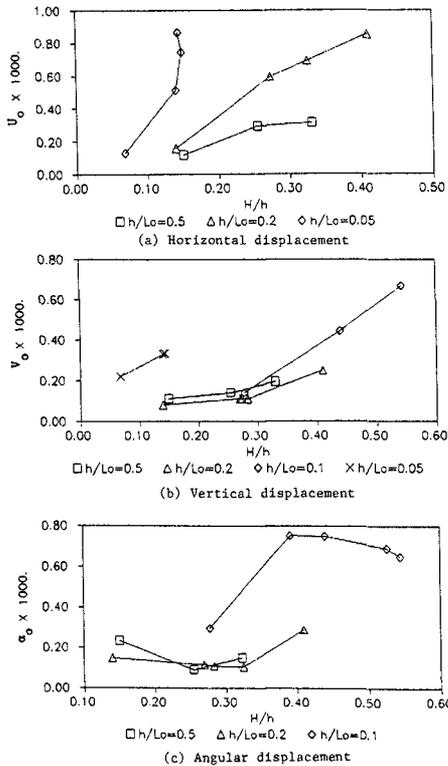


Figure 12. Dimensionless mudline displacements of measurements as a function of wave height for different water depths

Comparison of Theory and Measurements

The measurements of D5, D6, D7, S6, and S7 (cf. Fig. 8) were compared with the analytical model. Figure 13 shows the calculated pressure versus the measured pore pressure. Although the trend is predicted, there is considerable scatter. Figure 14 shows the computed contours of pore pressure and measurements. Again, the trend is in general agreement but there is considerable scatter. The deviation of the predicted porewater pressure from the measured may result from the assumption of a linear wave pressure distribution underneath the caisson. A more accurate pressure model being developed by Ward (1986) may be employed to obtain a more realistic pressure boundary condition. Unfortunately, this model was not completed in time to be used in the present study.

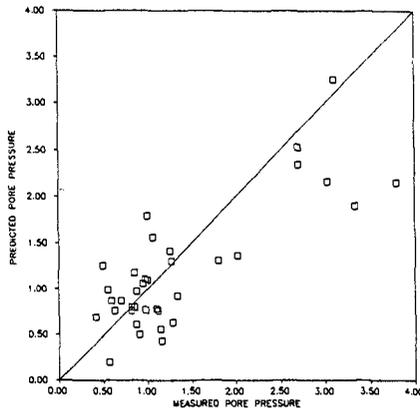
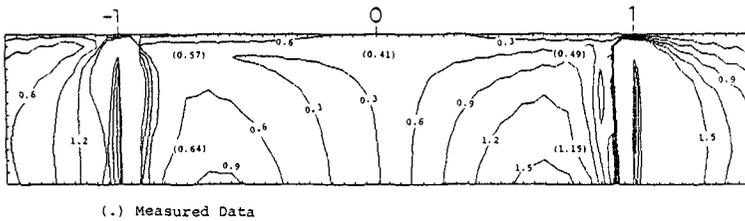


Figure 13. Comparison of theory and measurements for pore pressure



(.) Measured Data

Figure 14. Comparison of the predicted pressure contours and the measured data (T = 5.6 sec and H = 3.2 ft)

The measured and the predicted displacements are shown in Fig. 15. The dimensionless displacements were plotted to the dimensionless calculated horizontal wave force in the caisson F_1/wh^2 . The wave force, F_1 , was calculated by the method recommended in the Shore Protection Manual (1984). The measured displacements are rather scattered. The horizontal and the vertical displacement data are larger than the predicted. However, the measured rotational displacement data are in reasonably good agreement with the predicted. The predicted angular displacement is in better agreement with the laboratory results than both the horizontal and vertical displacements because the angular motion is the largest among the three degrees of the caisson motion. Thus, the noise effects on the measured angular displacement are less than on the horizontal and vertical displacement measurements.

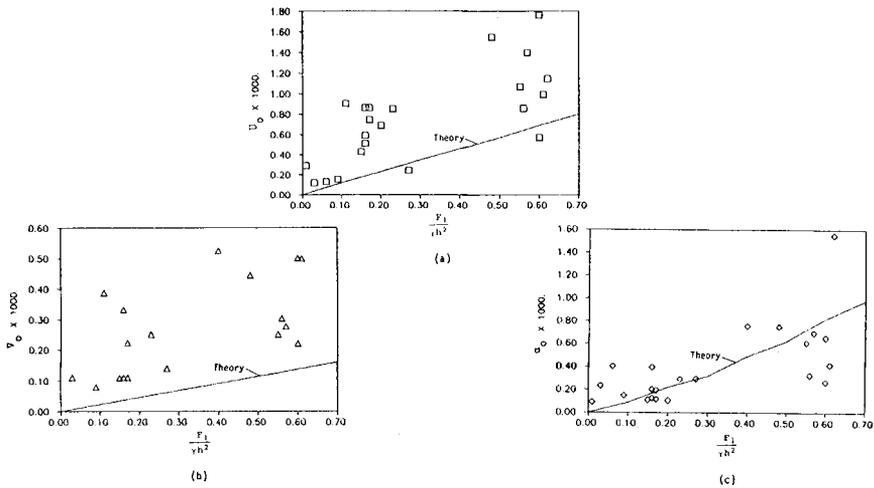


Figure 15. Comparison of the predicted and measured displacements: (a) horizontal, (b) vertical, and (c) angular

4. Conclusions

1. The analytical model agrees well with finite element model for all soils depths examined. This implies that the analytical model is not particularly sensitive to the thin layer assumption for the soil.
2. Under the caisson, the porewater pressure in the soil increases with the depth because of the confining of the impermeable rigid bed below. This is observed in both the analytical and physical models.

3. The predicted porewater pressure is in reasonable agreement with the measured data.
 4. Displacement data are unreliable due to the very poor signal-to-noise ratio. Thus, the following modifications to the experimental procedure are recommended:
 - a) Using an LVDT for displacement measurements.
 - b) Occupying entire wave channel width for caisson to avoid the effects on the measurements from both the side channels and dummy structures.
 - c) Making a stiffer caisson, e.g., using concrete.
 5. For comparison with data, more accurate methods to estimate the wave pressure on the mudline and forces on the caisson would be useful.
5. Acknowledgment

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Appendix II. English/SI Unit Conversions

Area	1 ft ²	= 0.0929 m ²
Density	1 slug/ft ³	= 515.4 kg/m ³
Force	1 lb	= 4.4483 N
Length	1 ft	= 0.305 m
Mass	1 slug	= 14.60 kg
Pressure	1 lb/ft ²	= 47.9 N/m ²
Specific Weight	1 lb/ft ³	= 157.1 N/m ³
Stress	1 lb/ft ²	= 47.9 N/m ²
Velocity	1 ft/s	= 0.305 m/s
Volume	1 ft ³	= 0.0283 m ³