Ocean waves propagating over cohesionless seabed deposits produce cyclic shear stresses within the deposit. Under certain conditions these stresses may cause a progressive build-up of pore pressure. Pore pressure accumulation can result in liquefaction or a substantial decrease in the effective stress with attendant large deformations of the seabed deposit.

In recognition of the need to study ocean wave-induced liquefaction, a series of large scale wave flume tests were conducted. The test program consisted of generating a series of uniform waves over a sand deposit and measuring the pore pressure response. The pore pressure response was studied for a variety of soil and wave conditions. Specific variables addressed in the study included drainage conditions near the surface of the deposit, relative density of the deposit, magnitude of wave loading, and previous wave loading history. Both the cyclic and mean pore pressure responses in the deposit were determined.

Cyclic shear stresses within the deposit are generated by wave-induced cyclic pressure fluctuations at the mudline. For a given wave length, the cyclic shear stresses increase with increasing wave height.
Wave-induced liquefaction of a fine sand was observed. A decrease in effective stress and liquefaction resulted from an increase in the mean pore pressure during undrained wave loading. The mean pore pressure increased owing to a transfer of intergranular stress to the pore fluid. Pore pressures cycled about the shifting mean pressure at the same frequency as the wave loading. Mudline pressure fluctuations were not affected by pore pressure changes occurring below the mudline. Liquefaction resulted in gross disturbance of the deposit.

Pore pressure accumulation was not observed during drained loading. Several drained tests were conducted for a range of initial relative density and wave conditions. During the drained tests, the relative density of the sand layer increased and settlement of the sand was observed. The increased density of the sand layer was accompanied by attenuation of the cyclic pore pressure fluctuations during the initial wave loading cycles of the test. Following attenuation, the cyclic pore pressure fluctuations remained constant to the end of the test.

The effect of previous wave loading was evaluated by comparing the response of tests subjected to previous wave loading with those not subjected to previous wave loading. Previous wave loading resulted in increased stability of the sand with respect to liquefaction potential. The increased stability was the result of an increase in relative density as well as changes in the structure of the deposit.
Measured mean pore pressure responses for all the tests were compared to the response predicted by a theoretical model. Theoretical predictions agreed with the measured response except for the case where the deposit was subjected to a previous wave loading history.

The potential for pore pressure accumulation and liquefaction of a seabed deposit decreases with: 1) increased drainage at the surface of the deposit, 2) increased relative density, 3) decreased magnitude of wave loading, and 4) previous wave loading.
EVALUATION OF OCEAN WAVE-INDUCED LIQUEFACTION

IN A LARGE SCALE WAVE FLUME

By

David Leo Thielen

A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of
Master of Science

Completed December 14, 1984

Commencement June 1985
APPROVED:  
-Redacted for privacy

Professor of Civil Engineering in charge of major

Redacted for privacy

Head of Department of Civil Engineering

Redacted for privacy

Dean of Graduate School

Date thesis presented: December 14, 1984

Typed by Linda L. D'Agostino and Carl Anderson for David Leo Thielen
ACKNOWLEDGEMENTS

This research was supported by the Oregon State University Sea Grant Program, National Oceanic and Atmospheric Administration Office of Sea Grant, Department of Commerce, under Grant No. NA79AA-D-00106.

Thank-you to my parents, Jim and Shirley Thielen, for giving me life and the ambition to make this contribution.

The author extends thanks to his major professor Dr. T. S. Vinson, and to Dr. W. G. McDougal, Dr. J. R. Bell, and Dr. C. K. Sollitt for their advice and guidance throughout this research project. The author is also grateful to the entire civil engineering staff at Oregon State University for providing an excellent education. A thank-you is extended to Wave Research Laboratory personnel Larry Crawford, Dave Stanley, and Terry Dibble for their patient assistance with this project.

The author extends thanks to Ken Robbins and to my other friends at the Portland office of Dames & Moore for their encouragement and support. A special thanks goes to Linda D'Agostino and Carl Anderson for typing this thesis.

A special thank-you also goes to the following people for their assistance and encouragement: Bob and Marianna Mattecheck, Ted Hammer, Jim Huddleston, and Paul Peri.
This document is dedicated to my wife, Katy, whose endless support and understanding played the key role in its completion.
# TABLE OF CONTENTS

**1.0 INTRODUCTION**  
1.1 Statement of the Problem  
1.2 Objectives and Scope  

**2.0 INTERACTION OF OCEAN WAVES WITH MARINE SOILS**  
2.1 Description of Wave-Induced Liquefaction  
2.2 Literature Review  
2.3 Description of Numerical Model  

**3.0 LARGE SCALE WAVE FLUME TESTS**  
3.1 General  
3.2 Test Configuration and Equipment  
3.3 Model Description  
3.3.1 Materials  
3.3.2 Pre-test Procedures  
3.4 Wave Flume Tests  
3.5 Summary of Test Soil Profiles and Wave Conditions  

**4.0 WAVE FLUME TEST RESULTS**  
4.1 Pore Water Pressure Measurements  
4.1.1 Tests Exhibiting No Pore Pressure Accumulation  
4.1.2 Test Exhibiting Pore Pressure Accumulation and Liquefaction  
4.1.3 Control Tests  
4.2 Comparison of Results from Wave Flume Tests and Numerical Model  
4.2.1 Theoretical versus Measured Pore Pressure Accumulation for Test Exhibiting Liquefaction
### TABLE OF CONTENTS (CONT.)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.2.2 Theoretical versus Measured Pore Pressure Accumulation for Tests not Exhibiting Liquefaction</td>
<td>94</td>
</tr>
<tr>
<td>4.3 Discussion of Wave Flume Test Results</td>
<td>95</td>
</tr>
<tr>
<td>4.3.1 Wave Conditions</td>
<td>98</td>
</tr>
<tr>
<td>4.3.2 Soil Response During Wave Loading</td>
<td></td>
</tr>
<tr>
<td>- Non-liquefied Tests</td>
<td>103</td>
</tr>
<tr>
<td>4.3.3 Soil Response During Wave Loading</td>
<td></td>
</tr>
<tr>
<td>- Liquefied Test</td>
<td>109</td>
</tr>
<tr>
<td>4.3.4 Soil Response During Wave Loading</td>
<td></td>
</tr>
<tr>
<td>- Control Tests</td>
<td>116</td>
</tr>
<tr>
<td>4.3.5 Previous Loading Conditions</td>
<td>118</td>
</tr>
<tr>
<td>5.0 SUMMARY AND CONCLUSIONS</td>
<td>121</td>
</tr>
<tr>
<td>5.1 Summary</td>
<td>121</td>
</tr>
<tr>
<td>5.2 Conclusions</td>
<td>122</td>
</tr>
<tr>
<td>5.3 Recommendations for Further Research</td>
<td>125</td>
</tr>
<tr>
<td>BIBLIOGRAPHY</td>
<td>128</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>2.1</td>
<td>Wave Characteristics and Mudline Pressure</td>
</tr>
<tr>
<td>2.2</td>
<td>Typical Shear Stress Ratio versus Depth</td>
</tr>
<tr>
<td>2.3</td>
<td>Rate of Pore Pressure Generation</td>
</tr>
<tr>
<td>2.4</td>
<td>Basic Equation and Solution Domain</td>
</tr>
<tr>
<td>3.1</td>
<td>In-place Photograph of the Test Section Before Addition of the Soil Layers</td>
</tr>
<tr>
<td>3.2</td>
<td>Typical Cross-Section of the Test Section</td>
</tr>
<tr>
<td>3.3</td>
<td>OSU Wave Flume Showing Location of Test Section</td>
</tr>
<tr>
<td>3.4</td>
<td>Pressure Transducer Locations</td>
</tr>
<tr>
<td>3.5</td>
<td>Grain Size Distribution and Properties of the Sand Layer</td>
</tr>
<tr>
<td>3.6</td>
<td>Permeability versus Void Ratio for Test Sand</td>
</tr>
<tr>
<td>3.7</td>
<td>Young's Modulus versus Void Ratio and Confining Pressure for the Test Sand</td>
</tr>
<tr>
<td>3.8</td>
<td>Poisson's Ratio versus Void Ratio for the Test Sand</td>
</tr>
<tr>
<td>3.9</td>
<td>Grain Size Distribution and Properties of the Gravel Layer</td>
</tr>
<tr>
<td>4.1</td>
<td>Graphical Representation of the Cyclic Pore Pressure Response for Tests 1, 2, 3, and 5</td>
</tr>
<tr>
<td>4.2</td>
<td>Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 1-I</td>
</tr>
<tr>
<td>4.3</td>
<td>Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 2-I</td>
</tr>
<tr>
<td>4.4</td>
<td>Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 3-I</td>
</tr>
<tr>
<td>4.5</td>
<td>Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 3-S</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>4.6</td>
<td>Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 5-I</td>
</tr>
<tr>
<td>4.7</td>
<td>Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 5-S.1</td>
</tr>
<tr>
<td>4.8</td>
<td>Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 5-S.2</td>
</tr>
<tr>
<td>4.9</td>
<td>Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 5-S.3</td>
</tr>
<tr>
<td>4.10</td>
<td>Cyclic Pressure Amplitude versus Number of Cycles for Test 1-I</td>
</tr>
<tr>
<td>4.11</td>
<td>Cyclic Pressure Amplitude versus Number of Cycles for Test 2-I</td>
</tr>
<tr>
<td>4.12</td>
<td>Cyclic Pressure Amplitude versus Number of Cycles for Test 3-I and 3-S</td>
</tr>
<tr>
<td>4.13</td>
<td>Cyclic Pressure Amplitude versus Number of Cycles for Test 5-I, 5-S Series</td>
</tr>
<tr>
<td>4.14</td>
<td>Mean Pore Pressure Response versus Number of Cycles for Test 4-I</td>
</tr>
<tr>
<td>4.15</td>
<td>Non-dimensionalized Cyclic Pore Pressure Response versus Depth for Test 4-I</td>
</tr>
<tr>
<td>4.16</td>
<td>Cyclic Pressure Amplitude versus Number of Cycles for Test 4-I</td>
</tr>
<tr>
<td>4.17</td>
<td>Cyclic Pressure Amplitude versus d/L for the Control Tests</td>
</tr>
<tr>
<td>4.18</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 1-I</td>
</tr>
<tr>
<td>4.19</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 2-I</td>
</tr>
<tr>
<td>4.20</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 3-I</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>4.21</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 3-S</td>
</tr>
<tr>
<td>4.22</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 4-I</td>
</tr>
<tr>
<td>4.23</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 5-I</td>
</tr>
<tr>
<td>4.24</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 5-S.1</td>
</tr>
<tr>
<td>4.25</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 5-S.2</td>
</tr>
<tr>
<td>4.26</td>
<td>Wave-Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 5-S.3</td>
</tr>
<tr>
<td>4.27</td>
<td>Field Corrected Cyclic Strength Curves for Initial Liquefaction, for Undrained Unidirectional Loading, for a Uniform Medium Sand</td>
</tr>
<tr>
<td>4.28</td>
<td>Developed Cyclic Strength Relationships for the Test Sand for Undrained, Unidirectional Wave Loading</td>
</tr>
<tr>
<td>4.29</td>
<td>Measured and Theoretical Pore Pressure Ratio versus Number of Cycles for the First Four Cycles of Test 4-I for all Depths within the Sand Layer</td>
</tr>
<tr>
<td>4.30</td>
<td>Measured and Theoretical Pore Pressure versus Number of Cycles for Test 5-S.2 for all depths within the Sand Layer</td>
</tr>
<tr>
<td>4.31</td>
<td>Measured and Theoretical Pore Pressure versus Number of Cycles for Test 5-S.3 for all depths within the Sand Layer</td>
</tr>
<tr>
<td>4.32</td>
<td>Average Cyclic Shear Stress Ratio versus H/L for the Sand Layer</td>
</tr>
<tr>
<td>4.33</td>
<td>Test Section Response to Liquefaction During Test 4-I</td>
</tr>
<tr>
<td>4.34</td>
<td>Analog Recording of Wave Pressure at the Mudline and Pore Pressure Response at a Depth of 1.9 Feet Below the Mudline for the First 15 Cycles of Test 4-I</td>
</tr>
</tbody>
</table>
### LIST OF FIGURES (CONCL.)

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.35</td>
<td>Observed Wave-Induced Liquefaction Response in which the Mean Pore Pressure Increases to the Initial Effective Stress</td>
<td>113</td>
</tr>
<tr>
<td>4.36</td>
<td>Developed Cyclic Strength Relationships for the Test Sand for Undrained, Unidirectional Wave Loading, Showing the Effect of Previous Wave Loading</td>
<td>120</td>
</tr>
</tbody>
</table>
### LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Calculation for Equivalent Number of Cycles</td>
<td>15</td>
</tr>
<tr>
<td>3.1</td>
<td>Geotextile Properties</td>
<td>37</td>
</tr>
<tr>
<td>3.2</td>
<td>Fluidization Procedure</td>
<td>37</td>
</tr>
<tr>
<td>3.3</td>
<td>Summary of Wave Test Designations</td>
<td>45</td>
</tr>
<tr>
<td>3.4</td>
<td>Control Test Wave Conditions</td>
<td>45</td>
</tr>
<tr>
<td>3.5</td>
<td>Range of Layer Thickness and Sand Layer Properties During Testing</td>
<td>50</td>
</tr>
<tr>
<td>3.6</td>
<td>Range of Wave Conditions During Testing</td>
<td>50</td>
</tr>
<tr>
<td>3.7</td>
<td>Test Conditions for Soil Profile No. 1</td>
<td>51</td>
</tr>
<tr>
<td>3.8</td>
<td>Test Conditions for Soil Profile No. 2</td>
<td>52</td>
</tr>
<tr>
<td>3.9</td>
<td>Test Conditions for Soil Profile No. 3</td>
<td>53</td>
</tr>
<tr>
<td>3.10</td>
<td>Test Conditions for Soil Profile No. 4</td>
<td>54</td>
</tr>
<tr>
<td>3.11</td>
<td>Test Conditions for Soil Profile No. 5</td>
<td>55</td>
</tr>
<tr>
<td>4.1</td>
<td>Comparison of Measured and Predicted Mudline Pressures.</td>
<td>100</td>
</tr>
<tr>
<td>4.2</td>
<td>Comparison of Average Wave-Induced Cyclic Shear Stress Ratios for the Sand Layer.</td>
<td>101</td>
</tr>
<tr>
<td>4.3</td>
<td>Comparison of the Decrease of Cyclic Pore Pressure Amplitude Over the Duration of Wave Loading.</td>
<td>106</td>
</tr>
</tbody>
</table>
NOTATION

d  still water depth
Dr  relative density
e  void ratio
H  wave height
Heq  equivalent wave height
k  permeability
L  wavelength
m_v  compressibility coefficient
N  number of cycles
Neq  number of equivalent cycles
N_l  number of cycles to liquefaction
N_w  number of waves
P_0  amplitude of wave-induced cyclic mudline pressure
P_z  cyclic pore pressure amplitude at a depth z below the mudline
r_u  pore pressure ratio
t  time
T  wave period
T_D  duration of wave action
u  excess pore pressure
u_g  pore pressure generation coefficient
u/σ'vo  pore pressure ratio
N/N_l  cycle ratio
x  coordinate in direction of wave travel
NOTATION

$z$  depth below the mudline
$Z$  depth below mudline to the impermeable boundary

$\theta$  pore pressure generation constant
$\sigma'_{vo}$  initial vertical effective stress
$\tau_c$  cyclic shear stress
$\tau_c/\sigma'_{vo}$  cyclic shear stress ratio
$\gamma_w$  unit weight of water
$\gamma_b$  buoyant unit weight of soil
$\pi$  numerical constant (3.14159)
EVALUATION OF OCEAN WAVE-INDUCED LIQUEFACTION
IN A LARGE SCALE WAVE FLUME

1.0 INTRODUCTION

1.1 Statement of the Problem

The design of structures founded on or buried beneath cohesionless marine deposits must address the stability of the deposit with respect to bearing capacity and settlement. These criteria should be examined with respect to the wave climate at the site to assess: 1) the effects of increased foundation pressures due to wave loading on the structure, and 2) the influence of wave-loading directly on the deposit. The research reported herein focuses on a specific concern related to the second topic; the instability of a cohesionless marine deposit resulting from pore pressure accumulation and liquefaction caused by ocean waves propagating over the deposit.

Although pore pressure accumulation and liquefaction in marine sediments have been recognized as important design considerations, very little experimental or field data are available to provide guidance for predicting their occurrence. No verified cases of ocean wave-induced liquefaction exist. Also, laboratory studies for evaluating liquefaction potential have generally been limited to traditional triaxial and simple shear tests with loading conditions simulating those occurring during strong motion earthquakes.
1.2 Objectives and Scope

The objectives of the research program reported herein are:

1) To present current procedures used to evaluate pore pressure accumulation and liquefaction of fine cohesionless marine deposits under wave loading conditions.

2) To present the results of a large scale experimental test program established to model wave loading of fine cohesionless marine deposits.

3) To compare the results of the test program with a numerical prediction model.

4) To assess the application of large scale testing to predict pore pressure accumulation and liquefaction in fine cohesionless marine deposits.

The scope of work to accomplish the objectives includes:

1) A review of technical literature addressing pore pressure accumulation and liquefaction in marine deposits.

2) An experimental test program to model pore pressure accumulation and liquefaction in a large scale wave flume.

3) An analysis of the experimental test results and a comparison of the results with predictions from a numerical prediction method.
2.0 INTERACTION OF OCEAN WAVES WITH MARINE SOILS

2.1 Description of Wave-Induced Liquefaction

As ocean waves pass over a marine soil deposit, variations in the water surface elevation cause pressure fluctuations to be transmitted through the water column to the sea bed. Although wave motions may be very complex, it is common practice to describe the sea surface as a simple sinusoid. Random sea states and directionality may be addressed with a simple periodic solution by employing Fourier transform methods. The sinusoidal waves produce simple harmonic pressure fluctuations within the water column which are transmitted to the seafloor.

Cyclic loading of the seafloor creates cyclic shear, normal, and neutral stresses within the sediment structure, and cyclic displacements of the soil. When a fine cohesionless soil is subjected to wave loading, the cyclic shear stresses may consolidate the soil depending on the magnitude of the shear stresses and soil structure. The tendency for consolidation transfers some or all of the load carried by the soil particles to the pore fluid. If pore pressures are not allowed to dissipate, due to restricted drainage, excess pore pressures develop. The amount of pore pressure build-up depends upon: 1) the characteristics of the deposit including soil structure, relative density, compressibility, permeability, lateral earth pressure coefficient, and previous strain history, (Seed, 1978; Seed and Rahman, 1977); 2) the magnitude of the shear stresses; 3) duration of wave loading; 4) drainage conditions;
and 5) the time allowed for drainage between loading cycles (defined by the period of loading).

The accumulation of excess pore pressure has a direct effect on strength reduction in the deposit. In the extreme case, the pore pressure accumulates until it equals the vertical intergranular stress. In this condition, the soil loses its ability to withstand significant shear stresses and behaves like a viscous fluid. This condition is termed liquefaction.

2.2 Literature Review

A substantial amount of literature is available on the interaction of ocean waves with marine soils. The bulk of the literature is from the ocean engineering professional community, with additional contributions from the fields of geotechnical engineering, oceanography, mechanical engineering, and groundwater hydrology. The topics cover a wide range of subject areas including: 1) wave damping, 2) pore-fluid response, 3) soil stresses, and 4) sea floor instability. A comprehensive historical review of this literature is given by McDougal, et al., (1981). The following review is focused on the literature dealing with pore pressure response and, in particular, pore pressure accumulation and liquefaction.

In the literature, treatment of pore pressure response of sediments to wave loading may be divided into two groups: 1) solutions that assume simple periodic porewater pressure responses that cycle about a constant mean, reflecting no change in the
average porewater pressure, and 2) solutions that time-average the cyclic porewater pressures to examine the mean pressure response, reflecting a change in the average porewater pressure. A comprehensive solution has not been published that incorporates both types of pore pressure response. This may be attributed to the complexity of the problem. As it is, most solutions for the two types of pore pressure response adopt extensive simplifying assumptions.

Simple periodic solutions result in a sinusoidal pore pressure response that cycles about a constant mean. The tendency for the mean pore pressure to increase or decrease during loading cannot be accommodated by these solutions. Researchers concerned with a progressive build-up of pore pressure, as reflected by a shift in the mean, would have to expand these solutions to an increasingly complex mathematical model. As an alternative, solutions for evaluating mean pore pressure accumulation and liquefaction under ocean wave loading conditions have been developed which employ the results from earthquake induced liquefaction studies.

Most analyses for earthquake induced liquefaction utilize models of pore pressure response to earthquake induced cyclic shear stresses developed from laboratory cyclic triaxial and cyclic simple shear tests. This is also the case for wave-induced liquefaction studies. Although the two phenomena are similar with respect to pore pressure build-up caused by shear stress reversals the following differences exist (Nataraja and Singh, 1979): 1) the cyclic shear stresses occurring during earthquakes are the result of shear
waves propagating upward through the soil, while cyclic shear stresses caused by waves are the result of a cyclic pressure change initiated above a soil deposit, 2) ocean wave frequencies are lower than the frequencies of earthquake cyclic loading, and 3) storm durations are substantially longer than the duration of earthquake loading. The modifications of earthquake induced liquefaction analyses to ocean wave induced liquefaction are based on these differences.

One of the first studies of wave induced liquefaction was reported by Christian et al. (1974) for the investigation of a large underwater pipeline for a nuclear power plant. In their analysis the induced shear stresses were calculated by treating the sea floor as a non-linear elastic half-space. The resulting shear stresses were compared to the minimum allowable shear stress required to cause liquefaction, between 1,000 and 10,000 cycles of loading, evaluated from "field adjusted" undrained cyclic triaxial test results. The pore pressure dissipation effects during the storm were evaluated by comparing undrained and drained analyses.

The liquefaction potential at Ekofisk Tank in the North Sea was investigated by Lee and Focht (1975). The cyclic shear stresses on the submarine deposit were induced by the cyclic wave forces on the tank transmitted to the sediment. The random wave-induced stresses were converted to a series of equivalent uniform stresses, using a modification of the method presented by Lee and Chan (1972) for earthquake liquefaction, to allow a comparison of the induced cyclic stresses with soil strength.
Pore pressure response during loading was evaluated by utilizing modified cyclic triaxial tests (standard cyclic triaxial tests are conducted undrained to failure). For their work, Lee and Focht interrupted the tests periodically and allowed 10 percent dissipation of excess pore pressure by opening the drain valve. The resulting cyclic strength curve was compared to predicted field induced stresses.

Seed and Rahman (1977) and Rahman, Seed and Booker (1977) were the first to develop an analysis that quantitatively predicts pore pressure build-up during ocean wave loading. Seed and Rahman (1977) evaluate the effects of cyclic shear stresses directly induced by waves, whereas, Rahman, Seed and Booker (1977) review the effects of shear stresses developed under offshore gravity structures. Both studies assume the seabed to be an elastic half-space with the storm wave loading converted to a series of equivalent uniform stresses. The pore pressure response is calculated using the consolidation equation with a term added to account for pore pressure dissipation. The dissipation term is determined from laboratory undrained cyclic triaxial tests and calculations presented in an earlier article by Seed et al (1976). The procedure utilizes the finite element method to determine pore pressure response with depth and time of loading.

Finn et al (1983) expanded the solution by Seed and Rahman (1977) to include the effects of decreasing shear and bulk moduli with pore pressure accumulation.
A simplified method of evaluating the liquefaction potential at an ocean site was presented by Nataraja and Singh (1979). This work was partially based on suspected liquefaction of an ocean outfall in Puerto Rico (Nataraja et al., 1978). The method was improved (Nataraja et al., 1980), and subsequently presented with applications (Nataraja and Gill, 1982). Their method is based on the simplified procedure developed by Seed and Idriss (1971) for earthquake induced liquefaction. The procedure involves comparison of the field corrected undrained cyclic strength (obtained from cyclic triaxial tests) to the estimated induced cyclic shear stresses. The induced stresses are calculated from a simplified elastic model and the cyclic shear strength is estimated from corrected standard penetration test resistance values.

2.3 Description of Numerical Model

The procedure developed by Seed and Rahman (1977) has been adopted in this research program to evaluate mean pore pressure response of cohesionless marine sediments to ocean wave loading. The method results in a quantitative evaluation of pore pressure response with depth and duration of loading.

The procedure by Seed and Rahman utilizes a one-dimensional finite element model to predict pore pressure response to wave loadings. The model is a modification of one developed to evaluate pore pressure response of sands during earthquake loading (Seed, et al., 1976). The steps in the analysis are as follows:
Step 1 - Determination of the Design Soil Profile.

The design profile may be a uniform deposit or layered system. The required soil properties are unit weight, relative density, permeability, and compressibility.

Step 2 - Determination of Storm Wave Characteristics at the Site.

Based on available oceanographic information or hindcasting from a meteorological study, the characteristics of the design storm may be established. The distribution of wave heights may be estimated using a design wave spectrum (Ippen, 1966). This results in a statistical classification of the waves into groups, with each group having a specific height, length, period, and number of waves.

Step 3 - Determination of the Wave Pressures.

The model uses linear wave theory to predict wave loading on the surface of the deposit (Ippen, 1966). The resulting pressure fluctuation is periodic and given by:

\[
p = \frac{H \gamma_w}{2} \cdot \frac{1}{\cosh \left( \frac{2\pi d}{L} \right)} \cdot \sin \frac{\pi x}{L} \cdot \sin \frac{\pi t}{T}
\]  

(2.1)

where \( H \) is the wave height, \( \gamma_w \) is the unit weight of water, \( d \) is the water depth, \( L \) is the wave length, \( T \) is the wave period, and \( x \) and \( t \) are the space and time coordinates in the direction of wave propagation measured from the positive zero crossing. These terms are shown in Figure 2.1. Equation 2.1 predicts the periodic fluctuation
\[ z = \text{DEPTH BELOW MUDLINE} \]
\[ d = \text{WATER DEPTH} \]
\[ L = \text{WAVELENGTH} \]
\[ H = \text{WAVE HEIGHT} \]
\[ T = \text{WAVE PERIOD} \]

**WATER SURFACE FLUCTUATION:**
\[ a = \frac{H}{2} \sin 2\pi \left( \frac{x}{L} - \frac{t}{T} \right) \]

**DYNAMIC MUDLINE PRESSURE FLUCTUATION:**
\[ p = H \frac{g}{2} \frac{1}{\cosh^2(2\pi d/L)} \sin 2\pi \left( \frac{x}{L} - \frac{t}{T} \right) \]

*Figure 2.1* Wave Characteristics and Mudline Pressure
of pressure; the maximum pressure fluctuation is given by:

\[ P_o = \frac{H \gamma w}{2 \cosh (2\pi d/L)} \]  

(2.2)

Step 4 - Determination of Wave-Induced Cyclic Shear Stresses.

The wave-induced cyclic shear stress distribution is determined utilizing the wave pressures at the mudline and the soil characteristics. The distribution is determined for each wave component of the storm. Solutions are presented by Seed and Rahman (1977) for two cases: a) a uniform deposit of infinite depth, and b) a layered deposit of finite depth. The layered finite depth solution can be used to model layered deposits of infinite depth and uniform deposits of finite depth. Both solutions assume the deposit to be represented as an elastic half space loaded at the surface by a static sinusoidal wave. The wave pressure amplitude is determined from Equation 2.2. The resulting shear stress distributions are determined as shear stress ratio with depth. The shear stress ratio is defined as the maximum horizontal cyclic shear stress divided by the initial effective overburden stress. Seed and Rahman state a static analysis is justified because the long periods of ocean storm waves result in negligible dynamic effects.

a) Uniform Deposit of Infinite Depth.

From the mudline pressure variation \( P_o \) given in Equation 2.2, the theory of elasticity predicts the maximum cyclic shear stress at a depth \( z \) within the deposit as:

\[ \tau_c = \frac{2 \pi P_o z}{L} \exp \left( \frac{-2 \pi z}{L} \right) \]  

(2.3)
The maximum cyclic stress ratio $\frac{\tau_c}{\sigma'_{vo}}$ is evaluated by determining the initial effective overburden stress as:

$$\sigma'_{vo} = Y_b z$$  \hspace{1cm} (2.4)

where $Y_b$ is the bouyant unit weight of the deposit. Combining Equations 2.3 and 2.4 results in:

$$\frac{\tau_c}{\sigma'_{vo}} = \frac{2\pi p_o}{\gamma_b L} \exp\left(-\frac{2\pi z}{L}\right)$$  \hspace{1cm} (2.5)

This equation predicts the maximum cyclic shear stress ratio distribution within the soil deposit. These maximum values occur under the crest and trough of the waves. A finite value of shear stress ratio is predicted at the mudline with an exponential decay occurring with depth. A plot of typical shear stress ratio versus depth for an homogeneous, infinite depth elastic half space for typical wave conditions is shown in Figure 2.2.

b) Layered Deposit of Finite Depth.

The solution for a layered deposit of finite depth utilizes the theory of elasticity applied to a layered system. The boundary conditions are as follows: 1) the mudline vertical stress equals the mudline pressure fluctuation given by equation 2.1, and the horizontal shear stress equals zero; 2) at the interfaces, the vertical stress, horizontal shear stresses, and horizontal and vertical displacements are equal above and below the interface; and 3) at the bottom boundary the horizontal and vertical displacements equal zero. Computation of the maximum values of shear stress ratio are accomplished using a finite element program.
Figure 2.2 Typical Shear Stress Ratio versus Depth

Shear Stress Ratio ($\tau_c / \sigma_{vo}$)

Depth in Feet

WAVE HEIGHT = 7.5 ft
WAVELENGTH = 100 ft
PERIOD = 6 sec
WATER DEPTH = 25 ft
SOIL BOUYANT WEIGHT = 48 lb/ft$^3$
developed by Seed and Rahman (1977).

**Step 5 - Determine an Equivalent Uniform Cyclic Stress History**

The random time history of wave loading is converted to an equivalent uniform cyclic stress history induced in the deposit. This is accomplished utilizing the storm wave distribution from Step 2, the stress ratios at the top of the soil deposit from Step 4, and a cyclic strength curve. The cyclic strength curve is an experimental curve relating applied cyclic shear stress ratio to the number of cycles required to induce liquefaction. The steps in this procedure are given below with an example calculation shown in Table 2.1.

a) Utilizing the storm wave distribution, the wave-induced mudline pressure, $P_0$, is calculated for each group of waves comprising the storm.

b) Using the mudline pressure, the shear stress ratio at the top of the soil deposit is calculated. This is accomplished using the methods presented in Step 4 for a uniform or layered soil profile.

c) From the cyclic strength curve, the number of cycles required to produce liquefaction ($N_1$), is determined for the shear stress ratios in Step b.

d) Based on the shear stress ratio distribution or corresponding wave height distribution, a value of shear stress ratio or wave height is selected to represent the equivalent loading
Table 2.1 CALCULATION FOR EQUIVALENT NUMBER OF CYCLES

<table>
<thead>
<tr>
<th>Wave Height (ft) $H_i$</th>
<th>Number of Waves $N_{w_i}$</th>
<th>Wave Period (secs) $T_i$</th>
<th>Wave Length (ft) $L_i$</th>
<th>Wave Pressure (kips/ft²) $\Delta p_i$</th>
<th>Shear Stress Ratio at Top of Soil Deposit $\tau_c/\sigma_v$</th>
<th>Number of Cycles to Liquef. $N_{l_i}$</th>
<th>Equivalent No. of Cycles $N_{eq}$ $= \frac{N_{w_i}}{N_{l_i}} \times N_{ref}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>50</td>
<td>7.0</td>
<td>130.7</td>
<td>.240</td>
<td>.198</td>
<td>3.2</td>
<td>53.13</td>
</tr>
<tr>
<td>8</td>
<td>80</td>
<td>6.5</td>
<td>120.3</td>
<td>.208</td>
<td>.196</td>
<td>3.4</td>
<td>80.00</td>
</tr>
<tr>
<td>6</td>
<td>155</td>
<td>6.0</td>
<td>109.8</td>
<td>.150</td>
<td>.163</td>
<td>7.2</td>
<td>73.19</td>
</tr>
<tr>
<td>4</td>
<td>180</td>
<td>5.0</td>
<td>80.6</td>
<td>.090</td>
<td>.130</td>
<td>24.0</td>
<td>25.19</td>
</tr>
<tr>
<td>2</td>
<td>200</td>
<td>4.0</td>
<td>66.57</td>
<td>.036</td>
<td>.071</td>
<td>10,000</td>
<td>.07</td>
</tr>
</tbody>
</table>

$N_{eq} = 232$

$N_{eq} = \Sigma N_{eq_i}$

$H_{eq} = 8.0$ ft

$\tau_c/\sigma_v = 0.196$

$N_{ref} = 3.4$

After Seed and Rahman (1977)
history of the storm.

Guidance for the selection of the representative shear stress ratio or wave height was not provided by Seed and Rahman; however, Lee and Focht (1975) provided such information. They recommended that an equivalent uniform stress (or corresponding wave height) be selected such that the final equivalent number of uniform cycles \( N_{eq} \) is between 0.2 and 0.5 times the total number of waves comprising the storm \( N_t \). This requires an iterative approach to alternately select a level of equivalent uniform stress, complete the subsequent calculations, and compare \( N_{eq} \) with \( N_t \). The range of 0.2 to 0.5 implies that a range of shear stresses and a corresponding number of equivalent uniform cycles should be considered for the design storm loading.

e) Using the information obtained in sub-steps "a" through "d" the number of equivalent uniform cycles is computed as follows.

\[
N_{eq} = \sum \left( N_1 \frac{N_{wi}}{N_{li}} \right)
\]  

(2.6)

Where \( N_1 \) is the number of cycles to liquefaction for the selected level of shear stress ratio from the cyclic strength curve; \( N_{wi} \) is the number of waves in each group from the storm wave distribution; and \( N_{li} \) is the number of cycles to liquefaction for the shear stress ratio at the top of the deposit for the waves in each group.

Step 6 - Determine the Rate of Pore Pressure Accumulation.

The model developed by Seed and Rahman (1977) to predict
the pore pressure response of a deposit utilizes the one-dimensional consolidation equation with a "generation" term added to incorporate the effects of pore pressure generation and dissipation. The basic equation is:

$$\frac{\partial}{\partial z} \left[ \frac{k_z}{\gamma_w} \frac{\partial u}{\partial z} \right] = m_v \left( \frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial t} \right)$$ (2.7)

where \( k_z \) is the permeability, \( m_v \) is the volume compressibility, \( \gamma_w \) is the unit weight of water, \( u \) is the excess pore pressure, and \( u_g \) is the generated or dissipated excess pore pressure. Equation 2.7 is the one-dimensional consolidation equation (Terzaghi, 1943) with a source term added to account for pore pressure accumulation.

The generation term \( \frac{\partial u_g}{\partial t} \) was developed in a previous study by Seed et al., (1975) and is given by:

$$\frac{\partial u_g}{\partial t} = \frac{\sigma_{vo}'}{\theta \pi T_D} \left( \frac{N_{eq}}{N_L} \right) \cdot \frac{1}{\sin 2\theta - 1} \left( \frac{\pi u}{2\sigma_{vo}'} \right) \cos \left( \frac{\pi u}{2\sigma_{vo}'} \right)$$ (2.8)

Where \( \sigma_{vo}' \) is the initial effective overburden pressure, \( \theta \) is an empirical constant with values given in Figure 2.3, \( T_D \) is storm duration, \( N_{eq} \) is the number of equivalent uniform stress cycles from Step 5, \( N_L \) is the number of cycles producing liquefaction under a stress of \( \sigma_{vo}' \) for undrained conditions, and \( u \) is the excess pore pressure.

The relationship between \( u_g/\sigma_{vo}' \) and \( N/N_L \) is determined from undrained cyclic laboratory tests; however, Seed et al., (1975)
have determined that for many soils this relationship can be expressed as:

$$\frac{\partial u_g}{\partial N} = \frac{\sigma_{vo}}{\theta n L} \cdot \frac{1}{\sin^2 \theta - 1} \left( \frac{\pi u}{2 \sigma_{vo}} \right) \cos \left( \frac{\pi u}{2 \sigma_{vo}} \right)$$  \hspace{1cm} (2.9)

This relationship is presented in Figure 2.3. In the absence of experimental data they recommend using a value of $\theta$ equal to 0.7, which has been found to represent the average curve for most soils.

In addition to the preceding considerations, Seed and Rahman also account for the increase in volume compressibility, $m_v$, with increasing pore pressure. The equation was given by Seed (1976) as:

$$m_v = m_{vo} \cdot \frac{\exp \left( A \left( \frac{u}{\sigma_{vo}} \right)^B \right)}{1 + A \left( \frac{u}{\sigma_{vo}} \right)^B + \frac{(A^2) \left( \frac{u}{\sigma_{vo}} \right)^{2B}}{2}}$$  \hspace{1cm} (2.10)

where $A = 5(1.5 - D_r)$, $B = 3/(2^{2D_r})$, $D_r$ is the relative density, $m_v$ is the volume compressibility at excess pore pressure $u$, and $m_{vo}$ is the initial compressibility at zero excess pore pressure, with the other terms as previously defined. Therefore, the pore pressure accumulation is computed by solving Equation 2.7 with the pore pressure generation term given in Equation 2.8 and the variation of compressibility given in Equation 2.10. The equation is solved for the boundary conditions shown in Figure 2.4. To solve
Figure 2.3 Rate of Pore Pressure Generation

After Seed and Rahman (1977)
After Seed and Rahman (1977)

**Figure 2.4 Basic Equation and Boundary Conditions**
this set of equations, Seed and Rahman developed a finite element computer program. The time history of pore pressure response for specified depths within the deposit are determined with the program.

Step 7 - Evaluation of the Results of the Analysis.

The results of the analysis are used to evaluate the degree of pore pressure accumulation with depth in the deposit and the time rate of accumulation. This information may be used in the design of structures supported on the deposit. In the case of buried pipelines, the analysis may be used to determine the depth of liquefaction so the pipeline can be positioned below this depth or at a depth sufficient to prevent flotation of the pipeline. The analysis can also be used to specify a cover material for the pipeline. Placement of a nonliquefiable coarse sand or gravel cover with a high permeability greatly reduces the accumulation of pore pressure.

In the case of foundations, the analysis can be used to determine the loss of strength of the underlying and surrounding soils. Even at a pore pressure build-up less than required to cause liquefaction, loss of soil strength can occur that results in excess settlement or soil failure under the foundation load.
3.0 LARGE SCALE WAVE FLUME TESTS

This chapter presents the procedures, equipment and results of large scale wave flume tests conducted to evaluate pore pressure response and liquefaction in fine sands.

3.1 General

To evaluate wave-induced liquefaction, large scale tests were conducted at the Oregon State University Wave Research Laboratory (OSU-WRL) in Corvallis, Oregon. The tests were designed to measure both the cyclic and mean pore pressure response of a sand deposit during wave loadings. The cyclic stress test results were presented by McDougal, et al., (1982). In the present study particular emphasis is given to the evaluation of mean pore pressure response.

Generally, the test program consisted of generating a series of uniform waves over a sand deposit and measuring the pore pressure response. The pore pressure response was studied for a variety of soil and wave conditions. The following parameters were investigated:

1. Wave Conditions - The effects of wave length, wave height and period were investigated. Water depth was constant at 8.0 ft (2.45 m) during the tests.

2. Soil Conditions - The effect of varying the initial relative density of the sand was investigated. Changes in relative density result in changes of permeability, stiffness, and compressibility under applied cyclic shear stresses.
3. Soil Drainage - The effects of a drained and an undrained sand layer were investigated.

4. Wave Loading History - Initial wave loading response was compared to subsequent wave loading response.

To investigate the above parameters a test program was designed that included five soil profiles subjected to various wave loadings.

3.2 Test Configuration and Equipment

The wave loading tests were conducted in a large outdoor wave flume at the OSU-WRL. The flume is 12 ft (3.66m) wide and 318 ft (97m) long. Wave generation at the OSU-WRL is accomplished by the angular rotation of a hinged-flap wave board. The wave board is driven by an MTS electrohydraulic closed-loop test system. The wave board is a reinforced welded aluminum plate 18 ft (5.49m) high and 12.0 ft (3.66m) wide. A water seal around the wave board keeps the back side of the board dry which reduces power requirements.

Periodic waves, standing waves, and random waves can be generated at the OSU-WRL. Breaking waves of up to 5.0 ft (1.52m) in height can be produced in the deep water section as well as waves of smaller height. The useful range of wave frequencies is from about .25 Hz to 1 Hz. Numerous bottom depths and shapes can be modeled by using precast concrete panels installed as a false bottom. For purposes of the subject tests, a 1V:12H sloping beach at the breaking end of the flume was employed to reduce wave reflection.

A PDP 11 minicomputer is available for digital recording
of instrument signals as well as for generation of random waves. Other instruments employed in this study include strip chart recorders, signal conditioners, amplifiers, filters, and sonic and ultrasonic wave profile measurement equipment. Digital equipment consists of analog to digital converters, multiplexers, and disc and tape recorders.

3.2 Model Description

A soil containment structure was constructed in the wave flume as shown in Figures 3.1 and 3.2. It consisted of three longitudinal sections separated by two walls. The two outside sections were filled with gravel to the mudline elevation. The middle section contained a layer of sand and gravel separated by a geotextile. The gravel in the outside sections prevented sidewall movement during wave loading. Because the transducers were mounted in the wall, any sidewall deflection would have had a major influence on the measured pore water pressures. A sectional elevation showing the bottom configuration and location of the test section within the flume is shown in Figure 3.3.

The test section was constructed of 3/4 in. (1.91 cm) plywood, reinforced and braced with 2 in. (5 cm) by 4 in. (10 cm) studs. A 3/4 in. (1.91 cm) plywood sheet was also placed on the concrete floor of the wave tank to seal the test section and prevent loss of sand. The walls were held in place with horizontal bracing to the sides, and bolts to the floor of the wave tank. The horizontal bracing was fastened to the side of the wave tank using 4 in. (10 cm) by 4 in. (10 cm) by 1/4 in. (.64 cm) steel angle iron. The
Figure 3.1 In Place Photograph of the Test Section Before Addition of the Soil Layers

Figure 3.2 Typical Cross-Section of the Test Section
Fig. 3.3 OSU Wave Flume Showing Location of Test Section
end walls of the section were braced with 2 in. (5 cm) by 4 in. (10 cm) timbers that angled up from the concrete floor of the flume. All joints within the test section were sealed with silicon caulking compound to prevent loss of sand and insure that drainage was restricted upward to the mudline.

Fourteen pore water pressure transducers were located in one wall of the test section, as shown in Figure 3.4. The transducers were fastened to aluminum mounts set into the plywood wall. The mounts were held in place by aluminum plates and sealed with silicon. Sand was kept from direct contact with the transducers to keep individual grains from interfering with the internal mechanism. This was accomplished using a porous carborundum filter held in place by an aluminum plate and sealed with an O-ring. The location of the pressure transducers was chosen to define the vertical pressure profile in the test section and to check for horizontal changes occurring along the length of the bed. The transducers were mounted along the direction of wave travel to reduce the system to two dimensions. Wave-induced velocities were anticipated to be small at and below the mudline; therefore, their effect on the pressure measurements was neglected. Mudline pressures were measured with a flush-mounted transducer pointing upwards at the top of the section.

Druck model PDCR10 pressure transducers were employed in the test section. The back of the pressure diaphragm was vented to the atmosphere to allow pressure measurement with respect to atmo-
Figure 3.4 Pressure Transducer Locations
spheric pressure. The transducer sensitivity was set at one volt of output approximately equal to 1 lb/sq in. (6895 Pa) of pressure. The transducer signal was transmitted to filters, analog to digital converters, and strip chart and digital recorders.

Water surface fluctuations were measured using a Sonic System Model 86 transducer. These transducers operate by measuring the time for a sonic pulse to propagate from the transducer to the water surface and back. This time is converted to a voltage level representing the distance between the sensing head and the water surface. The range of the instrument is from 2 in./volt (5.08 cm/volt) to 64 in./volt (162.6 cm/volt). The 64 in./volt (162.6 cm/volt) range was selected for the test program. Voltage output from the sonic profiler was recorded on both strip chart and digital recorders. The sonic profiler was placed above the center of the test section directly over the vertical line of pressure transducers.

3.2.1 Materials

Two soil types were utilized in the study, namely, a uniform fine sand and a "pea" gravel. Two commercially available geotextiles and a polyethylene sheet were employed in the various tests to provide separation between the fine sand and pea gravel.

The primary criterion for the sand employed was that it have a high liquefaction potential. Based on the results of Castro and Poulos (1976), the most likely soil to liquefy is a clean uniform fine sand. Grain size analyses were conducted on several
candidate soils from the Pacific Northwest. A clean uniform fine sand located in a natural deposit near Hammond, Oregon, a city on the Columbia River was selected for the test program. Approximately 26 cu yd (20 cu m) were needed to fill the test section.

During the test program, the density of the sand in the test section was determined. In order to evaluate additional properties, a laboratory program was established. Laboratory testing conformed to ASTM specifications or proposed test methods. The following tests were conducted:

1. Grain-size Analysis.
2. Specific Gravity Determination.
5. Resonant Column Testing.

The results of the grain size analysis, specific gravity determination, and maximum and minimum void ratio determination are shown in Figure 3.5. The permeability is plotted against void ratio in Figure 3.6. The resonant column tests were utilized to evaluate Young's modulus and Poisson's ratio. These properties are plotted against void ratio in Figures 3.7 and 3.8, respectively.

The gravel used in the test program served four purposes: 1) it was used as backfill material on either side of the test section to prevent sidewall movement, 2) it held the geotextile in place on top of the sand in the test section, 3) it acted as a surcharge on the sand bed, and 4) it provided a stable cover under
<table>
<thead>
<tr>
<th>Gravel</th>
<th>Sand</th>
<th>Silt</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse to medium</td>
<td>Fine</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

U.S. standard sieve sizes

- No. 4
- No. 10
- No. 20
- No. 40
- No. 100
- No. 200

Specific Gravity of Solids ($G_s$) = 2.75
Maximum Void Ratio ($e_{-\text{max}}$) = 1.09
Minimum Void Ratio ($e_{-\text{min}}$) = 0.65

Figure 3.5 Grain Size Distribution and Properties of the Sand Layer
Figure 3.6 Permeability vs. Void Ratio for the Test Sand
Figure 3.7 Young's Modulus vs. Void Ratio and Confining Pressure for the Test Sand

Extrapolated from:

\[ E = (1-u) \sigma_m^{1/2} \]

(Seed and Idriss, 1970)
Figure 3.8 Poisson's Ratio versus Void Ratio for the Test Sand
the large waves tested.

The basic requirement for the backfill material was that it be a clean gravel. A suitable "pea" gravel was located near Corvallis from a Willamette River deposit. The properties of the gravel were determined from limited laboratory tests including grain size analyses and permeability tests. Values of bulk modulus and poissons ratio were obtained from the literature (Seed and Idriss, 1970). The results of the grain size analysis and properties of the gravel layer are shown in Figure 3.9.

A geotextile was used in the tests as a separation layer between the sand and gravel layers. Separation was required to accurately evaluate consolidation and liquefaction of the sand layer, and reduce the potential for scour. The important characteristics of the geotextile for this study were: 1) the ability to separate the sand and gravel, and 2) the permeability across the fabric. Two geotextiles were used in the tests: one of relatively high permeability and one of relatively low permeability. An impermeable membrane (polyethylene sheet) was also employed in the test program. A list of the geotextile types and properties are given in Table 3.1.

3.2.2 Pre-test Procedures

Prior to the tests in the wave flume, the following pre-test procedures were required:

1. Calibration of the pressure transducers.
Gravel

<table>
<thead>
<tr>
<th>Coarse to medium</th>
<th>Fine</th>
<th>Silt</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.S. standard sieve sizes</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Grain diameter, mm

PERMEABILITY = 0.059 FT/SEC
BOUYANT UNIT WEIGHT = 69 LB/CU FT
POISSON'S RATIO = 0.35
YOUNG'S MODULUS = 7.6 KIPS/SQ IN

Figure 3.9 Grain size Distribution and Properties of the Gravel Layer
### TABLE 3.1 GEOTEXTILE PROPERTIES

<table>
<thead>
<tr>
<th>Geotextile Type</th>
<th>Permeability in Feet per Second</th>
<th>Thickness in Inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bidim-C42 Needle-Punch Nonwoven</td>
<td>0.130</td>
<td>0.180</td>
</tr>
<tr>
<td>Typar-3401 Heat Bonded Nonwoven</td>
<td>0.004</td>
<td>0.015</td>
</tr>
</tbody>
</table>

### TABLE 3.2 FLUIDIZATION PROCEDURE

<table>
<thead>
<tr>
<th>Direction of Travel</th>
<th>Depth of Fluidizer in Feet</th>
<th>Speed in Feet per Minute</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>1.0</td>
<td>1.3</td>
</tr>
<tr>
<td>S</td>
<td>1.0</td>
<td>1.3</td>
</tr>
<tr>
<td>N</td>
<td>2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>S</td>
<td>2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>N</td>
<td>3.0</td>
<td>1.8</td>
</tr>
<tr>
<td>S</td>
<td>3.0</td>
<td>1.8</td>
</tr>
<tr>
<td>N</td>
<td>3.8</td>
<td>2.0</td>
</tr>
<tr>
<td>S</td>
<td>3.8</td>
<td>2.0</td>
</tr>
<tr>
<td>N</td>
<td>2.0</td>
<td>3.0</td>
</tr>
<tr>
<td>S</td>
<td>2.0</td>
<td>3.0</td>
</tr>
<tr>
<td>N</td>
<td>3.7</td>
<td>3.0</td>
</tr>
<tr>
<td>S</td>
<td>3.7</td>
<td>3.0</td>
</tr>
</tbody>
</table>
2. Standardization of the fluidization and compaction procedures for the sand layer, and measurement of the resulting "initial conditions."

These are discussed below.

Calibrations of the pressure transducers were conducted with the transducers installed in the test section. Three different calibrations were performed prior to testing to verify the linearity of the transducers and determine if the soil or porous stones affected pressure readings. One test was conducted without porous stones or soil, one with the porous stones in place, and one with the porous stones and the soil in place. The results of the calibrations showed that the performance of the transducers was unaffected by the presence of the porous stones and soil. The overall accuracy of the transducers with respect to drift, linearity, and response for the wave frequencies utilized was within 0.01 lb/sq in. (70 Pa). This corresponds to an equivalent hydrostatic head of water of 0.25 in. (0.64 cm), which exceeds the accuracy to which the surface fluctuations can be controlled.

Preparation of the test section prior to testing involved fluidization of the sand layer by the injection of water. The water was injected in the saturated sand at a pressure of 250 lb/in\(^2\) (1720 kPa) utilizing a "fluidizer". The fluidizer was an inverted T-shaped manifold with 1/16 in. (0.16 cm) holes drilled at 0.5 in. (1.27 cm) centers. It was constructed of 4 in. (10 cm) diameter pipe.
To thoroughly fluidize the bed, the fluidizer was moved the entire length of the sand bed at different depths. The fluidization procedure was standardized for all tests and is given in Table 3.2.

Standardization of the fluidation procedure was accomplished prior to the conduct of the test program. The procedure was developed to insure that a level bed with a relatively uniform density was produced. A consistent means of preparing the sand layer was required because density testing before each test would alter the condition of the sand bed. The fluidization procedure resulted in a sand layer with a dry density of 92 lb/cu ft (1.47 g/cu cm), corresponding to a relative density of 51 percent.

Pore water pressure measurements were taken within the sand layer during the fluidization process. This was done to insure all excess pore pressures developed during fluidization had dissipated prior to testing. The results showed that excess pore pressures generated during fluidization dissipated almost immediately following placement of the gravel overburden.

For two of the tests the sand was compacted after fluidizing. This was accomplished by connecting a 5 ft (1.5 m) long 0.6 in. (1.5 cm) diameter aluminum rod to a standard concrete vibrator and vibrating the sand. The aluminum rod was placed into the sand bed at right angles to the sand surface. To insure a uniform compaction effort, the rod was inserted into the sand in a grid pattern starting at the center of the test section. The zone
of influence of compaction was 1.3 ft (.4 m) so the grid pattern was set on 1.0 ft (.3 m) centers. The vibrator was 2 in. (4.6 cm) in diameter by 12 in. (30 cm) in length and utilized a 0.75 kw electric motor. This provided 990 lbs (4,400 N) of centrifugal force at an unrestrained amplitude of 0.2 in. (0.5 cm) and a frequency of 160 Hz.

The compaction procedure was also standardized prior to the conduct of the test program to determine the change of density of the sand layer. Single and double coverage with the vibrator increased the dry density to 94 lb/cu ft (1.50 g/cu cm) and 98 lb/cu ft (1.57 g/cu cm), respectively, from the initial density of 92 lb/cu ft (1.47 g/cu cm) after fluidization.

3.4 Wave Flume Tests

The test phase of the program involved the following steps:

1. Preparation and evaluation of the test section prior to wave loading.

2. Generation of waves and pore water pressure measurements.

3. Evaluation of the test section following wave loading.

These steps are described in greater detail below. The wave conditions, as well as pre- and post-test soil profiles are presented in Section 3.4. Pore water pressure measurements are presented in Section 4.
Step 1 - Preparation of the Test Section.

A. Saturation of the Pressure Transducers and Porous Stones.

Prior to each test, the pressure transducers and porous stones were saturated to prevent trapped air bubbles from affecting the pressure measurements. This procedure was followed for each test. The initial step in the saturation procedure was to excavate the sand from around the transducers and raise the water level in the flume above them. A syringe with a small needle was used to inject water into the transducer port to displace air near the face of the pressure diaphragm. Saturation of the porous stones was accomplished by boiling them in water for 20 minutes. They were subsequently placed in the transducer mounts while submerged.

B. Fluidization of the Sand Bed.

The sand layer was "fluidized" by injecting high pressure water (following saturation of the pressure transducers). The standardized procedure presented in Section 3.2.3 was used for all tests.

During the fluidization process, the water level in the flume was maintained at the top of the test section. Following fluidization, the sand was allowed to settle and consolidate under its own weight until the surface of the sand layer was relatively level. This occurred almost instantaneously; however, the water in the flume was discolored by the small percentage of silt-sized particles in the sand. Therefore, the sand layer was allowed to
stabilize for 20 minutes until the water above the sand layer became clear. At this time, the water level in the flume was drawn down to within 1/8 in. (0.3 cm) of the sand surface.

C. Compaction of the Sand Layer.

For two of the tests the sand layer was compacted following fluidization. The compaction procedure described in Section 3.2.3 was used.

D. Sand Surface Survey Before Testing.

Following fluidization and compaction (if performed), the sand surface was surveyed. Surveying was accomplished by placing a straight-edge across the top of the test section and measuring down to the sand layer with a scale. This was done in a grid pattern on 2 ft (0.6 m) centers.

During Tests 4 and 5, five - 10 lb (2.3 kg) flat lead weights were added at one end of the test section and their elevation before testing was recorded. These weights aided in evaluating changes in the surface of the sand layer during the test and provided a qualitative measurement of loss of strength during testing.

E. Placement of Geotextile or Polyethylene Sheet.

After the survey, a geotextile or polyethylene sheet was carefully placed over the sand layer. When a polyethylene sheet was used, a geotextile was placed over the sheet to protect it from puncture during placement of the gravel layer.
F. Placement of the Gravel Layer.

The final step during preparation of the test section was placement of the gravel layer. This was accomplished by careful hand placement of 3 in. (8 cm) lifts. When the polyethylene sheet was used, gravel placement proceeded outward from the center of the test section to allow trapped water between the polyethylene sheet and the sand layer to dissipate out of the ends and sides of the test section.

Step 2 - Generation of Waves and Porewater Pressure Measurements.

Following preparation of the test section, the wave flume was filled to the still water level 8 ft above the top of the test section. Filling took approximately 8 hrs and required about 210,000 gal (810,000 l) of water. During this procedure, periodic water level and pressure readings were taken to verify calibration of the pressure transducers. The water level was held constant overnight (about 16 hrs) without generating waves. Pressure measurements were taken during this calm period to check for the presence of small excess pore pressures. The measurements showed a constant pressure during long-term monitoring corresponding to the hydrostatic head. This indicated all residual pore pressures developed during preparation of the test section had dissipated rapidly and no residual pore pressures were present prior to wave loading.
A. Wave Tests

After the flume was filled to the still water level and allowed to remain calm overnight, the wave tests were conducted. Three types of tests were conducted on each of the five test sections prepared:

1. Initial wave loading tests.
2. Subsequent wave loading tests.
3. Control tests.

A summary of the types of tests conducted and their designation is given in Table 3.3. The first number corresponds to the soil profile number, the letters correspond to the type of test; I, for initial wave loading, S for subsequent wave loading, and C, for control wave tests. The second number indicates that more than one test of a particular type was conducted.

The types of wave flume tests which were conducted are explained as follows:

1. Initial Wave Loading Tests. The initial wave loading tests subjected the test section to a series of uniform monochromatic waves; the first wave of this series being the first wave experienced by the soil deposit. These waves were discontinued following attainment of a steady state pore pressure response. The total number of waves for these tests varied, but ranged between 100 and 200.

2. Subsequent Wave Loading Tests. The subsequent wave loading tests were conducted for soil profiles 3 and 5 following completion of the initial wave loading tests. Between initial and
TABLE 3.3 SUMMARY OF WAVE TEST DESIGNATIONS

<table>
<thead>
<tr>
<th>Soil Profile Number (1)</th>
<th>Initial Loading Test and Type (2)</th>
<th>Subsequent Loading Test and Type (3)</th>
<th>Control Test and Type (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1-I (Drained)</td>
<td>Not Conducted</td>
<td>1-C (Drained)</td>
</tr>
<tr>
<td>2</td>
<td>2-I (Drained)</td>
<td>Not Conducted</td>
<td>2-C (Drained)</td>
</tr>
<tr>
<td>3</td>
<td>3-I (Drained)</td>
<td>3-S (Drained)</td>
<td>3-C (Drained)</td>
</tr>
<tr>
<td>4</td>
<td>4-I (Undrained)</td>
<td>Not Conducted</td>
<td>4-C (Undrained)</td>
</tr>
<tr>
<td>5</td>
<td>5-I (Drained)</td>
<td>5-S.1 (Undrained)</td>
<td>5-C.1 (Drained)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5-S.2 (Undrained)</td>
<td>5-C.2 (Undrained)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5-S.3 (Undrained)</td>
<td></td>
</tr>
</tbody>
</table>

TABLE 3.4 CONTROL TEST WAVE CONDITIONS

<table>
<thead>
<tr>
<th>Deans Wave Case (1)</th>
<th>Period in Seconds (2)</th>
<th>Wave Height in Feet (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4A</td>
<td>8.84</td>
<td>1.56</td>
</tr>
<tr>
<td>5A</td>
<td>5.59</td>
<td>1.55</td>
</tr>
<tr>
<td>5B</td>
<td>5.59</td>
<td>3.07</td>
</tr>
<tr>
<td>6A</td>
<td>3.95</td>
<td>1.47</td>
</tr>
<tr>
<td>6B</td>
<td>3.95</td>
<td>2.92</td>
</tr>
<tr>
<td>6C</td>
<td>3.95</td>
<td>4.40</td>
</tr>
<tr>
<td>7A</td>
<td>2.80</td>
<td>1.28</td>
</tr>
<tr>
<td>7B</td>
<td>2.80</td>
<td>2.52</td>
</tr>
<tr>
<td>7C</td>
<td>2.80</td>
<td>3.76</td>
</tr>
</tbody>
</table>
subsequent wave loading tests the water surface was allowed to calm. The test section was then subjected to a different wave loading than in the initial tests. The purpose of the subsequent wave loading tests was to evaluate differences in pore water pressure response caused by a previous loading history. Subsequent wave loadings also consisted of a series of uniform monochromatic waves. The number of waves ranged from 50 to 300.

For soil profile Number 3, subsequent wave test 3-S was accomplished without altering the soil profile. For soil profile Number 5 the subsequent Tests 5-S.1, 5-S.2, and 5-S.3 were conducted undrained whereas 5-I was a drained test. This was accomplished by
(a) lowering the water level in the flume to the top of the sand layer, (b) removing the gravel layer, (c) placing a polyethylene sheet over the geotextile, (d) replacing the gravel layer, and (e) refilling the flume. The undrained soil profile 5 was then subjected to three different wave conditions during tests 5-S.1, 5-S.2, and 5-S.3.

3. Control Wave Tests. The control wave loading tests were conducted following completion of either the initial wave loading tests or subsequent wave loading tests as shown on Table 3.4 Each control test consisted of a "mini-series" of uniform monochromatic waves of about 25 cycles. Nine such "mini-series" were generated with different wave lengths, wave heights and periods, corresponding to Dean's Stream Function Cases 4A, 5A, 5B, 6A, 6B, 6C, 7A, 7B, and 7C (Dean 1974). These are summarized in
Table 3.4. These tests were primarily used to generate data for a simultaneous study of simple periodic pore pressure response (McDougal et al., 1981); however, the results proved useful for this study as addressed in the discussion of results.

Step 3 - Evaluation of the Test Section Following Wave Loading

Following completion of the wave loading tests, the water was drained from the wave flume. During draining, pressure measurements and water level readings were taken to re-verify the calibration of the pressure transducers.

The test section was visually inspected to record obvious displacement or scour of the surface of the gravel layer. During all tests, minor scour of the gravel layer occurred near the ends of the test section; however, the interior (80 percent) gravel surface remained intact. No measurements were taken at the gravel surface to determine the extent of consolidation of the test section. The accuracy of these measurements would have been questionable due to the previously mentioned scour, as well as the size of the gravel particles. In addition to the above, the gravel layer consisted of rounded "pea" gravel that exhibited negligible densification following placement. Therefore, all settlement measurements for the test section were taken at the surface of the sand layer.

Following inspection of the gravel surface, the gravel was removed to expose the geotextile (or polyethylene sheet). The geotextile was examined for clogging or excessive wrinkling (which
would indicate lateral movement). Following inspection, the geotextile layer was removed to expose the sand bed.

The condition of the sand layer following a test was evaluated by surveying the surface of the layer and conducting density tests. Surveying was again accomplished by placing a straight-edge across the top of the test section and measuring down to the surface with a scale. Elevation measurements were made of the 10 lb (2.3 N) lead weights placed on the sand layer during Tests 4 and 5.

Density testing was limited to the central area of the test section, immediately adjacent to the pressure transducers. A minimum of nine tests were conducted, three each at three depths within the sand layer. As previously mentioned, the water level within the sand layer was lowered to about the midpoint of the layer between tests. Because of this, density tests were restricted to the upper half of the sand layer.

3.5 Summary of Test Soil Profiles and Wave Conditions

This section presents the results of measurements taken to evaluate wave conditions and the condition of the test section (soil profile) before and after the wave tests. Properties of the gravel layer and geotextile did not change as a result of wave loadings and are given in Figure 3.9 and Table 3.1, respectively. "Before" and "after" measurements of the thickness and density of the sand layer indicated that the sand consolidated. The range of layer thickness and properties of the sand layer during testing are
presented in Table 3.5. The range of wave conditions tested is given in Table 3.6. Specific sand layer thickness, soil properties, and wave conditions during testing are given in Tables 3.7 to 3.11. Values of permeability, Young's modulus and Poisson's ratio are interpolated from Figures 3.6, 3.7, and 3.8, respectively.

The sand layer properties presented in Tables 3.7 to 3.11 were computed from the measured values of dry density. For each test, the change in the "before" and "after" dry unit weights of the sand layer was compared to the measured settlement of sand layer. This comparison revealed that the "after" dry unit weights were between 0 and 4 percent higher than predicted by the measured settlement. This discrepancy is attributed to a slightly higher dry unit weight in the lower half of the test section. As previously addressed, density testing was restricted to the upper half of the test section.
TABLE 3.5  RANGE OF LAYER THICKNESS AND SAND LAYER PROPERTIES DURING TESTING

<table>
<thead>
<tr>
<th></th>
<th>Before</th>
<th>After</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel Layer Thickness (In.)</td>
<td>5.0 - 7.5</td>
<td>5.0 - 7.5</td>
</tr>
<tr>
<td><strong>Sand Layer Properties:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness (In.)</td>
<td>36.5 - 38.5</td>
<td>33.5 - 38.0</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.75 - 0.87</td>
<td>0.68 - 0.75</td>
</tr>
<tr>
<td>Dry Unit Weight (Lb/Cu Ft)</td>
<td>92 - 98</td>
<td>96 - 102</td>
</tr>
<tr>
<td>Buoyant Unit Weight (Lb/Cu Ft)</td>
<td>59 - 62</td>
<td>62 - 65</td>
</tr>
<tr>
<td>Relative Density (Percent)</td>
<td>51 - 77</td>
<td>77 - 93</td>
</tr>
<tr>
<td>Permeability (X 10^4 Ft/Sec)</td>
<td>4.8 - 7.6</td>
<td>3.8 - 4.8</td>
</tr>
<tr>
<td>Compressibility (X 10^6 Ft^2/Lb)</td>
<td>1.2 - 1.4</td>
<td>1.0 - 1.2</td>
</tr>
<tr>
<td>Young's Modulus (Kips/Sq In.)</td>
<td>5.0 - 5.3</td>
<td>6.0 - 6.7</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.34 - 0.35</td>
<td>0.33 0.34</td>
</tr>
</tbody>
</table>

TABLE 3.6  RANGE OF WAVE CONDITIONS DURING TESTING

<table>
<thead>
<tr>
<th></th>
<th>Initial Wave Tests</th>
<th>Subsequent Wave Tests</th>
<th>Control Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Depth (Ft)</td>
<td>8.0</td>
<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Wave Height (Ft)</td>
<td>2.2 - 4.0</td>
<td>2.0 - 4.0</td>
<td>1.3 - 4.4</td>
</tr>
<tr>
<td>Wavelength (Ft)</td>
<td>44 - 84</td>
<td>36 - 57</td>
<td>36 - 130</td>
</tr>
<tr>
<td>Period (Sec)</td>
<td>3.2 - 5.5</td>
<td>2.8 - 4.0</td>
<td>2.8 - 8.8</td>
</tr>
</tbody>
</table>
**TABLE 3.7 TEST CONDITIONS FOR SOIL PROFILE NO. 1**

**SOIL CONDITIONS**

Pre-Test Compaction Used - No  
Type of Geotextile Used - Bidium C-42 (Monsanto)  
Gravel Layer Thickness (In.) - 6.0

<table>
<thead>
<tr>
<th>Sand Layer Properties:</th>
<th>Before</th>
<th>After</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (In.)</td>
<td>38.0</td>
<td>37.0</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.87</td>
<td>0.82</td>
</tr>
<tr>
<td>Dry Unit Weight (Lb/Cu Ft)</td>
<td>92</td>
<td>94</td>
</tr>
<tr>
<td>Buoyant Unit Weight (Lb/Cu Ft)</td>
<td>59</td>
<td>65</td>
</tr>
<tr>
<td>Relative Density (Percent)</td>
<td>51</td>
<td>93</td>
</tr>
<tr>
<td>Permeability (X 10^4 Ft/Sec)</td>
<td>7.6</td>
<td>3.8</td>
</tr>
<tr>
<td>Compressibility (X 10^6 Ft^2/Lb)</td>
<td>1.4</td>
<td>1.0</td>
</tr>
<tr>
<td>Young's Modulus (Kips/Sq In.)</td>
<td>5.0</td>
<td>6.7</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.35</td>
<td>0.33</td>
</tr>
</tbody>
</table>

**WAVE CONDITIONS**

<table>
<thead>
<tr>
<th>WAVE Depth (Ft)</th>
<th>Wave Height (Ft)</th>
<th>Wave Length (Ft)</th>
<th>Period (Sec)</th>
<th>Number of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Wave Test (2-I)</td>
<td>8.0</td>
<td>4.0</td>
<td>44</td>
<td>3.8</td>
</tr>
<tr>
<td>Subsequent Wave Test</td>
<td>Not Conducted</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Control Tests</td>
<td>See Table 3.4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
# TABLE 3.8 TEST CONDITIONS FOR SOIL PROFILE NO. 2

## SOIL CONDITIONS

- **Pre-Test Compaction Used** - Yes
- **Type of Geotextile Used** - Bidium C-42 (Monsanto)
- **Gravel Layer Thickness (In.)** - 6.5

<table>
<thead>
<tr>
<th>Sand Layer Properties:</th>
<th>Before</th>
<th>After</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (In.)</td>
<td>37.5</td>
<td>36.5</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.83</td>
<td>0.72</td>
</tr>
<tr>
<td>Dry Unit Weight (Lb/Cu Ft)</td>
<td>94</td>
<td>100</td>
</tr>
<tr>
<td>Buoyant Unit Weight (Lb/Cu Ft)</td>
<td>60</td>
<td>64</td>
</tr>
<tr>
<td>Relative Density (Percent)</td>
<td>60</td>
<td>85</td>
</tr>
<tr>
<td>Permeability (X 10^4 Ft/Sec)</td>
<td>6.5</td>
<td>4.4</td>
</tr>
<tr>
<td>Compressibility (X 10^6 Ft^2/Lb)</td>
<td>1.3</td>
<td>1.1</td>
</tr>
<tr>
<td>Young's Modulus (Kips/Sq In.)</td>
<td>5.3</td>
<td>6.3</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.35</td>
<td>0.33</td>
</tr>
</tbody>
</table>

## WAVE CONDITIONS

<table>
<thead>
<tr>
<th>Water Depth (Ft)</th>
<th>Wave Height (Ft)</th>
<th>Wave-length (Ft)</th>
<th>Period (Sec)</th>
<th>Number of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Wave Test (1-I)</td>
<td>8.0</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
</tr>
</tbody>
</table>

- **Subsequent Wave Test** Not Conducted
- **Control Tests** See Table 3.4
### TABLE 3.9 TEST CONDITIONS FOR SOIL PROFILE NO. 3

**SOIL CONDITIONS**

Pre-Test Compaction Used - Yes  
Type of Geotextile Used - Bidium C-42 (Monsanto)  
Gravel Layer Thickness (In.) - 7.5 In.

<table>
<thead>
<tr>
<th>Sand Layer Properties:</th>
<th>Before</th>
<th>After</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (In.)</td>
<td>36.5</td>
<td>36.5</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>Dry Unit Weight (Lb/Cu Ft)</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>Buoyant Unit Weight (Lb/Cu Ft)</td>
<td>62</td>
<td>62</td>
</tr>
<tr>
<td>Relative Density (Percent)</td>
<td>77</td>
<td>77</td>
</tr>
<tr>
<td>Permeability (X 10^4 Ft/Sec)</td>
<td>4.8</td>
<td>4.8</td>
</tr>
<tr>
<td>Compressibility (X 10^6 Ft^2/Lb)</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>Young's Modulus (Kips/Sq In.)</td>
<td>6.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.34</td>
<td>0.34</td>
</tr>
</tbody>
</table>

**WAVE CONDITIONS**

<table>
<thead>
<tr>
<th>Wave Conditions</th>
<th>Water Depth (Ft)</th>
<th>Wave Height (Ft)</th>
<th>Wave-length (Ft)</th>
<th>Period (Sec)</th>
<th>Number of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Wave Test (3-I)</td>
<td>8.0</td>
<td>2.2</td>
<td>84</td>
<td>5.5</td>
<td>85</td>
</tr>
<tr>
<td>Subsequent Wave Test (3-P)</td>
<td>8.0</td>
<td>3.7</td>
<td>44</td>
<td>3.8</td>
<td>300</td>
</tr>
<tr>
<td>Control Tests</td>
<td>See Table 3.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### TABLE 3.10 TEST CONDITIONS FOR SOIL PROFILE NO. 4

#### SOIL CONDITIONS

Pre-Test Compaction Used - No  
Type of Geotextile Used - Polyethylene Sheet  
Gravel Layer Thickness (In.) - 7.5

<table>
<thead>
<tr>
<th>Sand Layer Properties:</th>
<th>Before</th>
<th>After</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (In.)</td>
<td>36.5</td>
<td>33.5 (Average)</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.87</td>
<td>0.68</td>
</tr>
<tr>
<td>Dry Unit Weight (Lb/Cu.Ft)</td>
<td>92</td>
<td>102</td>
</tr>
<tr>
<td>Buoyant Unit Weight (Lb/Cu Ft)</td>
<td>59</td>
<td>65</td>
</tr>
<tr>
<td>Relative Density (Percent)</td>
<td>51</td>
<td>93</td>
</tr>
<tr>
<td>Permeability (X 10^4 Ft/Sec)</td>
<td>7.6</td>
<td>3.8</td>
</tr>
<tr>
<td>Compressibility (X 10^6 Ft²/Lb)</td>
<td>1.4</td>
<td>1.0</td>
</tr>
<tr>
<td>Young's Modulus (Kips/Sq In.)</td>
<td>5.0</td>
<td>6.7</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.35</td>
<td>0.33</td>
</tr>
</tbody>
</table>

#### WAVE CONDITIONS

<table>
<thead>
<tr>
<th>Water Depth (Ft)</th>
<th>Wave Height (Ft)</th>
<th>Wave-length (Ft)</th>
<th>Period (Sec)</th>
<th>Number of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Wave Test (4-I)</td>
<td>8.0</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
</tr>
<tr>
<td>Subsequent Wave Test</td>
<td>Not Conducted</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Control Tests</td>
<td>See Table 3.4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### TABLE 3.11  TEST CONDITIONS FOR SOIL PROFILE NO. 5

#### SOIL CONDITIONS

Pre-Test Compaction Used - No  
Type of Geotextile Used - Typar 3401 (Test 5-I and 5-C.1)  
- Polyethylene Sheet (5-S Series and 5-C.2)  
Gravel Layer Thickness (In.) - 5.5

<table>
<thead>
<tr>
<th>Sand Layer Properties:</th>
<th>Before</th>
<th>After</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (In.)</td>
<td>38.5</td>
<td>38.0</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.87</td>
<td>0.75</td>
</tr>
<tr>
<td>Dry Unit Weight (Lb/Cu Ft)</td>
<td>92</td>
<td>98</td>
</tr>
<tr>
<td>Buoyant Unit Weight (Lb/Cu Ft)</td>
<td>59</td>
<td>62</td>
</tr>
<tr>
<td>Relative Density (Percent)</td>
<td>51</td>
<td>77</td>
</tr>
<tr>
<td>Permeability (X 10^4 Ft/Sec)</td>
<td>7.6</td>
<td>4.8</td>
</tr>
<tr>
<td>Compressibility (X 10^6 Ft^2/Lb)</td>
<td>1.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Young's Modulus (Kips/Sq In.)</td>
<td>5.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.35</td>
<td>0.34</td>
</tr>
</tbody>
</table>

#### WAVE CONDITIONS

<table>
<thead>
<tr>
<th>Wave Conditions</th>
<th>Water Depth (Ft)</th>
<th>Wave Height (Ft)</th>
<th>Wave Length (Ft)</th>
<th>Period (Sec)</th>
<th>Number of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Wave Test (5-I)</td>
<td>8.0</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
<td>100</td>
</tr>
<tr>
<td>Subsequent Wave Test</td>
<td>5-S.1</td>
<td>8.0</td>
<td>2.0</td>
<td>57</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>5-S.2</td>
<td>8.0</td>
<td>4.0</td>
<td>57</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>5-S.3</td>
<td>8.0</td>
<td>4.0</td>
<td>36</td>
<td>2.8</td>
</tr>
<tr>
<td>Control Tests</td>
<td>5-C.1</td>
<td>See Table 3.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5-C.2</td>
<td>See Table 3.4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.0 WAVE FLUME TEST RESULTS

4.1 Pore Water Pressure Measurements

Pore water pressure measurements were used to evaluate the mean and cyclic pressure response of the sand layer for the various wave and soil conditions considered. Pressures below the mudline have been normalized with respect to the mudline pressure amplitude. For all tests, the mudline pressures consisted of uniform, monochromatic pressure waves, in-phase with surface fluctuations. No tendency for mean pressure shift at the mudline was observed.

The study reported herein was directed at pore pressure accumulation and liquefaction, consequently the primary objective of the measurement program was to determine the mean pore pressure response. However, the cyclic pore pressure response results obtained provide additional information to identify changes occurring within the sand layer during wave loading. These results are further explained and presented in the following two sections.

4.1.1. Tests Exhibiting No Pore Pressure Accumulation

Four of the five test sections exhibited no pore pressure accumulation under both the initial and subsequent wave loadings. They were Soil Profiles 1, 2, 3 and 5, as presented in Tables 3.7, 3.8, 3.9, and 3.11. The mean pore pressure remained constant during these tests and no gross disturbance of the soil profile occurred. However, examination of the before and after layer thickness and densities revealed that the sand layer densified during the tests.
This was also verified during Test 5 by measurements taken on the lead weights. These weights remained on top of the sand layer but moved downward as the sand layer settled.

The increased density of the sand layer was reflected by a change in the cyclic pore pressure amplitude within the sand layer over the duration of the tests. In general, for the initial wave loading sequences, the cyclic pore pressure amplitude decreased as the number of cycles increased, until it reached a constant value. For the subsequent wave loadings, the cyclic pore pressure amplitude followed this trend during Test 3, but exhibited a steady-state response for Test 5. Graphical representation of this response is shown on Figure 4.1. This information has been quantified for each test and is presented as plots of 1) dynamic pore pressure amplitude versus depth within the test section, and 2) dynamic pore pressure amplitude versus number of cycles. These data are shown on Figures 4.2 through 4.13. In all cases, the dynamic pore pressure amplitude \( P_z \) has been non-dimensionalized with respect to the mudline dynamic pressure amplitude \( P_0 \).

The plots of dynamic pore pressure versus depth (Figures 4.2 to 4.9) have been constructed to define changes in the profile during the tests by presenting one or more relationships for each test. These relationships are labeled with the corresponding wave cycle(s) they represent. For cases where one relationship is given (e.g., Figure 4.4), no changes in the profile were observed over the duration of the test; for cases where more than one relationship is
Figure 4.1 Graphical Representation of the Cyclic Pore Pressure Response for Tests 1, 2, 3, and 5
Figure 4.2 Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 1-I

WAVE HEIGHT = 4.0 ft  PERIOD = 3.2 sec
WAVELENGTH = 44 ft  WATER DEPTH = 8.0 ft
Figure 4.3 Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 2-I

WAVE HEIGHT = 3.8 ft  PERIOD = 4.0 sec
WAVELENGTH = 57 ft  WATER DEPTH = 8.0 ft
Figure 4.4 Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 3-I

WAVE HEIGHT = 2.2 ft            PERIOD = 5.5 sec
WAVELENGTH = 84 ft               WATER DEPTH = 8.0 ft
Figure 4.5 Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 3-S
Figure 4.6 Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 5-I

WAVE HEIGHT = 3.8 ft PERIOD = 4.0 sec
WAVELENGTH = 57 ft WATER DEPTH = 8.0 ft
WAVE HEIGHT = 2.0 ft
WAVELENGTH = 57 ft
PERIOD = 4.0 sec
WATER DEPTH = 8.0 ft

Figure 4.7 Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 5-S.1
Figure 4.8 Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 5-S.2

WAVE HEIGHT = 4.0 ft  PERIOD = 4.0 sec
WAVELENGTH = 57 ft  WATER DEPTH = 8.0 ft
Figure 4.9  Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 5-S.3

WAVE HEIGHT= 4.0 ft        PERIOD= 2.8 sec
WAVELENGTH = 36 ft           WATER DEPTH = 8.0 ft
Figure 4.10 Cyclic Pressure Amplitude versus Number of Cycles for Test 1-1
CONSTANT TO END OF TEST
AT CYCLE 200

Figure 4.11 Cyclic Pressure Amplitude versus Number of Cycles for Test 2-1
Figure 4.12 Cyclic Pressure Amplitude versus Number of Cycles for Tests 3-I and 3-S
Figure 4.13 Cyclic Pressure Amplitude versus Number of Cycles for Tests 5-I and 5-S Series
shown (e.g., Figure 4.2), the first cycle profile is given together with the highest numbered cycle corresponding to the time at which the pressure fluctuations reached a steady state and no further changes were observed.

These relationships present instantaneous profiles observed during testing. To evaluate the gradients of change over the duration of the tests, these profiles should be used in conjunction with the plots of dynamic pore pressure amplitude versus number of cycles shown on Figures 4.10 to 4.13. The plots of dynamic pore pressure amplitude versus number of cycles have been constructed from data taken at the lowest transducer. This corresponds to a depth of 3.4 ft (1 m) or 90 percent of the total depth of the test section. The data from this depth is presented because it shows the greatest magnitude of change over the duration of the test. Relationships could have been developed for any depth within the profile.

4.1.2 Test Exhibiting Pore Pressure Accumulation and Liquefaction

The results of pore pressure measurements taken during Test 4 revealed that liquefaction of the sand layer occurred within the first three cycles of the initial wave loading sequence. This was further evidenced by gross disturbance of the test section following the test as well as settlement of the lead weights to the bottom of the test section. A complete discussion of Test 4 is included in Section 4.3.
The before and after soil conditions for Test 4 are presented in Table 3.10. Pore water pressure data for this test are presented on Figures 4.14 to 4.16. Figure 4.14 shows the mean pore pressure response in terms of pore pressure ratio versus number of cycles of wave loading. The pore pressure ratio is the ratio of the excess pore pressure (above hydrostatic) to the initial effective overburden pressure. The hydrostatic pressure corresponds to the still water level. The mean pore water pressures over a given wave cycle were used to compute the excess pressure values.

Figures 4.15 and 4.16 present the cycle pore pressure response during Test 4. These relationships were developed in the same manner as those presented previously for Tests 1, 2, 3 and 5 (not exhibiting liquefaction). These results are further discussed in Section 4.3.

4.1.3 Control Tests

Pressure measurements taken during the control wave tests were used primarily to evaluate the cyclic pore pressure response (McDougal et al., 1981). However, the results of these tests provided information with which to analyze whether the differences in cyclic pore pressure response were the result of differences in soil or wave conditions from test to test. Pressure measurements taken during the control tests revealed that the dynamic pressure fluctuations within the test section were dependent on the wave length. This information is presented in Figure 4.17 as a plot of dynamic pressure amplitude at a depth of 3.4 ft (1 m) in
Figure 4.14 Mean Pore Pressure Response versus Number of Cycles for Test 4-I
Figure 4.15 Non-dimensionalized Cyclic Pore Pressure Amplitude versus Depth for Test 4-I

WAVE HEIGHT = 3.8 ft.  PERIOD = 4.0 sec.
WAVELENGTH = 57 ft.  WATER DEPTH = 8.0 ft
Figure 4.16 Cyclic Pressure Amplitude versus Number of Cycles for Test 4-I
Figure 4.17 Cyclic Pressure Amplitude versus d/L for the Control Tests
the test section versus depth of water divided by wave length (d/L) for all the control test series.

4.2 Comparison of Results from Wave Flume Tests and Numerical Model

The mean pore pressure responses recorded during the wave flume tests were compared to the responses predicted by the Seed and Rahman model presented in Chapter 2. The comparison is presented utilizing the steps outlined in Chapter 2. Computer programs OCEAN1 and STR1 developed by Seed and Rahman were used to analyze the wave flume tests.

Steps 1 and 2 - Determination of the Soil Profile and Wave Conditions

The soil profile with properties of the layers and wave conditions were determined during the test program as described in Chapter 3. They are presented in Tables 3.7 to 3.11. Required input to describe the soil profile included; layer geometry, buoyant unit weights, bulk modulus of elasticity, Poisson's ratio, compressibility, permeability, and relative density. Wave conditions were described by water depth, wave height, and wave period.

Steps 3 & 4 - Determination of Mudline Wave Pressures and Cyclic Shear Stresses Induced in the Soil

Mudline pressures and wave-induced cyclic shear stresses were computed using the computer program STR1. Two layer analyses were conducted which accounted for the sand and gravel layers but neglected the influence of the geotextiles and polyethylene sheet.
This was justified because these membranes were flexible and constituted a small percentage of the total soil profile.

Mudline pressures and cyclic shear stresses for the tests are given in Figures 4.18 to 4.26. These figures present the horizontal cyclic shear stress and cyclic shear stress ratio plotted against depth. The cyclic shear stress ratio is the ratio of the cyclic shear stress to the initial effective overburden pressure. Also shown are the wave height, wave length, wave period, and water depth.

Step 5 - Determination of Equivalent Uniform Cyclic Stress History

This step was not required because the wave loadings were uniform and monochromatic, producing uniform cyclic shear stresses for the duration of a given test.

Step 6 - Determining the Rate of Pore Pressure Accumulation

This step involved numerical modeling of the test data by the program OCEAN1 developed by Seed and Rahman. Providing the appropriate input data were straightforward with the exception of two criteria; the pore pressure generation constant (g), and development of a cyclic strength information for the sand and gravel layers. The remaining input included soil profile geometry, relative density, compressibility, and permeability, as listed on Tables 3.7 to 3.11.

Laboratory testing was not conducted to determine the value of the pore pressure generation constant or cyclic strength
Figure 4.18 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 1-I

Po = 71 LB/SQ FT

H = 4.0 FT       d = 8.0 FT
L = 44 FT       T = 3.2 SEC
Figure 4.19 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 2-1

Po = 84 LB/SQ FT
H = 3.8 FT       d = 8.0 FT
L = 57 FT        T = 4.0 SEC
Figure 4.20 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 3-I

Po = 58 LB/SQ FT
H = 2.2 FT  d = 8.0 FT
L = 83 FT  T = 5.5 SEC
Figure 4.21 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 3-P

Po = 79 LB/SQ FT
H = 3.7 FT  d = 8.0 FT
L = 44 FT   T = 3.8 SEC
Figure 4.22 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 4-1

\[
\frac{\tau_c}{\sigma'_{vo}}
\]

\[
\begin{align*}
\tau_c \\
\tau_c/\sigma'_{vo}
\end{align*}
\]

\[
Po = 84 \text{ LB/SQ FT}
\]

\[
H = 3.8 \text{ FT} \quad d = 8.0 \text{ FT}
\]

\[
L = 57 \text{ FT} \quad T = 4.0 \text{ SEC}
\]
Figure 4.23 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 5-I
Figure 4.24 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 5-8.1

Po = 44 LB/SQ FT
H = 2.0 FT  d = 8.0 FT
L = 57 FT  T = 4.0 SEC
Figure 4.25 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 5-s.2

Po = 89 LB/SQ FT

H = 4.0 FT        d = 8.0 FT

L = 57 FT         T = 4.0 SEC
Figure 4.26 Wave Induced Cyclic Shear Stress Ratio and Cyclic Shear Stress for Test 5-S.3

$Po = 58 \text{ LB/SQ FT}$

$H = 4.0 \text{ FT}$  $d = 8.0 \text{ FT}$

$L = 36 \text{ FT}$  $T = 2.8 \text{ SEC}$
information. Based on findings by Seed, et al. (1975) that a value for $\phi$ of 0.7 represents an average value for most soils, this value was used in the model. The development of cyclic strength information is explained below.

The cyclic strength information required by the model is the number of cycles to liquefaction, for the undrained case, for the wave induced cyclic shear stress ratios determined in the previous step. This relationship is most commonly developed from cyclic strength testing with simple shear or triaxial stress apparatus. For this study, the cyclic strength relationships were determined by combining the wave flume test results with data from the literature.

Cyclic strengths are usually presented in terms of a family of relationships such as shown on Figure 4.27 from DeAlba, et al (1975). The relationships represent the field cyclic strengths for a range of relative densities. The number of cycles required to cause liquefaction is for the undrained case. The relationships shown were developed for a uniform medium sand with a $D_{50}$ equal to 0.014 in. (0.35mm) subjected to unidirectional shaking. The wave flume studies utilized a fine uniform sand with a $D_{50}$ equal to 0.008 in. (0.20mm) under unidirectional wave loading. Although the grain characteristics and state of stress may differ between the test conditions associated with Figure 4.27 and the wave flume tests, it was assumed that the general shape of the relationships shown could be used to develop cyclic strength relationships for the wave flume test sand.
After DeAlba et al., 1975

Figure 4.27 Field Corrected Cyclic Strength Curves for Initial Liquefaction, for Undrained Unidirectional Loading, for a Uniform Medium Sand
To evaluate the appropriate curves for the fine sand employed in the wave tests, field "points", or values of number of cycles to liquefaction for given applied cyclic stress, were required. This was provided from the undrained test that liquefied, Test 4-I. Unfortunately, only one data point was available to develop the curves; however, as will be shown subsequently, the response predicted from these curves matched both the drained and undrained response of the wave flume tests very well. In addition, had laboratory strength curves been developed for the test sand, they would have been corrected, if necessary, to reflect the observed liquefaction during Test 4-I.

The field curves for the test sand were developed as follows:

a) Based on the results of Test 4-I, the number of cycles to liquefaction for a relative density of 51% is 3. This is shown on Fig 4.14. From the results of the cyclic shear stress model presented on Fig 4.22, liquefaction occurred at an average cyclic shear stress ratio of 0.065. These values represent a point on the cyclic strength curve for the test sand at a relative density of 51%. It is important to note that the validity of this field point is based on, (1) the fact that the sand layer liquefied uniformly with depth, and (2) the shear stress ratio was relatively constant with depth.

b) Once this point was established, the remaining field curve at a relative density of 51% was determined by direct extrap-
olation from the relationship given by DeAlba, et al (re. Figure 4.27) for a relative density of 51%. The field value and resulting cyclic strength curve are shown on Fig 4.28.

c) The remaining two relationships required at relative densities of 60% and 77% were constructed based on the finding by DeAlba, et al, (1975) that the cyclic shear stress ratio at a given number of cycles is proportional to relative density. This applies to relative densities between 50% and 80%, which are within the range of initial relative densities tested. The resulting field relationships for the test sand are shown on Figure 4.28.

Cyclic strength for the gravel layer was not required as it was considered non-liquefiable based on the high permeability. The geotextile layers were not considered in the model because of their high permeability with respect to the sand. The drainage conditions during wave loading were controlled by the permeability of the sand layer except when the polyethylene sheet was used.

Utilizing the cyclic strength relationships (re. Figure 4.28) and soil properties determined during the test program, the theoretical rates of pore pressure accumulation were computed for the wave flume tests. These are presented below.

4.2.1 Theoretical Versus Measured Pore Pressure Accumulation for Test Exhibiting Liquefaction

The results from the numerical model compared to the wave flume test results for Test 4-I are presented on Figure 4.29, which shows the pore pressure ratio versus the number of cycles for the first four wave loading cycles. For the first four cycles, good
Figure 4.28 Developed Cyclic Strength Relationships for the Test Sand for Undrained, Unidirectional Wave Loading
Figure 4.29 Measured and Theoretical Pore Pressure Ratio versus Number of Cycles for the First Four Cycles of Test 4-I for All Depths within the Sand Layer
agreement between the theoretical and measured response occurs. After the fourth cycle, the measured pore pressure ratio decreases, as shown on Fig. 4.14, due to disturbance of the impermeable polyethylene sheet. The theoretical model predicts a constant pore pressure ratio equal to 1.0 (liquefaction) for all cycles beyond the fourth. If the effect of the gross disturbance of the sand layer on the soil properties, in particular the permeability, could have been measured, the model could have been used to predict this response. This was not possible or necessary to verify the model.

4.2.2 Theoretical Versus Measured Pore Pressure Accumulation for Tests not Exhibiting Liquefaction

The wave flume tests not exhibiting pore pressure accumulation were Tests 1-I, 2-I, 3-I, 3-S, 5-I, and the 5-S series. Utilizing the previously described soil properties and cyclic strength relationships, these tests were modeled for their theoretical response. The results of the model agreed with the measured results for Tests 1-I, 2-I, 3-I, 3-S, and 5-I in that no pore pressure accumulation was predicted. These results are not shown graphically because they are straight lines corresponding to values of zero pore pressure ratio for all cycles.

The theoretical results did not agree completely with the test results for the 5-S series. These tests were conducted to evaluate the effects of a previously drained loading history on a subsequently undrained deposit. Three wave loading conditions were imposed during the 5-S series: 5-S.1 produced relatively low cyclic shear stresses, and 5-S.2 and 5-S.3 produced relatively high cyclic
shear stresses. Liquefaction was not observed for any of these tests; however, the theoretical results predicted liquefaction for Tests 5-S.2 and 5-S.3. (Note: The increase in relative density of the sand layer during Test 5-I was reflected in the numerical models for Tests 5-S.1, 5-S.2 and 5-S.3. This is further discussed in Section 4.3 and shown in Table 4.3).

The comparison of theory versus measured mean pore pressure accumulation for Tests 5-S.2 and 5-S.3 are shown in Figures 4.30 and 4.31, respectively. This comparison is not shown for Test 5-S.1 because both the actual and measured responses produced an excess pore pressure ratio of zero.

The discrepancy between theory and measured response during the 5-S series is attributed to the previously undrained loading history. This is discussed further in Section 4.3.

4.3 Discussion of Wave Flume Test Results

The results of the wave testing phase were studied to aid in evaluating the conditions under which wave-induced pore pressure accumulation and liquefaction occur. The results also provided insights toward understanding wave-soil interaction.

The mechanics of a wave-soil system are very complex. This is particularly true when considering the interaction of wave-induced pore pressures and the soil structure. The causes and effects of either cannot be separated. The remainder of this section has been subdivided to discuss the following.

1) Wave Conditions.
Figure 4.30 Measured and Theoretical Pore Pressure Ratio versus Number of Cycles for Test 5-S.2 for All Depths Within the Sand Layer
Figure 4.31 Measured and Theoretical Pore Pressure Ratio versus Number of Cycles for Test 5-S.3 for All Depths Within the Sand Layer
2) Soil Response During Wave Loading - Non Liquefied Tests.

3) Soil Response During Wave Loading - Liquefied Test.

4) Soil Response During Wave Loading - Control Tests.

5) Wave Loading History.

4.3.1 Wave Conditions

The wave conditions were the forcing function for the system. The resulting pressure wave at the mudline induced cyclic shear stresses in the test section as well as cyclic pore pressure fluctuations. Mudline pressure fluctuations were measured to be uniform, monochromatic pressure waves in-phase with the surface waves.

Observations made during the tests revealed that both the density of the soil and the dynamic pore pressure fluctuations changed during wave loading. The specifics of these changes are discussed subsequently; however, it is important at this point to note that changes occurring below the mudline did not affect the mudline pressure fluctuations. The strongest supporting evidence for this statement was observed in Test 4-I, in which pore pressure accumulation and liquefaction occurred. From the data presented in Figures 4.14 to 4.16, it can be seen that significant changes in both the mean and the dynamic pressure occurred within the sand layer during Test 4-I. The corresponding mudline pressure fluctuations remained unchanged. The importance of this observation is that it permits the wave-soil system for these test conditions to be modeled by applying a pressure wave at the mudline that is
unaffected by changes occurring below mudline. This substantiates a basic assumption of the Seed and Rahman model.

The wave-induced mudline pressure fluctuations determine the magnitude of the induced cyclic shear stresses. The model developed by Seed and Rahman uses linear wave theory to predict the magnitude of the mudline wave pressures (Eq 2.2). A comparison of the measured and predicted values is presented in Table 4.1. The comparison shows that the predicted mudline pressure fluctuations were from 4 to 20 percent higher than the measured values. Since wave-induced cyclic shear stresses are directly proportional to mudline pressure fluctuations, the model overpredicts the shear stresses by the same amount. However, this results in a conservative estimate.

The effect of wave conditions on the cyclic shear stresses is presented in Table 4.2, in which the average cyclic shear stress ratio in the sand layer is given for various wave conditions. The data are plotted in Figure 4.32. The relationship shown represents an average condition for (1) the geometries of the test section, (2) a range of initial relative densities between 51% and 77%, and (3) a range of wave heights between 2 and 4 ft (.6 and 1.2m). The results indicate that for a water depth of 8.0 ft (2.4m), the highest cyclic shear stresses were produced by waves with a height to length ratio between 0.06 and 0.09 for the range of soil conditions and wave heights considered.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Wave Height (H) in feet</th>
<th>Wave-length (L) in feet</th>
<th>Period (T) in seconds</th>
<th>Measured P₀/H*</th>
<th>Predicted P₀/H*</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-I</td>
<td>4.0</td>
<td>44</td>
<td>3.8</td>
<td>0.48</td>
<td>0.58</td>
<td>17</td>
</tr>
<tr>
<td>2-I</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
<td>0.58</td>
<td>0.70</td>
<td>17</td>
</tr>
<tr>
<td>3-I</td>
<td>2.2</td>
<td>84</td>
<td>5.5</td>
<td>0.67</td>
<td>0.84</td>
<td>20</td>
</tr>
<tr>
<td>3-S</td>
<td>3.7</td>
<td>44</td>
<td>3.8</td>
<td>0.48</td>
<td>0.58</td>
<td>17</td>
</tr>
<tr>
<td>4-I</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
<td>0.58</td>
<td>0.70</td>
<td>17</td>
</tr>
<tr>
<td>5-I</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
<td>0.58</td>
<td>0.70</td>
<td>17</td>
</tr>
<tr>
<td>5-S.1</td>
<td>2.0</td>
<td>57</td>
<td>4.0</td>
<td>0.67</td>
<td>0.70</td>
<td>4</td>
</tr>
<tr>
<td>5-S.2</td>
<td>4.0</td>
<td>57</td>
<td>4.0</td>
<td>0.62</td>
<td>0.70</td>
<td>11</td>
</tr>
<tr>
<td>5-S.3</td>
<td>4.0</td>
<td>36</td>
<td>2.8</td>
<td>0.41</td>
<td>0.48</td>
<td>15</td>
</tr>
</tbody>
</table>

* P₀ = Dynamic Pressure Amplitude at the Mudline

** Water depth constant at 8.0 ft.
TABLE 4.2 COMPARISON OF AVERAGE WAVE-INDUCED CYCLIC SHEAR STRESS RATIOS FOR THE SAND LAYER

**WAVE CONDITIONS**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Wave Height(H) in feet</th>
<th>Wave-length(L) in feet</th>
<th>Period(T) in seconds</th>
<th>H/L</th>
<th>Average Cyclic Shear Stress Ratio($T_{c}/\sigma_{vo}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-I</td>
<td>4.0</td>
<td>44</td>
<td>3.8</td>
<td>.091</td>
<td>.069</td>
</tr>
<tr>
<td>2-I</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
<td>.067</td>
<td>.066</td>
</tr>
<tr>
<td>3-I</td>
<td>2.2</td>
<td>84</td>
<td>5.5</td>
<td>.026</td>
<td>.032</td>
</tr>
<tr>
<td>3-S</td>
<td>3.7</td>
<td>44</td>
<td>3.8</td>
<td>.084</td>
<td>.062</td>
</tr>
<tr>
<td>4-I</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
<td>.067</td>
<td>.065</td>
</tr>
<tr>
<td>5-I</td>
<td>3.8</td>
<td>57</td>
<td>4.0</td>
<td>.067</td>
<td>.067</td>
</tr>
<tr>
<td>5-S.1</td>
<td>2.0</td>
<td>57</td>
<td>4.0</td>
<td>.035</td>
<td>.036</td>
</tr>
<tr>
<td>5-S.2</td>
<td>4.0</td>
<td>57</td>
<td>4.0</td>
<td>.070</td>
<td>.071</td>
</tr>
<tr>
<td>5-S.3</td>
<td>4.0</td>
<td>36</td>
<td>2.8</td>
<td>.111</td>
<td>.062</td>
</tr>
</tbody>
</table>

* Water depth constant at 8.0 ft.*
Figure 4.32 Average Cyclic Shear Stress Ratio versus H/L for the Sand Layer
4.3.2 Soil Response During Wave Loading - Non-liquefied Tests

The response of the sand layer during the wave flume tests was studied to evaluate whether correlations existed between the observed responses and the parameters considered in the program (i.e., wave, soil, drainage and loading history conditions). The response of the mean pore pressure was of primary interest in the research program; however, the cyclic pore pressure response provided valuable insight into the response of the sand layer during the tests. The cyclic pore pressure response can be used to evaluate changes occurring in the soil over the duration of the tests. This is important because the density tests used to evaluate soil conditions were only conducted before and after the complete series of wave tests (i.e., initial, subsequent and control wave tests). The total number of loading cycles applied between density tests ranged from 650 to 860. The cyclic pressure amplitude measurements provided a means for determining the cycles over which changes in the soil conditions occurred and the time rate of these changes. This was particularly important in evaluating the effects of loading history, where changes in the soil conditions during the initial wave tests directly affected the response during subsequent wave tests.

Data pertaining to changes in cyclic pore pressure fluctuations are presented on Figs. 4.1 through 4.13. These plots were explained in Section 4.1. The test results illustrate two basic patterns of cyclic pore pressure response (with the exception of Test 4-I which will be discussed subsequently), as follows:
1. The amplitude of the cyclic pore pressure decreased during the loading cycles and reached a constant value (re. Tests 1-I, 2-I, 3-I, 3-S, 5-I).

2. The amplitude of the cyclic pore pressure remained unchanged during the tests (re. Tests 5-S.1, 5-S.2, 5-S.3).

The observed attenuation in cyclic pore pressure amplitude for Tests 1-I, 2-I, 3-I, 3-S and 5-I was accompanied by a measured decrease in sand layer thickness and an increase in relative density. This response indicates that a direct correlation exists between changes in cyclic pore pressure amplitude and relative density. Moreover, it indicates that additional correlations exist between changes in cyclic pore pressure and soil properties that are directly related to relative density.

The relationship between changes in relative density and changes in the cyclic pore pressure amplitude was particularly important for evaluating the "before" test relative density of the subsequent wave loading tests. For the 5-S series, where no change in the cyclic pore pressure amplitude was observed, the relative density remained constant and corresponded to the value measured following this series of tests. Based on the response of the cyclic pore pressure amplitude during Test 5-I, a decrease during the initial cycles followed by a constant response, the soil had densified and remained at a constant density to the end of the test. The wave induced stresses produced by the 5-S series did not densify
the sand layer further. The results of Test 3-S, where with time, a slight decrease in the cyclic pore pressure amplitude was observed, indicate that the before test density was somewhat less than the value measured following this test.

A comparison of the decrease in cyclic pore pressure amplitude at the bottom of the test section, with initial and final relative densities (for all tests except 4-I) is shown in Table 4.3. The table indicates tests for which relative densities were inferred from the cyclic pore pressure response. Also shown are the average cyclic shear stress ratios within the sand layer.

This information shows the following:

1. The greatest decrease in cyclic pore pressure amplitude occurred for the initial loading tests with the lowest initial relative densities (re. Tests 1-I, 2-I and 5-I).

2. In Test 5-I, the initial drained loading produced significant cyclic shear stresses, however a decrease in the dynamic pressure fluctuation did not occur during the subsequent undrained loading tests even though the cyclic shear stresses in Test 5-S.2 exceeded those for Test 5-I.

3. In Test 3-I, the decrease in cyclic pore pressure amplitude was very small for a low wave-induced stress and higher initial relative density.
TABLE 4.3 COMPARISON OF THE DECREASE OF CYCLIC PORE PRESSURE AMPLITUDE OVER THE DURATION OF WAVE LOADING

<table>
<thead>
<tr>
<th>Test</th>
<th>Type</th>
<th>Decrease of Cyclic Pore Pressure Amplitude at z = 3.4 ft in Percent</th>
<th>Average Cyclic Shear Stress Ratio Within The Sand Layer</th>
<th>Initial Relative Density in Percent</th>
<th>Final Relative Density in Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-I</td>
<td>Drained</td>
<td>10</td>
<td>0.069</td>
<td>51</td>
<td>93</td>
</tr>
<tr>
<td>2-I</td>
<td>Drained</td>
<td>5</td>
<td>0.066</td>
<td>60</td>
<td>85</td>
</tr>
<tr>
<td>3-I</td>
<td>Drained</td>
<td>1 1/2</td>
<td>0.032</td>
<td>77</td>
<td>77*</td>
</tr>
<tr>
<td>3-S</td>
<td>Drained</td>
<td>1/2</td>
<td>0.062</td>
<td>77*</td>
<td>77*</td>
</tr>
<tr>
<td>5-I</td>
<td>Drained</td>
<td>4</td>
<td>0.067</td>
<td>51</td>
<td>77*</td>
</tr>
<tr>
<td>5-S.1</td>
<td>Undrained</td>
<td>0</td>
<td>0.036</td>
<td>77*</td>
<td>77*</td>
</tr>
<tr>
<td>5-S.2</td>
<td>Undrained</td>
<td>0</td>
<td>0.071</td>
<td>77*</td>
<td>77*</td>
</tr>
<tr>
<td>5-S.3</td>
<td>Undrained</td>
<td>0</td>
<td>0.062</td>
<td>77*</td>
<td>77*</td>
</tr>
</tbody>
</table>

* Relative densities determined from interpolating between before and after test section densities based on cyclic pore pressure response.
4. The decrease in cyclic pore pressure amplitude for subsequent load Test 3-S was less than that resulting from initial loading tests with a comparable magnitude of induced stresses (re. Tests 1-I and 2-I).

As previously discussed, the amplitude of the cyclic pore pressure changed while the amplitude of the cyclic mudline pressure remained constant. This confirms that cyclic pore pressure changes must be occurring in response to changes within the soil structure. This is supported by the results of the "before and after" tests that show an increase in density of the sand layer and a decrease in sand layer thickness during the tests. Another possible explanation of the attenuation of cyclic pore pressures could have been the partial clogging of the geotextile layer. However, visual inspection of the geotextiles following testing revealed that significant clogging did not occur.

The pre-testing laboratory studies showed that increased density of the sand layer results in the following:

1. A decrease in the compressibility.
2. An increase in the stiffness.
3. A decrease in the permeability.
4. An increase in Poisson's ratio.

Although a discussion of the effects of these variables on the cyclic pressure amplitudes is beyond the scope of this thesis, the above changes represent an increase in the stability of the layer. The layer stabilized under the given wave and soil conditions and resisted further change.
The effect that the aforementioned variables has on the mean pore pressure accumulation was evaluated by comparing the results of the two undrained Tests 4-I and 5-S.2, conducted with similar wave conditions. Test 4-I exhibited pore pressure accumulation and liquefaction while Test 5-S.2 did not. The significant difference between the two tests is that 5-S.2 was subjected to a previous loading history while 4-I was not. The results of the cyclic pore pressure attenuation for the wave loading prior to 5-S.2 (5-I and 5-S.1) show that the sand layer had reached a stable condition before Test 5-S.2. This condition was reflected by the increased density and corresponding changes in soil properties. Therefore, the following soil properties increase the stability of the deposit against pore pressure accumulation:

1) High relative density.
2) High permeability.
3) Low compressibility.
4) High stiffness.
5) High Poisson's ratio.

For the drained tests the permeability of the geotextiles was much greater than the permeability of the sand layer. Based on the observation that pore pressure accumulation did not occur for the drained tests, it may be concluded that the permeability of the sand layer was the controlling factor. Therefore, the importance of permeability, compressibility, stiffness, relative density and Poisson's ratio is not significant to pore pressure accumulation.
unless the permeability is less than a "critical" value. The critical value of permeability for these test conditions was not determined, but the value is lower than the permeability of the sand employed. This implies that pore pressure accumulation will not occur in the fine sand employed in the test program for the range of test conditions considered, unless drainage conditions are imposed that reduce the permeability of the sand below its natural range. A reduction in permeability was accomplished in the test program with the polyethylene sheet. A similar situation may occur in the field when the sand is overlain by a deposit of lower permeability such as a silt or clay, or drainage is artificially impeded by a structure.

4.3.3 Soil Response During Wave Loading - Liquefied Test

The response of the dynamic pressure fluctuations for wave Test 4-I was dramatically different than the other tests. This was due to liquefaction of the sand bed. As previously stated, Test 4-I was the only test to demonstrate liquefaction. It was the only initial wave test that was undrained. The other undrained tests were the 5-S series, but they were subjected to previous wave loadings.

The pore pressure response during wave Test 4-I is shown on Figures 4.14 to 4.16. The results presented show the response of the dynamic and mean pore pressure during the test. This response can be divided into three groups of wave loading cycles, as shown on Figure 4.33.
a) CYCLE 0
EXCESS PORE PRESSURE EQUALS ZERO

b) CYCLE 3
EXCESS PORE PRESSURE EQUALS 100%

c) CYCLE 4 TO 35
EXCESS PORE PRESSURE DECREASES FROM 100 TO 75%

d) CYCLE 36 TO 200
EXCESS PORE PRESSURE EQUALS ZERO AT CYCLE 90

Figure 4.33 Test Section Response to Liquefaction During Test 4-I
1. **Initial Three Cycles.** During these cycles the mean pore pressure built up to a value equal to the effective stress resulting in complete liquefaction of the sand layer. Ninety percent of the mean pressure increase occurred within the first two cycles. This is shown on Figure 4.33 (b). The analog pore pressure recording taken during these cycles is shown on Figure 4.34.

The test results for the first two cycles confirm that wave-induced liquefaction resulted from the mean pore pressure equaling the effective overburden stress, as opposed to liquefaction resulting from the peak cyclic pore pressure equaling the effective stress. The observed mechanism of liquefaction is given on Figure 4.35. This was verified by measuring the mean pore pressure increase and comparing it to the initial effective overburden pressure. If liquefaction had resulted from peak cyclic pressures the measured mean pore pressure increase would have been substantially less than the effective overburden pressure.

Based on these observations, the mechanism of pore pressure response during liquefaction may be described as follows. The total pore pressure response is dependent on the superimposed response of 1) the transfer (or no transfer) of intergranular stress to the pore fluid, and 2) the wave-induced cyclic pressure fluctuations. Accumulation of excess pore pressure and liquefaction is associated with the transfer of intergranular stress to the pore fluid, observed as an increase in the mean pressure. The wave-
Figure 4.34 Analog Recording of Wave Pressure at the Mudline and Pore Pressure Response at a Depth of 1.9 Feet Below the Mudline for the First 15 Cycles of Test 4-I
Figure 4.35 Observed Wave-induced Liquefaction Response—in which the Mean Pore Pressure Increases to the Initial Effective Stress
induced cyclic pressure does not represent a transfer of intergranular stress to the pore fluid, but rather a transfer of wave-induced fluid pressure to the pore fluid.

This mechanism explains why peak pore pressures in excess of the initial effective overburden pressure were observed following liquefaction. If the peak pressures had represented a transfer of intergranular stress to the pore fluid, for the undrained conditions of Test 4-I, these pressures would not have dissipated and the pressure wave would have appeared as a series of troughs without crests. The mean pressure would not have increased to the point of equaling the initial effective overburden pressure.

During the initial three cycles, the cyclic pressure amplitude attenuated significantly as the mean pore pressure increased. The magnitude of attenuation was dependent on depth, as shown on Figure 4.15. The cyclic variation at the bottom boundary is shown on Figure 4.16. This occurrence is believed to be caused by the damping action of the liquefying sand. As the sand liquefied it became a slurry, behaving as a dense viscous fluid to dampen the cyclic pore pressures by as much as 45 percent at the bottom of the test section for cycle number three.

2. Cycles 3 to 35. During these cycles, the mean pore pressure decreased slowly from 100 to 75 percent of the initial effective stress. This response is shown on Figure 4.16. A graphical representation of the test section is given on Figure
4.33(c). Although visual examination was not possible during the test, the reduction in mean pressure was probably caused by drainage around the sides of the impermeable polyethylene sheet above the sand layer.

During these cycles, the dynamic pressure component actually increased to a value higher than experienced at the mudline. This was a gradual phenomenon and a possible explanation would be that the low effective stress allowed the sand slurry to undulate at the same frequency as the waves with increasing amplitude. Undulations of the sand slurry imposed an increased pressure on the pore fluid resulting in higher than expected dynamic pressure fluctuations. After the 20th cycle (approximately) the effective stress increased to a value at which this effect began to reverse.

3. Cycles 36 to 200. Between Cycles 36 and 90 the mean excess pore pressure decreased at an increasing rate to a value of zero and remained at this value until the end of the test (Cycle 200), as shown on Figure 4.16. The condition of the test section during these cycles is illustrated on Figure 4.33(d). The increased rate of drainage was the result of increased disturbance of the polyethylene sheet allowing a greater area for dissipation of excess pore pressure.

During Cycles 36 to 90, the dynamic pressure fluctuations followed the same trend as observed in Tests 1-I, 2-I, 3-I, 3-P and 5-I. That is, the amplitude decreased and reached a constant
value. Based on the knowledge that 1) the sand layer liquefied, and 2) the final density of the sand layer following Test 4-I was comparable to the other final test densities, this trend clearly supports the assumption that a more stable configuration of the sand layer was achieved.

4.3.4. Soil Response During Wave Loading - Control Tests

The test results showed a significant difference in the final magnitude and attenuation of cyclic pore pressure during testing. This is shown in Table 4.3. The results of the control tests were used to evaluate these differences and are presented on Figure 4.17, which shows the nondimensionalized cyclic pore pressure \( \frac{p_z}{p_o} \) at the bottom boundary versus \( d/L \). The information presented in Figure 4.17 shows that greater cyclic pore pressure attenuation occurred for the shorter wavelengths.

The last cycle values of \( \frac{p_z}{p_o} \) for the initial and subsequent wave tests from Figures 4.10 to 4.13 fall very close to the control test values at their respective wavelengths. The similarity of these values indicates that the soil conditions during the control tests did not change from the condition achieved during the initial and subsequent wave tests. The stabilized condition was verified by checking the cyclic pore pressure attenuation with number of cycles of wave loading during the control tests and no attenuation was observed. The small deviation of some of the last cycle values is attributed to a slight dependence of attenuation to wave height, observed during the control tests. Although a trend
existed, the dependence on wave height was inconclusive because the range of wave heights tested produced a difference of only about 2 percent. In general, the greater wave heights produced higher attenuation values.

The results from the control wave tests indicate the following:

1) The change of attenuation with number of wave cycles during the initial and subsequent wave tests was the result of changes occurring within the soil. If these changes had been attributable to the wave conditions, the same pattern of attenuation would have been observed during the control wave loading tests.

2) The control wave test results indicate that the attenuation of cyclic pore pressure with depth is controlled by both wave and soil conditions. This is illustrated on Figure 4.17. The dependence on wave conditions is given by the shape of the relationships, indicating greater attenuation with depth for shorter wavelengths. This dependence on soil conditions is shown by the fact that the control relationships do not coincide. The difference in attenuation between the control relationships was reviewed to attempt a correlation with soil and drainage conditions in the test section. However, no direct correlation was found. This may be attributed to the small range of sand layer density during the control tests. It is interesting to note that the curve for the undrained Test 5-C.2 is flatter than those for Tests 1-C, 2-C, 3-C, 4-C and 5-C.1 and has a significantly higher attenuation for longer
waves. This response, in part, may have been due to partial upward reflection of the pressure waves off the impermeable polyethylene sheet. This response was not observed in the other undrained control test, Test 4-C, due to complete disturbance of the polyethylene sheet.

4.3.5 Previous Loading Conditions

Previous wave loading on the test section played a significant role in determining the response of the sand layer to subsequent wave loading. This is evidenced by the results of the undrained Test 3-S. During Test 3-S the wave loading produced significantly higher shear stresses than for Test 3-I, as shown in Table 4.2. However, as shown in Table 4.3, the degree of cyclic pore pressure attenuation with number of cycles for Test 3-S was significantly less than that for initial undrained wave tests (Tests 1-I, 2-I and 5-I) of comparable shear stresses. This indicates that previous loading has a significant effect on stabilizing the sand layer.

The stabilizing effect of previous wave loading was demonstrated by comparison of the results of Test 4-I and Test 5. Test 5 consisted of a drained initial test followed by three undrained loading tests. As shown on Figure 4.13, the initial wave loading for Test 5-I produced cyclic pore pressure attenuation with number of wave cycles but the subsequent tests did not. The results of the density testing and constant cycle pore pressure response indicate the relative density of the sand layer was
stabilized to a value of 77% following Test 5-I and remained at this value through the 5-S series (see Table 4.3).

Test 4-I was conducted undrained, at an initial relative density of 51%, and liquefied in 3 cycles under an imposed cyclic shear stress ratio of 0.065. During Test 5-S.2, the imposed cyclic shear stress ratio due to wave loading was 0.071, and pore pressure accumulation was not observed. Despite the increased relative density of the sand layer in Test 5-S.2, the model predicted liquefaction in 12 cycles as shown in Figure 4.30. Based on the fact that liquefaction was not observed, it is concluded that previous wave loading (previous strain history from shear stress reversals) had the effect of increasing the cyclic strength of the sand layer. This results in a shift of the cyclic strength curve as illustrated in Figure 4.36.

This same difference between the measured and theoretical results occurred by undrained Test 5-S.3; liquefaction was predicted but pore pressure accumulation did not occur. The theoretical and observed results for Test 5-S.3 are shown on Figure 4.31.
Figure 4.36 Developed Cyclic Strength Relationships for the Test Sand for Undrained, Unidirectional Wave Loading, Showing Effect of Previous Wave Loading
5.0 SUMMARY AND CONCLUSIONS

5.1 Summary

Ocean waves propagating over marine soil deposits transmit pressure fluctuations to the sea floor. Although wave motions may be very complex, it is common practice to describe the sea surface and resulting pressure fluctuations as simple period functions. The resulting cyclic wave loading on the seafloor creates cyclic shear, normal, and neutral stresses within the sediment structure, and cyclic displacements of the soil. When fine cohesionless soils are subjected to cyclic shear stresses they may consolidate, depending on the magnitude of the shear stresses and soil structure. The tendency for consolidation transfers some or all of the intergranular stress to the pore fluid, resulting in an increase in pore pressure. If pore pressures are not allowed to dissipate, due to restricted drainage, they may accumulate. Accumulation of pore pressure results in a reduction of strength for the deposit and in the extreme case liquefaction; where the excess pore pressure accumulates until it equals the initial effective stress.

To model wave-induced pressure accumulation and liquefaction, a series of large scale wave flume tests were conducted. Testing consisted of generating a series of uniform waves over a sand deposit and measuring the pore pressure response. The pore pressure response was studied for a variety of wave and soil conditions. Both the cyclic and mean pore pressure responses were measured, where the mean response was the time-averaged cyclic pore
pressure response. The measured mean pore pressure response was compared to a theoretical model developed by Seed and Rehman (1977).

5.2 Conclusions

The conclusions of this study are as follows:

1. Wave loadings on cohesionless soil deposits induce cyclic shear stresses that may cause pore pressure accumulation within the deposit. Moreover, the magnitude of the cyclic shear stresses can be great enough to cause pore pressure accumulation resulting in liquefaction for the undrained case. Wave-induced liquefaction occurred for a fine sand deposit at an initial relative density of 51 percent. Evidence supporting liquefaction included: (1) the pore pressure response, which showed that the mean pore pressure gradually increased over the first three wave cycles of the test until it was equal to the initial effective overburden pressure, and (2) observations made after the test which revealed complete disturbance of the sand layer.

2. Pore pressure accumulation and liquefaction are controlled by the mean pore pressure response and not the cyclic pore pressure response. The complete pore pressure response is the sum of the mean and cyclic pore pressure response. Under stable conditions the cyclic pore pressures fluctuate about a constant mean pore pressure corresponding to the hydrostatic mean pressure as measured from the still water level. During pore pressure accumulation, the mean pore pressure increases above the hydrostatic mean pressure and the cyclic pore pressures fluctuate about the shifting mean pressure. The increase in mean pore pressure represents a
transfer of intergranular stress to the pore fluid, whereas the cyclic response represents fluid pressures that are transferred to the pore fluid only. This is significant because instantaneous "peak" cyclic pore pressures (representing the wave crests) that are greater than the initial effective overburden pressure can be induced in fine cohesionless deposits without causing liquefaction. In order for liquefaction to occur, effective soil pressures must be transferred to the pore fluid.

3. The liquefaction susceptibility of cohesionless deposits depends upon the drainage condition of the deposit. Two tests were conducted with identical wave and soil conditions: One test was conducted undrained and the other drained. The drained test did not exhibit pore pressure accumulation, while the undrained test liquefied in three cycles. The rapid liquefaction for the undrained case implies that impeded drainage of the sand may cause pore pressure accumulation or liquefaction in more than three cycles. This is important where a sand deposit contains layers with permeabilities significantly less than the sand deposit. The evidence developed in this program suggests that several models should be considered to evaluate the effects of impeded drainage.

4. Previous loading history plays an important role in determining the susceptibility of a cohesionless sand deposit to pore pressure accumulation. This is based on the results of two tests conducted under similar initial soil and wave conditions but for one test the sand deposit was subjected to a previous wave loading history. Test 4-I was conducted undrained, at an initial
relative density of 51%, and liquefied in 3 cycles. Test 5-S.2 was subjected to a previous drained loading history that consolidated the sand bed to a relative density of 77%. The subsequent wave loading during Test 5-S.2 did not produce mean pore pressure accumulation.

The numerical model predicted the observed liquefaction of Test 4-I in 3 cycles, however it also predicted liquefaction of Test 5-S.2 in 12 cycles, where no accumulation of pore pressure was observed. The evidence implies that the previous loading history increased the cyclic strength of the deposit above that achieved by the increase in relative density alone. Although the test results do not provide a means to evaluate the effect of previous loading history in a quantitative manner, the results clearly show that previous wave loading history increases the magnitude of shear stress required to produce liquefaction. Furthermore, these results indicate that the effect of previous wave loading on a deposit cannot be evaluated solely on the relative density of the deposit.

5. The wave-soil system for cohesionless sediments can be modeled by imposing a pressure wave at the mudline that is unaffected by changes occurring below the mudline. This was verified during Test 4 which liquefied. During the test, both the mean and cyclic pore pressures showed extreme variation. The cyclic pore pressures increased above those measured at the mudline. The mean pore pressure within the test section increased an average of 20 percent during liquefaction. These changes occurred while the cyclic and mean mudline pressures remained unchanged.
6. Large scale wave flume tests can be successfully utilized to model the potential for mean pore pressure accumulation and liquefaction of cohesionless marine deposits. Large scale wave flume tests were shown to be particularly important to establishing wave conditions that will induce liquefaction for the undrained case and, therefore, provide a "field value", or values, that can be used directly in subsequent numerical modeling. The majority of numerical models for wave-induced liquefaction utilize cyclic shear strength relationships obtained from laboratory cyclic shear or cyclic triaxial tests, corrected to field conditions. Large scale wave flume tests can be a powerful tool to provide input that more closely resembles actual field conditions. This study shows that the following parameters may be investigated with large scale wave flume tests: (1) variable wave loading conditions, (2) variable relative density of the sand, (3) undrained and drained loading conditions, and (4) effects of previous loading history.

7. The numerical model presented by Seed and Rahman (1977) can be used to accurately model the response of cohesionless marine deposits to wave loading. Their theoretical results closely matched the observations except for the case in which the deposits were subjected to a previous wave loading history.

5.3 Recommendations for Further Research

Large scale wave flumes provide an excellent means for studying pore pressure accumulation and liquefaction resulting from wave loading. By utilizing a wave flume such as that located at the
Oregon State University Wave Research Laboratory, and the soil containment structure and pore pressure transducers described in this study, a wide range of soil and wave conditions may be considered. The following studies are suggested and would provide laboratory data where a severe shortage now exists, namely, in the evaluation of pore pressure accumulation and liquefaction of marine soils due to wave loading:

1. Drained and undrained wave loading tests directed at producing pore pressure accumulation and liquefaction should be conducted. Variables in these studies would include:

   a) Wave Conditions

   Variable wave conditions would include monotonic and random waves. Where liquefaction was observed, this data could be used to verify present methods to convert an irregular time history of wave loading to an equivalent uniform storm.

   b) Soil Conditions

   Variable soil conditions would include soil type and soil profile. Layered and uniform soil profiles would be tested with additional variable soil index properties, such as relative density, permeability, and compressibility, for each soil type.

   c) Drainage Conditions

   Variable drainage conditions would include drained and undrained tests as well as artificially imposed drainage from
structures or layers of lower permeability constructed within the main deposit. An example of this would be the placement of a silt layer between the sand and gravel layers described in the present study. The silt layer would be separated from the surrounding sand and gravel by utilizing geotextiles. The effect of variable silt layer thickness could be evaluated. The effect of permeability of the imposed "silt" drainage layer could be evaluated by mixing variable amounts of clean sand with the silt. Another means of limiting drainage would be to utilize a polyethylene sheet provided with small drain holes.

d) Previous Wave Loading History

The effect of wave loading history would be studied by subjecting deposits to a wide range of wave-induced stresses caused by small to large waves.

2. The potential for wave-induced liquefaction from standing waves should be evaluated.

3. The effect of combined structural and wave loading should be evaluated. This could be accomplished by the installation of a) a pile or group of piles into the deposit, b) a gravity structure above the deposit, or c) a pipeline or similar structure buried below the deposit. These studies would include monitoring structural movements in addition to measuring pore water pressures.

4. Studies should be conducted to evaluate the effect of pore water accumulation on marine slopes. Variable wave, soil, and drainage conditions could be studied as outlined in the previous recommendation.
BIBLIOGRAPHY


