April 11, 2014

Project No. 10594-001

Lennar Multifamily Investors, Inc.
25 Enterprise, Suite 305
Aliso Viejo, California 92656

Attention: Mr. Ethen Thacher

Subject: Report of Geotechnical Exploration
Proposed Oceanaire Project
150 West Ocean Boulevard
Long Beach, California

In accordance with our revised December 11, 2013 proposal, Leighton and Associates Inc. (Leighton) is pleased to present this geotechnical exploration report in support of the subject project. Our scope of work for this study included research, subsurface exploration, laboratory testing, engineering analysis, and preparation of this report.

Geotechnical aspects that require special consideration include the presence of undocumented fill that will require removal and shallow groundwater. Development of the site is considered feasible from a geotechnical standpoint provided the recommendations in this report are incorporated in the design and construction of the project.
We appreciate the opportunity to be of service to Lennar Multifamily Investors, LLC. If you have any questions or if we can be of further service, please call us at your convenience at (866) LEIGHTON, at the direct extensions listed below, or e-mail us as listed below.

Respectfully submitted,

LEIGHTON and ASSOCIATES, INC.

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JAR/CCK/Ir

Distribution: (4) Addressee
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Authorization</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Scope of Work</td>
<td>1</td>
</tr>
<tr>
<td>1.3 Study Area</td>
<td>2</td>
</tr>
<tr>
<td>1.4 Project Description</td>
<td>3</td>
</tr>
<tr>
<td>2.0 GEOTEchnICAL FINDINGS</td>
<td>4</td>
</tr>
<tr>
<td>2.1 Geologic Setting</td>
<td>4</td>
</tr>
<tr>
<td>2.2 Geologic Structure</td>
<td>4</td>
</tr>
<tr>
<td>2.2.1 Wilmington Oil Field</td>
<td>5</td>
</tr>
<tr>
<td>2.3 Subsurface Soil Conditions</td>
<td>6</td>
</tr>
<tr>
<td>2.4 Expansive Soil</td>
<td>7</td>
</tr>
<tr>
<td>2.5 Soil Corrosivity</td>
<td>8</td>
</tr>
<tr>
<td>2.6 Groundwater</td>
<td>9</td>
</tr>
<tr>
<td>3.0 GEOLOGIC/SEISMIC HAZARDS</td>
<td>10</td>
</tr>
<tr>
<td>3.1 Surface Fault Rupture</td>
<td>10</td>
</tr>
<tr>
<td>3.2 Historical Seismicity</td>
<td>10</td>
</tr>
<tr>
<td>3.3 Secondary Seismic Hazards</td>
<td>11</td>
</tr>
<tr>
<td>3.4 Slope Stability Analysis</td>
<td>13</td>
</tr>
<tr>
<td>3.4.1 Backcut Stability</td>
<td>14</td>
</tr>
<tr>
<td>4.0 CONCLUSIONS</td>
<td>15</td>
</tr>
<tr>
<td>5.0 DESIGN RECOMMENDATIONS</td>
<td>16</td>
</tr>
<tr>
<td>5.1 Earthwork</td>
<td>16</td>
</tr>
<tr>
<td>5.2 Site Preparation</td>
<td>16</td>
</tr>
<tr>
<td>5.3 General Grading Recommendations</td>
<td>16</td>
</tr>
<tr>
<td>5.4 Fill Placement</td>
<td>17</td>
</tr>
<tr>
<td>5.5 Pipe Bedding</td>
<td>18</td>
</tr>
<tr>
<td>5.6 Trench Backfill</td>
<td>18</td>
</tr>
<tr>
<td>5.7 Surface Drainage</td>
<td>18</td>
</tr>
<tr>
<td>5.8 Foundation Recommendations</td>
<td>19</td>
</tr>
<tr>
<td>5.9 Slab-On-Grade</td>
<td>20</td>
</tr>
<tr>
<td>5.10 Lateral Earth Pressures</td>
<td>20</td>
</tr>
<tr>
<td>5.11 Seismic Design Parameters</td>
<td>22</td>
</tr>
<tr>
<td>5.12 Hydrostatic Uplift</td>
<td>22</td>
</tr>
<tr>
<td>5.13 Temporary Excavations</td>
<td>23</td>
</tr>
<tr>
<td>5.14 Shoring</td>
<td>24</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS Continued

Section                                      Page

5.15 County of Los Angeles Building Code Section 111 Statement .......... 29
5.16 Additional Geotechnical Services ......................................................... 29

6.0 LIMITATIONS ..................................................................................................... 31
7.0 REFERENCES ................................................................................................... 32

ATTACHMENTS:

Important Information Regarding Your Geotechnical Exploration Report

Figures

Figure 1 – Site Location Map ........................................................................ Rear of Text
Figure 2 – Geologic Cross Section A-A’ .........................................................Rear of Text
Figure 3 – Regional Geology Map .................................................................. Rear of Text
Figure 4 – Regional Fault Map ....................................................................... Rear of Text
Figure 5 – Historical Seismicity Map .............................................................. Rear of Text
Figure 6 – Seismic Hazard Map ..................................................................... Rear of Text
Figure 7 – Flood Hazard Zone Map ............................................................... Rear of Text
Figure 8 – Retaining Wall for EI<50 ............................................................. Rear of Text

Appendices

Appendix A – Geophysical Survey
Appendix B – Boring and Cone Penetrometer Data
Appendix C – Slope Stability Analysis
Appendix D – Laboratory Data
Appendix E – Liquefaction Analysis
Appendix F – General Earthwork and Grading

Plate

Plate 1 – Geotechnical Map
1.0 INTRODUCTION

1.1 Authorization

In accordance with our December 11, 2013 proposal, which you authorized on January 24, 2014, Leighton and Associates, Inc. (Leighton) has performed document review, subsurface exploration, laboratory testing, and engineering analysis for the proposed Oceanaire residential development project. The project is located at 150 West Ocean Boulevard in the City of Long Beach, California (Figure 1, Site Location Map). Site coordinates are Latitude 33.76659 and Longitude -118.193174.

1.2 Scope of Work

- **Review of Available Data:** We reviewed documentation describing the proposed project, including the Planning Submittal Set of drawings for the project by Togawa Smith Martin Architects Inc., dated February 6, 2014, and the Concept Basis of Design by John Labib and Associates, dated October 21, 2013. Additionally, we reviewed our prior reports prepared for the site and adjacent projects. Material reviewed in preparation of this report is listed in Section 7.0, References.

- **Geophysical Survey:** We performed seismic refraction surveys along two lines within the project site to develop the shear wave velocity profile for subsurface materials down to 100 feet. The geophysical survey is included herein as Appendix A, Geophysical Survey. Survey lines RL-1 and RL-2 are shown on Plate 1, Geotechnical Map.

- **Supplementary Geotechnical Exploration:** We excavated four hand auger borings (HA-1 through HA-4) behind an existing retaining wall along the northern portions of the site within the coastal bluff material (Plate 1). Bulk samples were collected from the hand auger borings and transferred to our lab for geotechnical laboratory testing. The borings were backfilled with the excavated material. These hand auger borings and borings performed during previous geotechnical and environmental investigations (Leighton, 2007a, b and d) are shown on Plate 1 and are included in Appendix B, Boring and Cone Penetrometer Data.
• **Slope Stability Analysis:** We performed slope stability analysis along a representative geologic cross section (Section A-A’) to evaluate the stability of various backcut slopes to accommodate construction of retaining walls along the north end of the site.

Our subsurface interpretations are shown on Figure 2, *Geologic Cross Section A-A’*. Shoring is anticipated at the northeast and northwest corners of the site to protect adjacent existing improvements. Results of the stability analysis are included in Appendix C, *Slope Stability Analysis*.

• **Laboratory Testing:** We performed geotechnical laboratory testing on bulk samples recovered during the investigation to determine moisture contents of recovered earth material from the hand auger borings. Laboratory test results performed during the current and previous geotechnical studies (Leighton, 2007a) are included in Appendix D, *Laboratory Data*.

• **Engineering Analysis:** We developed updated and optimized geotechnical recommendations for design and construction based on our understanding of the current project for compliance with the 2013 California Building Code (CBC).

• **Report:** This report documents the results of our current and previous geotechnical studies and provides recommendations for design and earthwork construction of the project.

### 1.3 Study Area

The project site encompasses an area of approximately 1.6 acres. The upper area of the project site (Victory Park) is bordered by West Ocean Boulevard on the north, a high-rise complex and South Pine Avenue on the east, a three story parking structure and Pacific Avenue on the west, and West Seaside Way on the south (Figure 1, *Site Location Map*). The lower site currently is used as an asphalt concrete parking lot.

The site topography over most of the site is generally flat and gently sloped from about Elevation +8 feet mean sea level (msl) at the northern retaining wall to about Elevation +5 feet msl adjacent to West Seaside Way (Plate 1). A small slope descends from near West Ocean Boulevard, which is at about Elevation +25 feet msl adjacent to the project site, at an angle of approximately 2:1 (horizontal to vertical) in the northwestern portion of the property. The remaining
northern portion of the site adjacent to Victory Park includes an approximately 20-foot-high concrete retaining wall containing numerous lateral cracks within the face of the wall. An access ramp descends to the site from the northeastern corner fronting West Ocean Boulevard.

1.4 Project Description

Based on our review of the referenced project documents, Leighton understands that the proposed structure consists of a new 5- to 7-story residential building above a two story parking garage. The northern portion of the proposed building will be benched into the existing slope adjacent to West Ocean Boulevard. Our understanding of the project in profile view is shown on Figure 2.

We understand that dead plus live column loads will average around 750 kips with heavier columns at 750 kips and wall loads of 12 to 25 kips.
2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting

The project site is located along the southern boundary of the Long Beach Plain, a slightly elevated mesa-like feature between the San Gabriel and Los Angeles Rivers. The Long Beach Plain is part of the larger southwestern block of the Los Angeles Basin, characterized as a deep structural trough that evolved over time through deposition and tectonic disturbance.

About 7 million years ago, the boundary between the Pacific and North American plates shifted to its present position and the geologically modern Los Angeles basin began to form. The deepest part of the Los Angeles basin is north and northwest of the site, where Tertiary to Quaternary age (65 million years and younger) marine and nonmarine sedimentary rocks are about 24,000 feet thick (Yerkes, et al, 1965; Wright, 1991). The City of Long Beach rests on a stratigraphic succession of 14,000 feet of Pliocene, Miocene and lower Pleistocene clastic sediments.

The northern terraced portion of the project site (existing Victory Park) is located along an east-west trending arcuate shaped coastline with the lower southern portion of the site topographically lower and underlain at shallow depths by unconsolidated Quaternary alluvium deposited by local erosion of the terrace material and by sediment from the Los Angeles River. For the past 15,000 years, the Los Angeles River has been intermittently transporting material eroded from the upland areas to San Pedro Bay. Much of this sediment was deposited as sand, silt, and clay as the river meandered across the floodplain of the Los Angeles basin. Local wave erosion of the underlying San Pedro Formation results in a high percentage of marine alluvial deposition primarily consisting of unconsolidated, fine to coarse grained sand with occasional gravels.

2.2 Geologic Structure

Evolution of the basin through deposition and tectonic disturbance has resulted in pronounced structural trends marked by a chain of elongated low lying hills and mesas that extend northwest from Newport Beach to Baldwin Hills along the Newport Inglewood Structural Fault Zone (NIFZ). The NIFZ is northwest-trending, right–lateral, strike-slip zone of approximately a 2- to 4-mile-wide belt of anticlinal folds and faults disrupting early Holocene to Late Pleistocene-age and older
deposits (Barrows, 1974) characterized by structural trends attributable to right-lateral shearing of basement rocks at depth (Moody and Hill, 1956). The zone defines the boundary between the western basement complex of Catalina type schist and related rocks to the southwest and the eastern basement complex of metasedimentary, metavolcanic, and plutonic rocks to the northeast (Yerkes, et al., 1965). Right-lateral, strike-slip displacement of 3,000 to 5,000 feet has been measured in Lower Pliocene strata along the Newport-Inglewood structural zone (Dudley, 1954). Apparent vertical offset across faults of the Newport-Inglewood structural zone ranges from 4000 feet at the basement interface, to 1000 feet in the Pliocene strata, and 200 feet at the Plio-Pliocene boundary (Yerkes, et al., 1965). Movement along this structural zone is inferred to have been initiated during middle Miocene time (approximately 15 million years ago), with seismic activity continuing up to present time. Tilted and structurally deformed sediments have also been observed within the Newport-Inglewood structural zone (Barrows, 1974).

2.2.1 Wilmington Oil Field

Delineation and interpretation of the Wilmington Oil Field as a result of oil exploration defines the complex structural arrangement of the field as a highly faulted anticline within San Pedro Bay and the harbor areas of Long Beach and Los Angeles (Randall, et al., 1983). Attributed to northwest to southeast shearing between the Pacific and North American tectonic plates the faults that comprise the Wilmington structural trend are considered to be inactive (Long Beach City Planning Department, 1975) as present day shearing has been accommodated along the active Newport Inglewood fault zone (Randall, et al, 1983).

Land subsidence within the Wilmington Oil Field is well documented beginning with surveys taken in 1940 and 1941. Originally thought to be related to groundwater withdrawal the subsidence continued after groundwater pumping was stopped. The deepest portion of the bowl shaped elliptical depression lies within the main channels of Los Angeles Harbor with lesser amounts near the outer edges. Based on subsidence contours (Figure 22 in Randall, et al, 1983) the project site lies near the outer edges of the depression with total subsidence since measurements began ranging between 2 to 4 feet. Mitigation of subsidence in Long Beach is achieved directly by water injection initiated by the California Division of Oil and Gas (CDOG, 1980). As a result of this repressurization
subsidence in the Wilmington Oil Field has largely been arrested reducing the affected area from approximately 20 square miles to 3 square miles. Some areas of subsidence have shown up to 10 inches of rebound. Subsidence is not expected to pose a constraint to long term performance of the proposed structures.

2.3 Subsurface Soil Conditions

The site is underlain by undocumented artificial fill, coastal beach deposits (Quaternary alluvium), and Quaternary age Pleistocene terrace deposits (Figure 3, Regional Geology Map). Historically, the site was developed between the late 1800’s and 1976 when the Pike Amusement Park closed. Review of historical Sanborn maps indicate the site has been developed with numerous commercial and recreational structures, above ground and below ground fuel storage tanks prior to the late 1970’s when the area was redeveloped as the a parking lot.

The artificial fill soils form a relatively thin mantle (2 to 7 feet thick) and consist primarily of dark brown, loose to medium dense, fine to medium grained silty sand to sand with occasional gravel and manmade debris. Fill was likely placed during construction and buildup of the lower bluff area to increase the land area during the early 1920’s. Fill should be expected to vary in thickness and consistency.

Quaternary Alluvium: Map Symbol (Qal): Underlying the fill are recent (Holocene age<11,000 years old) alluvial and coastal beach deposits consisting of medium dense, wet, fine to coarse grained beach sands with numerous shell fragments (Plate 1 and Figure 2). Primarily of fluvial and coastal tideland origin, the material is generally composed of unconsolidated silt, gravel, and sand formed by coalescence of alluvial fans of the San Gabriel and Los Angeles Rivers (Poland and Piper, 1956). The alluvium is intermixed with beach deposits typically of fine to medium grained sands occurring in a narrow strip along the coast as a result of erosion of the underlying San Pedro Formation.

Quaternary Terrace Deposits: Map Symbol (Qt): The middle to early Pleistocene age (1.8 million to 500,000 year old) terrace deposits which make the bluff area in the north end of the site (Plate 1) consist mostly of consolidated, interbedded, poorly sorted, moderately permeable, reddish brown, iron oxide stained marine and non-marine deposits composed of medium dense to dense, silty sand to clayey sand with minor gravel including very stiff to hard sandy clay with fine to
coarse grained sand. Thickness of this unit ranges from 0 to 700 feet (Randall, et al, 1983). Local foundation studies indicate the fine-grained soils within these deposits are generally preconsolidated exhibiting moderate to high shear strength and moderate to low compressibility.

**Quaternary San Pedro Formation: Map Symbol (Qsp):** Based on review of the boring logs, cone penetrometer (CPT) data (Appendix B) and shear wave velocities (Appendix A) the lower Pleistocene San Pedro Formation is interpreted below the site at depths ranging from approximately 34 to 55 feet below current grade in the southern portion of the site. The San Pedro sand unit is characterized as dense, regularly bedded to cross bedded fine grained sand with occasional gravel capped with cohesive fine grained sandy silts and clay marking the transition from marine to non-marine deposition as a result of lowering sea levels.

A more detailed description of the subsurface soils encountered in the borings is presented in the boring logs (Appendix B). Some of the engineering properties of these soils are described in the following subsections.

### 2.4 Expansive Soil

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result. Based on our explorations (Leighton, 2007a, 2007b), the near surface onsite soils in the lower parking area consist predominantly of silty sand to sand. The onsite soils are generally considered to have a low potential for expansion. Material contained within the coastal bluff is likely more variable in composition and is expected to consist of moderately expansive clayey material within the upper 5 to 10 feet as a result of paleo soil development processes.

It is our opinion that the proposed structure will not be adversely impacted by soils expansion provided recommendations in this report are included in design and followed during construction. Expansion testing should be performed on bearing surfaces within the terrace materials at or near the completion of overexcavation to confirm the assumptions made in this report.
2.5 Soil Corrosivity

One sample of silty sand was tested (Leighton, 2007a) for corrosivity to evaluate corrosion potential to buried concrete (e.g., footings, retaining walls). The chemical analysis test results for the near surface onsite soil are summarized below.

**Corrosivity Test Results**

<table>
<thead>
<tr>
<th>Test Parameter</th>
<th>Test Results</th>
<th>General Classification of Hazard</th>
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<tbody>
<tr>
<td></td>
<td>Boring B-2 0-5'</td>
<td></td>
</tr>
<tr>
<td>Water-Soluble Sulfate in Soil (ppm)</td>
<td>132</td>
<td>Negligible sulfate exposure to buried concrete</td>
</tr>
<tr>
<td>Water-Soluble Chloride in Soil (ppm)</td>
<td>80</td>
<td>Non-corrosive to buried concrete (per Caltrans Specifications)</td>
</tr>
<tr>
<td>pH</td>
<td>8.17</td>
<td>Mildly alkaline</td>
</tr>
<tr>
<td>Minimum Resistivity (saturated, ohm-cm)</td>
<td>2,660</td>
<td>Moderately Corrosive to buried ferrous pipes (per ASTM²)</td>
</tr>
</tbody>
</table>

²ASTM STP 1013 titled Effect of Soil Characteristics on Corrosion (February, 1989)

Based on the available water soluble sulfate results, the corrosion potential to buried concrete is considered “negligible”. The sample tested for water-soluble chloride content indicates a low potential for corrosion of reinforcing steel in concrete due to the chloride content of the soil. However, any concrete element extending below Elevation +2 feet msl should be designed to accommodate corrosion induced by sea water.

The soils are considered moderately corrosive to ferrous metal.

Typical recommendations for mitigation of the corrosive potential of the soil in contact with building materials are the following:

- Below grade ferrous metals should be given a high quality protective coating, such as an 18 mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar.
• Below grade ferrous metals should be electrically insulated (isolated) from above grade ferrous metals and other dissimilar metals, by means of dielectric fittings in utilities and exposed metal structures breaking grade.

• Steel and wire reinforcement within concrete in contact with the site soils should have at least two inches of concrete cover.

If ferrous building materials are expected to be placed in contact with site soils, it may be desirable to consult a corrosion specialist regarding chosen construction materials, and/or protection design for the proposed structure.

2.6 Groundwater

Groundwater was encountered during our previous investigations at about mean sea level. The groundwater level at the site can be expected to rise and fall in response to tidal influence and/or during storm and flooding events. The groundwater level should be assumed to be at Elevation +2 feet msl for design.
3.0 GEOLOGIC/SEISMIC HAZARDS

Geologic and seismic hazards include surface faulting, seismic shaking, landslides, liquefaction, seismically induced settlement, lateral spreading, slope stability and seismically induced landslides, seiches and tsunamis, and flooding. The following sections discuss these hazards and their potential impact at the project site.

3.1 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 2007). Therefore, a surface fault rupture hazard evaluation is not mandated for this site. There are no currently known active surface faults at this site (Figure 4, Regional Fault Map), therefore, the potential risk for surface fault rupture at this site is currently deemed low.

The location of the closest active faults to the site was generated using the United States Geological Survey (USGS) Earthquake Hazards Program (USGS, 2008c) and site decimal degree (latitude-longitude) coordinates N33.7670° and W118.1932°. The closest active faults to the site are the Newport-Inglewood Fault Zone and the Palos Verdes Fault, located approximately 2.9 miles and 3.8 miles, respectively, from the site. The San Andreas fault, which is the largest active fault in California, is approximately 51 miles northeast of the site.

3.2 Historical Seismicity

Although Southern California has been seismically active during the past 200 years, written accounts of only the strongest shocks survive the early part of this period. Early descriptions of earthquakes are rarely specific enough to allow an association with any particular fault zone. It is also not possible to precisely locate epicenters of earthquakes that have occurred prior to the twentieth century.

A search of historical earthquakes was performed using the computer program EQ Search (Blake, 2000) for the time period between 1800 and 2012. Within that time frame 1,012 earthquakes were found within a 62-kilometer (100-mile) radius of the Site. Of these earthquakes, the closest was located offshore 1.2 miles south of the site and occurred on August 4, 1933. Based on its epicentral
location, the suspect fault is the Newport-Inglewood fault zone which registered a 4.0 Mw and induced recorded peak ground acceleration (PGA) of 0.121g.

At least five earthquakes with magnitude of 4.9 or greater have been associated with the NIFZ since 1920 (Barrows, 1974). The first reported earthquake was magnitude 4.9 earthquake occurring on the June 21, 1920 causing moderate damage in the town of Inglewood. The largest instrumentally recorded magnitude 6.3 Long Beach earthquake occurred on March 11, 1933 and represents the most dramatic example of the consequences of disregard for seismic hazards associated with the NIFZ (Richter, 1958, Barrows, 1974) resulting in passage of the Field Act which regulates construction of school buildings. The Long Beach earthquake was followed by a significant aftershock of magnitude 5.4 near Signal Hill on October 2, 1933. In 1941; two earthquakes of magnitude 5.0 and 5.4 caused damage in the Torrance-Gardena area (Richter, 1958).

The largest recorded PGA at the site is estimated to have been roughly 0.28g from the magnitude 6.3 Long Beach earthquake that shook the region on March 11, 1933. For a general view of recorded historical seismic activity see Figure 5, Historical Seismicity Map.

### 3.3 Secondary Seismic Hazards

In general, secondary seismic hazards for the site could include soil liquefaction, seismically induced settlement, lateral spreading, seismically induced landsliding, seiches and tsunamis. These potential secondary seismic hazards are discussed below.

**Liquefaction Potential:** Liquefaction is the loss of soil strength or stiffness due to increasing pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils.

As shown on the State of California Seismic Hazard Zones Map for the Long Beach Quadrangle (CGS, 1999), this site is located within an area that has been identified by the State of California as being potentially susceptible to liquefaction (Figure 6, Seismic Hazard Map). Results of our liquefaction analysis indicate that the potential for liquefaction at the site is low (Appendix E).
**Seismically Induced Settlement:** During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, unsaturated granular soils, separate from liquefaction. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Seismically induced settlement under the structure is anticipated to be less than 1 inch (Appendix E).

**Lateral Spreading:** Lateral spreading is a phenomenon in which large blocks of intact, non-liquefied soil move downslope on a liquefied soil layer. Lateral spreading is often a regional event. For lateral spreading to occur, the liquefiable soil zone must be laterally continuous, unconstrained laterally, and free to move along sloping ground. Due to the low susceptibility for liquefaction, the potential for lateral spreading is considered low.

**Slope Stability and Seismically Induced Landslides:** Significant slopes are not located at the site. Based on the State of California Seismic Hazard Zones Map for the Long Beach Quadrangle (CGS, 1999), the site is not located within an area that has been identified by the State of California as being potentially susceptible to seismically induced landslides (Figure 6).

The upper Pleistocene terrace deposit in the northern portion of the site assumes a topographically higher mesa like position above the southern alluvial plain. Deep seated failure of the bluff is rare, rather the materials are more susceptible to sloughing off of wet material during prolonged seasonal precipitation. The potential for seismically induced landslides to affect the site prior to and after construction is low.

**Seiches and Tsunamis:** Seiches are large waves generated in very large enclosed bodies of water or partially enclosed arms of the sea in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. According to the State of California Tsunami Inundation Map for Emergency Planning Long Beach Quadrangle (CGS, 2009) the Site is situated within the tsunami inundation line.

Tsunamis and seiches have both caused historic damage in the Long Beach area. A tsunami arrived in the Los Angeles-Long Beach Harbor as a result of the 1960 Chilean Earthquake inflicting damage on boats and harbor facilities. Seiche movements caused by the tsunami wave caused 5-foot waves to surge back and forth in the Cerritos Channel (Long Beach City Planning, 1975).
However, considering the amount of seaward development of the low lying harbor areas the outer harbor, breakwater and coastal strand are expected to take the brunt of any large tsunami wave, therefore the potential for a tsunami or sieche to affect the site is considered low.

**Flooding Hazards:** According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the site is located within a flood zone (Figure 7, Flood Hazard Zone Map). The Los Angeles and San Gabriel Rivers are major flood control projects which are concrete and rip rap lined carrying their water to the Pacific Ocean. The probability of flooding caused by failure of dams or levees is considered to be low.

3.4 **Slope Stability Analysis**

Slope stability in Long Beach is not a major geologic constraint. Most natural slopes within the City are stable and not susceptible to deep seated failure. Erosion of the coastal terrace generally occurs as sloughing material during and after prolonged rainfall events of high intensity. Based on review of the conceptual drawings for the site the footprint of concrete parking structure encroaches into and below the northern terrace (Victory Park) by a linear distance of approximately 35 feet from the current property line.

Slope stability analysis was performed to evaluate the stability of the proposed cut required for the site grading and wall construction. We analyzed the stability of the proposed construction backcut slope along cross-section A-A’ (Figure 2). The inclination of the backcut was analyzed at 1.5:1 (horizontal:vertical) and 1:1. The daylight line for the 2:1 backcut when projected to the surface along section A-A’ encroaches into the city sidewalk, therefore making this approach unfeasible due to boundary constraints.

The results of the analyses indicate that geologic conditions do not pose a major constraint to the stability of the proposed cut and site grading. Stockpiling at the top of the cut is not recommended. The backcut inclined at 1.5:1 exhibited a factor of safety greater than 1.25, which is acceptable for temporary conditions. The results of our stability analysis are presented in Appendix C.
3.4.1 Backcut Stability

Surficial stability of the temporary slope is dependent primarily on the cohesive properties of the earth materials that comprise the terrace. If non-cohesive, running sands are encountered they will be susceptible to heavy erosion during rainfall events. Therefore the backcut is recommended to be observed and geologically mapped on a full-time basis by the Engineering Geologist during backcut operations. The purpose of this mapping is to substantiate the geologic conditions that we have assumed in our analysis. In order to expedite the mapping of the temporary slopes, we recommend that the grading contractor trim the cut with a slope board to be free of loose material as it is brought downward.

Due to site access constraints, the east and west sides of the proposed backcut will require shoring to protect adjacent structures. A temporary shoring system consisting of soldier beam ad lagging may be used to support the excavation. The recommendations for shoring are presented in Section 5.14.
4.0 CONCLUSIONS

The currently proposed project is deemed feasible from a geotechnical standpoint, provided the recommendations presented in this report are implemented in the design and construction.

- The northern region of the site (Victory Park) is underlain by terrace deposits consisting of middle to early Pleistocene age (1.8 million to 500,000 year old) consolidated, interbedded, poorly sorted, moderately permeable, reddish brown, iron oxide stained marine and non-marine silty sand to clayey sand with minor gravel including very stiff to hard sandy clay.

- The proposed temporary construction backcut into the northern terrace materials exhibits a calculated factor of safety (FOS) greater than 1.25 for slope inclinations of 1.5:1 (horizontal:vertical) or flatter. The east and west sides of the northern backcut excavation will require shoring to protect adjacent existing structural improvements and facilitate construction of the lower level parking structure.

- The existing undocumented fill at the site is deemed unsuitable for support of proposed improvements and should be removed and replaced as engineered fill.

- Groundwater was encountered during our subsurface exploration at about mean sea level. The groundwater level should be assumed at Elevation +2 feet msl for design.

- The potential for liquefaction at the site is considered to be low and not a significant consideration for site development.

- The on-site soils are expected to have low expansion potential. Reuse of the existing undocumented fill as engineered fill may require segregation/sorting of debris or other unsuitable materials.

- Based on the laboratory testing, concrete in contact with the on-site soil is expected to have negligible exposure to water-soluble sulfates. The on-site soil is considered moderately corrosive to buried ferrous metal. Any improvements extending below the design groundwater level should be designed to accommodate corrosion induced by sea water.

- The proposed structure may be supported on conventional shallow foundations established in undisturbed natural soils or on engineered fill. Floor slabs may be supported on grade.
5.0 DESIGN RECOMMENDATIONS

Geotechnical recommendations for the proposed development are presented in the following sections and are intended to provide sufficient geotechnical information to develop the project in general accordance with 2013 CBC requirements. The following recommendations are considered preliminary and should be considered minimal from a geotechnical viewpoint as there may be more restrictive requirements of the architect, structural engineer, governing agencies and the City of Long Beach.

The geotechnical consultant should review the grading plan, foundation plan and specifications as they become available to verify that the recommendations presented in this report have been incorporated into the plans prepared for the project.

5.1 Earthwork

We recommend all earthwork for the project be performed in accordance with the following recommendations, future grading plan review report(s), the City of Long Beach and County of Los Angeles grading requirements and the General Earthwork and Grading Specifications included in Appendix F. In case of conflict the following recommendations shall supersede those provided in Appendix F.

5.2 Site Preparation

Prior to construction, the areas proposed for residential development and improvements should be cleared of any existing improvements associated with the former land use and properly disposed of offsite. Efforts should be made to locate any existing utility lines to be removed or rerouted where interfering with the proposed construction. Any resulting cavities should be properly backfilled and compacted. After the areas are cleared, the soils should be carefully observed for the removal of all potentially unsuitable deposits.

5.3 General Grading Recommendations

The existing undocumented artificial fill should be removed to expose competent native deposits and replaced as engineered fill. For budgeting purposes, it may be assumed that average depth of undocumented fill at the site is 5 feet. The actual thickness varies across the site and will require confirmation during grading.
If excavation to remove fill and unsuitable bearing soils extends to the groundwater level or otherwise unstable soil conditions, stabilization of the subgrade and temporary dewatering using sump pits may be required. Subgrade stabilization may consist of a bridging layer of crushed rock or a waste concrete slab.

Overexcavation and recompaction should extend a minimum horizontal distance equal to the vertical distance between the proposed footing bottom and depth of overexcavation.

After completion of the overexcavation and prior to fill placement or other improvements such as flatwork and hardscape, the exposed soils should be scarified to a minimum depth of six inches, moisture conditioned 2 to 4 percentage points above optimum moisture content and compacted to a minimum of 95 percent relative compaction (ASTM D1557-12).

5.4 Fill Placement

The onsite soils, less any deleterious material (construction debris) or organic matter, can be used in required fills. Oversized material greater than 6-inches in maximum dimension should not be placed in the fill. Areas prepared to receive structural fill and/or other surface improvements should be scarified, brought to at least optimum moisture content and recompacted to at least 95 percent relative compaction per ASTM Test Method D1557-12.

Any required import material should consist of non-corrosive and relatively non-expansive soils with an Expansion Index (EI) less than 20. The imported materials should contain sufficient fines (binder material) so as to result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being placed at the site.

All fill soil should be placed in thin, loose lifts, with each lift properly moisture conditioned 2 to 4 percentage points above optimum moisture content and compacted to a minimum of 95 percent relative compaction (ASTM D1557-12). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, lift thickness for granular fill should not exceed 8 inches in compacted thickness. Aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D1557-12).
5.5 **Pipe Bedding**

Any proposed pipe should be placed on properly placed bedding materials. Pipe bedding should extend to a depth in accordance with the pipe manufacturer’s specification. The pipe bedding should extend to at least 12-inches over the top of the pipeline. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock and should be densified by mechanical means. Due to the predominately granular nature of the subsurface soils and porous nature of the cohesive soils flooding or jetting may be considered. Pipe bedding material should have a Sand Equivalent (SE) of at least 30 per California Test Method CTM-217. A 5-foot-long seepage plug consisting of clay soil or CLSM slurry should be placed as backfill where the trench enters under the building slab, with the purpose of preventing water from within the trench bedding from seeping into/under the building pad.

5.6 **Trench Backfill**

Trench excavations above pipe bedding zone may be backfilled with onsite soils under the observation of the geotechnical consultant. All fill soils should be placed in loose lifts, moisture conditioned as required and compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D 1557-12. Lift thickness will be dependent on the equipment used as suggested in the latest edition of the Standard Specifications for Public Works Construction (Greenbook).

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the “competent person” required by OSHA standards to evaluate soil conditions. Soil types will vary, but Type C soils can be expected at shallow depths. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

5.7 **Surface Drainage**

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings. Positive drainage may be accomplished by providing drainage away from buildings a minimum of 2 percent for a lateral distance of at least five feet and further maintained by a swale or
drainage path at a gradient of at least 1 percent. Eave gutters are recommended and should reduce water infiltration into the subgrade materials. Downspouts should be connected to appropriate outlet devices.

5.8 Foundation Recommendations

Proposed structures may be supported on shallow spread footings established in undisturbed natural soils or engineered fill.

*Allowable Bearing Pressure:* Footings established on undisturbed natural soils or engineered fill may be designed to impose an allowable bearing pressure of 4,000 pounds per square foot (psf). A one-third increase in the bearing value for short duration loading, such as wind or seismic forces, may be used.

The ultimate bearing capacity can be taken as 12,000 psf, which does not incorporate a factor of safety. A resistance factor of 0.5 should be used for bearing capacity evaluation with factored loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 150 pounds per cubic foot (pcf); the weight of soil backfill can be neglected when determining the downward loads.

*Footing Embedment:* Footings should have a minimum embedment of 18 inches and have a minimum width of 12 inches.

*Estimated Settlement:* The estimated settlement of columns supported on spread footings as recommended above due to dead plus live loads is less 1 inch. Most of this anticipated settlement will occur during construction.

The differential settlement over a span of 30 feet may be assumed to be about half of the total settlement.

Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. The settlement estimates should be reviewed by Leighton when final foundation plans and loads for the proposed structures become available.
5.9 Slab-On-Grade

Parking Garage Floor Slabs: Concrete floor slabs subjected to special loads should be designed by the structural engineer in accordance with the 2013 CBC. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the 2013 CBC.

- A minimum slab thickness of 5 inches reinforced with a minimum of No. 4 rebar placed at 16 inches on center in each direction and placed in the middle third of the slab thickness.

Exterior Flatwork: The exterior concrete flatwork should be a minimum of 4 inches thick and provided with construction or weakened plane joints at a maximum spacing of 10 feet. The flatwork subgrades should be wetted prior to placing concrete. Exterior concrete slabs should also be reinforced.

Construction Considerations: Minor cracking of the concrete as it cures, due to drying and shrinkage, is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete (not exceeding 4 inches at the time of placement) can reduce the potential for shrinkage cracking. In addition, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential for concrete cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals, typically on the order of 10 feet for a 4-inch thick slab. Joints should be laid out to form approximately square panels.

5.10 Lateral Earth Pressures

Earth Pressures: The design of the retaining structures will be dependent upon the location (i.e., type of material retained) and applicable earth pressure condition. Walls that are free to rotate to mobilize the active earth pressure condition may be designed for a lower soil pressure than walls that are fixed or
restrained from movement where the at-rest earth pressure distribution should be used in design.

The following table summarizes the values of equivalent fluid pressure that are recommended to be used to design retaining walls that retain on-site soils.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equivalent Fluid Unit Weight (psf/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level Backfill</td>
</tr>
<tr>
<td>Active</td>
<td>38</td>
</tr>
<tr>
<td>Seismic Increment*</td>
<td>20</td>
</tr>
<tr>
<td>At-Rest</td>
<td>60</td>
</tr>
<tr>
<td>Passive</td>
<td>400</td>
</tr>
<tr>
<td>Coefficient of Friction</td>
<td>0.35</td>
</tr>
</tbody>
</table>

*to be added to active earth pressure

The parameters stated above are based upon drained conditions behind the walls. Retaining structures should be provided with an appropriate drainage system to prevent buildup of hydrostatic pressure behind the wall. The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

**Surcharge Loads:** In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or pavement, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the base of retaining structures should be considered as a surcharge. For surcharges located behind retaining structures that are large in plan/aerial extent, the surcharge may be modeled as a uniform lateral pressure with a horizontal pressure intensity equivalent to 50 percent or 33 percent of vertical pressure acting on the ground surface behind the wall for the at-rest and active earth pressure conditions, respectively. The surcharge due to surface loads of limited lateral extent such as a foundation will be dependent upon the size and shape of the loaded area, and the distance from the retaining structure. Surcharges due to areas limited dimension can be analyzed on a case-specific basis.
Walls adjacent to streets and areas of traffic should also be designed to accommodate surcharge loads. For traffic surcharge, a uniform lateral pressure of 100 pounds per square foot acting as a result of an assumed 300 pounds per square foot surcharge behind the wall due to normal traffic; the traffic surcharge load may be neglected provided a minimum of 10 foot clearance between the wall and the traffic is maintained.

5.11 Seismic Design Parameters

The following values may be used for the seismic design method based on the 2013 California Building Code:

<table>
<thead>
<tr>
<th>Categorization/Coefficient</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Latitude</td>
<td>N33.7670</td>
</tr>
<tr>
<td>Site Longitude</td>
<td>W118.1932</td>
</tr>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td>Mapped spectral response acceleration parameter at short period, ( S_S )</td>
<td>1.610g</td>
</tr>
<tr>
<td>Mapped spectral response acceleration parameter at a period of 1 sec, ( S_1 )</td>
<td>0.606g</td>
</tr>
<tr>
<td>Short Period (0.2 sec) Site Coefficient, ( F_a )</td>
<td>1.000</td>
</tr>
<tr>
<td>Long Period (1.0 sec) Site Coefficient, ( F_v )</td>
<td>1.500</td>
</tr>
<tr>
<td>Design spectral response acceleration parameter at short period, ( S_{DS} )</td>
<td>1.073g</td>
</tr>
<tr>
<td>Design spectral response acceleration parameter at a period of 1 sec, ( S_{D1} )</td>
<td>0.606</td>
</tr>
</tbody>
</table>

5.12 Hydrostatic Uplift

We recommend that portions of structures below Elevation +2 feet msl be designed to resist hydrostatic uplift pressures unless a permanent drainage system is provided to prevent the buildup of hydrostatic pressure.

Uplift pressure may be resisted by the dead weight of the structure. Hydrostatic pressures may be calculated using a water density of 64 pounds per cubic foot.
(pcf) with a design groundwater level at Elevation +2 feet msl. Backfill may be assumed to have a unit weight of 120 pcf.

5.13 **Temporary Excavations**

All temporary excavations, including footings and utility trenches should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 4 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the cut, unless the cut is shored appropriately.

We analyzed the stability of the proposed construction backcut slope along northern portion of the site. The inclination of the backcut was analyzed at 1.5:1 (horizontal:vertical) and 1:1. The results indicate that geologic conditions do not pose a major constraint to the stability of the proposed 1.5:1 cut and site grading. Stockpiling at the top of the cut is not recommended. The backcut inclined at 1.5:1 exhibited a factor of safety greater than 1.25, which is acceptable for temporary conditions. The results of our stability analysis are presented in Appendix C.

Surficial stability of the temporary slope is dependent primarily on the cohesive properties of the earth materials that comprise the terrace. If non-cohesive, running sands are encountered they will be susceptible to heavy erosion during rainfall events. Therefore the backcut is recommended to be observed and geologically mapped on a full-time basis by the Engineering Geologist during backcut operations. The purpose of this mapping is to substantiate the geologic conditions that we have assumed in our analysis. In order to expedite the mapping of the temporary slopes, we recommend that the grading contractor trim the cut with a slope board to be free of loose material as it is brought downward.

During construction, soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.
5.14  **Shoring**

Shoring for the site will likely consist of soldier piles and lagging. Soldier piles may consist of steel H-beams set in predrilled holes and backfilled with lean-mix concrete to the ground surface. If the depth of the excavation is less than approximately 15 feet, tieback anchors, or internal bracing are not expected to be required. Deeper excavations will require some form of bracing.

The potential raveling and caving of sand layers may pose difficulties in the drilling of the soldier piles and tie-back anchors. Accordingly, the shoring contractor should be prepared to use special techniques and measures, if necessary, to permit the proper installation of the soldier piles and tie-back anchors.

**Lateral Earth Pressures:** For design of cantilevered shoring, where the surface of the backfill is level, it can be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot (pcf).

In addition to the recommended earth pressure, the shoring should be designed to resist any applicable surcharge loads due to foundation, storage, traffic, or other anticipated loads.

For the design of braced shoring, a trapezoidal distribution of lateral earth pressure plus any surcharge loadings occurring as a result of traffic and adjacent foundations should be used. The recommended pressure distribution for the case where the grade is level behind the walls is illustrated in the following diagram, where the maximum lateral pressure will be $28H$ in pounds per square foot (psf), where $H$ is the height of the wall in feet:
In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to streets should be designed to resist a uniform lateral pressure 100 psf, acting as a result of an assumed 100 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. We can determine lateral surcharge pressures for specific cases, such as construction crane, concrete trucks, and other heavy construction equipment adjacent to shoring, if requested.

**Surcharge Pressure from Adjacent Buildings:** Where existing building foundations are within a 1:1 plan projected upward from the bottom of the planned shoring and basement walls, a lateral surcharge load should be applied to the earth pressure to account for the pressure imposed by the foundation. The surcharge from adjacent footings may be modeled as a uniform lateral pressure with a horizontal pressure intensity equivalent to 33 percent of vertical pressure acting on the ground surface behind the wall.

**Design of Soldier Piles:** For the design of soldier piles spaced at least two diameters on centers (OC), the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 600 psf at the excavated surface, up to a maximum of 6,000 psf. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient
strength to adequately transfer the imposed loads from the soldier pile to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the design load. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.4. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth. In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 500 psf.

**Lagging:** Continuous lagging will be required between the soldier piles. Careful installation of the lagging will be necessary to achieve bearing against the retained earth.

The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. For clear spans up to 8 feet, we recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 400 psf at the midline between soldier piles, and 0 psf at the soldier piles.

**Anchor Design:** Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation. The anchors should extend at least 40 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in the following section, Anchor Testing. For design purposes, it may be estimated that drilled friction anchors will develop an average friction value of 1,000 psf. For post-grouted anchors, it may be estimated that the anchors could develop an average friction of up to 3,000 psf. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors need be considered due to group action.
**Anchor Installation:** The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. Mining (removal of soils from the anchor holes without advancing the drilling auger) of the sandy and gravelly soils could occur and the shoring contractor should take special care to prevent, or at least minimize, such mining.

Conventional anchors should be filled with concrete placed by pumping from the tip outward, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

**Anchor Testing:** Our representative should select at least ten percent of the anchors for quick 200% tests. Twenty-four hour tests should be performed on at least two of those 200% test anchors. The purpose of the 200% test is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

For post-grouted anchors where concrete is used to backfill the anchor along its entire length, the test load should be computed as that required to develop the appropriate friction along the entire bonded length of the anchor. The test load should therefore be computed as:

\[ P_{\text{test}} = P_{\text{design}} * \frac{L_t}{L_b} * M \]

where \( L_t = \) Total Length of Anchor  
\( L_b = \) Post-grouted Length of Anchor  
\( M = 150\% \) or 200\% depending on test performed

However, we understand that for this project, the unbonded length of anchors within the active wedge may be encased in PVC sheathing to prevent load transfer to surrounding soil. Accordingly, the test loads need not be increased
using the criteria described above if the unbounded length of anchors is thus isolated from surrounding soil.

The total deflection during the 24-hour 200% tests should not exceed 12 inches during loading; the anchor deflection should not exceed ¼ inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than ½ inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the 200% test load should be maintained for at least 15 minutes. The total deflection of the anchor during the 200% quick tests should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed 0.2 inch during the 15-minute period. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

All of the production anchors should be pretested to at least 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% tests should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by our firm.

**Internal Bracing:** Raker bracing, if used, could be supported laterally by temporary concrete footings (deadmen). For design of such temporary footings, poured with the bearing surface normal to rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 4,000 psf may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

**Deflection:** It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. To
help protect adjacent existing buildings and infrastructure, the maximum allowable horizontal shoring deflection as measured at the top of the excavation is ½ inch.

If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent structures and of any utilities in the adjacent streets. To reduce the deflection of the shoring, if desired, a greater active pressure could be used in the shoring design.

**Monitoring:** Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system is finalized.

We recommend that the adjacent existing streets be surveyed for horizontal and vertical locations. Also, a careful survey of existing cracks and offsets in the streets should be performed and recorded along with photographic records. A pre-construction benchmark survey establishing horizontal locations and vertical elevations for the adjacent buildings combined with documentation of existing cracks and offsets may be useful in responding to claims of building distress and damage (if any).

### 5.15 County of Los Angeles Building Code Section 111 Statement

Provided that the recommendations in this report are implemented, it is Leighton’s opinion that the proposed improvements will be safe from the hazards of landslide, settlement, or slippage, and that the completed grading and proposed improvements will not adversely affect the stability of adjacent properties.

### 5.16 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited geologic mapping. Our conclusions and recommendations presented in this report should be reviewed and verified by Leighton during site construction and revised accordingly if exposed geotechnical conditions vary from our preliminary findings and interpretations. The recommendations presented in this report are only valid if
Leighton verifies the site conditions during construction. Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- Overexcavation and compaction;
- Compaction of all fill materials;
- Shoring installation;
- Excavation and installation of foundations;
- After excavation of all slabs and footings and prior to placement of steel or concrete to confirm the slabs and footings are founded in firm, compacted fill;
- Utility trench backfilling and compaction; and
- When any conditions are encountered that varies significantly from the conditions described in this report.

Leighton should review the grading and foundation plans and specifications, when available, to comment on the geotechnical aspects. Our recommendations should be revised, as necessary, based on future plans and incorporated into the final design plans and specifications.
6.0 LIMITATIONS

This research report was based in part on available published data, limited non-invasive and invasive subsurface exploration. Such information is, therefore, incomplete. The nature of many projects is such that differing earth materials and/or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, findings, conclusions and recommendations presented in this geotechnical report are based on the assumption that Leighton will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Lennar Multi Family Investors LLC and their design team, for their use in assessing the proposed Oceanaire Improvements, in accordance with generally accepted geotechnical engineering practices at this time in the City of Long Beach and County of Los Angeles.
7.0 REFERENCES


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Important Information about Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.
While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects
Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared solely for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. And no one—not even you—should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report
Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors
Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client’s goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:
- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:
- the function of the proposed structure, as when it’s changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, always inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

Subsurface Conditions Can Change
A geotechnical engineering report is based on conditions that existed at the time the study was performed. Do not rely on a geotechnical engineering report whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. Always contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions
Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report’s Recommendations Are Not Final
Do not overly rely on the construction recommendations included in your report. Those recommendations are not final, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual
subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report’s recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members’ misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team’s plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer’s Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, but prefix it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report’s accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled “limitations” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. Read these provisions closely. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a geoenvironmental study differ significantly from those used to perform a geotechnical study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Unanticipated environmental problems have led to numerous project failures. If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. Do not rely on an environmental report prepared for someone else.

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant. none of the services performed in connection with the geotechnical engineer’s study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.

Rely, on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/ THE BEST PEOPLE ON EARTH exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.
Figure 1

Base Map: ESRI ArcGIS Online 2014

Thematic Information: Leighton

Author: (mmurphy)

Oceanaire
150 West Ocean Boulevard
Long Beach, California
GEOLOGIC CROSS SECTION A-A'

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Reference:

Scale: 1"=30'
Date: 04/2014

Proj: 10594.001
Eng/Geol: CK/JAR

Reference:

GEOLOGIC CROSS SECTION A-A'

1. Existing Retaining Wall
2. Proposed 2-Level Podium Residential
3. Proposed 2-Level Parking
4. Proposed Backcut Inclination
   Shown at 1.5x1 (Horizontal : Vertical)

North
**Legend**

- **af** Artificial Fill
- **Qyf** Young alluvial fan deposits
- **Qya** Young channel deposits
- **Qt** Terrace deposits

**REGIONAL GEOLOGY MAP**

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Scale: 1" = 2,000'

Date: April 2014

Base Map: Esri ArcGIS Online 2014
USGS, 2006, Geologic map of the San Bernardino and Santa Ana 30' x 60' quadrangles, California, Version 1.0, Open File Report 2006-1217

Author: (mmurphy)
NEWPORT-INGLEWOOD-ROSE CANYON FAULT ZONE,
NORTH LOS ANGELES BASIN SECTION

PALOS VERDES FAULT ZONE,
SAN PEDRO SHELF SECTION
(PALOS VERDES FAULT)

THUMS-HUNTINGTON BEACH

Legend
- Historic (since 1769)
- Holocene (last 11,000 years)
- Pleistocene (11,000 to 1.6 million years)
- Pre-Quaternary (before 1.6 million years)

Approximate Site Location

REGIONAL FAULT MAP
Oceanaire
150 West Ocean Boulevard
Long Beach, California

Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
HISTORICAL SEISMICITY MAP

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Legend
Earthquake Events
- 4 - 5
- 5 - 6
- 6 - 7

Approximate Site Location

Sources: Esri, DeLorme, NAVTEQ, TomTom, Intermap, increment P Corp.,
GEBCO, USGS, FAO, NPS, NRCan, GeoBase, IGN, Kadaster NL,
Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo,
and the GIS User Community

Figure 5

Leighton Base Map: ESRI ArcGIS Online 2014
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)
FLOOD HAZARD ZONE MAP

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Legend

- 500 Year Flood Plain
- 100 Year Flood Plain

Scale: 1" = 2,000'

Project: 10594.001
Eng/Geol: CK/JR

Date: April 2014

Base Map: ESRI ArcGIS Online 2014
FEMA, Q3 Flood data, Los Angeles County, CA
Author: (mmurphy)
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF <50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

- WITH PROPER SURFACE DRAINAGE
- SLOPE OR LEVEL
- WATERPROOFING (SEE GENERAL NOTES)
- 12" MINIMUM
- CLASS 2 PERMEABLE FILTER MATERIAL (SEE GRADATION)
- LEVEL OR SLOPE
- WEEP HOLE (SEE NOTE 5)
- 4 INCH DIAMETER PERFORATED PIPE (SEE NOTE 3)

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC

- WITH PROPER SURFACE DRAINAGE
- SLOPE OR LEVEL
- WATERPROOFING (SEE GENERAL NOTES)
- 1/4 TO 1 1/2 INCH SIZE GRAVEL WRAPPED IN FILTER FABRIC
- LEVEL OR SLOPE
- WEEP HOLE (SEE NOTE 5)

Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

<table>
<thead>
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<th>Sieve Size</th>
<th>Percent Passing</th>
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</thead>
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<tr>
<td>1&quot;</td>
<td>100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>90-100</td>
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<tr>
<td>3/8&quot;</td>
<td>40-100</td>
</tr>
<tr>
<td>No. 4</td>
<td>25-40</td>
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<tr>
<td>No. 8</td>
<td>18-33</td>
</tr>
<tr>
<td>No. 30</td>
<td>5-15</td>
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<tr>
<td>No. 50</td>
<td>0-7</td>
</tr>
<tr>
<td>No. 200</td>
<td>0-3</td>
</tr>
</tbody>
</table>

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
* Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
* Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weep hole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF <50

Leighton Figure 8
APPENDIX A
February 14, 2014
Project No. 114053

Mr. Joe Roe
Leighton and Associates
17781 Cowan
Irvine, CA 92614

Subject: Geophysical Evaluation
150 West Ocean Boulevard
Long Beach, California

Dear Mr. Roe:

In accordance with your authorization, we have performed geophysical survey services pertaining to the proposed Long Beach Oceanaire project located at 150 West Ocean Boulevard in Long Beach, California (Figure 1). The purpose of our survey was to develop Shear-wave velocity profiles for two locations at the project site. This report presents the survey methodology, equipment used, analysis, and findings.

Our scope of services included the performance of a refraction microtremor (ReMi) survey at two preselected areas at the property (Figure 2). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a Shear-wave velocity profile of the study area down to a depth, in this case, of approximately 100 feet. The ReMi survey included the use of a 24-channel Geometrics Strataview seismograph and 24 4.5-Hz vertical component geophones. The geophones were spaced 10 feet apart, for a total line length of 230 feet. Fifteen records, each roughly 32 seconds long, were recorded for each profile line and then downloaded to a computer. The data were later processed using SeisOpt® ReMi™ software. Figure 3 depicts the general site conditions in the area of the ReMi lines.

Figures 4a and 4b, and Table 1 present the results from our ReMi survey. Based on our analysis of the collected data, the average characteristic site Shear-wave velocity down to a depth of 100
feet is 1,086 ft/sec for RL-1 and 1,138 ft/sec for RL-2. Both these values correspond to a site classification of D (CBC, 2010).

<table>
<thead>
<tr>
<th>Line No.</th>
<th>Depth (feet)</th>
<th>Shear Wave Velocity (feet/second)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RL-1</td>
<td>0 – 14</td>
<td>602</td>
</tr>
<tr>
<td></td>
<td>14 – 47</td>
<td>1,179</td>
</tr>
<tr>
<td></td>
<td>47 – 94</td>
<td>1,222</td>
</tr>
<tr>
<td></td>
<td>94 – 100</td>
<td>2,577</td>
</tr>
<tr>
<td>RL-2</td>
<td>0 – 10</td>
<td>602</td>
</tr>
<tr>
<td></td>
<td>10 – 20</td>
<td>749</td>
</tr>
<tr>
<td></td>
<td>20 – 37</td>
<td>1,024</td>
</tr>
<tr>
<td></td>
<td>37 – 88</td>
<td>1,394</td>
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<tr>
<td></td>
<td>88 – 100</td>
<td>2,763</td>
</tr>
</tbody>
</table>

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties’ sole risk.
We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,

SOUTHWEST GEOPHYSICS, INC.

Edward R. Verdugo  
Senior Staff Geophysicist

Principal Geologist/Geophysicist

Attachments:  
Figure 1 – Site Location Map  
Figure 2 – Line Location Map  
Figure 3 – Site Photographs  
Figure 4a – ReMi Results, RL-1  
Figure 4b – ReMi Results, RL-2

Distribution:  
Addressee (electronic)
SITE LOCATION MAP

Figure 1

150 West Ocean Boulevard
Long Beach, California

Project No.: 114053
Date: 02/14

SOUTHWEST
ENGINEERING INC.
Shear-Wave Velocity
(ft/sec)

ReMi RESULTS
RL-1

150 West Ocean Boulevard
Long Beach, California

Project No.: 114053  Date: 02/14

Figure 4a
Vs Model

Shear-Wave Velocity (ft/sec)

Vs100' = 1,138 ft/s
IBC site class 'D'

ReMi RESULTS
RL-2
150 West Ocean Boulevard
Long Beach, California

Figure 4b
APPENDIX B
**GEOTECHNICAL BORING LOG HA-1**

<table>
<thead>
<tr>
<th>Project No.</th>
<th>10594.001</th>
<th>Date Drilled</th>
<th>2-4-14</th>
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<tbody>
<tr>
<td>Project</td>
<td>Oceanaire</td>
<td>Logged By</td>
<td>EBP</td>
</tr>
<tr>
<td>Drilling Co.</td>
<td>Leighton and Associates, Inc.</td>
<td>Hole Diameter</td>
<td>3&quot;</td>
</tr>
<tr>
<td>Drilling Method</td>
<td>Hand Auger - Hand Tools</td>
<td>Ground Elevation</td>
<td>27'</td>
</tr>
<tr>
<td>Location</td>
<td>See Plate 1: Geotechnical Map</td>
<td>Sampled By</td>
<td>EBP</td>
</tr>
</tbody>
</table>

---

**SOIL DESCRIPTION**

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

<table>
<thead>
<tr>
<th>Soil Class, (U.S.C.S.)</th>
<th>Sampled By</th>
<th>Type of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>SC</td>
<td>EBP</td>
<td>Sample Types, Type of Tests:</td>
</tr>
<tr>
<td>SC+SM</td>
<td>EBP</td>
<td>B BULK SAMPLE</td>
</tr>
<tr>
<td></td>
<td>EBP</td>
<td>C CORE SAMPLE</td>
</tr>
<tr>
<td></td>
<td>EBP</td>
<td>G GRAB SAMPLE</td>
</tr>
<tr>
<td></td>
<td>EBP</td>
<td>R RING SAMPLE</td>
</tr>
<tr>
<td></td>
<td>EBP</td>
<td>S SPLIT SPOON SAMPLE</td>
</tr>
<tr>
<td></td>
<td>EBP</td>
<td>T TUBE SAMPLE</td>
</tr>
</tbody>
</table>

**Sample Descriptions:**

- **Artificial Fill, undocumented:** (Afu)
- **Quaternary Terrace Deposits (Qt):**
  - @1: Clayey Sand and Silty SAND (SC+SM), dark yellowish brown, dry, dense to very dense, fine grained, pinhole porosity in clayey sand, lenses of dark reddish brown sandy clay.
  - @2: Refusal while sampling.
  - @4: Refusal while sampling.
- **SM @7.5': Silty SAND (SM), yellowish brown, dry, medium dense, fine to medium grained.**

Total Depth = 10.0 feet
No groundwater encountered in boring.
Backfilled with cuttings and tampt 2/4/14.

---

***This log is a part of a report by Leighton and should not be used as a stand-alone document.***
# Geotechnical Boring Log HA-2

**Project No.** 10594.001  
**Date Drilled** 2-4-14

**Project** Oceanaire
**Logged By** EBP

**Drilling Co.** Leighton and Associates, Inc.
**Hole Diameter** 3"

**Drilling Method** Hand Auger - Hand Tools
**Ground Elevation** 18'

**Location** See Plate 1: Geotechnical Map

**Logged By** EBP

## Soil Description

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

### Soil Type: Artificial Fill: undocumented (Afu)

- **@0**: Clayey SAND with silt (SC), dark yellowish-brown, dry, fine to medium grained, few gravel, few cobbles, trace debris consisting of gravel to cobbles sized concrete, brick, and asphalt.
- **@2**: Refusal on cobble/concrete.

**Total Depth** = 2.0 feet


### Sample Types:

- **B** = Bulk Sample
- **C** = Core Sample
- **G** = Grab Sample
- **R** = Ring Sample
- **S** = Split Spoon Sample
- **T** = Tube Sample

### Type of Tests:

- **DS** = Direct Shear
- **EI** = Expansion Index
- **H** = Hydrometer
- **MD** = Maximum Density
- **PP** = Pocket Penetrometer
- **CU** = Undrained Triaxial
- **RV** = R Value

---

**Note:** This log is a part of a report by Leighton and should not be used as a stand-alone document. **Page 1 of 1**
**GEOTECHNICAL BORING LOG HA-3**

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<th>Date Drilled</th>
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<td>Project</td>
<td>Oceanaire</td>
<td>Logged By</td>
<td>EBP</td>
</tr>
<tr>
<td>Drilling Co.</td>
<td>Leighton and Associates, Inc.</td>
<td>Hole Diameter</td>
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<tr>
<td>Drilling Method</td>
<td>Hand Auger - Hand Tools</td>
<td>Ground Elevation</td>
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<tr>
<td>Location</td>
<td>See Plate 1: Geotechnical Map</td>
<td>Sampled By</td>
<td>EBP</td>
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<table>
<thead>
<tr>
<th>Elevation Feet</th>
<th>Depth Feet</th>
<th>Graphic Log</th>
<th>Attitudes</th>
<th>Sample No.</th>
<th>Blows Per 6 Inches</th>
<th>Dry Density, %</th>
<th>Moisture Content, %</th>
<th>Soil Class (U.S.C.S.)</th>
<th>Type of Tests</th>
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<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>B-1</td>
<td>SC</td>
<td></td>
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<td></td>
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<td></td>
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</tbody>
</table>

**SOIL DESCRIPTION**

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Artificial Fill: undocumented (Afu)

@0': Clayey SAND with silt (SC), dark yellowish brown, dry, fine to medium grained, few gravel, few cobbles, trace debris consisting of gravel to cobbles sized concrete, brick, and asphalt.

@2.0': Refusal on cobble/concrete.

Total Depth = 2.0 feet
No groundwater encountered in boring.
Backfilled with cuttings and tampt 2/4/14.

**SAMPLE TYPES:**
- BULK SAMPLE
- CORE SAMPLE
- GRAB SAMPLE
- RING SAMPLE
- SPLIT SPOON SAMPLE
- TUBE SAMPLE

**TYPE OF TESTS:**
- ATTERBERG LIMITS
- CONSOLIDATION
- COLAPSE
- CORROSION
- UDRAINED TRIAXIAL
- DIRECT SHEAR
- EXPANSION INDEX
- HYDROMETER
- POCKET PENETROMETER
- SPECIFIC GRAVITY
- UNCONFINED COMPRRESSIVE STRENGTH
- R VALUE

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***
**GEOTECHNICAL BORING LOG HA-4**

**Project No.** 10594.001  
**Date Drilled** 2-4-14  
**Logged By** EBP  
**Drilling Co.** Oceanaire  
**Drilling Method** Leighton and Associates, Inc. Hole Auger - Hand Tools  
**Hole Diameter** 3”  
**Ground Elevation** 16'  
**Sampled By** EBP

### SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

- **Artificial Fill: undocumented (Afu)**
  - *@0*: Clayey SAND with silt (SC), dark yellowish brown, dry, fine to medium grained, few gravel, few cobbles, trace debris consisting of gravel to cobbles sized concrete, brick, and asphalt.
  - *@2.5*: Refusal on cobble/concrete.

  Total Depth = 2.5 feet
  No groundwater encountered in boring.
  Backfilled with cuttings and tamp 2/4/14.

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<tr>
<th>Elevation (Feet)</th>
<th>Depth (Feet)</th>
<th>Graphic Log</th>
<th>Attitudes</th>
<th>Sample No.</th>
<th>Blows Per 6 Inches</th>
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<th>Moisture Content (%)</th>
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</table>

**SAMPLE TYPES:**
- BULK SAMPLE
- CORE SAMPLE
- GRAB SAMPLE
- RING SAMPLE
- SPLIT SPOON SAMPLE
- TUBE SAMPLE

**TYPE OF TESTS:**
- DS: DIRECT SHEAR
- SA: SIEVE ANALYSIS
- EI: EXPANSION INDEX
- SE: SAND EQUIVALENT
- H: HYDROMETER
- SG: SPECIFIC GRAVITY
- MD: MAXIMUM DENSITY
- UC: UNCONFINED COMPRESSIVE STRENGTH
- PP: POCKET PENETROMETER
- RV: R VALUE

---

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***
# Geotechnical Boring Log B-1

Date: 1-22-07

**Project:** 012120-001 Intergulf/Oceanaire

**Logged / Sampled By:** ACS/ATR

**Date:**

**Sheet of 2**

**Location:** See Plate 1 Geotechnical Map

**Drive Weight:** 140 lbs Auto-hammer

**Drop:** 30°

---

## Soil Description

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

### Soil Types:

- **0:** 3-inches Asphalt Concrete (AC)
- **0.25:** Clayey SAND, dark brown, moist, fine to coarse grained sand
- **0.5:** SAND, grayish brown, medium dense, wet, fine-medium sand
- **0.75:** Quaternary Alluvium: (Qal)
  - SAND, loose, grayish brown, medium dense, wet, fine-medium sand
- **1:** Same as above, medium dense, with some shells
- **1.5:** SAND with silt, grayish brown, medium dense, wet, fine to medium grained sand, some shells
- **2:** Same as above, medium dense
- **2.5:** SAND, grayish brown, medium dense, wet, fine grained sand

---

## Sample Types:

- **S** Split Spoon
- **G** Grab Sample
- **R** Ring Sample
- **C** Core Sample
- **B** Bulk Sample
- **T** Tube Sample

## Type of Tests:

- **DS** Direct Shear
- **MD** Maximum Density
- **CN** Consolidation
- **CR** Corrosion
- **UC** Unconfined Compressive Strength
- **SA** Sieve Analysis
- **SE** Sand Equivalent
- **EI** Expansion Index
- **CO** Collapse
- **PP** Pocket Penetrometer

---

**Notes:**

- This log is a part of a report by Leighton and should not be used as a stand-alone document. **

---

**Page 1 of 2**
GEOTECHNICAL BORING LOG B-1

Date: 1-22-07

Project: 012120-001 Intergulf/Oceanaire

Logged / Sampled By: ACS/SR

Drilling Co.: Martini Drilling Corp

Type of Rig: CME-75

Hole Diameter: 8-inches

Drive Weight: 140 lbs Auto-hammer

Elevation Top of Hole: 5'

Location: See Plate 1 Geotechnical Map

---

### SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

@30': Silty SAND, dark brown, medium dense, wet, fine to coarse grained sand

Total depth of boring = 31.5 feet.

Groundwater encountered at 6 feet during drilling.

Hole was backfilled with soil cuttings and bentonite and capped with asphalt upon completion of drilling.

---

**Type of Tests:**

- OS DIRECT SHEAR
- SA SIEVE ANALYSIS
- -200 % FINES PASSING
- MD MAXIMUM DENSITY
- SE SAND EQUIVALENT AL
- ATTERBERG LIMITS
- CN CONSOLIDATION
- EI EXPANSION INDEX
- CO COLLAPSE
- CR CORROSION
- RV R VALUE
- PP POCKET PENETROMETER
- UC UNCONFINED COMPRESSION STRENGTH

---

**Sample Types:**

- S SPLIT SPOON
- G GRAB SAMPLE
- DS DIRECT SHEAR
- SA SIEVE ANALYSIS
- -200 % FINES PASSING
- R RING SAMPLE
- C CORE SAMPLE
- MD MAXIMUM DENSITY
- SE SAND EQUIVALENT AL
- ATTERBERG LIMITS
- B BULK SAMPLE
- T TUBE SAMPLE
- CN CONSOLIDATION
- EI EXPANSION INDEX
- CO COLLAPSE
- CR CORROSION
- RV R VALUE
- PP POCKET PENETROMETER
- UC UNCONFINED COMPRESSIVE STRENGTH

---

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***
GEOTECHNICAL BORING LOG B-2

Date: 1-22-07
Project: 012120-001 Intergulf/Oceanaire
Logged / Sampled By: ACS/SR
Type of Rig: CME-75

Hole Diameter: 8-inches
Drive Weight: 140 lbs Auto-hammer
Location: See Plate 1 Geotechnical Map

SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

<table>
<thead>
<tr>
<th>Elevation Feet</th>
<th>Depth Feet</th>
<th>Graphic Log</th>
<th>Attitudes</th>
<th>Sample No.</th>
<th>Blows Per 8 inches</th>
<th>Dry Density pcf</th>
<th>Moisture Content %</th>
<th>Soil Class. (U.S.C.S.)</th>
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<tbody>
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<td>0</td>
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<td>13</td>
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<td>S-3</td>
<td>SP-SM</td>
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</table>

**SAMPLE TYPES:**
- S: SPLIT SPOON
- G: GRAB SAMPLE
- R: RING SAMPLE
- C: CORE SAMPLE
- B: BULK SAMPLE
- T: TUBE SAMPLE

**TYPE OF TESTS:**
- DS: DIRECT SHEAR
- MD: MAXIMUM DENSITY
- CR: CORROSION
- UC: UNCONFINED COMpressive STRENGTH
- CN: CONSOLIDATION
- EI: EXPANSION INDEX
- RV: R VALUE
- PP: POCKET PENETROMETER
- SA: SIEVE ANALYSIS
- SE: SAND EQUIVALENT
- AL: ATTERBERG LIMITS
- CO: COLLAPSE
- MD: MAXIMUM DENSITY
- PP: POCKET PENETROMETER
- UC: UNCONFINED COMpressive STRENGTH

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## SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

<table>
<thead>
<tr>
<th>Elevation Feet</th>
<th>Depth Feet</th>
<th>Graphic Log</th>
<th>Attitudes</th>
<th>Sample No.</th>
<th>Blows Per 6 Inches</th>
<th>Dry Density pcf</th>
<th>Moisture Content, %</th>
<th>Soil Class, (U.S.C.S.)</th>
<th>Type of Tests</th>
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<td>@ 30'</td>
<td></td>
<td>S-4</td>
<td></td>
<td>6</td>
<td>8</td>
<td></td>
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<td>SP-SC</td>
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<tr>
<td></td>
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<td>S-5</td>
<td></td>
<td>4</td>
<td>8</td>
<td></td>
<td></td>
<td>SP-SM</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>S-6</td>
<td></td>
<td>11</td>
<td>13</td>
<td></td>
<td></td>
<td>SP-SM</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>S-7</td>
<td></td>
<td>4</td>
<td>24</td>
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<td></td>
<td>SP-SM</td>
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<td></td>
<td></td>
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<td></td>
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<td>25</td>
<td>41</td>
<td>50/4</td>
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<td>SM</td>
<td></td>
</tr>
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</table>

**SAMPLE TYPES:**
- S: Split Spoon
- G: Grab Sample
- R: Ring Sample
- C: Core Sample
- B: Bulk Sample
- T: Tube Sample

**TYPE OF TESTS:**
- DS: Direct Shear
- MD: Maximum Density
- CN: Consolidation
- CR: Corrosion
- UC: Unconfined Compressive Strength
- SA: Sieve Analysis
- SE: Sand Equivalent
- EI: Expansion Index
- PP: Pocket Penetrometer
- AL: Atterberg Limits

---

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### GEOTECHNICAL BORING LOG B-2

**Date**: 1-22-07  
**Project**: 012120-001 Intergulf/Oceanaire  
**Logged / Sampled By**: ACS/SR  
**Drilling Co.**: Martini Drilling Corp  
**Type of Rig**: CME-75  
**Hole Diameter**: 8-inches  
**Drive Weight**: 140 lbs Auto-hammer  
**Drop**: 30"  
**Elevation Top of Hole**: 4'  
**Location**: See Plate 1 Geotechnical Map

#### Soil Description

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

<table>
<thead>
<tr>
<th>Elevation Feet</th>
<th>Depth Feet</th>
<th>Graphic Log</th>
<th>Attitudes</th>
<th>Sample No.</th>
<th>Blows Per 6 inches</th>
<th>Dry Density Pcf</th>
<th>Moisture Content, %</th>
<th>Soil Class, (U.S.C.S.)</th>
<th>Type of Tests</th>
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<tbody>
<tr>
<td>60</td>
<td></td>
<td></td>
<td>S-9</td>
<td>6</td>
<td>18, 20</td>
<td></td>
<td></td>
<td>SM</td>
<td>@60°: Silty SAND, brown, dense, wet, fine grained sand</td>
</tr>
</tbody>
</table>
| -60            |            |             | S-10      | 6          | 10, 14            |                 |                     | SM                   | @65°: Top - Same as above, medium dense  
|                |            |             |           |            |                   |                 |                     | Bottom - Silty SAND, grey, medium dense, wet, fine grained sand |
| -65            |            |             | S-11      | 5          | 12, 24            |                 |                     | SM                   | @70°: Same as above, dense |
| -70            |            |             | S-12      | 4          | 18, 16            |                 |                     | ML                   | @75°: Sandy SIL.T, gray, hard, moist, fine grained sand |
| -75            |            |             |           |            |                   |                 |                     |                      | Total depth of boring = 76.5 feet.  
|                |            |             |           |            |                   |                 |                     |                      | Groundwater encountered at 6.5 feet during drilling.  
| -80            |            |             |           |            |                   |                 |                     |                      | Hole was backfilled with soil cuttings and bentonite and capped with asphalt patch upon completion of drilling. |

**Sample Types:**
- S: Split Spoon
- R: Ring Sample
- B: Bulk Sample
- T: Tube Sample

**Type of Tests:**
- OS: Direct Shear
- DAM: Maximum Density
- CN: Consolidation
- CR: Corrosion
- UC: Unconfined Compressive Strength
- DS: Sieve Analysis
- MD: Sand Equivalent
- EI: Expansion Index
- PP: Pocket Penetrometer
- SA: Atterberg Limits
- CO: Collapse
- RV: R Value

---

**Page 3 of 3**
**GEOTECHNICAL BORING LOG B-3**

**Date**: 1-22-07  
**Project**: 012120-001 Intergulf/Oceanaire  
**Logged/Sampled By**: ACS/SR  
**Drilling Co.**: Martini Drilling Corp  
**Type of Rig**: CME-75  
**Hole Diameter**: 8-inches  
**Drive Weight**: 140 lbs Auto-hammer  
**Elevation Top of Hole**: 5'  
**Location**: See Plate 1 Geotechnical Map

<table>
<thead>
<tr>
<th>Elevation Feet</th>
<th>Depth Feet</th>
<th>Graphic Log</th>
<th>Attitudes</th>
<th>Sample No.</th>
<th>Blows Per 6 inches</th>
<th>Dry Density,pcf</th>
<th>Moisture Content, %</th>
<th>Soil Class, (U.S.C.S.)</th>
<th>Type of Tests</th>
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<td></td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>R-2</td>
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<td>R-2</td>
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<td>6</td>
<td></td>
<td>SP</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-5</td>
<td>R-3</td>
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<td>R-3</td>
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<td>12</td>
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</tbody>
</table>

**SAMPLE TYPES:**  
S SPLIT SPOON  
R RING SAMPLE  
B BULK SAMPLE  
T TUBE SAMPLE  

**TYPE OF TESTS:**  
DS DIRECT SHEAR  
MD MAXIMUM DENSITY  
CN CONSOLIDATION  
CR CORROSION  
UC UNCONFINED COMPRESSIVE STRENGTH  
SA SIEVE ANALYSIS  
SE SAND EQUIVALENT  
EI EXPANSION INDEX  
PP POCKET PENETROMETER

**SOIL DESCRIPTION**

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

- @ 0': 5 inches of Asphalt Concrete
- Artificial Fill, Undocumented: (Afp)
- Clayey SAND, dark brown, moist, fine-coarse grained
- @ 0.4': Clayey SAND, dark brown, moist, fine to coarse grained sand
- @ 2.5': Quaternary alluvium: (Qal)
- SAND, grayish brown, medium dense, very moist, fine to medium grained
- @ 5': SAND, grayish brown, medium dense, wet, fine to medium grained
- @ 7.5': SAND, grayish brown, medium dense, wet, fine to medium grained, with trace of shells
- @ 10': SAND, grayish brown, medium dense, wet, fine to medium grained
- @ 15': SAND, grayish brown, medium dense, wet, fine to medium grained
- @ 20': Silty SAND, brown, medium dense, wet, fine grained
- @ 25': Silty SAND, brown, medium dense, wet, fine grained

***This log is a part of a report by Leighton and should not be used as a stand-alone document.***
**GEOTECHNICAL BORING LOG B-3**

<table>
<thead>
<tr>
<th>Date</th>
<th>1-22-07</th>
<th>Sheet</th>
<th>2 of 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>012120-001 Intergulf/Oceanaire</td>
<td>Logged / Sampled By</td>
<td>ACS/SR</td>
</tr>
<tr>
<td>Drilling Co.</td>
<td>Martini Drilling Corp</td>
<td>Type of Rig</td>
<td>CME-75</td>
</tr>
<tr>
<td>Hole Diameter</td>
<td>8-inches</td>
<td>Drive Weight</td>
<td>140 lbs Auto- hammer</td>
</tr>
<tr>
<td>Elevation Top of Hole</td>
<td>5'</td>
<td>Location</td>
<td>See Plate 1 Geotechnical Map</td>
</tr>
</tbody>
</table>

### SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

- **@ 30':** Silty CLAY, brown, stiff, moist, low plasticity clay
- **Total depth of boring = 31.5 feet.**
- **Groundwater encountered at 6 feet during drilling.**
- **Hole was backfilled with soil cuttings and bentonite and patched with asphalt upon completion of drilling.**

---

**SAMPLE TYPES:**
- S: SPLIT SPOON
- R: RING SAMPLE
- B: BULK SAMPLE
- T: TUBE SAMPLE

**TYPE OF TESTS:**
- DS: DIRECT SHEAR
- MD: MAXIMUM DENSITY
- CN: CONSOLIDATION
- CR: CORROSION
- UC: UNCONFINED COMpressive STRENGTH

- SA: SIEVE ANALYSIS
- SE: SAND EQUIVALENT
- EI: EXPANSION INDEX
- RV: R VALUE
- PP: POCKET PENETROMETER

---

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***
### Monitoring Well Construction Log

**Project Number:** 012120-002  
**Leighton Group Company**  
**Project Name:** Interqulf/OceanAire  
**Location:** 150 W. Ocean Blvd.

**Boring/Well Number:** B-4  
**Date Drilled:** 1/22/07  
**Casing Type/ Diameter:** 3"  
**Screen Type/Slot:** N/A  
**Gravel Pack Type:** N/A  
**Ground Water Elevation:** N/A

**Sampling Method:** Hand Auger  
**Ground Elevation:** Hand Auger  
**Top of Casing:** 7.5 ft  
**Logged By:** MWL  
**Remarks:**...

<table>
<thead>
<tr>
<th>Depth (ft B.G.L.)</th>
<th>Blow Counts</th>
<th>Recovery Extent</th>
<th>Sample ID</th>
<th>U.S.C.S.</th>
<th>Lithologic Description</th>
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<tbody>
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<td>B-4/1'</td>
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<td>SM</td>
<td>Dirt Surface</td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td>@ 0': Brown silty SAND with gravel, slightly moist, medium dense</td>
</tr>
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<td></td>
<td>@ 1': Same as above</td>
</tr>
<tr>
<td>B-4/5'</td>
<td></td>
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<td>SP</td>
<td>Alluvium</td>
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<td>@ 3': Light grayish-brown, very fine to fine SAND</td>
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<td>@ 7': Same as above</td>
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</table>

Total Depth = 7.5 feet below ground surface  
No groundwater encountered at time of drilling  
Borehole backfilled with soil cuttings on 1/22/07

**WELL DIAGRAM:**  
No Monitoring Well Installed
**MONITORING WELL CONSTRUCTION LOG**

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<tr>
<th>PROJECT NUMBER</th>
<th>012120-002</th>
<th>BORINGWELL NUMBER</th>
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<td>DEPTH TO WATER</td>
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<td>LOGGED BY</td>
<td>MWL</td>
<td>GROUND WATER ELEVATION</td>
<td>ft</td>
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<table>
<thead>
<tr>
<th>DEPTH (ft BGL)</th>
<th>BLOW COUNTS</th>
<th>RECOVERY (inches)</th>
<th>SAMPLE ID</th>
<th>P.I.D (psi)</th>
<th>U.S.C.S.</th>
<th>LITHOLOGIC DESCRIPTION</th>
<th>CONTACT DEPTH</th>
<th>WELL DIAGRAM</th>
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<td>SM FILL Dirt Surface</td>
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<td></td>
<td></td>
<td></td>
<td>@ 0&quot;: Brown sily SAND with gravel, slightly moist, medium dense</td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 1&quot;: Same as above</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-5/5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SP ALUMINUM Light grayish-brown, very fine to fine SAND</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 3&quot;: Same as above</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-5/7</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Total Depth = 7.5 feet below ground surface
No groundwater encountered at time of drilling
Borehole backfilled with soil cuttings on 1/22/07
MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER 012120-002
PROJECT NAME Intergulf/OceanAire
LOCATION 150 W. Ocean Blvd.
BORING/WELL NUMBER B-7
DATE DRILLED 3/6/07
CASING TYPE/DIAMETER 8"
SCREEN TYPE/SLOT N/A
DRILLING METHOD Hollow-Stem Auger
GRATE PACK TYPE N/A
GROUND ELEVATION ft.
SPAMING METHOD Hand Sampler
GROUT TYPE/QUANTITY Bentonite Chips
GROUND ELEVATION ft.

DEPTH (ft BSL) BLOW COUNTS RECOVERY (inches) SAMPLE ID. PID (gpm) U.S.C.S. GRAPHIC LOG
B-7/1
B-7/3
B-7/5
B-7/7

LITHOLOGIC DESCRIPTION
@ 0.3: Asphalt (0-5' Hand Augered)
@ 0.5: Dark brown, silty fine SAND, asphalt and brick fragments
@ 5': Same as above
ALLUVIUM
@ 5.5: Light brown, silty fine SAND, moist, loose, no hydrocarbon odor noted
@ 7": Brown, trace shell fragments, same as above

CONTACT DEPTH: 0.3 - 7.5

Total Depth = 7.5 feet below ground surface
Backfilled with 2.6 ft of hydrated bentonite chips and capped with asphalt on 2/16/07

WELL DIAGRAM
No Monitoring Well Installed

PAGE 1 OF 1
## Monitoring Well Construction Log

### Project Details
- **Project Number**: 012120-32
- **Project Name**: Interquif/OceanAire
- **Location**: 150 W. Ocean Blvd.
- **Drilling Method**: Hollow-Stem Auger
- **Sampling Method**: Split Spoon
- **Ground Elevation**: ft.
- **Top of Casing**: N/A
- **Logged By**: MWL

### Boring/Well Number Details
- **Boring/Well Number**: B-9
- **Date Drilled**: 4/17/07
- **Casing Type/Diameter**: 8"
- **Screen Type/Slot**: N/A
- **Gravel Pack Type**: N/A
- **Depth to Water**: 10.00 ft.
- **Groundwater Elevation**: ft.

### Lithologic Description

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>Blow Counts</th>
<th>Recovery (inches)</th>
<th>P.D. (ppm)</th>
<th>U.S.C.S.</th>
<th>Graphic Log</th>
<th>Contact</th>
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<tbody>
<tr>
<td>8</td>
<td>7</td>
<td>0</td>
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<td>SM</td>
<td>3&quot; Asphalt</td>
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<tr>
<td>10</td>
<td>28</td>
<td>62.6</td>
<td>8.3</td>
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<td>37</td>
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<tr>
<td>14</td>
<td>50/6&quot;</td>
<td>0</td>
<td>0.0</td>
<td></td>
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<td>0.0</td>
<td></td>
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</tr>
<tr>
<td>29</td>
<td>50/6&quot;</td>
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<td></td>
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<tr>
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<td></td>
<td>0</td>
<td>0.0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Remarks
- **3" Asphalt**
  - @ 0.5': Brown, silty fine to medium SAND, bricks, loose, slightly moist
- **Fill**
  - @ 4": Same as above
- **Alluvium**
  - @ 5": Grayish-brown, silty fine to medium SAND, moist, medium dense, no hydrocarbon odor noted
  - @ 8": Dark gray, hydrocarbon odor noted from cuttings
  - @ 10": Dark gray, fine SAND, moist, hydrocarbon odor noted, very dense
  - @ 11": Groundwater encountered
  - @ 15": Light brown, trace hydrocarbon odor noted, wet
  - @ 16": No hydrocarbon odor noted
  - @ 20": Light grayish-brown, fine to medium SAND, wet, very dense, no hydrocarbon odor noted

**Total Depth = 21 feet below ground surface**
- Backfilled on 4/17/07
- Groundwater encountered at 10 feet below ground surface
- **Quantities Used**
  - Concrete: 0.2 ft³
  - Bentonite Chips: 7.2 ft³

---

**A Leighton Group Company**

**No Monitoring Well Installed**
## Monitoring Well Construction Log

**Project Number:** 012120-002  
**Boring/Well Number:** B-10  
**Location:** 150 W Ocean Blvd.  
**Project Name:** InterGulf/OceanAire  
**Drilling Method:** Hollow-Stem Auger  
**Sample Method:** Split Spoon  
**Ground Elevation:** N/A  
**Top of Casing:** N/A  
**Logged By:** MWL  
**Remarks:**

<table>
<thead>
<tr>
<th>Depth (ft BS)</th>
<th>Blow Counts</th>
<th>Recovery (inches)</th>
<th>Extent</th>
<th>PID (psm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td></td>
<td></td>
<td>SM</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td></td>
<td>FILL</td>
<td>0.3</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>S-1</td>
<td>0.0</td>
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<td>21</td>
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<td>16</td>
<td>18</td>
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</tr>
<tr>
<td>21</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Lithologic Description

- **3' Asphalt Fill**
  - @ 0.5': Brown, silty fine to medium sand, debris, loose, dry, no hydrocarbon odor noted
  - @ 2'-4': Pieces of concrete 2-3" in diameter, same as above

- **Alluvium**
  - @ 5': Light brown, fine-grained sand, slightly moist, medium dense, no hydrocarbon odor noted
  - @ 7': Gray, same as above

- @ 10': Gray with thin lenses of reddish-brown, wet, dense, no hydrocarbon odor noted

- @ 15': Grayish-brown, fine- to medium-grained sand, wet, very dense, no hydrocarbon odor noted

- @ 20': Same as above

**Total depth = 21 feet below ground surface**  
Backfilled on 4/17/07  
Groundwater encountered at 10 feet below ground surface  
**Quantities Used**
- Concrete: 0.2ft³  
- Bentonite Chips: 7.2 ft³
**MONITORING WELL CONSTRUCTION LOG**

**PROJECT NUMBER** 012120-002
**BORING/WELL NUMBER** B-11

**PROJECT NAME** Intergulf/OceanAire
**DATE DRILLED** 4/17/07

**LOCATION** 150 W. Ocean Blvd.
**CASING TYPE/DIAMETER** 8"

**DRILLING METHOD** Hollow-Stem Auger
**SCREEN TYPE/SLOT** N/A

**SAMPLING METHOD** Split Spoon
**GRAVEL PACK TYPE** N/A

**GROUND ELEVATION** ft.
**GROUNDELEVATION** ft.

**TOP OF CASING** N/A
**DEPTH TO WATER** 10.00 ft.

**LOGGED BY** MWL

### DEPTH (ft. BGL) | BLOW COUNTS | RECOVERY (inches) | SAMPLE ID. | EXTENT | PID (ppm) | U.S.C.S. | GRAPHIC LOG | LITHOLOGIC DESCRIPTION | CONTACT (ft.) | WELL DIAGRAM
---|---|---|---|---|---|---|---|---|---|---
8 | 0.0 | S-1 | 0.0 | SM | 3" Asphalt FILL | 0.3 | 0.3 | No Monitoring Well Installed
17 | 31 | 0.0 | ML | @ 0.5': Brown, silty fine to medium SAND, brick fragments, debris, loose
44 | 10 | 0.0 | S-2 | @ 8': Concrete approximately 3-4", pieces in thickness
37 | 14 | 0.0 | S-3 | @ 10': Grayish-brown, silty fine SAND, wet, very dense, no hydrocarbon odor noted
30 | 11 | 0.0 | S-4 | @ 16': Light grayish-brown, fine to medium SAND, wet, very dense, no hydrocarbon odor noted
50/2" | 28 | 0.0 | SP | @ 19': Light brown, same as above

Total Depth = 21 feet below ground surface
Backfilled on 4/17/07
Groundwater encountered at 10 feet below ground surface at time of drilling
Quantities Used
Concrete: 0.2 ft³
Bentonite Chips: 7.2 ft³
<table>
<thead>
<tr>
<th>DEPTH (ft. B.G.L.)</th>
<th>BLOW COUNTS</th>
<th>RECOVERY (inches)</th>
<th>SAMPLE ID.</th>
<th>EXTENT</th>
<th>PH (ppm)</th>
<th>U.S.C.S.</th>
<th>GRAPHIC LOG</th>
<th>LITHOLOGIC DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>SM</td>
<td>0.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3&quot; Asphalt FILL</td>
</tr>
<tr>
<td>S-2</td>
<td>SP</td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 0.5&quot; Dark gray, silty fine to medium SAND, asphalt</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 1&quot; Same as above</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
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<td>ALLUVIUM</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 1.5&quot; Light gray fine SAND, moist, medium dense, no</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>hydrocarbon odor noted</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 2&quot; Same as above</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 5&quot; Same as above</td>
</tr>
<tr>
<td>S-3</td>
<td></td>
<td>5.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total Depth = 5.5 feet below ground surface</td>
</tr>
</tbody>
</table>

**Remarks:**

- Backfilled on 9/17/07
- Quantities Used
  - Concrete: 0.2 ft³
  - Bentonite Chips: 1.7 ft³

**Well Diagram:**

No Monitoring Well Installed
## MONITORING WELL CONSTRUCTION LOG

**Project Number**: 012120-002  
**Boring/Well Number**: B-13  
**Date Drilled**: 4/17/07  
**Location**: 150 W. Ocean Blvd.  
**Drilling Method**: Hollow-Stem Auger  
**Sampling Method**: Split Spoon  
**Casing Type/ Diameter**: 8"  
**Screen Type/Slot**: N/A  
**Gravel Pack Type**: N/A  
**Ground Elevation**: ft.  
**Top of Casing**: N/A  
**Logged By**: MWL  
**Remarks**:

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>Blow Counts</th>
<th>Recovery (inches)</th>
<th>Sample ID</th>
<th>PID (rpm)</th>
<th>U.S.C.S.</th>
<th>Lithologic Description</th>
<th>Contact Depth</th>
<th>Well Diagram</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3&quot; Asphalt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Fill @ 0.5: Brown, silty fine SAND, asphalt fragments, loose, moist</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Alluvium @ 1.5: Light brown, very fine SILT, moist, medium dense, no hydrocarbon odor noted</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td></td>
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<td></td>
<td></td>
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<td>ML @ 5: Same as above</td>
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<tr>
<td>10.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SP @ 10: Light brown, fine grained SAND, wet, very dense, no hydrocarbon odor noted</td>
<td>15.5</td>
<td></td>
</tr>
<tr>
<td>15.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>S-3 @ 15: Same as above</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Total Depth = 15.5 Feet  
Backfilled on 4/17/07  
Groundwater encountered at 10 feet below ground surface at time of drilling  
Quantities Used  
Concrete: 0.2 ft³  
Bentonite Chips: 5.2 ft³  

No Monitoring Well Installed
### MONITORING WELL CONSTRUCTION LOG

**Project Number:** 012120-002  
**Boring/Well Number:** B-14

**Location:** 150 W. Ocean Blvd.  
**Date Drilled:** 5/2/07

**Drilling Method:** Hand Auger  
**Sampling Method:** Hand Auger

**Ground Elevation:** N/A  
**Top of Casing:** N/A

**Logged By:** MWL

**Remarks:**
- @ 0.5': Brown, silty fine SAND, brick fragments, dry, loose, no hydrocarbon odor noted
- @ 1.5': Same as above with concrete debris
- @ 2.5': Refusal on solid concrete, borehole abandoned

**Lithologic Description:**
- 0.3' Asphalt FILL
- 2.5' Total Depth = 2.5 feet below ground surface
- No groundwater encountered at time of drilling
- Borehole backfilled with soil cuttings on 5/2/07

**Well Diagram:**
- No Monitoring Well Installed

---

**Depth (ft. bgl) | Blow Counts | Recovery (inches) | Sample ID | PHD (ppm) | U.S.C.S. | Graphic Log**
<table>
<thead>
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<td>B-14/2</td>
<td>3</td>
<td>N/A</td>
<td>0.3</td>
<td>2.5</td>
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</table>

---

**Contact Depth:**

---

**Well Diagram:**

---

**Ground Water Elevation:** N/A  
**Casing Type/Diameter:** 3"  
**Screen Type/Slot:** N/A  
**Gravel Pack Type:** N/A  
**Grout Type/Quantity:** N/A

---

**Lithologic Description:**
- Total Depth = 2.5 feet below ground surface
- No groundwater encountered at time of drilling
- Borehole backfilled with soil cuttings on 5/2/07
### MONITORING WELL CONSTRUCTION LOG

<table>
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<th>PROJECT NUMBER</th>
<th>012120-002</th>
<th>BORING/WELL NUMBER</th>
<th>B-15</th>
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</thead>
<tbody>
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<td>PROJECT NAME</td>
<td>Intergulf/OceanAire</td>
<td>DATE DRILLED</td>
<td>5/2/07</td>
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<tr>
<td>LOCATION</td>
<td>150 W. Ocean Blvd.</td>
<td>CASING TYPE/DIAMETER</td>
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<tr>
<td>DRILLING METHOD</td>
<td>Hand Auger</td>
<td>SCREEN TYPE/SLOT</td>
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<tr>
<td>SAMPLING METHOD</td>
<td>Hand Auger</td>
<td>GRAVEL PACK TYPE</td>
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<td>GROUND ELEVATION</td>
<td>ft.</td>
<td>GROUT TYPE/QUANTITY</td>
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<tr>
<td>TOP OF CASING</td>
<td>N/A</td>
<td>DEPTH TO WATER</td>
<td>ft.</td>
</tr>
<tr>
<td>LOGGED BY</td>
<td>MWL</td>
<td>GROUND WATER ELEVATION</td>
<td>ft.</td>
</tr>
<tr>
<td>REMARKS</td>
<td></td>
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</tr>
</tbody>
</table>

#### Lithological Description

- **FILL**
  - @ 0.5': Dark brown, silty fine SAND, brick fragments, dry, medium dense
  - @ 1.5': Same as above
  - @ 2': Refusal on solid brick and concrete debris, borehole abandoned

Total Depth = 2 feet below ground surface
No groundwater encountered at time of drilling
Borehole backfilled with soil cuttings on 5/2/07

#### WELL DIAGRAM

No Monitoring Well Installed
### Monitoring Well Construction Log

**Project Number:** 012120-002  
**Boring/Well Number:** MW-1  
**Project Name:** Intrigue/OceanAire  
**Date Drilled:** 3/6/07  
**Location:** 150 W. Ocean Blvd.  
**Casing Type/Diameter:** 2"  
**Screen Type/Slot:** 0.01"  
**Gravel Pack Type:** 2/12 Sand  
**Casing Type/Quantity:** Bentonite Chips  
**Sampling Method:** Hand Sampler  
**Drilling Method:** Hollow-Stem Auger  
**Boring/Well Number:** MW-1  
**Logged By:** RAJ  
**Remarks:**

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<td>FILL</td>
<td>0.0</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
<td>Asphalt (0 to 2' Hand Auger)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 0.5': Dark brown, moist, silty SAND, no hydrocarbon odor noted</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 3': Gray, moist, fine SAND, no hydrocarbon odor noted</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 5': Gray, moist, fine SAND, no hydrocarbon odor noted</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 8': Groundwater encountered</td>
</tr>
</tbody>
</table>

**Total Depth:** 17 feet below ground surface  
**Backfilled on:** 3/6/07  
**Quantities Used:**  
- Concrete: 0.9 ft³  
- Bentonite Chips: 0.6 ft³  
- No. 2/12 Sand: 3.9 ft³
**MONITORING WELL CONSTRUCTION LOG**

**PROJECT NUMBER** 012120-002  
**BO~ING/WELL NUMBER** MW-2

**DATE DRILLED** 4/17/07  
**CASING TYPE/ DIAMETER** 2"

**PROJECT NAME** Intergulf/OceanAire  
**SCREEN TYPE/SLOT** 0.01"

**LOCATION** 150 W. Ocean Blvd.  
**GRAVEL PACK TYPE** #2/12 Sand

**DRILLING METHOD** Hollow-Stem Auger  
**GROUT TYPE/QUANTITY** Bentonite Chips

**SAMPLING METHOD** Split Spoon  
**DEPTH TO WATER** 10.00 ft.

**GROUND ELEVATION** ft.  
**GROUND WATER ELEVATION** ft.

**TOP OF CASING** N/A  
**LOGGED BY** MWL

**REMARKS**

---

**LITHOLOGIC DESCRIPTION**

**SM**

@ 3': Asphault  
Fill

@ 0.5': Dark brown, silty fine to medium SAND, loosely compacted, debris, slightly moist

@ 2.5': Medium brown, cobbles, same as above, brick fragments

**ML**

@ 6': Light brown, very fine to fine SILT, moist, medium dense

**SP**

@ 10': Grayish-brown, silty fine SAND, wet, very dense, no hydrocarbon odor noted

@ 15': Gray, fine to medium SAND, wet, dense, no hydrocarbon odor noted

@ 20': Light brown, same as above

**WELL DIAGRAM**

- @0'-2': Concrete
- @2'-3': Bentonite Chips
- @0'-5': Blank PVC
- @5'-15': Schedule 40 0.01' Slotted Casing
- @3'-21.5': #2/12 Sand

---

Total Depth = 21.5 feet below ground surface  
Backfilled on 4/17/07  
Borehole backfilled from 21.5' to 15' below ground surface with sand. Monitoring Well installed from 15' to surface.

**Quantities Used**

- Concrete: 0.5 ft³
- Bentonite Chips: 0.3 ft³
- #2/12 Sand: 5.9 ft³
Site: INTERGULF - OCEANAIRE
Engineer: R. STROH
Sounding: CPT-01
Date: 2/16/2007 10:08

LEIGHTON

Max. Depth: 35.100 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)
Sounding: CPT-03
Date: 2/16/2007 11:56

Max. Depth: 50.200 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)
Site: INTERGULF - OCEANAIRE
Engineer: R. STROH
Sounding: CPT-04
Date: 2/16/2007 11:20

Max. Depth: 35.100 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)
APPENDIX C
Section A-A' - 1.5H:1V Slope Backcut

Temporary Slope Stability

End of Sidewalk
West Ocean Boulevard

Method: bishop simplified
Factor of Safety: 1.37
Center: 157.950, 44.666
Radius: 44.359
Left Slip Surface Endpoint: 117.278, 26.961
Right Slip Surface Endpoint: 155.257, 0.389

<table>
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<tr>
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<th>Color</th>
<th>Unit Weight (lbs/ft³)</th>
<th>Strength Type</th>
<th>Cohesion (lb/ft²)</th>
<th>Phi</th>
<th>Water Surface</th>
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Proposed Residential Development - Oceanaire

Leighton Consulting, Inc.  
A Leighton Group Company

Analyzed By: SP  
Units: feet  
Scale: 1:120  
Project No.: 10594.001

Date: 3/3/2014  
Condition: Static

P:\Infocus Projects\10594 Lennar Oceanaire\001\Analyses\Slope Stability\Section A Static Temporary Slope Stability.slim
Slide Analysis Information

Proposed Residential Development - Oceanaire

Project Summary

File Name: Section A Static Temporary Slope Stability
Slide Modeler Version: 6.008
Project Title: Proposed Residential Development - Oceanaire
Analysis: Section A-A’ - 1.5H:1V Slope Backcut
Author: SP
Date Created: 3/3/2014
Comments:

Static
10594.001
Temporary Slope Stability

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Left to Right
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used
Bishop simplified
Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malfa < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116
Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Slope Search
Number of Surfaces: 5000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Material Properties

Section A Static Temporary Slope Stability.slim

3/3/2014
Global Minimums

Method: bishop simplified

FS: 1.366200
Center: 157.950, 44.666
Radius: 44.359
Left Slip Surface Endpoint: 117.278, 26.961
Right Slip Surface Endpoint: 155.257, 0.389
Resisting Moment=910071 lb-ft
Driving Moment=666134 lb-ft

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces: 4987
Number of Invalid Surfaces: 13

Error Codes:

Error Code -103 reported for 7 surfaces
Error Code -113 reported for 6 surfaces

Error Codes

The following errors were encountered during the computation:

-103 = Two surface / slope intersections, but one or more surface / nonslope external polygon intersections lie between them. This usually occurs when the slip surface extends past the bottom of the soil region, but may also occur on a benched slope model with two sets of Slope Limits.
-113 = Surface intersects outside slope limits.

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.3662
### Interslice Data

**Global Minimum Query (bishop simplified) - Safety Factor: 1.3662**

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### List Of Coordinates

**Water Table**

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**External Boundary**

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Material Boundary

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Project Name: Intergulf Oceanaire
Tested By: GB
Date: 01/31/07
Input By: LF
Date: 02/01/07
Project No.: 012120-001
Checked By: LF
Boring No.: B-2
Sample No.: S-8
Depth (ft.): 50.0
Soil Identification: Olive brown lean clay (CL)

<table>
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<th>TEST NO.</th>
<th>PLASTIC LIMIT</th>
<th>LIQUID LIMIT</th>
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<td>Wet Wt. of Soil + Cont. (g)</td>
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<td>Wt. of Container (g)</td>
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<td>Moisture Content (%) [Wn]</td>
<td>25.50</td>
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Liquid Limit 42
Plastic Limit 26
Plasticity Index 16.07
Classification CL

PI at "A" - Line = 0.73(LL-20) 16.06
One-Point Liquid Limit Calculation
LL = Wn(N/25)³/¹²

PROCEDEURES USED
- Wet Preparation
  Multipoint - Wet
- Dry Preparation
  Multipoint - Dry
- Procedure A
  Multipoint Test
- Procedure B
  One-point Test
ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
( ASTM D 2435)

Project Name: Intergulf Oceanaire
Tested By: FT,ESS Date: 01/29/07
Project No.: 012120-001
Checked By: LF Date: 02/07/07
Boring No.: B-2
Depth (ft.): 52.0
Sample No.: R-6
Sample Type: Drive

Soil Identification: Olive brown lean clay with sand (CL)s

Sample Diameter (in.): 2.416
Sample Thickness (in.): 1.000
Weight of Sample + ring (g): 191.93
Weight of Ring (g): 44.23
Height after consol. (in.): 0.9640

Before Test
Wt. of Wet Sample+Cont. (g): 182.40
Wt. of Dry Sample+Cont. (g): 148.82
Weight of Container (g): 38.83
Initial Moisture Content (%): 30.5
Initial Dry Density (pcf): 94.0
Initial Saturation (%): 104
Initial Vertical Reading (in.): 0.1030

After Test
Wt. of Wet Sample+Cont. (g): 228.99
Wt. of Dry Sample+Cont. (g): 194.16
Weight of Container (g): 39.21
Final Moisture Content (%): 31.46
Final Dry Density (pcf): 95.4
Final Saturation (%): 111
Final Vertical Reading (in.): 0.1390
Specific Gravity (assumed): 2.70
Water Density (pcf): 62.43

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<tr>
<th>Pressure (p) (ksf)</th>
<th>Final Reading (in.)</th>
<th>Apparent Thickness (in.)</th>
<th>Load Compliance (%)</th>
<th>Deformation % of Sample Thickness</th>
<th>Void Ratio</th>
<th>Corrected Deformation (%)</th>
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No Time Readings

Log of Time (min.)

Square Root of Time (min. \(10^{2}\))

Deformation (%)

Pressure, \(p\) (ksf)

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<th>Sample No.</th>
<th>Depth (ft.)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Void Ratio</th>
<th>Degree of Saturation (%)</th>
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Soil Identification: Olive brown lean clay with sand (CL)s
**DIRECT SHEAR TEST**

**Consolidated Undrained**

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**Soil Identification:** Olive poorly graded sand (SP)

| Sample Diameter (in.): | 2.415 | 2.415 | 2.415 |
| Sample Thickness (in.): | 1.000 | 1.000 | 1.000 |
| Weight of Sample + ring (gm): | 176.14 | 176.13 | 176.15 |
| Weight of Ring (gm): | 45.51 | 45.50 | 45.52 |

**Before Shearing**

| Weight of Wet Sample + Cont. (gm): | 222.33 | 222.33 | 222.33 |
| Weight of Dry Sample + Cont. (gm): | 198.34 | 198.34 | 198.34 |
| Weight of Container (gm): | 38.39 | 38.39 | 38.39 |
| Vertical Rdg. (in.): Initial | 0.1063 | 0.1022 | 0.1065 |
| Vertical Rdg. (in.): Final | 0.1235 | 0.1253 | 0.1324 |

**After Shearing**

| Weight of Wet Sample + Cont. (gm): | 173.73 | 171.29 | 173.59 |
| Weight of Dry Sample + Cont. (gm): | 146.91 | 145.81 | 148.46 |
| Weight of Container (gm): | 38.81 | 39.15 | 39.06 |
| Specific Gravity (Assumed): | 2.70 | 2.70 | 2.70 |
| Water Density (pcf): | 62.43 | 62.43 | 62.43 |
**Sample No.** Bag-1  
**Boring No.** B-1  
**Depth (ft)** 0-5  
**Sample Type:** 90% Remold  
**Soil Identification:** Olive poorly graded sand (SP)

**Strength Parameters**

<table>
<thead>
<tr>
<th></th>
<th>C (psf)</th>
<th>$\phi$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak</td>
<td>93.0</td>
<td>32.2</td>
</tr>
<tr>
<td>Ultimate</td>
<td>77.5</td>
<td>32.0</td>
</tr>
</tbody>
</table>

**Normal Stress (kip/ft²)**

<table>
<thead>
<tr>
<th></th>
<th>2.000</th>
<th>4.000</th>
<th>8.000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Shear Stress (kip/ft²)</td>
<td>1.357</td>
<td>2.598</td>
<td>5.126</td>
</tr>
<tr>
<td>Shear Stress @ End of Test (ksf)</td>
<td>1.326</td>
<td>2.576</td>
<td>5.073</td>
</tr>
<tr>
<td>Deformation Rate (in./min.)</td>
<td>0.0500</td>
<td>0.0500</td>
<td>0.0500</td>
</tr>
</tbody>
</table>

**Depth (ft)**

<table>
<thead>
<tr>
<th></th>
<th>0.0</th>
<th>1.0</th>
<th>2.0</th>
<th>3.0</th>
<th>4.0</th>
<th>5.0</th>
<th>6.0</th>
<th>7.0</th>
<th>8.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Stress @ End of Test (ksf)</td>
<td>0.9828</td>
<td>0.9769</td>
<td>0.9741</td>
<td></td>
<td></td>
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<td></td>
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**Ultimate**

<table>
<thead>
<tr>
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<th>77.5</th>
<th>32.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Stress (kip/ft²)</td>
<td>24.8</td>
<td>23.9</td>
</tr>
<tr>
<td>Final Moisture Content (%)</td>
<td>24.8</td>
<td>23.9</td>
</tr>
</tbody>
</table>

**Normal Stress (kps/ft²)**

<table>
<thead>
<tr>
<th></th>
<th>2.000</th>
<th>4.000</th>
<th>8.000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Shear Stress (kip/ft²)</td>
<td>1.357</td>
<td>2.598</td>
<td>5.126</td>
</tr>
<tr>
<td>Shear Stress @ End of Test (ksf)</td>
<td>1.326</td>
<td>2.576</td>
<td>5.073</td>
</tr>
<tr>
<td>Deformation Rate (in./min.)</td>
<td>0.0500</td>
<td>0.0500</td>
<td>0.0500</td>
</tr>
</tbody>
</table>

**Project No.:** 012120-001

**Leighton**

**DIRECT SHEAR TEST RESULTS**

*Consolidated Undrained*

**Intergulf Oceanaire**

**01-07**
**DIRECT SHEAR TEST**

**Consolidated Undrained**

Project Name: **Intergulf Oceanaire**  
Project No.: **012120-001**  
Boring No.: **B-2**  
Sample No.: **R-4**  
Sample Type: **Drive**  
Depth (ft.): **10.0**

Soil Identification: Gravish Brown Silty Sand (SM) with shells

<table>
<thead>
<tr>
<th></th>
<th>2.415</th>
<th>2.415</th>
<th>2.415</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Diameter(in):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample Thickness(in.):</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>Weight of Sample + ring(gm):</td>
<td>190.72</td>
<td>190.75</td>
<td>195.11</td>
</tr>
<tr>
<td>Weight of Ring(gm):</td>
<td>43.63</td>
<td>42.27</td>
<td>45.86</td>
</tr>
</tbody>
</table>

**Before Shearing**

<table>
<thead>
<tr>
<th></th>
<th>173.71</th>
<th>173.71</th>
<th>173.71</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of Wet Sample+Cont.(gm):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weight of Dry Sample+Cont.(gm):</td>
<td>150.90</td>
<td>150.90</td>
<td>150.90</td>
</tr>
<tr>
<td>Weight of Container(gm):</td>
<td>39.15</td>
<td>39.15</td>
<td>39.15</td>
</tr>
<tr>
<td>Vertical Rdq.(in): Initial</td>
<td>0.0000</td>
<td>0.2494</td>
<td>0.2633</td>
</tr>
<tr>
<td>Vertical Rdq.(in): Final</td>
<td>-0.0146</td>
<td>0.2704</td>
<td>0.2843</td>
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</table>

**After Shearing**

<table>
<thead>
<tr>
<th></th>
<th>184.87</th>
<th>183.40</th>
<th>183.26</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of Wet Sample+Cont.(gm):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weight of Dry Sample+Cont.(gm):</td>
<td>159.35</td>
<td>156.04</td>
<td>159.98</td>
</tr>
<tr>
<td>Weight of Container(gm):</td>
<td>38.85</td>
<td>38.52</td>
<td>39.14</td>
</tr>
<tr>
<td>Specific Gravity (Assumed):</td>
<td>2.70</td>
<td>2.70</td>
<td>2.70</td>
</tr>
<tr>
<td>Water Density(pcf):</td>
<td>62.43</td>
<td>62.43</td>
<td>62.43</td>
</tr>
</tbody>
</table>
**DIRECT SHEAR TEST RESULTS**

**Consolidated Undrained**

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>Sample No.</th>
<th>Sample Type:</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>10</td>
<td>R-4</td>
<td>Drive</td>
</tr>
</tbody>
</table>

**Soil Identification:**
Grayish Brown Silty Sand (SM) with shells

**Strength Parameters**

<table>
<thead>
<tr>
<th></th>
<th>C (pcf)</th>
<th>φ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak</td>
<td>24.0</td>
<td>40.2</td>
</tr>
<tr>
<td>Ultimate</td>
<td>178.0</td>
<td>34.3</td>
</tr>
</tbody>
</table>

**Normal Stress (kip/ft²)**

<table>
<thead>
<tr>
<th></th>
<th>2.000</th>
<th>4.000</th>
<th>8.000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Shear Stress (kip/ft²)</td>
<td>1.875</td>
<td>3.047</td>
<td>6.845</td>
</tr>
<tr>
<td>Shear Stress @ End of Test (ksf)</td>
<td>1.860</td>
<td>2.431</td>
<td>5.795</td>
</tr>
<tr>
<td>Deformation Rate (in./min.)</td>
<td>0.0500</td>
<td>0.0500</td>
<td>0.0500</td>
</tr>
</tbody>
</table>

| Initial Sample Height (in.) | 1.000   |
| Diameter (in.)              | 2.415   |
| Initial Moisture Content (%)| 20.41   |
| Dry Density (pcf)           | 101.6   |
| Saturation (%)              | 83.6    |
| Soil Height Before Shearing (in.) | 0.9854  |
| Final Moisture Content (%)  | 21.2    |

**Project No.:** 012120-001

Intergulf Oceanaire

01-07
MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Project Name: Intergulf Oceanaire
Tested By: RDS/GB
Project No.: 012120-001
Input By: JHW
Boring No.: B-1
Date: 01/26/07
Sample No.: Bag-1
Depth (ft.): 0-5

Soil Identification: Olive Poorly-graded Sand (SP)

Preparation Method: ☑ Moist
[ ] Dry

Mold Volume (ft³): 0.03321
Ram Weight = 10 lb.; Drop = 18 in.

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wt. Compacted Soil + Mold (g)</td>
<td>3474.0</td>
<td>3509.0</td>
<td>3559.0</td>
<td>3535.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weight of Mold (g)</td>
<td>1786.0</td>
<td>1786.0</td>
<td>1786.0</td>
<td>1786.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Net Weight of Soil (g)</td>
<td>1688.0</td>
<td>1723.0</td>
<td>1773.0</td>
<td>1749.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wet Weight of Soil + Cont. (g)</td>
<td>395.00</td>
<td>357.70</td>
<td>387.30</td>
<td>407.10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dry Weight of Soil + Cont. (g)</td>
<td>360.90</td>
<td>322.50</td>
<td>339.90</td>
<td>350.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weight of Container (g)</td>
<td>51.40</td>
<td>51.40</td>
<td>54.70</td>
<td>51.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moisture Content (%)</td>
<td>11.02</td>
<td>12.98</td>
<td>16.62</td>
<td>18.96</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wet Density (pcf)</td>
<td>100.9</td>
<td>101.2</td>
<td>100.9</td>
<td>97.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dry Density (pcf)</td>
<td>102.1</td>
<td>101.2</td>
<td>100.9</td>
<td>97.6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Maximum Dry Density (pcf) 101.5
Optimum Moisture Content (%) 15.0

PROCEDURE USED

☑ Procedure A
Soil Passing No. 4 (4.75 mm) Sieve
Mold: 4 in. (101.6 mm) diameter
Layers: 5 (Five)
Blows per layer: 25 (twenty-five)
May be used if +#4 is 20% or less

☐ Procedure B
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold: 4 in. (101.6 mm) diameter
Layers: 5 (Five)
Blows per layer: 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

☐ Procedure C
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold: 6 in. (152.4 mm) diameter
Layers: 5 (Five)
Blows per layer: 56 (fifty-six)
Use if +3/8 in. is >20% and +3/4 in. is <30%

Particle-Size Distribution:
GR:SA:FI

Atterberg Limits:
LL,PL,PI
## Tests for Sulfate Content, Chloride Content, and pH of Soils

<table>
<thead>
<tr>
<th>Project Name:</th>
<th>Intergulf Oceanaire</th>
<th>Tested By:</th>
<th>VJ</th>
<th>Date: 01/29/07</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project No.:</td>
<td>012120-001</td>
<td>Data Input By:</td>
<td>LF</td>
<td>Date: 02/01/07</td>
</tr>
</tbody>
</table>

### Sample Information

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>B-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample No.</td>
<td>Bag-1</td>
</tr>
<tr>
<td>Sample Depth (ft)</td>
<td>0-5</td>
</tr>
</tbody>
</table>

### Soil Identification

<table>
<thead>
<tr>
<th>Soil Identification:</th>
<th>SM</th>
</tr>
</thead>
</table>

### Weight Measurements

<table>
<thead>
<tr>
<th>Wet Weight of Soil + Container (g)</th>
<th>236.52</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Weight of Soil + Container (g)</td>
<td>215.85</td>
</tr>
<tr>
<td>Weight of Container (g)</td>
<td>56.63</td>
</tr>
<tr>
<td>Moisture Content (%)</td>
<td>12.98</td>
</tr>
<tr>
<td>Weight of Soaked Soil (g)</td>
<td>100.84</td>
</tr>
</tbody>
</table>

### Sulfate Content, DOT California Test 417, Part II

<table>
<thead>
<tr>
<th>Beaker No.</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crucible No.</td>
<td>22</td>
</tr>
<tr>
<td>Furnace Temperature (°C)</td>
<td>840</td>
</tr>
<tr>
<td>Time In / Time Out</td>
<td>7:45 / 8:30</td>
</tr>
<tr>
<td>Duration of Combustion (min)</td>
<td>45</td>
</tr>
<tr>
<td>Wt. of Crucible + Residue (g)</td>
<td>18.7802</td>
</tr>
<tr>
<td>Wt. of Crucible (g)</td>
<td>18.7774</td>
</tr>
<tr>
<td>Wt. of Residue (g) (A)</td>
<td>0.0028</td>
</tr>
<tr>
<td>PPM of Sulfate (A) x 41150</td>
<td>115.22</td>
</tr>
<tr>
<td><strong>PPM of Sulfate, Dry Weight Basis</strong></td>
<td><strong>132</strong></td>
</tr>
</tbody>
</table>

### Chloride Content, DOT California Test 422

<table>
<thead>
<tr>
<th>ml of Chloride Soln. For Titration (B)</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>ml of AgNO3 Soln. Used in Titration (C)</td>
<td>0.9</td>
</tr>
<tr>
<td>PPM of Chloride (C -0.2) * 100 * 30 / B</td>
<td>70</td>
</tr>
<tr>
<td><strong>PPM of Chloride, Dry Wt. Basis</strong></td>
<td><strong>80</strong></td>
</tr>
</tbody>
</table>

### pH Test, DOT California Test 532/643

<table>
<thead>
<tr>
<th>pH Value</th>
<th>8.17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature °C</td>
<td>20.3</td>
</tr>
</tbody>
</table>
**SOIL RESISTIVITY TEST**
**DOT CA TEST 532 / 643**

- **Project Name:** Intergulf Oceanaire
- **Project No.:** 012120-001
- **Boring No.:** B-2
- **Sample No.:** Bag-1
- **Soil Identification:** SM

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Water Added (ml) (Wa)</th>
<th>Adjusted Moisture Content (MC)</th>
<th>Resistance Reading (ohm)</th>
<th>Soil Resistivity (ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>12.98</td>
<td>510</td>
<td>3440</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>21.67</td>
<td>400</td>
<td>2698</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>30.36</td>
<td>410</td>
<td>2766</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **Tested By:** VJ  **Date:** 01/29/07
- **Data Input By:** LF  **Date:** 02/01/07
- **Depth (ft.):** 0-5

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Moisture Content (%) (MC)</th>
<th>Wet Wt. of Soil + Cont. (g)</th>
<th>Dry Wt. of Soil + Cont. (g)</th>
<th>Wt. of Container (g)</th>
<th>Container No.</th>
<th>Initial Soil Wt. (g) (Wt)</th>
<th>Box Constant</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.98</td>
<td>236.52</td>
<td>215.85</td>
<td>56.63</td>
<td></td>
<td>1300.00</td>
<td>6.746</td>
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<tr>
<td>2</td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>

**MC = (((1+Mci/100)x(Wa/Wt+1))-1)x100**

<table>
<thead>
<tr>
<th>Min. Resistivity (ohm-cm)</th>
<th>Moisture Content (%)</th>
<th>Sulfate Content (ppm)</th>
<th>Chloride Content (ppm)</th>
<th>Soil pH</th>
</tr>
</thead>
<tbody>
<tr>
<td>DOT CA Test 532 / 643</td>
<td>Dot CA Test 417 Part II</td>
<td>DOT CA Test 422</td>
<td>DOT CA Test 532 / 643</td>
<td></td>
</tr>
<tr>
<td>2660</td>
<td>23.8</td>
<td>132</td>
<td>80</td>
<td>8.17</td>
</tr>
</tbody>
</table>

**Graph:**

- **Y-axis:** Soil Resistivity (ohm-cm)
- **X-axis:** Moisture Content (%)
Project title: Oceanaire
Location: 150 West Ocean Boulevard, Long Beach, CA

Overall Liquefaction Potential Index report

CPTU name

<table>
<thead>
<tr>
<th>CPTU</th>
<th>LPI value</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT-01</td>
<td>17.00</td>
</tr>
<tr>
<td>CPT-02</td>
<td>16.00</td>
</tr>
<tr>
<td>CPT-03</td>
<td>15.00</td>
</tr>
<tr>
<td>CPT-04</td>
<td>14.00</td>
</tr>
</tbody>
</table>

LPI color scheme
- Very high risk
- High risk
- Low risk

Basic statistics
- Total CPT number: 4
- 100.00% low risk
- 0.00% high risk
- 0.00% very high risk

CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software
Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
Overall vertical settlements report

<table>
<thead>
<tr>
<th>CPTU name</th>
<th>Vertical settlement (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT-01</td>
<td>0.48</td>
</tr>
<tr>
<td>CPT-02</td>
<td>0.374</td>
</tr>
<tr>
<td>CPT-03</td>
<td>0.308</td>
</tr>
<tr>
<td>CPT-04</td>
<td>0.707</td>
</tr>
</tbody>
</table>
Project title: Oceanaire
Location: 150 West Ocean Boulevard, Long Beach, CA

Overall Probability for Liquefaction report

Probability color scheme
- Very High Probability
- High Probability
- Low Probability

Basic statistics
Total CPT number: 4
100.00% low probability
0.00% high probability
0.00% very high probability
<table>
<thead>
<tr>
<th>TABLE OF CONTENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CPT-01 results</strong></td>
</tr>
<tr>
<td>Summary data report</td>
</tr>
<tr>
<td>Transition layer algorithm summary report</td>
</tr>
<tr>
<td>Vertical settlements summary report</td>
</tr>
<tr>
<td><strong>CPT-02 results</strong></td>
</tr>
<tr>
<td>Summary data report</td>
</tr>
<tr>
<td>Transition layer algorithm summary report</td>
</tr>
<tr>
<td>Vertical settlements summary report</td>
</tr>
<tr>
<td><strong>CPT-03 results</strong></td>
</tr>
<tr>
<td>Summary data report</td>
</tr>
<tr>
<td>Transition layer algorithm summary report</td>
</tr>
<tr>
<td>Vertical settlements summary report</td>
</tr>
<tr>
<td><strong>CPT-04 results</strong></td>
</tr>
<tr>
<td>Summary data report</td>
</tr>
<tr>
<td>Transition layer algorithm summary report</td>
</tr>
<tr>
<td>Vertical settlements summary report</td>
</tr>
</tbody>
</table>
Project title: Oceanaire
CPT file: CPT-01

Input parameters and analysis data

<table>
<thead>
<tr>
<th>Analysis method:</th>
<th>NCEER (1998)</th>
<th>Use fill:</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>G.W.T. (in-situ):</td>
<td>5.00 ft</td>
<td>Fill height:</td>
<td>N/A</td>
</tr>
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<td>Average results interval:</td>
<td>3</td>
<td>Fill weight:</td>
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<tr>
<td>Earthquake magnitude Mw:</td>
<td>7.20</td>
<td>Limit depth applied:</td>
<td>No</td>
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<tr>
<td>Peak ground acceleration:</td>
<td>0.63</td>
<td>MSF method:</td>
<td>Method based</td>
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</table>

**Cone resistance & Friction Ratio**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>qt (tsf)</th>
<th>Rf (%)</th>
</tr>
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<tbody>
<tr>
<td>35</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>34</td>
<td>200</td>
<td>2</td>
</tr>
<tr>
<td>33</td>
<td>400</td>
<td>4</td>
</tr>
<tr>
<td>32</td>
<td>600</td>
<td>6</td>
</tr>
<tr>
<td>31</td>
<td>800</td>
<td>8</td>
</tr>
<tr>
<td>30</td>
<td>1,000</td>
<td>10</td>
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**SBTn Plot**

<table>
<thead>
<tr>
<th>Ic (Robertson 1990)</th>
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</tr>
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<tbody>
<tr>
<td>2</td>
<td>2</td>
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<tr>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
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**CRR plot**

<table>
<thead>
<tr>
<th>Cyclic Stress Ratio (CSR)</th>
<th>0.1</th>
</tr>
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<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>0.2</td>
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</tr>
<tr>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>0.5</td>
<td>0.5</td>
</tr>
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</table>

**FS Plot**

<table>
<thead>
<tr>
<th>Factor of safety</th>
<th>0</th>
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<tbody>
<tr>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>1.5</td>
<td>1.5</td>
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**Summary of liquefaction potential**

<table>
<thead>
<tr>
<th>Zone</th>
<th>Liquefaction potential</th>
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</thead>
<tbody>
<tr>
<td>A1</td>
<td>Cyclic liquefaction likely depending on size and duration of cyclic loading</td>
</tr>
<tr>
<td>A2</td>
<td>Cyclic liquefaction and strength loss likely depending on loading and ground geometry</td>
</tr>
<tr>
<td>B</td>
<td>Liquefaction and post-earthquake strength loss unlikely, check cyclic softening</td>
</tr>
<tr>
<td>C</td>
<td>Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry</td>
</tr>
</tbody>
</table>

**Friction Ratio by HAND AUGER**

<table>
<thead>
<tr>
<th>Rf (%)</th>
<th>Qt (tsf)</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>86</td>
</tr>
<tr>
<td>3</td>
<td>64</td>
</tr>
<tr>
<td>4</td>
<td>20</td>
</tr>
<tr>
<td>5</td>
<td>8</td>
</tr>
</tbody>
</table>

**Mw=7.25, sigma = 1 atm base curve**

**Liquefaction Potential**

- Zone A1: Cyclic liquefaction likely depending on size and duration of cyclic loading
- Zone A2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
- Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
- Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

**Normalized friction ratio (%)**

- Zone A: No liquefaction
- Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
- Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

**Normalized CPT penetration resistance**

- Zone A: No liquefaction
- Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
- Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry
**Liquefaction analysis overall plots (intermediate results)**

**Total cone resistance**

**SBTn Index**

**Norm. cone resistance**

**Grain char. factor**

**Corrected norm. cone resistance**

---

**Input parameters and analysis data**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tr>
<td>Analysis method</td>
<td>NCEER (1998)</td>
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<tr>
<td>Fines correction method</td>
<td>NCEER (1998)</td>
</tr>
<tr>
<td>Points to test</td>
<td>Based on Ic value</td>
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<tr>
<td>Earthquake magnitude $M_w$</td>
<td>7.20</td>
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<tr>
<td>Peak ground acceleration</td>
<td>0.63</td>
</tr>
<tr>
<td>Depth to water table (in situ)</td>
<td>5.00 ft</td>
</tr>
<tr>
<td>Average results interval</td>
<td>3</td>
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<tr>
<td>Ic cut-off value</td>
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<tr>
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<td>Clay like behavior applied</td>
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CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:29 PM

Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
Input parameters and analysis data

- Fines correction method: NCEER (1998)
- Points to test: Based on Ic value
- Earthquake magnitude $M_w$: 7.20
- Peak ground acceleration: 0.63
- Depth to water table (insitu): 5.00 ft
- Depth to water table (erthq.): 0.63
- Average results interval: 3
- Ic cut-off value: 2.60
- Unit weight calculation: Based on SBT
- Use fill: No
- Fill height: N/A
- Fill weight: N/A
- Transition detect. applied: Yes
- $K$ applied: Yes
- Clay like behavior applied: Sands only
- Limit depth applied: No
- Limit depth applied: N/A
- N/A
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy
- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots

- CRR plot
- FS Plot
- LPI

Vertical settlements

Lateral displacements
Check for strength loss plots (Robertson (2010))

Input parameters and analysis data

- Fines correction method: NCEER (1998)
- Points to test: Based on Ic value
- Earthquake magnitude $M_w$: 7.20
- Peak ground acceleration: 0.63
- Depth to water table (insitu): 5.00 ft
- Use fill: No
- Fill height: N/A
- Transition detect. applied: Yes
- Kc applied: Yes
- Clay like behavior applied: Sands only
- Limit depth applied: No
- Limit depth: N/A
- Vertical results interval: Ic cut-off value:
- Unit weight calculation: Based on SBT
- Kc applied: Yes
- Transition detect. applied: Yes
- Use fill: No
- Fill height: N/A
- Limit depth applied: No
- Limit depth: N/A

Depth to water table (erthq.): 3.00 ft
Fill weight: N/A
Peak Su ratio
Liquefied Su/Sig’v

SBTn Index

Corrected norm. cone resistance

Grain char. factor

Norm. cone resistance

SBTn Index

Liquefied Su/Sig’v

CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:29 PM
Project file: C:sers\car\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
TRANSITION LAYER DETECTION ALGORITHM REPORT

Summary Details & Plots

Short description
The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of $I_c$ values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of $I_c$. Transitions typically occur when the rate of change of $I_c$ is fast (i.e. delta $I_c$ is small).

The SBT$_n$ plot below, displays in red the detected transition layers based on the parameters listed below the graphs.

Transition layer algorithm properties
- $I_c$ minimum check value: 1.70
- $I_c$ maximum check value: 3.00
- $I_c$ change ratio value: 0.0250
- Minimum number of points in layer: 4

General statistics
- Total points in CPT file: 214
- Total points excluded: 10
- Exclusion percentage: 4.67%
- Number of layers detected: 2
Estimation of post-earthquake settlements

Cone resistance

SBTn Plot

FS Plot

Strain plot

Vertical settlements

Abbreviations

q_c: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
FS: Calculated Factor of Safety against liquefaction

Volumetric strain: Post-liquefaction volumetric strain
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude Mw: 7.20
Peak ground acceleration: 0.63

Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Limit depth: N/A
MSF method: Method based

Cyclic stress ratio (CSR)

Liquefaction

Zone A1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Summary of liquefaction potential

Normalized CPT penetration resistance

Normalized friction ratio (%)

Factor of safety

3/5/2014, 3:21:30 PM

CPT basic interpretation plots

Cone resistance

Friction Ratio

Pore pressure

SBT Plot

Soil Behaviour Type

Input parameters and analysis data

Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude Mw: 7.20
Peak ground acceleration: 0.63
Depth to water table (insitu): 5.00 ft
Depth to water table (erthq.): 5.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Transition detect. applied: Yes
K, applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A
Fill weight: N/A
Use fill: No
Fill height: N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:30 PM
Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
**CPT basic interpretation plots (normalized)**

- **Norm. cone resistance**
- **Norm. friction ratio**
- **Norm. pore pressure ratio**
- **SBTn Plot**

**Input parameters and analysis data**

- **Analysis method:** NCEER (1998)
- **Fines correction method:** NCEER (1998)
- **Points to test:** Based on Ic value
- **Earthquake magnitude $M_w$:** 7.20
- **Peak ground acceleration:** 0.63
- **Depth to water table (in situ):** 5.00 ft
- **Depth to water table (earthq.):** 5.00 ft
- **Average results interval:** 3
- **Ic cut-off value:** 2.60
- **Unit weight calculation:** Based on SBT
- **Clay like behavior applied:** Sands only
- **Transition detect. applied:** Yes
- **K$_s$ applied:** Yes
- **Ic value:** N/A
- **Fill weight:** N/A
- **Transition detect. applied:** Yes
- **K$_s$ applied:** Yes
- **Ic value:** N/A
- **Fill weight:** N/A

**SBTn legend**

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to sand
9. Very stiff fine grained sand
10. Very dense/stiff soil
11. Very dense/stiff soil
12. Very dense/stiff soil
13. Very dense/stiff soil
14. Very dense/stiff soil
15. Very dense/stiff soil
16. Very dense/stiff soil
17. Very dense/stiff soil
18. Very dense/stiff soil
19. Very dense/stiff soil
**Input parameters and analysis data**

- **Analysis method:** NCEER (1998)
- **Fines correction method:** NCEER (1998)
- **Points to test:** Based on Ic value
- **Earthquake magnitude \( M_w \):** 7.20
- **Peak ground acceleration:** 0.63
- **Depth to water table (insitu):** 5.00 ft

- **Depth to water table (erthq.):** 5.00 ft
- **Average results interval:** 3
- **Ic cut-off value:** 2.60
- **Unit weight calculation:** Based on SBT
- **Use fill:** No
- **Fill height:** N/A

- **Fill weight:** N/A
- **Transition detect. applied:** Yes
- **K\(\sigma\) applied:** Yes
- **Clay like behavior applied:** Sands only
- **Limit depth applied:** No
- **Limit depth:** N/A

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<table>
<thead>
<tr>
<th>Total cone resistance</th>
<th>SBTn Index</th>
<th>Norm. cone resistance</th>
<th>Grain char. factor</th>
<th>Corrected norm. cone resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 - 4.0</td>
<td>0.5 - 4.0</td>
<td>0.5 - 4.0</td>
<td>0.5 - 4.0</td>
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</tr>
</tbody>
</table>

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**CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:30 PM**

Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
Input parameters and analysis data

Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude Mw: 7.20
Peak ground acceleration: 0.63
Depth to water table (insitu): 5.00 ft
Depth to water table (earthq.): 7.20
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A
Fill weight: N/A
Transition detect. applied: Yes
Ks applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

F.S. color scheme
- Red: Almost certain it will liquefy
- Orange: Very likely to liquefy
- Yellow: Liquefaction and no liq. are equally likely
- Green: Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Red: Very high risk
- Orange: High risk
- Yellow: Low risk
- Green: Almost certain it will not liquefy

Vertical settlements
- Settlement (in)
- Depth (ft)

Lateral displacements
- Displacement (in)
- Depth (ft)
Input parameters and analysis data

Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude $M_w$: 7.2
Peak ground acceleration: 0.63
Depth to water table (in situ): 5.00 ft
Depth to water table (erthq.): 3.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Fill weight: N/A
Transition detect. applied: Yes
K_c applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A
Use fill: No
Fill height: N/A

Peak Su ratio
Liq. Su ratio

Check for strength loss plots (Robertson (2010))

Grain char. factor
Corrected norm. cone resistance
SBTn Index
Liquefied Su/Sig'v

CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:30 PM
Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
Short description

The software will delete data when the cone is in transition from either clay to sand or vice-versa. To do this, the software requires a range of Ic values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of $I_c$. Transitions typically occur when the rate of change of $I_c$ is fast (i.e., delta $I_c$ is small).

The SBTn plot below displays in red the detected transition layers based on the parameters listed below the graphs.

### Transition layer algorithm properties

- $I_c$ minimum check value: 1.70
- $I_c$ maximum check value: 3.00
- $I_c$ change ratio value: 0.0250
- Minimum number of points in layer: 4

### General statistics

- Total points in CPT file: 305
- Total points excluded: 15
- Exclusion percentage: 4.92%
- Number of layers detected: 3
Estimation of post-earthquake settlements

Abbreviations

qt: Total cone resistance (cone resistance qc corrected for pore water effects)
Ic: Soil Behaviour Type Index
FS: Calculated Factor of Safety against liquefaction
Volumetric strain: Post-liquefaction volumetric strain
LIQUEFACTION ANALYSIS REPORT

Project title: Oceanaire
CPT file: CPT-03

Location: 150 West Ocean Boulevard, Long Beach, CA

Input parameters and analysis data

- Fines correction method: NCEER (1998)
- Points to test: Based on Ic value
- Earthquake magnitude Mw: 7.20
- Peak ground acceleration: 0.63

Use fill: No
Fill height: N/A
Fill weight: N/A
Limit depth applied: No
MSF method: Method based

Friction Ratio

Cone resistance

Friction Ratio

SBTn Plot

CRR plot

FS Plot

Summary of liquefaction potential

Zone A1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:31 PM
Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
**Input parameters and analysis data**

- **Analysis method:** NCEER (1998)
- **Fines correction method:** NCEER (1998)
- **Points to test:** Based on Ic value
- **Earthquake magnitude Mw:** 7.2
- **Peak ground acceleration:** 0.63
- **Depth to water table (insitu):** 5.00 ft

- **Depth to water table (earthq.):** 5.00 ft
- **Average results interval:** 3 ft
- **Ic cut-off value:** 2.60
- **Unit weight calculation:** Based on SBT
- **Transition detect. applied:** Yes
- **Limit depth applied:** No
- **Fill weight:** N/A
- **Fill height:** N/A
- **Kσ applied:** Yes
- **Clay like behavior applied:** No
- **Limit depth:** N/A
- **Sands only:** No
- **No N/A:** Yes

---

**SBTn legend**

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

---

**CPT basic interpretation plots (normalized)**

- **Norm. cone resistance**
- **Norm. friction ratio**
- **Norm. pore pressure ratio**
- **SBTn Plot**

---

**SBTn legend**

- Sand & silty sand
- Silty sand & sandy silt
- Silty sand & sandy silt
- Sand
- Sand & silty sand
- Very dense/stiff soil
- Very dense/stiff soil
- Clay & silty clay
- Clay & silty clay

---

**NCEER (1998)**

- Based on Ic value

---

**Ic (Robertson 1990)**

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained
**Liquefaction analysis overall plots (intermediate results)**

**Total cone resistance**

**SBTn Index**

**Norm. cone resistance**

**Grain char. factor**

**Corrected norm. cone resistance**

---

**Input parameters and analysis data**

- **Analysis method:** NCEER (1998)
- **Fines correction method:** NCEER (1998)
- **Points to test:** Based on Ic value
- **Earthquake magnitude \(M_w\):** 7.20
- **Peak ground acceleration:** 0.63
- **Depth to water table (erthq.):** 5.00 ft
- **Average results interval:** 3
- **Ic cut-off value:** 2.60
- **Use fill:** No
- **Fill height:** N/A
- **Unit weight calculation:** Based on SBT
- **Transition detect. applied:** Yes
- **\(K_c\) applied:** Yes
- **Clay like behavior applied:** Sands only
- **Limit depth applied:** No
- **Limit depth:** N/A
- **Fill weight:** N/A
- **Transition detect. applied:** Yes
- **\(K_c\) applied:** Yes
- **Clay like behavior applied:** Sands only
- **Limit depth applied:** No
- **Limit depth:** N/A

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This software is licensed to: Carl Kim

CPT name: CPT-03
Input parameters and analysis data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
<td>Analysis method</td>
<td>NCEER (1998)</td>
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<td>Earthquake magnitude ( M_w )</td>
<td>7.20</td>
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<td>Peak ground acceleration</td>
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<tr>
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<td>5.00 ft</td>
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<td>Clay like behavior applied</td>
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<td>Liquefaction and no liq. are equally likely</td>
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</tr>
<tr>
<td>Limit depth applied</td>
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<td>Fines correction method</td>
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<tr>
<td>Earthquake magnitude ( M_w )</td>
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<tr>
<td>Peak ground acceleration</td>
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<td>Clay like behavior applied</td>
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<td>Liquefaction and no liq. are equally likely</td>
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<tr>
<td>Limit depth</td>
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</tr>
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</table>

F.S. color scheme
- Red: Almost certain it will liquefy
- Orange: Very likely to liquefy
- Yellow: Liquefaction and no liq. are equally likely
- Green: Unlike to liquefy
- Black: Almost certain it will not liquefy

LPI color scheme
- Red: Very high risk
- Orange: High risk
- Yellow: Low risk
Input parameters and analysis data

- **Analysis method:** NCEER (1998)
- **Fines correction method:** NCEER (1998)
- **Points to test:** Based on Ic value
- **Earthquake magnitude Mw:** 7.20
- **Peak ground acceleration:** 0.63
- **Depth to water table (ethiq.):** 5.00 ft
- **Average results interval:** 3
- **Ic cut-off value:** 2.60
- **Unit weight calculation:** Based on SBT
- **Use fill:** No
- **Fill height:** N/A
- **Fill weight:** N/A
- **Transition detect. applied:** Yes
- **Kσ applied:** Yes
- **Clay like behavior applied:** Sands only
- **Limit depth applied:** No
- **Limit depth:** N/A
- **SBTn Index:** Ic (Robertson 1990)
- **Corrected norm. cone resistance:** Qtn,cs
- **Check for strength loss plots (Robertson 2010))**
- **Grain char. factor:** Kc
- **Norm. cone resistance**
- **SBTn Index**
- **Liquefied Su/Sig'v**

CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:31 PM
**TRANSITION LAYER DETECTION ALGORITHM REPORT**

**Summary Details & Plots**

**Short description**

The software will delete data when the cone is in transition from either clay to sand or vice-versa. To do this, the software requires a range of Ic values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of $I_c$. Transitions typically occur when the rate of change of $I_c$ is fast (i.e., delta $I_c$ is small).

The SBTn plot below, displays in red the detected transition layers based on the parameters listed below the graphs.

**Transition layer algorithm properties**

- $I_c$ minimum check value: 1.70
- $I_c$ maximum check value: 3.00
- $I_c$ change ratio value: 0.0250
- Minimum number of points in layer: 4

**General statistics**

- Total points in CPT file: 306
- Total points excluded: 0
- Exclusion percentage: 0.00%
- Number of layers detected: 0

---

**SBTn Index**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Norm. Soil Behaviour Type</th>
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</thead>
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<tr>
<td>50</td>
<td>Sand &amp; silty sand</td>
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<td>48</td>
<td>Silty sand &amp; sandy silt</td>
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<tr>
<td>28</td>
<td>Silty sand &amp; sandy silt</td>
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---

**SBTn (Robertson 1990)**

<table>
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<tr>
<th>Depth (ft)</th>
<th>Norm. Soil Behaviour Type</th>
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</thead>
<tbody>
<tr>
<td>50</td>
<td>Sand &amp; silty sand</td>
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<td>Silty sand &amp; sandy silt</td>
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<td>0</td>
<td>Sand</td>
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</tbody>
</table>
Estimation of post-earthquake settlements

Abbreviations

- \( q_t \): Total cone resistance (cone resistance \( q \) corrected for pore water effects)
- \( I_c \): Soil Behaviour Type Index
- \( FS \): Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:31 PM
Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq

CPT name: CPT-03
Project title: Oceanaire  
Location: 150 West Ocean Boulevard, Long Beach, CA

CPT file: CPT-04

Input parameters and analysis data

- Fines correction method: NCEER (1998)
- Points to test: Based on Ic value
- Earthquake magnitude $M_w$: 7.20
- Peak ground acceleration: 0.63

Use fill: No  
Fill height: N/A  
Fill weight: N/A  
Limit depth applied: No  
Trans. detect. applied: Yes  
$K_a$ applied: Yes  
MSF method: Method based

CPT penetration resistance

Friction Ratio

SBTn Plot

CRR plot

FS Plot

Summary of liquefaction potential

$M_w = 7^{1/2}$, $\sigma' = 1$ atm base curve

Liquefaction

Cyclic Stress Ratio ($\sigma'$)
CPT basic interpretation plots

Input parameters and analysis data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis method</td>
<td>NCEER (1998)</td>
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<tr>
<td>Fines correction method</td>
<td>NCEER (1998)</td>
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<tr>
<td>Points to test</td>
<td>Based on Ic value</td>
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<tr>
<td>Earthquake magnitude $M_w$</td>
<td>7.20</td>
</tr>
<tr>
<td>Peak ground acceleration $g$</td>
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</tr>
<tr>
<td>Depth to water table ($d_{w}$)</td>
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<tr>
<td>Ic cut-off value</td>
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<tr>
<td>Unit weight calculation</td>
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<tr>
<td>Limit depth applied</td>
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<tr>
<td>Limit depth</td>
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<tr>
<td>Fill weight</td>
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</tr>
</tbody>
</table>

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained
CPT basic interpretation plots (normalized)

**Norm. cone resistance**

**Norm. friction ratio**

**Norm. pore pressure ratio**

**SBTn Plot**

**Norm. Soil Behaviour Type**

**Input parameters and analysis data**

- **Analysis method:** NCEER (1998)
- **Fines correction method:** NCEER (1998)
- **Points to test:** Based on Ic value
- **Earthquake magnitude Mw:** 7.20
- **Depth to water table (erthq.):** 5.00 ft
- **Depth to water table (insitu):** 5.00 ft
- **Peak ground acceleration:** 0.63
- **Average results interval:** 3
- **Unit weight calculation:** Based on SBT
- **Ic cut-off value:** 2.60
- **Kc applied:** Yes
- **Transition detect. applied:** Yes
- **Clay like behavior applied:** Sands only
- **Fill height:** N/A
- **Fill weight:** N/A
- **Limit depth applied:** No
- **Limit depth:** N/A

**SBTn legend**

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty sand
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Very dense/stiff soil
8. Very dense/stiff soil
9. Very stiff fine grained
10. Very stiff fine grained
11. Very stiff fine grained
12. Very dense/stiff soil
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99. Very dense/stiff soil
100. Very dense/stiff soil
Input parameters and analysis data

Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude Mw: 7.2
Peak ground acceleration: 0.63
Depth to water table (in situ): 5.00 ft

Depth to water table (ethq.): 5.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Fill weight: N/A
Transition detect. applied: Yes
Kσ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBTn Index

Corrected norm. cone resistance

CLiq v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:32 PM
Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
**Liquefaction analysis overall plots**

### Input parameters and analysis data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
<td>Analysis method</td>
<td>NCEER (1998)</td>
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<tr>
<td>Fines correction method</td>
<td>NCEER (1998)</td>
</tr>
<tr>
<td>Points to test</td>
<td>Based on Ic value</td>
</tr>
<tr>
<td>Earthquake magnitude $M_w$</td>
<td>7.20</td>
</tr>
<tr>
<td>Peak ground acceleration</td>
<td>0.63</td>
</tr>
<tr>
<td>Depth to water table (in situ)</td>
<td>5.00 ft</td>
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<tr>
<td>Depth to water table (earthq.)</td>
<td>5.00 ft</td>
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<tr>
<td>Average results interval</td>
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<td>Ic cut-off value</td>
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<td>Very likely to liquefy</td>
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<td>Liquefaction and no liq. are equally likely</td>
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<td>Unlike to liquefy</td>
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<td>Almost certain it will not liquefy</td>
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Clique v.1.7.5.5 - CPT Liquefaction Assessment Software - Report created on: 3/5/2014, 3:21:32 PM

Project file: C:\Users\carl\SkyDrive\Documents\2014 projects\lennar urban\Oceanaire.clq
**Short description**

The software will delete data when the cone is in transition from either clay to sand or vice-versa. To do this the software requires a range of Ic values over which the transition will be defined (typically somewhere between 1.80 < Ic < 3.0) and a rate of change of Ic. Transitions typically occur when the rate of change of Ic is fast (i.e. delta Ic is small).

The SBTn plot below, displays in red the detected transition layers based on the parameters listed below the graphs.

<table>
<thead>
<tr>
<th>Transition layer algorithm properties</th>
<th>General statistics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ic minimum check value: 1.70</td>
<td>Total points in CPT file: 214</td>
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<tr>
<td>Ic maximum check value: 3.00</td>
<td>Total points excluded: 9</td>
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<tr>
<td>Ic change ratio value: 0.0250</td>
<td>Exclusion percentage: 4.21%</td>
</tr>
<tr>
<td>Minimum number of points in layer: 4</td>
<td>Number of layers detected: 2</td>
</tr>
</tbody>
</table>
Estimation of post-earthquake settlements

Abbreviations

qt: Total cone resistance (cone resistance q_c corrected for pore water effects)
Ic: Soil Behaviour Type Index
FS: Calculated Factor of Safety against liquefaction
Volumetric strain: Post-liquefaction volumetric strain
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEE Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart:

- $q_c$: tip resistance, $f_s$: sleeve friction
- $\sigma_{vo}$, $\sigma_{vo}'$: in-situ vertical total and effective stress
- units: all in kPa

**Initial Stress Exponent**
- $n = 1.0$ and calculate $Q$, $F$, and $I_c$
  - if $I_c \leq 1.64$, $n = 0.5$
  - if $1.64 < I_c < 3.30$, $n = (1 - 1.64)0.3 + 0.5$
  - if $I_c \geq 3.30$, $n = 1.0$
- Iterate until the change in $n$, $\Delta n < 0.01$
- if $\sigma_{vo}' > 300$ kPa, let $n = 1.0$ for all soils

**Equations**
- $Q = \frac{(q_c - \sigma_{vo})}{100} \cdot C_n$
- $F = \frac{f_s}{(q_c - \sigma_{vo})} \cdot 100$
- $I_c = \sqrt{(3.47 - \log Q)^2 + (1.22 + \log F)^2}$

- if $I_c \leq 1.64$, $K_c = 1.0$
- if $1.64 < I_c < 2.60$, $K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88$
- if $I_c \geq 2.60$, evaluate using other criteria; likely nonliquefiable if $F > 1%$

**BUT:** if $1.64 < I_c < 2.36$ and $F < 0.5\%$, set $K_c = 1.0$

**$K_c$ Factor**
- $(q_{cl})_{ex} = K_c Q$

**CRR$_{1,5}$**
- $CRR_{1.5} = 93 \cdot \left[\frac{(q_{cl})_{ex}}{1000}\right]^3 + 0.08$, if $50 \leq (q_{cl})_{ex} < 160$
- $CRR_{1.5} = 0.833 \cdot \left[\frac{(q_{cl})_{ex}}{1000}\right] + 0.05$, if $(q_{cl})_{ex} < 50$

if $I_c \geq 2.60$, evaluate using other criteria; likely nonliquefiable if $F > 1%$

---

1 "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart:

Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)

\[ q_c: \text{tip resistance, } f_s: \text{sleeve friction} \]
\[ \sigma_{vo}, \sigma_{vo}': \text{in situ vertical total and effective stress} \]

\[ m = 1.338 - 0.249 \times (q_{c1N})^{0.254} \]
iterate until change in \( m \), \( \Delta m < 0.01 \)

\[ C_N = \left( \frac{P_o}{\sigma_{vo}} \right)^m \leq 1.7 \]

\[ q_{c1N} = \frac{C_N \times q_c}{P_o} \]

\[ I_c < 2.60 \]
\[ I_c > 2.60 \]

\[ q_{c1Nes} = q_{c1N} + \Delta q_{c1N} \]
where:
\[ \Delta q_{c1N} = \left( 5.4 + \frac{q_{c1N}}{16} \right) \times e^{\left( 1.63 - \frac{9.7}{FC+0.01} - \frac{15.7}{FC+0.01} \right)} \]

\[ C_{R_{M-7.5,\sigma_{vo}^{-1}}} = 0.80 \times \frac{s_{ul}}{\sigma_{vo}} \times K_o \]

\[ C_{R_{M-7.5,\sigma_{vo}^{-1}}} = e^{\left( \frac{q_{c1Nes}}{540} - \frac{q_{c1Nes}}{67} \right) - \left( \frac{q_{c1Nes}}{80} \right) - \left( \frac{q_{c1Nes}}{114} \right) - 3} \]
Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)

- CPT
  - $q_t$, $f_u$, $I_t$

- $I_t < I_t$ cut-off

Initial estimate using raw tip measurements, friction ratio. Calculate $q_t$. Repeat until an acceptable convergence tolerance is achieved.

$$c = f_1 \cdot \left( \frac{R_f}{f_3} \right)^{f_2}$$

$$c_q = \left( \frac{p_a}{\sigma_v} \right)^c$$

$$q_{t,1} = c_q \cdot q_t$$

$$CRR = \exp \left[ \frac{t_{0.45} + 0.1 \cdot (0.110 \cdot R_f) + 0.001 \cdot R_f \cdot c \cdot 1.053 \cdot R_f - 0.848 \cdot h \cdot \gamma_w - 0.002 \cdot \ln\left( \sigma_v' \right) - 20.923 + 1.632 \cdot \phi \cdot \gamma_w}{7.177} \right]$$
Procedure for the evaluation of liquefaction-induced lateral spreading displacements

1 Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach

![Flowchart](image)

1 "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman
Average shear stress, $\tau_{av}$

$$\tau_{av} = CSR \cdot \sigma_0 = 0.65 \cdot \frac{q_{new}}{\sigma_0} \cdot \tau_1$$

Estimate small shear strain modulus, $G_0$

$$G_0 = 0.0188 \left[10^{0.551 \cdot 1.681}\right] (q_1 - \sigma_0)$$

Estimate shear strain amplitude, $\gamma$

(based on Pradel (1998))

$$\gamma = \left[1 + a \cdot e^{bR} \right] \cdot R \cdot 100 \text{ (\%)}$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note $\tau_{av}$ and $G_0$ same units)}$$

$$a = 0.0389 \cdot \left(\frac{\sigma_v}{P_s}\right) + 0.124$$

$$b = 6400 \left(\frac{\sigma_v}{P_s}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \left[\frac{(N_1)^{0.663}}{20}\right]^{1.20}$$

$$q_{max} = \text{Qmax} = 8.5 \cdot \left[1 - \frac{\varepsilon_c}{4.6}\right]$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, $s$

$$s = 2 \cdot \left[\frac{\Sigma_{vol}}{d_0} \cdot dz\right]$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA
Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methodology developed by Iwasaki (1982) and is adopted by AFPS.

To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

\[ \text{LPI} = \int_0^{20} (10 - 0.5z) \times F_L \times dz \]

where:
- \( F_L = 1 \) - F.S. when F.S. less than 1
- \( F_L = 0 \) when F.S. greater than 1
- \( z \) depth of measurement in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- \( \text{LPI} = 0 \) : Liquefaction risk is very low
- \( 0 < \text{LPI} \leq 5 \) : Liquefaction risk is low
- \( 5 < \text{LPI} \leq 15 \) : Liquefaction risk is high
- \( \text{LPI} > 15 \) : Liquefaction risk is very high

Graphical presentation of the LPI calculation procedure
References


- Iwasaki, T., 1986, Soil liquefaction studies in Japan: state-of-the-art, Soil Dynamics and Earthquake Engineering, Vol. 5, No. 1, 2-70


- Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT” FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, SAN diego, CA

APPENDIX F
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Appendix F Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-1.0</td>
<td>GENERAL</td>
</tr>
<tr>
<td>F-1.1</td>
<td>Intent</td>
</tr>
<tr>
<td>F-1.2</td>
<td>Role of Leighton and Associates, Inc.</td>
</tr>
<tr>
<td>F-1.3</td>
<td>The Earthwork Contractor</td>
</tr>
<tr>
<td>F-2.0</td>
<td>PREPARATION OF AREAS TO BE FILLED</td>
</tr>
<tr>
<td>F-2.1</td>
<td>Clearing and Grubbing</td>
</tr>
<tr>
<td>F-2.2</td>
<td>Processing</td>
</tr>
<tr>
<td>F-2.3</td>
<td>Overexcavation</td>
</tr>
<tr>
<td>F-2.4</td>
<td>Benching</td>
</tr>
<tr>
<td>F-2.5</td>
<td>Evaluation/Acceptance of Fill Areas</td>
</tr>
<tr>
<td>F-3.0</td>
<td>FILL MATERIAL</td>
</tr>
<tr>
<td>F-3.1</td>
<td>Fill Quality</td>
</tr>
<tr>
<td>F-3.2</td>
<td>Oversize</td>
</tr>
<tr>
<td>F-3.3</td>
<td>Import</td>
</tr>
<tr>
<td>F-4.0</td>
<td>FILL PLACEMENT AND COMPACTION</td>
</tr>
<tr>
<td>F-4.1</td>
<td>Fill Layers</td>
</tr>
<tr>
<td>F-4.2</td>
<td>Fill Moisture Conditioning</td>
</tr>
<tr>
<td>F-4.3</td>
<td>Compaction of Fill</td>
</tr>
<tr>
<td>F-4.4</td>
<td>Compaction of Fill Slopes</td>
</tr>
<tr>
<td>F-4.5</td>
<td>Compaction Testing</td>
</tr>
<tr>
<td>F-4.6</td>
<td>Compaction Test Locations</td>
</tr>
<tr>
<td>F-5.0</td>
<td>EXCAVATION</td>
</tr>
<tr>
<td>F-6.0</td>
<td>TRENCH BACKFILLS</td>
</tr>
<tr>
<td>F-6.1</td>
<td>Safety</td>
</tr>
<tr>
<td>F-6.2</td>
<td>Bedding and Backfill</td>
</tr>
<tr>
<td>F-6.3</td>
<td>Lift Thickness</td>
</tr>
</tbody>
</table>
F-1.0 GENERAL

F-1.1 Intent
These Earthwork and Grading Guide Specifications are for grading and earthwork shown on the current, approved grading plan(s) and/or indicated in the Leighton and Associates, Inc. geotechnical report(s). These Guide Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the project-specific recommendations in the geotechnical report shall supersede these Guide Specifications. Leighton and Associates, Inc. shall provide geotechnical observation and testing during earthwork and grading. Based on these observations and tests, Leighton and Associates, Inc. may provide new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

F-1.2 Role of Leighton and Associates, Inc.
Prior to commencement of earthwork and grading, Leighton and Associates, Inc. shall meet with the earthwork contractor to review the earthwork contractor’s work plan, to schedule sufficient personnel to perform the appropriate level of observation, mapping and compaction testing. During earthwork and grading, Leighton and Associates, Inc. shall observe, map, and document subsurface exposures to verify geotechnical design assumptions. If observed conditions are found to be significantly different than the interpreted assumptions during the design phase, Leighton and Associates, Inc. shall inform the owner, recommend appropriate changes in design to accommodate these observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include (1) natural ground after clearing to receiving fill but before fill is placed, (2) bottoms of all "remedial removal" areas, (3) all key bottoms, and (4) benches made on sloping ground to receive fill.

Leighton and Associates, Inc. shall observe moisture-conditioning and processing of the subgrade and fill materials, and perform relative compaction testing of fill to determine the attained relative compaction. Leighton and Associates, Inc. shall provide Daily Field Reports to the owner and the Contractor on a routine and frequent basis.

F-1.3 The Earthwork Contractor
The earthwork contractor (Contractor) shall be qualified, experienced and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor
shall review and accept the plans, geotechnical report(s), and these Guide Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

The Contractor shall inform the owner and Leighton and Associates, Inc. of changes in work schedules at least one working day in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that Leighton and Associates, Inc. is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish earthwork and grading in accordance with the applicable grading codes and agency ordinances, these Guide Specifications, and recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of Leighton and Associates, Inc., unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, Leighton and Associates, Inc. shall reject the work and may recommend to the owner that earthwork and grading be stopped until unsatisfactory condition(s) are rectified.

F-2.0 PREPARATION OF AREAS TO BE FILLED

F-2.1 Clearing and Grubbing
Vegetation, such as brush, grass, roots and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies and Leighton and Associates, Inc. Care should be taken not to encroach upon or otherwise damage native and/or historic trees designated by the Owner or appropriate agencies to remain. Pavements, flatwork or other construction should not extend under the “drip line” of designated trees to remain.

Leighton and Associates, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D 2974-00). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area. As presently defined by the State of California, most refined petroleum products
(gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

F-2.2 Processing
Existing ground that has been declared satisfactory for support of fill, by Leighton and Associates, Inc., shall be scarified to a minimum depth of 6 inches (15 cm). Existing ground that is not satisfactory shall be over-excavated as specified in the following Section C-2.3. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

F-2.3 Overexcavation
In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by Leighton and Associates, Inc. during grading. All undocumented fill soils under proposed structure footprints should be excavated.

F-2.4 Benching
Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), (>20 percent grade) the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet (4.5 m) wide and at least 2 feet (0.6 m) deep, into competent material as evaluated by Leighton and Associates, Inc. Other benches shall be excavated a minimum height of 4 feet (1.2 m) into competent material or as otherwise recommended by Leighton and Associates, Inc. Fill placed on ground sloping flatter than 5:1 (horizontal to vertical units), (<20 percent grade) shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

F-2.5 Evaluation/Acceptance of Fill Areas
All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by Leighton and Associates, Inc. as suitable to receive fill. The Contractor shall obtain a written acceptance (Daily Field Report) from Leighton and Associates, Inc. prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys and benches.
F-3.0 Fill Material

F-3.1 Fill Quality
Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by Leighton and Associates, Inc. prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to Leighton and Associates, Inc. or mixed with other soils to achieve satisfactory fill material.

F-3.2 Oversize
Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches (15 cm), shall not be buried or placed in fill unless location, materials and placement methods are specifically accepted by Leighton and Associates, Inc. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet (3 m) measured vertically from finish grade, or within 2 feet (0.61 m) of future utilities or underground construction.

F-3.3 Import
If importing of fill material is required for grading, proposed import material shall meet the requirements of Section C-3.1, and be free of hazardous materials ("contaminants") and rock larger than 3-inches (8 cm) in largest dimension. All import soils shall have an Expansion Index (EI) of 20 or less and a sulfate content no greater than (≤) 500 parts-per-million (ppm). A representative sample of a potential import source shall be given to Leighton and Associates, Inc. at least four full working days before importing begins, so that suitability of this import material can be determined and appropriate tests performed.

F-4.0 Fill Placement and Compaction

F-4.1 Fill Layers
Approved fill material shall be placed in areas prepared to receive fill, as described in Section C-2.0, above, in near-horizontal layers not exceeding 8 inches (20 cm) in loose thickness. Leighton and Associates, Inc. may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers, and only if the building officials with the appropriate jurisdiction approve. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.
F-4.2 Fill Moisture Conditioning
Fill soils shall be watered, dried back, blended and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM) Test Method D 1557-09.

F-4.3 Compaction of Fill
After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than ($\geq$) 90 percent of the maximum dry density as determined by ASTM Test Method D 1557-09. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at-least ($\geq$) 95 percent of the ASTM D 1557-09 modified Proctor laboratory maximum dry density. For fills thicker than (> ) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D 1557-09 laboratory maximum density. Compaction equipment shall be adequadly sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

F-4.4 Compaction of Fill Slopes
In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton and Associates, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D 1557-09 laboratory maximum density.

F-4.5 Compaction Testing
Field-tests for moisture content and relative compaction of the fill soils shall be performed by Leighton and Associates, Inc. Location and frequency of tests shall be at our field representative(s) discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

F-4.6 Compaction Test Locations
Leighton and Associates, Inc. shall document the approximate elevation and horizontal coordinates of each density test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton
and Associates, Inc. can determine the test locations with sufficient accuracy. Adequate grade stakes shall be provided.

**F-5.0 EXCAVATION**

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by Leighton and Associates, Inc. during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by Leighton and Associates, Inc. based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, then observed and reviewed by Leighton and Associates, Inc. prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by Leighton and Associates, Inc.

**F-6.0 TRENCH BACKFILLS**

F-6.1 Safety
The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2003 Edition or more current (see also: [http://www.dir.ca.gov/title8/sb4a6.html](http://www.dir.ca.gov/title8/sb4a6.html)).

F-6.2 Bedding and Backfill
All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2012 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, and conforming to Section 201-6 of the 2012 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D 1557-09) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall not be jetted. Jetting of the bedding around the conduits shall be observed by Leighton and Associates, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton and Associates, Inc.
F-6.3 **Lift Thickness**
Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to Leighton and Associates, Inc. that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method, and only if the building officials with the appropriate jurisdiction approve.
**FILL SLOPE**

Projected Plane 1:1 (Horizontal: Vertical) Maximum from toe of slope to approved ground.

- Existing ground surface
- 2% Min. lowest bench (key)
- 15 feet min. lowest bench (key)
- 2 feet min. key depth

**FILL-OVER-CUT SLOPE**

- Existing ground surface
- 2% Min.
- 15 feet min. lowest bench (key)
- 2 feet min. key depth
- Cut face shall be constructed prior to fill placement to allow viewing of geologic conditions

**CUT-OVER-FILL SLOPE**

- Projected plane 1 to 1 maximum from toe of slope to approved ground
- Design slope
- Overbuild and trim back
- Existing ground surface
- Bench
- Bench height (4 feet typical)
- Cut face shall be constructed prior to fill placement

**Keying and Benching**

General Earthwork and Grading Specifications Standard Details A

Leighton
**OVERSIZE ROCK DISPOSAL**

- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.

**PROFILE ALONG WINDROW**

**SECTION A-A’**

**JETTED OR FLOODED APPROVED SOIL**

**GENERAL EARTHWORK AND GRADING SPECIFICATIONS**

**STANDARD DETAILS B**
FILTER MATERIAL
FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE.
CLASS 2 GRADING AS FOLLOWS:

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
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<tr>
<td>1&quot;</td>
<td>100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>90-100</td>
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<td>No. 200</td>
<td>0-3</td>
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SUBDRAIN ALTERNATE A
- FILTER MATERIAL (9FT ³/FT)
- PERFORATED PIPE SURROUNDED WITH FILTER MATERIAL
- 6" Ø MIN.

SUBDRAIN ALTERNATE A-1
- 4" MIN. BEDDING
- 4" MIN.

SUBDRAIN ALTERNATE A-2

SUBDRAIN ALTERNATE B
- 3/4" GRAVEL WRAPPED IN FILTER FABRIC 12" MIN. OVERLAP
- FILTER FABRIC (MIRAFT 1401NC OR APPROVED EQUIVALENT)

ALTERNATE B-1
- 3/4" MAX. GRAVEL OR APPROVED EQUIVALENT (9FT ³/FT)
- PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

ALTERNATE B-2

DETAIL OF CANYON SUBDRAIN TERMINAL
- DESIGN FINISHED GRADE
- 10" MIN. BACKFILL
- FILTER FABRIC (MIRAFT 1401NC OR APPROVED EQUIVALENT)
- 3/4" OPEN GRADED GRAVEL OR APPROVED EQUIVALENT

NON-PERFORATED 6" Ø MIN.

15" MIN.

3/4" OPEN GRADED GRAVEL OR APPROVED EQUIVALENT
• **SUBDRAIN INSTALLATION** - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.

• **SUBDRAIN PIPE** - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.

• All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.
OVERBURDEN OR UNSUITABLE MATERIAL

CUT-FILL TRANSITION LOT OVEREXCAVATION

- REMOVE UNSUITABLE GROUND
- OVEREXCAVATE AND RECOMPACT
- UNWEATHERED BEDROCK OR MATERIAL APPROVED BY THE GEOTECHNICAL CONSULTANT

SIDE HILL FILL FOR CUT PAD

- NATURAL GROUND
- RESTRICTED USE AREA
- OVEREXCAVATE AND RECOMPACT (REPLACEMENT FILL)
- OVERBURDEN OR UNSUITABLE MATERIAL
- 2' MIN. KEY DEPTH
- 3% MIN.
- 9' MIN.
- TYPICAL BENCHING
- SEE STANDARD DETAIL FOR SUBDRAINS WHEN REQUIRED BY GEOTECHNICAL CONSULTANT
- UNWEATHERED BEDROCK OR MATERIAL APPROVED BY THE GEOTECHNICAL CONSULTANT

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAILS E
APPROXIMATE LOCATION OF HOLLOW STEM BORING
BY LEIGHTON & ASSOCIATES, 2007a and b

APPROXIMATE LOCATION OF GEOTECHNICAL CPT
BY LEIGHTON & ASSOCIATES, 2007a

APPROXIMATE LOCATION OF HAND AUGER BORING
PERFORMED BY LEIGHTON & ASSOCIATES, THIS INVESTIGATION

APPROXIMATE LOCATION OF MONITORING WELL
INSTALLED BY LEIGHTON & ASSOCIATES, 2007c

APPROXIMATE LOCATION OF HAND AUGER BORING
PERFORMED BY LEIGHTON & ASSOCIATES, 2007b

ARTIFICIAL FILL, UNDOCUMENTED
QUATERNARY ALLUVIAL DEPOSITS, SAND, SILT, AND CLAY WITH GRAVEL, UNCONSOLIDATED, CIRCLED WHERE BURIED
QUATERNARY SAN PEDRO FORMATION, FINE GRAINED, DENSE SAND WITH OCCASIONAL GRAVEL. CAPPED WITH SILT AND CLAY, SEE FIGURE 2 FOR SUBSURFACE DISTRIBUTION
GEOLOGIC CROSS SECTION, SEE FIGURE 2


QUATERNARY TERRACE DEPOSITS, IRON OXIDE STAINED SAND, SILT AND SOIL, CIRCLED WHERE BURIED
GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
GEOLOGIC CROSS SECTION, SEE FIGURE 2

ALL EXCAVATIONS SHOWN WITH TOTAL DEPTH (T.D.), DEPTH OF EARTH UNITS AND DEPTH TO GROUNDWATER (G.W.) WHERE APPLICABLE IN FEET BELOW EXISTING GROUND SURFACE.