Geology-Based Probabilistic Liquefaction Potential Mapping of the 7.5-Minute Charleston Quadrangle, South Carolina for Resilient Infrastructure Design

Lawrence Simonson
Clemson University, lasimon@clemson.edu

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GEOLOGY-BASED PROBABILISTIC LIQUEFACTION POTENTIAL MAPPING OF THE 7.5-MINUTE CHARLESTON QUADRANGLE, SOUTH CAROLINA FOR RESILIENT INFRASTRUCTURE DESIGN

A Thesis
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
Of the Requirements for the Degree
Master of Science
Civil Engineering

by
Lawrence A. Simonson
May 2012

Accepted by:
Dr. Ronald D. Andrus, Committee Chair
Dr. C. Hsein Juang
Dr. Wei Chiang Pang
This research was supported by the National Science Foundation, under grant number NSF-1011478. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the author and do not necessarily reflect the views of the National Science Foundation.
ABSTRACT

Two geology-based probabilistic liquefaction potential maps are developed for the 7.5-minute Charleston, South Carolina quadrangle in this thesis. Creation of the maps extends the previous liquefaction potential mapping work of the Charleston peninsula by Hayati and Andrus (2008) and Mount Pleasant by Heidari and Andrus (2010), and improves upon the previous maps by using peak ground accelerations that vary with local site conditions. The GIS software package ArcGIS 10 is used to develop the maps.

Development of the liquefaction potential maps involves the creation of four additional maps needed as inputs. The four additional maps are (1) depth to the top of the Tertiary-age Cooper Marl (d_{Marl}); (2) average shear wave velocity in the top 30 m (V_{S30}); soft-rock peak ground surface acceleration (PGA_{B-C}) for about a 500-year return period; and site-adjusted peak ground surface acceleration (PGA_{Site}). The d_{Marl} map is created using the elevation contour maps by Weema and Lemon (1993) and Fairbanks et al. (2008), and topographic information from the South Carolina Department of Natural Resources GIS Data Clearinghouse. Values of d_{Marl} range from 2 m to 24 m.

The V_{S30} map is created by combining the d_{Marl} map, the surficial geology map by Weems et al. (2011) and the average shear wave velocity values reported in Andrus et al. (2006), and assuming a simple two layer model. The initial V_{S30} map is refined locally with calculated V_{S30} values from specific test sites. Mapped values of V_{S30} range from \leq 140 m/s to 350 m/s. These V_{S30} values correspond to seismic Site Classes D and E, assuming no special Site Class F conditions.

The V_{S30} map is used with site factors derived by Aboye et al. (2011, 2012) to adjust the map of PGA_{B-C} created from the 2008 USGS National Seismic Hazard Map.
The resulting map of peak ground surface acceleration adjusted for site conditions (\(PGA_{\text{Site}}\)) consists of values 15 to 50% higher than values of \(PGA_{B-C}\).

Liquefaction potential for the area is expressed in terms of the liquefaction potential index (\(LPI\)) and calculated using relationships by Heidari (2011). These relationships correlate the probability of \(LPI \geq 5\) (\(P_{LPI>5}\)) with the ratio \(PGA_{\text{Site}}/MSF\) (where \(MSF\) is the magnitude scaling factor), depth to groundwater table (\(GWT\)), and \(d_{\text{Marl}}\). To match relationships by Heidari (2011), the geology of the area is grouped into the categories of artificial fill, younger natural sediments, and the Wando Formation.

An analysis of \(GWT\) for the quadrangle is conducted using data from Fairbanks et al. (2004) and Mohanan et al. (2006). From this data, a conservative estimate of 1.0 m is initially used for the \(GWT\) depth for all areas.

The first liquefaction potential map is based on a moment magnitude (\(M_W\)) = 7.3 and \(GWT = 1.0\) m for all areas. The second liquefaction potential map is based on \(M_W = 6.9\), \(GWT = 2.0\) m for the Wando Formation, and \(GWT = 1.0\) m for areas covered by younger materials. Liquefaction potential values for the \(M_W = 7.3\) map are too high when compared to field performance during the 1886 earthquake. Values of liquefaction potential for the \(M_W = 6.9\) map coincide more closely with observed field behavior and previous maps for the Charleston peninsula and Mount Pleasant. The highest risk of liquefaction on both maps is found to be in areas with the largest \(d_{\text{Marl}}\) depths and covered with artificial fill and the younger natural sediments.

A potential use for the liquefaction potential maps is discussed with respect to the resiliency of the roadway and bridge infrastructure of Charleston. All bridges in the quadrangle have abutments located on areas with \(P_{LPI>5} = 60 – 100\%\). The roads with highest risk of liquefaction-induced (i.e., areas of \(P_{LPI>5} = 80 – 100\%\)) are located on the
Southern end of the peninsula, James Island, and western Mount Pleasant. General recommendations are given for improving the resiliency of bridge infrastructure by taking preventative measures with existing and future structures, and by insuring that inspection and repair of damaged bridge structures take place in a timely manner after an earthquake. The liquefaction potential maps can be used to prioritize areas to be inspected following the next strong earthquake event.
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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TITLE PAGE</td>
<td>i</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>iii</td>
</tr>
<tr>
<td>ACKNOWLEDGMENTS</td>
<td>vi</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>x</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>xi</td>
</tr>
<tr>
<td>CHAPTER</td>
<td></td>
</tr>
<tr>
<td>1. INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Background and Purpose of Research</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Scope and Objectives</td>
<td>3</td>
</tr>
<tr>
<td>1.3 Organization</td>
<td>4</td>
</tr>
<tr>
<td>2. AVERAGE SHEAR-WAVE VELOCITY AND PEAK GROUND ACCELERATION MAPS</td>
<td>5</td>
</tr>
<tr>
<td>2.1 Introduction</td>
<td>5</td>
</tr>
<tr>
<td>2.2 Software</td>
<td>5</td>
</tr>
<tr>
<td>2.3 Surficial Geology</td>
<td>6</td>
</tr>
<tr>
<td>2.4 Depth to Cooper Marl</td>
<td>8</td>
</tr>
<tr>
<td>2.5 Average Shear Wave Velocity for Top 30 m</td>
<td>9</td>
</tr>
<tr>
<td>2.6 Peak Ground Surface Acceleration</td>
<td>11</td>
</tr>
<tr>
<td>2.7 Summary</td>
<td>13</td>
</tr>
</tbody>
</table>
### Table of Contents (Continued)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3. LIQUEFACTION POTENTIAL MAPS</td>
<td>26</td>
</tr>
<tr>
<td>3.1 Introduction</td>
<td>26</td>
</tr>
<tr>
<td>3.2 Liquefaction Potential Probability Relationships</td>
<td>26</td>
</tr>
<tr>
<td>3.3 Groundwater Table</td>
<td>30</td>
</tr>
<tr>
<td>3.4 Liquefaction Potential Maps</td>
<td>31</td>
</tr>
<tr>
<td>3.5 Summary</td>
<td>34</td>
</tr>
<tr>
<td>4. IMPROVING RESILIENCY OF CHARLESTON'S CIVIL INFRASTRUCTURE</td>
<td>46</td>
</tr>
<tr>
<td>4.1 Introduction</td>
<td>46</td>
</tr>
<tr>
<td>4.2 Roadway Liquefaction Evaluation and Hazard Evaluation</td>
<td>46</td>
</tr>
<tr>
<td>4.3 Improving Resilience of Bridges in the Charleston Area</td>
<td>47</td>
</tr>
<tr>
<td>4.4 Summary</td>
<td>52</td>
</tr>
<tr>
<td>5. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS</td>
<td>55</td>
</tr>
<tr>
<td>5.1 Average Shear-Wave Velocity and Peak Ground Acceleration Maps</td>
<td>55</td>
</tr>
<tr>
<td>5.2 Liquefaction Potential Maps</td>
<td>56</td>
</tr>
<tr>
<td>5.3 Application of Liquefaction Potential Maps for Improving Resilience</td>
<td>57</td>
</tr>
<tr>
<td>5.4 Limitations</td>
<td>57</td>
</tr>
<tr>
<td>5.5 Recommendations for Future Work</td>
<td>58</td>
</tr>
<tr>
<td>REFERENCES</td>
<td>60</td>
</tr>
</tbody>
</table>
## LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Description of geologic units (adapted from Weems and Lemon 1993; Weems et al. 2011)</td>
<td>16</td>
</tr>
<tr>
<td>2.2</td>
<td>NEHRP site classes based on $V_{S30}$ (BSSC 1995)</td>
<td>20</td>
</tr>
<tr>
<td>2.3</td>
<td>Relationships by Aboye et al. (2011, 2012) for estimating $F_{PGA}$</td>
<td>24</td>
</tr>
<tr>
<td>3.1</td>
<td>$P_{L_{Pb-5}}$ curve coefficients used for the Wando Formation (Heidari 2011)</td>
<td>37</td>
</tr>
<tr>
<td>3.2</td>
<td>$P_{L_{Pb-5}}$ curve coefficients used for younger natural sediments (Heidari 2011)</td>
<td>37</td>
</tr>
<tr>
<td>3.3</td>
<td>$P_{L_{Pb-5}}$ curve coefficients for type III artificial fills (Heidari 2011)</td>
<td>37</td>
</tr>
</tbody>
</table>
## LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Flowchart showing methodology used for developing the liquefaction potential maps</td>
<td>14</td>
</tr>
<tr>
<td>2.2</td>
<td>Geologic map of the Charleston quadrangle (modified from Weems and Lemon 1993; Weems et al. 2011)</td>
<td>15</td>
</tr>
<tr>
<td>2.3</td>
<td>Map of the Charleston quadrangle showing depth to top of Cooper Marl (modified from Weems and Lemon 1993; Fairbanks et al. 2008)</td>
<td>17</td>
</tr>
<tr>
<td>2.4</td>
<td>7.5-minute Charleston quadrangle DEM obtained from the South Carolina Department of Natural Resources GIS Data Clearinghouse (<a href="https://www.dnr.sc.gov/GIS/gisdownload.html">https://www.dnr.sc.gov/GIS/gisdownload.html</a>)</td>
<td>18</td>
</tr>
<tr>
<td>2.5</td>
<td>Elevation contour map of the top of Marl beneath Charleston peninsula by Fairbanks et al. (2008)</td>
<td>19</td>
</tr>
<tr>
<td>2.6</td>
<td>Map of the Charleston quadrangle showing $V_{S30}$</td>
<td>21</td>
</tr>
<tr>
<td>2.7</td>
<td>Map of the Charleston quadrangle showing $PGA_{B-C}$ for 475-year return period based on the 2008 USGS National Seismic Hazard Maps (<a href="http://geohazards.usgs.gov/">http://geohazards.usgs.gov/</a>)</td>
<td>22</td>
</tr>
<tr>
<td>2.8</td>
<td>Site factors for adjusting $PGA_{B-C}$ in Charleston (modified from Aboye et al. 2011, 2012)</td>
<td>23</td>
</tr>
<tr>
<td>2.9</td>
<td>Map of the Charleston quadrangle showing $PGA_{Site}$ for 475-year return period</td>
<td>25</td>
</tr>
<tr>
<td>3.1</td>
<td>Flowchart showing methodology used for developing the liquefaction potential maps</td>
<td>36</td>
</tr>
<tr>
<td>3.2</td>
<td>Distribution of groundwater table depths measured at CPT sites grouped by geology</td>
<td>38</td>
</tr>
<tr>
<td>3.4</td>
<td>Liquefaction potential map of the Charleston quadrangle for a 475-year return period accelerations and assuming $M_W=7.3$ and $GWT=1.0$ m for all areas</td>
<td>40</td>
</tr>
</tbody>
</table>
3.5 Geology map of the Charleston Peninsula and Drum Island with locations of 1886 liquefaction and ground deformation sites (Hayati and Andrus 2008). .............................................................................41

3.6 Geology map of Mount Pleasant with locations of 1886 liquefaction and ground deformation (Heidari and Andrus 2010). .........................................................42

3.7 Liquefaction potential map of the Charleston quadrangle for a 475-year return period accelerations and assuming $M_w=6.9$ and $GWT=2.0$ m for the Wando and $1.0$ m for all other areas ..............................................................................................43

3.8 Liquefaction potential map of Charleston Peninsula and Drum Island by Hayati and Andrus (2008) based on 1886 field performance and diagenetic-corrected $LPI$ for $M_w=7.1$ and $PGA_{Site}=0.3$ g. .................................................44

3.9 Liquefaction potential map of Mount Pleasant by Heidari and Andrus (2010) based on 1886 field performance and diagenetic-corrected $LPI$ for $M_w=6.9$ and $PGA_{Site}=0.25$ g. .................................................45

4.1 Liquefaction potential map of the Charleston quadrangle for 475-year-return-period accelerations and $M_w=6.9$, with roadways maintained by SCDOT (dbw.scdot.org/GISMapping/default.aspx). .................................................................53

4.2 Typical performance response curve of an infrastructure system following a disruptive event, as illustrated by Ouyang et al. (2012). .......................54
CHAPTER 1: INTRODUCTION

1.1 Background and Purpose of Research

Earthquake-induced ground liquefaction is the process by which loose, saturated granular soil loses shear strength due to collapse of the soil structure and pore pressure buildup. As pore-water pressure approaches the total overburden stress, the effective stress approaches zero and the soil begins behaving like a fluid. The loss of strength results in ground failures such as lateral spreads, flow failure, ground oscillation, and a decrease of bearing capacity. Water damage can also occur due to ejected sand and water flows into sublevels of structures (Coduto 2009).

The Charleston, South Carolina earthquake of 1886, with moment magnitude ($M_w$) of about 6.9, demonstrates the damage that can be caused in the study area by ground liquefaction (Dutton 1889; Robinson and Talwani 1983; Hayati and Andrus 2008; Hedari and Andrus 2010). This earthquake occurred on August 31, and is considered the most damaging historic earthquake to occur in the southeast United States (Bollinger 1977), causing 124 deaths and approximately $460$ million (2006 dollars) in damage (Côté 2006). A prominent source of the damage was from ground failure caused by the liquefaction. Over 40 different cases of liquefaction-induced ground failure during the 1886 earthquake have been identified in Charleston and Mount Pleasant (Hayati and Andrus 2008; Hedari and Andrus 2010).

Based on paleoliquefaction studies, Talwani and Gassman (2008) estimated that similar size earthquakes occur in the Charleston area every 500 years. Because of population growth, it is believed a future earthquake of similar magnitude to the event in
1886 would cause significantly more damage and loss of life, with a predicted 900 deaths, 44,000 injuries, and $20 billion worth of economic loss for South Carolina (Wong et al. 2005).

Four general types of maps have been used to express the liquefaction hazard of a region. As explained by Power and Holzer (1996), these maps include: (1) historic liquefaction maps, which identify areas where liquefaction has occurred and is likely to occur again; (2) liquefaction susceptibility maps, which inform the user on vulnerability to liquefaction based on geology, historic information, depth to groundwater table, and material properties; (3) liquefaction-induced ground failure maps, which show estimated ground displacements; and (4) liquefaction potential maps, which show the opportunity for liquefaction triggering given a scenario earthquake or an exposure time period.

Efforts to map the liquefaction potential in Charleston have been conducted by several researchers. The first map was developed using 67 standard penetration test (SPT) borings on the Charleston peninsula by Elton and Hadj-Hamou (1990). Balon and Andrus (2006) predicted moderate to very high potentials for much of the Charleston and Mount Pleasant quadrangles based on 87 cone penetration tests (CPT) performed in the greater Charleston area. Medium to high potential for the study area was also predicted by Juang and Li (2007) through the use of 28 CPT’s, many of which were also used by Balon and Andrus (2006). These initial studies, however, were limited as they did not take into account the instances of historic liquefaction, the geology, or the influence of soil age on liquefaction resistance. Hayati and Andrus (2008) and Heidari and Andrus (2010) noted these limitations in their liquefaction potential mapping efforts of the southern half of the Charleston peninsula and the western half of Mount Pleasant, respectively.
This thesis expands and improves the mapping efforts of Hayati and Andrus (2008) and Heidari and Andrus (2010) to the entire 7.5-minute Charleston quadrangle. One improvement involves estimating site peak ground surface acceleration based on soil stiffness. Another improvement involves considering the depth to bottom of the Quaternary (or depth to top of the Tertiary).

1.2 Scope and Objectives

The scope of the research presented in this thesis is the creation of a geology-based probabilistic liquefaction potential map of the 7.5-minute Charleston quadrangle and a brief discussion of how the map might improve resiliency of civil infrastructure. The specific objectives are as follows:

1. To create liquefaction potential maps for a 10% probability of exceedence of ground acceleration in 50 years (i.e. about 475-year return period);

2. To create maps of (1) the depth to top of the non-liquefiable Tertiary-age Cooper Marl; (2) the shear wave velocity for the top 30 m ($V_{S30}$); and (3) the site adjusted peak ground acceleration ($PGA_{site}$), which are needed inputs for creating the liquefaction potential maps;

3. Compare the roughly 475-year-return-period liquefaction potential maps with field performance during the 1886 earthquake; and

4. Briefly discuss how the liquefaction potential maps can be used to evaluate resilience of civil infrastructure in Charleston.
1.3 Organization

This thesis is organized into 4 chapters. Following this introduction, the development of maps of average shear wave velocity and site peak ground surface acceleration is presented in Chapter 2. Presented in Chapter 3 is the development of the liquefaction potential maps. Presented in Chapter 4 is a brief discussion of how the maps can be used to evaluate resilience of infrastructure in Charleston. Major conclusions are summarized in Chapter 5.
CHAPTER 2:

AVERAGE SHEAR-WAVE VELOCITY AND PEAK GROUND ACCELERATION MAPS

2.1 Introduction

Average shear wave velocity in the top 30 m ($V_{S30}$) and site peak ground surface acceleration ($PGA_{Site}$) maps of the 7.5-minute Charleston quadrangle are developed in this chapter. Presented in Figure 2.1, above the dashed line, is a visual representation of the inputs needed to develop the maps. The development of the $V_{S30}$ map involves combining a depth to the top of the Tertiary-age Cooper Marl map, a surficial geology map, and knowledge of the average shear-wave velocity for each geologic unit. The development of the $PGA_{Site}$ map involves combining the $V_{S30}$ map, a map of soft-rock peak ground acceleration for a 475-year return period, and peak ground acceleration site factors to adjust to local site conditions.

2.2 Software

The software used for creating the maps is the ESRI ArcGIS 10 package (http://www.esri.com/software/arcgis/arcgis10/index.html), with most of the data processing performed by the ArcMap program of the package. ArcMap allows the user to view and manipulate the many map layers necessary for this project and to perform all calculations between layers. ArcGIS software has been used in other studies to map liquefaction of large areas, including Santa Clara Valley, California (Holzer et al. 2009), Edessa, Northern Greece (Papathanassiou and Valkaniotis 2009), and Saint Louis, Missouri (Jae-Won and Rogers 2011).
Layers brought into ArcGIS can be of two different data model types: vector or raster. Layers in vector format use x and y coordinates to define specific locations in explicit relation to other features. Vector format is useful for capturing and storing spatial details. Layers in raster format, on the other hand, use a matrix of square areas (or cells) to define where features in the layer are located. Each cell contains its own specific information, but cannot capture and store the amount of detail a layer in vector format can. However, rasters are useful for viewing continuous data that change across a landscape or surface, such as elevation or slope (Bolstad 2008).

For this study, a raster analysis is performed. This involves bringing in all layers, both vector and raster formats, and converting them into grid rasters with the same cell size (30 m x 30 m) then snapping the layers so that the cells of each layer lie directly on top of one another. As most of the layers are in the NAD 1927 Datum in UTM Zone 17N coordinate system, this coordinate system is used for the entire analysis. Once all layers were converted into grid rasters, calculations can be performed to determine $V_{S30}$, $PGA_{Site}$, and liquefaction potential for the area.

2.3 Surficial Geology

Presented in Figure 2.2 is a surficial geology map of the 7.5-minute Charleston quadrangle. This geology map is taken from a 1:100,000 scale digital map of Charleston and parts of Berkeley, Dorchester, Colleton, and Georgetown Counties, as developed by the U.S. Geological Survey (Weems and Lemon 1993; Weems et al. 2011). The original digital geology map layer was in vector form as a polygon shapefile with a NAD 1983 Datum in UTM Zone 17N. The digital map is converted to raster form and clipped to include only the area within the 7.5-minute Charleston quadrangle.
As can be seen in Figure 2.2, the area within the quadrangle is split into several regions by (from west to east) the Ashley River, the Cooper River, and the Wando River. The three rivers converge in the southwest region of the quadrangle to form the Charleston Harbor. West of the Ashley River is West Ashley and James Island. Between the Ashley River and the Cooper River is North Charleston and the Charleston peninsula. The Charleston peninsula is the location of downtown Charleston, which includes the historic district. Between the Cooper and Wando Rivers is Daniel Island, which is currently the least developed area in the quadrangle. East of the Wando River is Mount Pleasant.

The age and material description of major geologic units within the quadrangle are summarized in Table 2.1. The youngest materials are artificial fill (af), most of which were placed after the 1886 earthquake (Heidari 2011) and includes fill for roads, dams, and other construction (Weems et al. 2011). Approximately half of the Charleston peninsula is made up of artificial fill, particularly adjacent to the shoreline. The youngest natural material is Qht, with an age of <5,000 years. Qht is found abundantly next to the rivers and harbor. Sediments that are part of the Silver Bluff terrace (Qsbc and Qsbs) flank the higher ground and range from 6,000 to 85,000 years old. The oldest deposits found at the ground surface are sediments of the Wando Formation (Qws, Qwc, and Qwls), ranging from 70,000 to 130,000 years old. The thickness and material properties of each unit have a significant impact on the $V_{S30}$, $PGA_{site}$, and liquefaction potential maps.
2.4 Depth to Cooper Marl

Underlying the entire area shown in Figure 2.2 is the Ashley Formation of the Cooper Group, locally known as the Cooper Marl. Within the Charleston quadrangle, the Cooper Marl lies between 2 and 25 m beneath the ground surface (Weems and Lemon 1993). The Cooper Marl is a well compacted calcarenite which consists of silty clay to clayey silt. Generally, the Marl is considered to be nonsusceptible to liquefaction due to its material properties and behavior when excavated (Li et al. 2007; Hayati and Andrus 2008). Because the Marl will not liquefy the depth to Marl is assumed to be the limiting depth for liquefaction.

Figure 2.3 shows the depth to top of Marl below the ground surface. This map is constructed primarily using elevation contours of the top of Marl from the 1:24,000 scale map of “Geology of the Cainhoy, Charleston, Fort Moultrie, and North Charleston Quadrangles, Charleston and Berkeley Counties, South Carolina” by Weems and Lemon (1993) and topographic information from a 7.5-minute digital elevation model (DEM) layer obtained from the South Carolina Department of Natural Resources GIS Data Clearinghouse (https://www.dnr.sc.gov/GIS/gisdownload.html). The DEM is presented in Figure 2.4. The elevation contours of the top of Marl beneath the Charleston peninsula are refined from Weems and Lemon (1993) using the map by Fairbanks et al. (2008) shown in Figure 2.5.

Analysis of the depth to top of Marl map in Figure 2.3 reveals several features. The Marl is at its shallowest in North Charleston and the northern section of Daniel Island, coming within 2 m of the ground surface at some points. Because the Marl is non-liquefiable, these areas likely have the lowest liquefaction potential. The deepest
mapped sections of the Marl follow the Cooper River downstream, with one branch cutting across the peninsula. The deeper sections of the Marl on land (about 24 m) are at the lower end of the Charleston peninsula and the southwestern tip of Mount Pleasant. The deepest sections of the Cooper Marl demonstrate the path of surface water movement at some time prior to the deposition of the Wando Formation.

2.5 Average Shear Wave Velocity for Top 30 m

Small strain shear wave velocity \( (V_s) \) and thickness of the near surface geology have been shown to be controlling factors for site response during seismic events (Kramer 1996). \( V_{S30} \) is often used as a proxy variable for the \( V_s \) profile. \( V_{S30} \) is defined as (Borcherdt 1994):

\[
V_{S30} = \frac{30}{\sum_{i=1}^{n} \frac{H_i}{V_{Si}}} 
\]

where \( H_i \) is the thickness in meters of layer \( i \), \( V_{Si} \) is the shear wave velocity in m/s of layer \( i \), and \( n \) is the number of layers in the top 30 m.

Values of \( V_{S30} \) were used by the National Hazard Reduction Program (NEHRP) to define seismic site class (BSSC 1995). Summarized in Table 2.2 are the values of \( V_{S30} \) corresponding to NEHRP defined site classes.

For this study, Equation 2.1 is simplified to represent a two layered system. The first layer is represented by the Quaternary deposits; and the second layer is represented by the Marl. The average \( V_s \) value of the Marl is 390 m/s based on seismic
cone penetration test (SCPT) $V_S$ profiles analyzed by Andrus et al. (2006). Thus, the simplified $V_{S30}$ equation is defined as:

$$V_{S30} = \frac{30}{d_{Marl}} + \frac{30 - d_{Marl}}{V_{S(Quaternary)}} 2.2$$

where $d_{Marl}$ is the depth to top of Marl at a given location, and $V_{S(Quaternary)}$ is the average shear wave velocity for the Quaternary sediments above the Marl. The average $V_{S(Quaternary)}$ values above the Marl are based on the statistics presented in Andrus et al. (2006): 140 m/s for af; 110 m/s for younger natural sediments (including Holocene aged soils and Silver Bluff terrace units), and 190 m/s for units of the Wando Formation. It was assumed that the surficial Quaternary units extended to the Marl. Using Equation 2.2 and the respective values for $V_{S(Quaternary)}$, an initial map of $V_{S30}$ was created.

The initial $V_{S30}$ map was adjusted locally using $V_{S30}$ values from SCPT profiles reported by Fairbanks et al. (2004). Differences between the initial $V_{S30}$ map and the SCPT profile values are most likely due to the assumption made in the two-layered system calculations that the surficial geology units extend the entire depth above the top of Marl. $V_{S30}$ values are modified by averaging values reported by Fairbanks et al. (2004) based on geology and location. These $V_{S30}$ modified values include: 230 m/s for $Q_{sbs}$ and $Q_{sbc}$; 180 m/s for $Q_{ht}$; 147 m/s for af on Drum Island; and 184 m/s for af in the remainder of the quadrangle. The final $V_{S30}$ map is presented in Figure 2.6.

Several observations can be made from the final $V_{S30}$ map presented in Figure 2.6. First, the areas with the lowest $V_{S30} \leq 180$ m/s are located predominately in low-lying areas bordered by water, particularly where $Q_{ht}$ is present. Artificial fill on Drum Island also has a $V_{S30} \leq 180$ m/s. Most of the other areas of artificial fill have a $V_{S30}$
values between 180 to 240 m/s. Silver Bluff terrace areas exhibit $V_{s30}$ between 240 to 340 m/s. Areas of Qwc and Qwls have $V_{s30}$ between 240 to 360 m/s, whereas areas of Qws vary from 180 to 360 m/s. From the map, $V_{s30}$ is highest (300 to 360 m/s) where the top of the Marl is at relative shallow depths (2 to 10 m). This is logical, as average $V_s$ of the Marl is 390 m/s, and would therefore result in the highest $V_{s30}$ when close to the surface.

The map of $V_{s30}$ in Figure 2.6 is useful for several purposes. Within the context of this study, the $V_{s30}$ map can be used to adjust the soft-rock peak ground acceleration ($PGA_{B-C}$) to account for site conditions. Figure 2.6 can also be used to determine NEHRP site classification. The entire site consists of Site Classes E and D, assuming no special Site Class F conditions.

### 2.6 Peak Ground Surface Acceleration

A map of $PGA_{B-C}$ is created for a 475-year return period (i.e. 10% probability of $PGA_{B-C}$ exceedence in 50 years) using predicted values from the 2008 USGS National Seismic Hazard Maps (http://geohazards.usgs.gov/). $PGA_{B-C}$ corresponds to the peak ground acceleration at sites where $V_{s30}$ is equal to 760 m/s, or the B-C boundary. Within ArcGIS the selected $PGA_{B-C}$ values are interpolated using a 12-point ordinary kriging interpolation with linear drift assuming earthquake motions to be constant parallel to the source zone assumed by USGS. The final $PGA_{B-C}$ map is shown in Figure 2.7 with a range of 0.168 g to 0.140 g, decreasing as one moves from the northwest corner to the southeast corner.
Because surficial geology can significantly affect the amplitude of seismic motion felt at the ground surface (Kramer 1996), a site factor is needed to adjust $PGA_{B-C}$ to the local site conditions. This adjustment can be made as follows:

$$PGA_{Site} = F_{PGA} \times PGA_{B-C}$$  \hspace{1cm} (2.3)

where $F_{PGA}$ is the peak ground surface acceleration site factor.

Presented in Figure 2.8 are the relationships between $F_{PGA}$, $PGA_{B-C}$, and $V_{S30}$ based on the modeling study of Aboyé et al. (2011, 2012) for conditions in Charleston. For $PGA_{B-C} = 0.1 \text{ g}$ and $0.2 \text{ g}$, these relationships are defined by the linear equations given in Table 2.3. Because all values of $PGA_{B-C}$ shown in Figure 2.7 fall between $0.1 \text{ g}$ and $0.2 \text{ g}$, values for $F_{PGA}$ are interpolated using the following equation:

$$F_{PGA} = F_{PGA,0.2} + (F_{PGA,0.1} - F_{PGA,0.2}) \left[ \frac{0.2 - PGA_{B-C}}{0.1} \right]$$  \hspace{1cm} (2.4)

where $F_{PGA,0.2}$ is the value of $F_{PGA}$ for a given $V_{S30}$ along the $PGA_{B-C} = 0.2 \text{ g}$ curve, and $F_{PGA,0.1}$ is the value of $F_{PGA}$ for a given $V_{S30}$ along the $PGA_{B-C} = 0.1 \text{ g}$ curve.

Presented in Figure 2.9 is the resulting map of $PGA_{Site}$. The entire area exhibits peak ground acceleration 15 to 50% higher than $PGA_{B-C}$. The lowest $PGA_{Site}$ values, between 0.16 and 0.18 g, are located on Drum Island because of very low values of $V_{S30}$. The highest $PGA_{Site}$ values, between 0.26 and 0.28 g, are located at the northwest corner of the map in North Charleston where $V_{S30}$ values are between 240 to 300 m/s. The rest of the mapped area has a $PGA_{Site}$ between 0.20 and 0.26 g. These values are slightly lower than the values of 0.3 g assumed by Hayati and Andrus (2008) for the
Charleston peninsula, and 0.25 g assumed by Heidari and Andrus (2010) for Mount Pleasant.

2.7 Summary

Discussed in this chapter are the GIS software package used, and the layers and steps taken to create a $V_{S30}$ map and a $PGA_{Site}$ map. All calculations performed in the analysis were raster calculations executed using ArcGIS. The coordinate system used was the NAD 1927 Datum in UTM Zone 17N. All raster layers created are snapped to the same coordinates and consist of grids with 30 m x 30 m area cells.

Surficial soils within Charleston quadrangle range from the relatively young (<300 years) artificial fill to the 100,000-year-old material of the Wando Formation. Underlying the entire area is the Cooper Marl. The Marl is generally considered non-liquefiable.

A two layered system was assumed to estimate $V_{S30}$. The two layers were the surficial Quaternary geology and the Marl. Mean shear wave velocity values reported by Andrus et al. (2006) were assumed for the Quaternary deposits and the Marl. Computed values of $V_{S30}$ were adjusted locally to match $V_{S30}$ values for SCPT profiles compiled by Fairbanks et al (2004). Values of $V_{S30}$ range from 143 m/s to 352 m/s. The final $V_{S30}$ map was used to adjust the B-C boundary peak ground acceleration for site specific conditions.

A map of $PGA_{B-C}$ was created using the 2008 USGS National Seismic Hazard Maps for a 475-year return period. Because the values of $PGA_{B-C}$ are for the B-C condition (i.e., $V_{S30} = 760$ m/s), a site adjustment was required. Site factors provided by Aboye et al. (2011, 2012) were used to obtain the $PGA_{Site}$ map. Values of $PGA_{Site}$ were as much as 50% higher than $PGA_{B-C}$.
Figure 2.1: Flowchart showing methodology used for developing the liquefaction potential maps.
Figure 2.2: Geologic map of the Charleston quadrangle (modified from Weems and Lemon 1993; Weems et al. 2011).
Table 2.1: Description of geologic units (adapted from Weems and Lemon 1993; Weems et al. 2011).

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Name</th>
<th>Age (years)</th>
<th>Soil type</th>
</tr>
</thead>
<tbody>
<tr>
<td>af</td>
<td>Artificial fill</td>
<td>&lt;300</td>
<td>Sand to clayey sand</td>
</tr>
<tr>
<td>Qht</td>
<td>Holocene tidal-marsh deposit</td>
<td>&lt;5 k</td>
<td>Clayey sand to clay, organic rich</td>
</tr>
<tr>
<td>Qhs</td>
<td>Holocene beach and barrier-island sands</td>
<td>&lt;10 k</td>
<td>Fine-grained quartz sand</td>
</tr>
<tr>
<td>Qal</td>
<td>Holocene alluvium</td>
<td>&lt;12 k</td>
<td>Sand and clayey sand, muck and peat veneer at surface</td>
</tr>
<tr>
<td>Qhm</td>
<td>Holocene to upper Pleistocene freshwater swamp deposit</td>
<td>&lt; 70 k</td>
<td>Muck and peat, organic rich</td>
</tr>
<tr>
<td>Qsbc</td>
<td>Silver Bluff terrace Holocene to Pleistocene estuarine deposit (previously Qhec)</td>
<td>6 – 85 k</td>
<td>Silty to sandy clay, quartz sand</td>
</tr>
<tr>
<td>Qsbs</td>
<td>Silver Bluff terrace Pleistocene beach deposit (previously Qhes)</td>
<td>33 – 85 k</td>
<td>Fine-grained, well-sorted quartz sand</td>
</tr>
<tr>
<td>Qws</td>
<td>Wando Formation barrier-island upper sand facies</td>
<td>70 – 130 k</td>
<td>Fine-grained quartz sand</td>
</tr>
<tr>
<td>Qwc</td>
<td>Wando Formation estuarine to fluvial facies</td>
<td>70 – 130 k</td>
<td>Clayey sand to clay</td>
</tr>
<tr>
<td>Qwls</td>
<td>Wando Formation barrier-island lower sand facies</td>
<td>70 – 130 k</td>
<td>Very fine-grained, quartz sand</td>
</tr>
</tbody>
</table>
Figure 2.3: Map of the Charleston quadrangle showing depth to top of Cooper Marl (modified from Weems and Lemon 1993; Fairbanks et al. 2008).
Figure 2.4: 7.5-minute Charleston quadrangle DEM obtained from the South Carolina Department of Natural Resources GIS Data Clearinghouse (https://www.dnr.sc.gov/GIS/gisdownload.html).
Figure 2.5: Elevation contour map of the top of Marl beneath Charleston peninsula by Fairbanks et al. (2008).
Table 2.2: NEHRP site classes based on $V_{S30}$ (BSSC 1995)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Soil Profile Name</th>
<th>$V_{S30}$ (m/s)</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Hard rock</td>
<td></td>
<td>&gt;1500</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Rock</td>
<td></td>
<td>&gt;760</td>
<td>1500</td>
</tr>
<tr>
<td>C</td>
<td>Very dense soil and soft rock</td>
<td></td>
<td>&gt;360</td>
<td>760</td>
</tr>
<tr>
<td>D</td>
<td>Stiff soil</td>
<td></td>
<td>180</td>
<td>360</td>
</tr>
<tr>
<td>E</td>
<td>Soft soil</td>
<td></td>
<td></td>
<td>&lt;180</td>
</tr>
</tbody>
</table>
Figure 2.6: Map of the Charleston quadrangle showing $V_{S30}$.
Figure 2.7: Map of the Charleston quadrangle showing $PGA_{B-C}$ for 475-year return period based on the 2008 USGS National Seismic Hazard Maps (http://geohazards.usgs.gov/).
Figure 2.8: Site factors for adjusting $PGA_{B-C}$ in Charleston (modified from Aboyé et al. 2011, 2012).
Table 2.3: Relationships by Aboye et al. (2011, 2012) for estimating $F_{PGA}$.

<table>
<thead>
<tr>
<th>PGA (g)</th>
<th>Peak $F_{PGA}$</th>
<th>$V_{S30}$ at Peak $F_{PGA}$ (m/s)</th>
<th>$F_{PGA}$ Linear Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>1.828</td>
<td>188</td>
<td>$V_{S30} &lt; V_{S30}$ at Peak $F_{PGA}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$0.009723 \times V_{S30}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$2.1 - 0.001448 \times V_{S30}$</td>
</tr>
<tr>
<td>0.2</td>
<td>1.556</td>
<td>245</td>
<td>$V_{S30} \geq V_{S30}$ at Peak $F_{PGA}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$0.006351 \times V_{S30}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$1.82 - 0.00108 \times V_{S30}$</td>
</tr>
</tbody>
</table>
Figure 2.9: Map of the Charleston quadrangle showing $PGA_{Site}$ for 475-year return period.
CHAPTER 3:
LIQUEFACTION POTENTIAL MAPS

3.1 Introduction

Liquefaction potential maps for about a 475-year return period are created in this chapter assuming two possible magnitudes and groundwater conditions. Liquefaction potential represents the likelihood of liquefaction occurring in a particular area for a given earthquake loading. Liquefaction potential can be expressed as a factor of safety, a probability, or other variable such as the liquefaction potential index (LPI) proposed by Iwasaki et al. (1978, 1982).

Presented in Figure 3.1, below the dashed line, is a visual representation of the inputs needed to create the liquefaction potential maps. These inputs include geology, depth to top of Marl ($d_{Marl}$), depth to groundwater table ($GWT$), earthquake moment magnitude ($M_W$), and $PGA_{Site}$. The inputs are used in relationships to predict liquefaction potential probability.

3.2 Liquefaction Potential Probability Relationships

The probability relationships derived by Heidari (2011) are used to estimate liquefaction potential in Charleston for 475-year-return-period ground shaking. Heidari (2011) used $LPI$ calculated from cone penetration tests to represent liquefaction potential. $LPI$ is based on conditions in the top 20 m and is defined as (Iwasaki et al. 1978):

$$LPI = \int_{0}^{20} F \cdot w(z)dz$$

3.1
where \( F \) is a function of factor of safety against liquefaction (FS) defined as \( F = 1 - FS \) for \( FS \leq 1 \) and \( F = 0 \) for \( FS > 1 \); \( z \) is the depth below the ground surface in meters; and \( w(z) \) is a depth-weighing factor equal to \( 10 - 0.5z \). For discrete layered profiles, Equation 3.1 can be represented in summation form by:

\[
LPI = \sum_{i=1}^{n} F_i w_i(z) H_i
\]

where \( F_i \) is the function of \( FS \) over the \( i \)th layer; \( w_i(z) \) is the depth-weighting factor for the \( i \)th layer; \( H_i \) is the thickness of the \( i \)th layer in meters; and \( n \) is the number of layers within the top 20 m of depth below the ground surface.

Theoretically, \( LPI \) values can range from 0 to 100. A value of 0 is obtained when \( FS > 1 \) over the entire 20 m of depth. A value of 100 is obtained when \( FS = 0 \) over the entire 20 m of depth.

\( FS \) is defined as the ratio between the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR). CSR represents the seismic demand or loading put on the soil by dynamic shaking, and is expressed by (Seed and Idriss 1971; Youd et al. 2001):

\[
CSR = 0.65 \left( \frac{\sigma_v}{\sigma'_v} \right) \left( \frac{PGA_{Site}}{g} \right) \left( \frac{r_d}{MSF \cdot K_\sigma} \right)
\]

where \( \sigma_v \) is the vertical total stress in the soil at a given depth; \( \sigma'_v \) is the vertical effective stress at the same depth; \( PGA_{Site} \) is the peak ground surface acceleration at the site in units of \( g \); \( g \) is the acceleration of gravity; \( r_d \) is the shear reduction factor dependent on depth; \( MSF \) is the magnitude scaling factor which accounts for the effects of shaking duration; and \( K_\sigma \) is the overburden correction factor.
Procedures recommended by Youd et al. (2001) were used by Heidari (2011) to calculate each variable in Equation 3.3. The recommended lower bound MSF equation can be expressed by:

\[
MSF = \frac{10^{2.24}}{M_{W}^{2.56}}
\]

where \(M_{W}\) is the moment magnitude.

\(CRR\) represents the soil’s capacity to resist liquefaction. The relationship proposed by Robertson and Wride (1998) and recommended by Youd et al. (2001) for computing \(CRR\) from cone penetration test (CPT) measurements is as follows:

\[
\begin{align*}
\text{If } (q_{t1N})_{cs} < 50 & \quad CRR = 0.833 \frac{(q_{t1N})_{cs}}{1000} + 0.05 & 3.5a \\
\text{If } 50 \leq (q_{t1N})_{cs} < 160 & \quad CRR = 93 \frac{(q_{t1N})_{cs}^3}{1000} + 0.08 & 3.5b
\end{align*}
\]

where \((q_{t1N})_{cs}\) is the equivalent clean-sand corrected cone tip resistance normalized to a reference stress of 100 kPa.

Heidari (2011) made corrections to \(CRR\) to account for diagenetic (e.g. age, cementation) effects in the soil. The correction is applied by (Seed 1979; Arango et al. 2000; Andrus et al. 2004):

\[
CRR_{k} = CRR^{*} K_{DR}
\]

where \(CRR_{k}\) is the diagenesis-corrected cyclic resistance ratio; and \(K_{DR}\) is the correction factor applied to the \(CRR\). \(K_{DR}\) can be estimated by (Hayati and Andrus 2009):
where $MEVR$ is the ratio of measured shear wave velocity divided by shear wave velocity estimated from $q_{1N}$.

Heidari (2011) followed the approach of Holzer et al. (2006, 2010) and Rix and Romero-Hudok (2007) to create liquefaction potential probability relationships between probability of $LPI \geq 5$ and the ratio $PGA_{Site}/MSF$. $LPI = 5$ is assumed as the threshold for sand boil generation (Toprak and Holzer 2003). Probability of $LPI \geq 5$ ($P_{LPI>5}$) relationships were derived from the complementary log-normal cumulative distribution of $LPI$ values for various geologic groups and multiple values of $PGA_{Site}$ and $M_W$. The cumulative distributions were then represented as functions of $PGA_{Site}$ for specific earthquake magnitudes through the repetition of the calculation for a given $M_W$ and $0.1 \, g \leq PGA_{Site} \leq 0.5 \, g$ in $0.05 \, g$ increments. The $P_{LPI>5}$ as a function of $PGA_{Site}/MSF$ was obtained by repeating the calculation for $5.0 \leq M_W \leq 7.5$ in $0.5$ magnitude increments.

The following equation describes the $P_{LPI>5}$ relationships created by Heidari (2011):

$$K_{DR} = \frac{1.08 MEVR - 0.08}{3.7}$$

$$P_{LPI>5} = \frac{a}{b + \left[ \frac{PGA_{Site}}{MSF} \right]^d}$$

where $a$, $b$, $c$, and $d$ are curve fitting coefficients. Values for the curve fitting coefficients based on depth to groundwater table ($GWT$) and depth to top of Marl are given in Tables 3.1, 3.2, and 3.3 for three geology groups: (1) Wando Formation (Qws, Qwc, Qwls); (2) younger natural sediments (Qal, Qhm, Qhs, Qht, Qsbc, and Qsbs); and (3) type III artificial fills (af).
Heidari (2011) split artificial fill into three categories. Type I consisted of sites where af was present at the ground surface and overlies Qht deposits that extend to depths $\geq 10$ m. Type II represents sites where af is present at the ground surface and underlain by Qht deposits extending to depth $< 10$ m. The curves for type III af were recommended for areas covered by af where Qhes and other younger natural sediments are likely present within the top 10 m of subsurface. Use of the type III af curves allows for conservative values of $P_{LPI>5}$ (Heidari 2011).

3.3 Groundwater Table

A map of average GWT depth is not available for the Charleston quadrangle. An initial analysis of GWT information is possible using measurements at CPT sites compiled by Fairbanks et al. (2004) and Mohanan et al. (2006). For this thesis, 174 sites in the Charleston quadrangle and two neighboring quadrangles are considered.

Presented in Figure 3.2 are histograms of GWT depths for the three geology groups. From Figure 3.2, approximately 19% of the points for the Wando Formation have GWT depths $< 1.0$ m; 47% have GWT depths between 1.0 and 2.0 m; and 34% of points have GWT depths $> 2.0$ m. For the younger natural sediments, 11% of the points have GWT depths $< 1.0$ m; 47% have GWT depths between 1.0 and 2.0 m; and 42% have GWT depths $> 2.0$ m. For artificial fill, 25% of the points have GWT depths $< 1.0$ m; 38% have GWT depths between 1.0 and 2.0 m; and 37% have GWT depths $> 2.0$ m. The date of GWT measurements were not given by Fairbanks et al. (2004) and Mohanan et al. (2006). Additional work is needed to quantify seasonal GWT depth fluctuations and to develop seasonal GWT maps.
Based on the GWT depth measurements from Fairbanks et al. (2004) and Mohanan et al. (2006), a GWT depth of 1.0 m is initially assumed as a conservative estimate for all geology groups. It should be noted that the 1886 earthquake took place in late August when GWT levels would be expected to be lower. Based on this knowledge, a second liquefaction potential map will be prepared assuming a GWT depth of 2.0 m for the Wando Formation.

Presented in Figures 3.3a, 3.3b, and 3.3c are $P_{LPI>5}$ relationship curves for: Figure 3.3a with curves by Heidari (2011) for the Wando Formation with $GWT = 1$ m and 3 m, younger natural sediments with $GWT = 1$ m, and type III with $GWT = 1$ m, respectively. In order to use these figures within ArcGIS, a linear regression analysis is conducted for each set of curves using the coefficients for Equation 3.8 given in Tables 3.1, 3.2 and 3.3. Thus, sets of equations for each soil group that account for the $PGA_{Site}/MSF$, $d_{Marl}$, and GWT are created for use in ArcGIS.

### 3.4 Liquefaction Potential Maps

Presented in Figure 3.4 is the $P_{LPI>5}$ map for a 475-year-return-period and $M_W = 7.3$, the modal moment magnitude for the seismic hazard assumed to develop the 2008 NEHRP Sesimic Hazard maps (http://earthquake.usgs.gov/hazards/apps/). Earthquakes with $M_W \geq 7.3$ contribute about 40% to the probability of exceedance for a 475-year return period. The $P_{LPI>5}$ map is divided into five liquefaction probability potential zones: 0 - 20%; 20 - 40%; 40 - 60%; 60 - 80%; and 80 - 100%.

The areas in Figure 3.4 with the least probability of liquefying (0 - 20%) are located in North Charleston and north Daniel Island where the Wando Formation is present near the ground surface. Areas with 20 - 60% are located in West Ashley,
James Island, North Charleston, Daniel Island, and in patches in north Mount Pleasant and Charleston peninsula. A majority of the quadrangle is within the $P_{LPI>5}$ range of 60 - 100%. The areas with highest probability of liquefying correspond to areas where $d_{Marl}$ is at greater depths (10 - 26 m). Nearly all artificial fill is in the 80 -100% category, regardless of $d_{Marl}$.

The $P_{LPI>5}$ values plotted in Figure 3.4 for the Wando Formation are significantly higher than values estimated by Hayati and Andrus (2008) for the Charleston peninsula and Heidari and Andrus (2010) for the town of Mount Pleasant in 1886. Shown in Figures 3.5 and 3.6 are mapped locations of historic liquefaction. From these figures, it can be seen that many of the reported instances of liquefaction occurred within af, Qhes, and Qht material. Little liquefaction occurred within Wando Formation soils. Therefore, the high $P_{LPI>5}$ values of Figure 3.4 may be attributed to the high $M_W$ (7.3) and the high $GWT$ (1.0 m) assumed for the Wando Formation.

To evaluate the influence of $M_W$ and $GWT$, a second $P_{LPI>5}$ map is created to better match ground shaking and groundwater conditions during the 1886 earthquake. $M_W = 6.9$ is considered a better estimate for the 1886 earthquake (Heidari 2011; Bollinger 1977; Johnston 1996; Bakun and Hopper 2004; Hayati and Andrus 2008; Talwani and Gassman 2008; Boyd and Cramer 2012). A value of $GWT = 2.0$ m for the Wando Formation more likely existed in August (Hossain 2010). Earthquakes with $M_W \geq 6.9$ contribute approximately 56% to the 475-year-return-period ground motions represented by the 2008 NEHRP Seismic Hazard maps.

The second liquefaction potential map is presented in Figure 3.7. From Figure 3.7, it can be observed that the probability of liquefaction decreases from Figure 3.4 throughout the quadrangle due the decrease in $M_W$ and $GWT$ for the Wando Formation.
Within North Charleston, West Ashley, and north Daniel Island the probability decreases to include more areas with 0 - 40% probability \( LPI \geq 5 \). Much of the Wando Formation’s probability drops down to the next lower probability category, with areas on the Charleston peninsula and Mount Pleasant dropping down to 20 - 60% \( P_{LPI>5} \). Many areas of af and Qsbs are still within the 80 - 100% probability range, while areas of Qht drop down a category into the 60 - 80% range.

The \( P_{LPI>5} \) values for the liquefaction potential map for \( M_W = 6.9 \) in Figure 3.7 match well with observed field behavior and \( P_{LPI>5} \) maps created for the Charleston peninsula by Hayati and Andrus (2008) and the town of Mount Pleasant by Heidari and Andrus (2010). Shown in Figures 3.8 and 3.9 are liquefaction potential maps created by Hayati and Andrus (2008) and Heidari and Andrus (2010). These maps take into account 1886 field performance and diagenetic-corrected \( LPI \) for \( M_W = 7.1 \) and \( PGA_{Site} = 0.3 \text{ g} \), and \( M_W = 6.9 \) and \( PGA_{Site} = 0.25 \text{ g} \), respectively. Corrected \( LPI \) values are shown for CPT sites.

The patch of \( P_{LPI>5} = 0 – 20\% \) in the western Mount Pleasant area is taken directly from the map in Figure 3.9 by Heidari and Andrus (2010) and is based on several CPT’s. This is the only area of the map in Figure 3.7 that has been adjusted based on specific cone tests.

It should be noted the assumption of the type III af curves allows for conservative estimates of \( P_{LPI>5} \) in areas where less liquefiable fill material is located. Fills in Charleston can range from liquefiable granular soils to non-liquefiable high plasticity fine-grained soils. It has been noted that several areas within the quadrangle, particularly the west tip of Mount Pleasant, the south end of Daniel Island, Clouter Island (between Daniel Island and North Charleston), and the old Naval Base on the Cooper River (west
of Daniel Island) are the locations of former dredge spoil basins consisting of high plasticity fine-grained soils. Additionally, these dredge spoil materials were placed on top of marsh deposits that also consisted of high plasticity fine-grained soils. These areas likely have lower liquefaction potential than predicted. Fills on the Charleston peninsula are known to contain mostly granular soils deposited on top of natural deposits with granular soils, and likely predict liquefaction potential more accurately (William M. Camp III written correspondence, April 16, 2012).

The liquefaction potential maps in Figures 3.4 and 3.7 show relative probability values. Therefore, $P_{LPI>5}$ values indicate the likelihood of liquefaction occurring in a given area relative to the rest of the map. For example, areas of $P_{LPI>5} = 80 – 100\%$ are 4 times as likely to liquefy then areas of $P_{LPI>5} = 0 – 20\%$, or twice as likely to liquefy than areas of $P_{LPI>5} = 40 – 60\%$.

### 3.5 Summary

Discussed in this chapter are the steps taken and layers used to create two liquefaction potential maps for the 7.5-minute Charleston quadrangle. The geology of the area was grouped into the categories of artificial fill, younger natural sediments, and Wando Formation to correspond to available $P_{LPI>5}$ curves derived by Heidary (2011). A conservative initial estimate of $GWT$ (1.0 m) was assumed based on measurements at CPT sites.

The $P_{LPI>5}$ relationships derived by Heidari (2011) use $LPI$ to represent liquefaction potential as a function of factor of safety. The relationships take into account diagenetic effects in the 100,000-year-old Wando Formation. The relationships assumed
LPI \geq 5 as the threshold for sand boil generation. The curves correlate the $P_{LPI>5}$ with $PGA_{Site}/MSF$, GWT, and $d_{Marl}$.

The two $P_{LPI>5}$ maps were created for a 475-year-return-period assuming two values of $M_W$. One map was based on $M_W = 7.3$ and $GWT = 1.0$ m for all areas. The second map was based on $M_W = 6.9$ (a more reasonable estimate for the 1886 event) and $GWT = 2.0$ m for the Wando Formation (a more likely depth in August). The values of $P_{LPI>5}$ based on $M_W = 7.3$ and high GWT are too high compared to actual ground behavior in 1886. Values of $P_{LPI>5}$ based on $M_W = 6.9$ and lower GWT agree well with 1886 ground behavior and previous maps created for the Charleston peninsula by Hayati and Andrus (2008) and Mount Pleasant by Heidari and Andrus (2010).
Figure 3.1: Flowchart showing methodology used for developing the liquefaction potential maps.
Table 3.1: $P_{LPI>5}$ curve coefficients used for the Wando Formation (Heidari 2011)

<table>
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<tr>
<th>Depth to GWT (m)</th>
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Table 3.2: $P_{LPI>5}$ curve coefficients used for younger natural sediments (Heidari 2011)

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Table 3.3: $P_{LPI>5}$ curve coefficients for type III artificial fills (Heidari 2011)

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Figure 3.2 Distribution of groundwater table depths measured at CPT sites grouped by geology.
Figure 3.3: Liquefaction probability curves developed by Heidari (2011) for (a) the Wando Formation with $GWT=1\text{m}$ (solid curves) and $3\text{ m}$ (dashed curves); (b) younger natural sediments with $GWT=1\text{ m}$; and (c) artificial fill with $GWT=1\text{ m}$.
Figure 3.4: Liquefaction potential map of the Charleston quadrangle for a 475-year return period accelerations and assuming $M_w=7.3$ and $GWT=1.0$ m for all areas.
Figure 3.5: Geology map of the Charleston Peninsula and Drum Island with locations of 1886 liquefaction and ground deformation sites (Hayati and Andrus 2008).
Figure 3.6: Geology map of Mount Pleasant with locations of 1886 liquefaction and ground deformation (Heidari and Andrus 2010).
Figure 3.7: Liquefaction potential map of the Charleston quadrangle for 475-year return period accelerations and assuming $M_w=6.9$ and $GWT=2.0$ m for the Wando and 1.0 m for all other areas.
Figure 3.8: Liquefaction potential map of Charleston Peninsula and Drum Island by Hayati and Andrus (2008) based on 1886 field performance and diagenetic-corrected LPI for $M_W=7.1$ and $PGA_{site}=0.3$ g.
Figure 3.9: Liquefaction potential map of Mount Pleasant by Heidari and Andrus (2010) based on 1886 field performance and diagenetic-corrected $LPI$ for $M_W=6.9$ and $PGA_{Site}=0.25 \text{ g}$. 
CHAPTER 4:

IMPROVING RESILIENCY OF CHARLESTON’S CIVIL INFRASTRUCTURE

4.1 Introduction

Liquefaction hazard maps can be used by planners and engineers to identify areas most likely to experience damage during future seismic events. In particular, any structures or utilities within areas of high liquefaction potential are prone to damage from settlement and ground movement induced by liquefaction. Structures or utilities that straddle areas of two dissimilar potential are prone to extensional or compressional damage from lateral displacement.

This chapter discusses potential uses for the liquefaction potential hazard maps created in this study with respect to roadways. Resiliency of roadway infrastructure against liquefaction is discussed in the context of bridge design and construction. While this discussion does not include metrics for measuring resiliency, methods for improving resiliency against liquefaction induced damages are considered.

4.2 Roadway Liquefaction Hazard Evaluation

To demonstrate one potential use of liquefaction potential maps, a map of roads maintained by SCDOT in 2011 (dbw.scdot.org/GISMapping/default.aspx) is added to the $M_W = 6.9$ liquefaction potential map in Figure 4.1. From Figure 4.1, a picture of roadways most vulnerable to liquefaction induced damage can be seen. Roads likely to have the most damage (i.e., roads in areas where $P_{LPI>5} = 80-100\%$) are located on the southern end of the Charleston peninsula, James Island, and the west shoreline of Mount Pleasant. Much of the area of historic downtown Charleston on the southern end of the
Charleston peninsula is also at a high risk of liquefaction-induced failures, particularly the roads adjacent to the rivers where af and young natural sediments are present.

Attention can also be drawn to the bridges (including the approaches) in the Charleston quadrangle. The approaches to the Arthur Ravenel Jr. Bridge between the Charleston peninsula and Mount Pleasant rest on areas of artificial fill with $P_{L_{PB5}} = 80 - 100\%$. The Savannah Highway and James Island Expressway bridges between the Charleston peninsula and West Ashley have approaches that rest on $P_{L_{PB5}} = 60 - 100\%$. The SC-7 bridge between North Charleston and West Ashley has approaches that rest on soil with $60 - 80\% \ P_{L_{PB5}}$. The approaches to the 526 Mark Clark Expressway bridge between Daniel Island and Mount Pleasant also rests on soil with $P_{L_{PB5}} = 60 - 80\%$.

4.3 Improving Resilience of Bridges

Resilience, when describing infrastructure, is the ability to reduce the magnitude and/or duration of disruptive events. Resiliency of infrastructure is measured by its ability to anticipate, absorb, adapt to, and/or rapidly recover from a potentially disruptive event (NIAC 2009). When discussing the resiliency of bridges against potential liquefaction effects in Charleston, the following must be considered: (1) the effectiveness of bridge and abutment design and construction to withstand ground failures; (2) and the readiness to rapidly institute recovery efforts should damage occur.

To present a method of measuring a system’s resiliency, Figure 4.2 is used to demonstrate a performance response curve for an infrastructure system after an emergency. This curve demonstrates a method of measuring resiliency before, during, and after a disaster event. As explained by Ouyang et al. (2012), the line from A to B demonstrates a baseline of 100%, or the total resistant capacity of the system. The first
stage \((0 \leq t \leq t_0)\) represents the disaster prevention stage, during which normal operation occurs. This stage also reflects the total resistant capacity of the system to reduce damage from a disaster event by beginning at the 100% baseline. The second stage \((t_0 \leq t \leq t_1)\) represents the occurrence of a disaster event and the ability of the system to absorb damage from the event and minimize consequences of damage. The final stage \((t_1 \leq t \leq t_E)\) represents the recovery process during which the damage to the system is assessed and recovery is enacted to restore the system to working capacity.

From Figure 4.2, one can observe that the controllable aspects of resiliency are (a) the disaster prevention stage and (b) the recovery stage. In the context of improving resiliency of bridges against liquefaction, this means constructing bridge foundations (or retrofitting existing bridge foundations) to possess a high resistance to damages and having a system in place to inspect and repair bridges and abutments after a seismic event. Resilience is increased for a system by decreasing the area of the shape under the line from A to B in Figure 4.2. Therefore, the bridge resilience can be increased by either decreasing the loss to performance level of the bridges from an earthquake or by decreasing the amount of time the assessment and recovery stage takes to perform.

The effects of soil liquefaction on bridges can depend greatly upon the design and location. If not properly designed, bridge pile foundations constructed through liquefiable layers can be adversely affected by liquefaction-induced lateral displacement, particularly when the piles are not stabilized to create a “pile-pinning” effect. Lateral displacements of bridge abutments from liquefaction displacement can cause permanent longitudinal displacement of the bridge structure (Ledezma and Bray 2010). Additionally, case histories of earthquakes in Japan have shown damage to bridges
where small diameter piles are constructed. Failures occur in the form of pile cracking and plastic hinge formation (Bhattacharya et al. 2011).

The foundations of existing structures can be improved to withstand liquefaction through the use of various ground improvement techniques. Andrus and Chung (1995) discussed methods of compaction grouting, permeation grouting, jet grouting, *in situ* soil mixing, and drain piles for mitigating damage of existing structures. Only jet grouting and *in situ* soil mixing have been found to be effective as remedies for all liquefiable soil types. Compaction grouting is marginally effective for the treatment of silts. Chemical grout is not effective for soils with 25% fines due to an inability to permeate past this fines content. Vertical drains extending through the full length of a liquefiable material can be used, particularly with deeper soils, to relieve excess pore pressures before they can increase to a level that causes liquefaction (Brennan and Madabhushi 2005). These vertical drains can consist of coarse soil or a permeable synthetic material (Bhattacharya et al. 2011). Drains are found to be ineffective in soil with low permeability.

In new construction, the use of sand compaction piles can be used to induce densification and compaction of soil (Akiyoshi et al. 1993). Use of stone columns (or gravel drains) also offers a method of mitigating liquefaction (Adailier and Elgamal 2004).

Successful bridge design against liquefaction effects was observed by Bhattacharya et al. (2011) after the recent March 11, 2011 Tokyo earthquake. Downtime of many of the Tokyo elevated highway bridges was low and little damage to superstructures and piers was observed. While liquefaction was observed around the foundations of several elevated highways, little damage was inflicted on the pier foundations. Much of this success can be attributed to the use of liquefaction mitigation
techniques and good bridge foundation designs. Rapid inspection and repair work can also be cited as reasons for little downtime where damage did occur.

Quantifying resilience of the bridge infrastructure can be put into terms of four properties: robustness, resourcefulness, rapidity, and redundancy (Bruneau et al. 2006). Robustness in this case would refer to the bridge structure’s (including the foundation) ability to withstand damage from liquefaction-induced ground failure. Improving the robustness of bridge structures would involve implementing the discussed pre- or post-construction remediation techniques and sufficient bridge design (e.g., pile stabilization, use of sufficient diameter piles).

Resourcefulness can be quantified in terms of material, labor, or monetary resources. An example of measuring this property is by determining whether the resources are available to implement bridge or abutment repair in event of an earthquake. Specialized construction equipment, labor, and funds must exist in sufficient quantities for repairs to be enacted. Similarly, determining whether the resources exist to modify existing bridge foundations in preparation for liquefaction quantifies resourcefulness.

Rapidity refers to the response time to react after a disaster event. This is best illustrated in the case of the response time after an earthquake to assess and repair damage to bridge structures. Increased rapidity equates to less time a bridge must be closed to traffic for inspection and repair.

Redundancy is the use of multiple components to prevent a system from failing should one component become unusable. Concerning bridge infrastructure in Charleston, the number of routes onto and off of the Charleston peninsula is an example. The peninsula is accessible via five routes: the Arthur Ravenel Jr. Bridge; the
Savannah Highway bridge; the James Island Expressway bridge; the SC-7 bridge; and local land route roads from North Charleston. Should one of these routes be inaccessible due to liquefaction effects, another route may be used. Problems arise when it becomes necessary to close multiple routes to traffic.

Increasing the redundancy of the bridges could be accomplished in several ways. More bridges could be built to connect to the Charleston peninsula, though this may be difficult to achieve. In the event of an earthquake event in which multiple bridges are damaged and must be closed, ferries or boats could be used to bring disaster relief or evacuate population and decrease the traffic load of accessible routes. Additionally, redundancy can be used to increase robustness by implementing multiple methods of resistance to liquefaction damage (e.g. designing a bridge to resist liquefaction effects while also using ground remediation during construction).

The difficulty of improving the redundancy by building new bridges suggests that improvements for resilience in Charleston should focus on the robustness property. Emphasis should be placed on the mitigation of damage from liquefaction and rechecking the condition of existing bridges to determine the necessity of ground improvement techniques.

To improve the resiliency of Charleston’s bridges in the event of liquefaction it is recommended that: (1) construction of future bridges take into account their ability to withstand damage from ground failure; (2) existing bridges be rechecked and, if necessary, retrofitted to withstand liquefaction through various ground improvement techniques; and (3) inspection and repair efforts should take effect immediately after an earthquake. The use of the liquefaction potential maps offers insight into areas where
liquefaction is most likely to occur, and allows for the design and placement of new structures and mitigation of existing structures to be more informed on potential risk.

It should be noted that bridges in South Carolina did not have enforced design criteria for seismic loads until 2001. Currently, only the Arthur Ravenel Jr. Bridge has been built since seismic design criteria have been enforced. Therefore, while the new Arthur Ravenel Jr. Bridge is designed to resist seismic loading and liquefaction-induced damage, the older bridges of the area may not have been designed with any seismic loading in mind. The liquefaction potential maps can be used to prioritize which bridges to inspect and retrofit before an earthquake, and to prioritize and save time during a post-earthquake assessment.

4.4 Summary

Discussed in this chapter is a potential use for the liquefaction potential hazard maps created in this study. A map of SCDOT roads was added to the $M_W = 6.9$ liquefaction potential map. Roads at the most risk of damage, with $P_{LPI > 5} = 80 - 100\%$, were located in areas at the southern end of Charleston peninsula, James Island, and western Mount Pleasant. All bridges in the quadrangles were located in areas with $P_{LPI > 5} = 60 - 100\%$.

Resiliency of Charleston’s bridge infrastructure against liquefaction was briefly discussed. To improve the resilience of bridges against liquefaction, it is recommended that: (1) construction of future bridges take into account their ability to withstand damage from liquefaction; (2) existing bridges be rechecked and, if necessary, retrofitted to withstand liquefaction through various ground techniques; and (3) inspection and repair efforts should take effect immediately after an earthquake.
Figure 4.1: Liquefaction potential map of the Charleston quadrangle for 475-year-return-period accelerations and $M_w=6.9$, with roadways maintained by SCDOT (dbw.scdot.org/GISMapping/default.aspx).
Figure 4.2 Typical performance response curve of an infrastructure system following a disruptive event, as illustrated by Ouyang et al. (2012).
Maps showing distribution of (1) $V_{S30}$, (2) $PGA_{Site}$, and (3) liquefaction potential for the 7.5-minute Charleston quadrangle were developed in this study using the GIS software package ArcGIS 10. All calculations performed in the analysis were raster calculations executed using ArcGIS. All created raster layers were snapped to the same coordinates and consisted of grids with 30 m x 30 m area cells.

5.1 Average Shear-Wave Velocity and Peak Ground Acceleration Maps

Surficial soils within Charleston quadrangle ranged from relatively young (<300 years) artificial fill to about 100,000-year-old material of the Wando Formation. Underlying the entire area is the Cooper Marl, a non-liquefiable Tertiary-aged deposit.

To estimate $V_{S30}$, a two layered geology model was assumed. The two layers consisted of the Quaternary deposits and the Marl. For the Quaternary deposits and the Marl, mean shear wave velocity values reported by Andrus et al. (2006) were assumed. Computed values of $V_{S30}$ were adjusted locally to match $V_{S30}$ values for SCPT profiles compiled by Fairbanks et al. (2004). The final $V_{S30}$ map was used to adjust $PGA_{B-C}$ to local site conditions. The entire Charleston quadrangle classifies as D and E seismic site classes, assuming no special Site Class F conditions.

A map of the $PGA_{B-C}$ was created from the 2008 USGS National Seismic Hazard Maps for a 475-year-return-period. Site factors provided by Aboye et al. (2011, 2012) were used to obtain the $PGA_{Site}$ map. Values of $PGA_{Site}$ were as much as 50% higher than $PGA_{B-C}$. 
5.2 Liquefaction Potential Maps

Two geology-based probabilistic maps of liquefaction potential were developed in Chapter 3. This work extends the work of Hayati and Andrus (2008) involving the Charleston peninsula and Heidari and Andrus (2010) involving Mount Pleasant to include the entire 7.5-minute Charleston quadrangle. This work improves upon the previous maps by using peak ground accelerations that were adjusted for varying local site effects.

The study area was split into the three categories of (1) artificial fill, (2) younger natural sediments, and (3) Wando Formation to match available $P_{LPI>5}$ curves developed by Heidari (2011). The $P_{LPI>5}$ curves correlate the $P_{LPI>5}$ with $PGA_{Site/MSF}$, $GWT$, and $d_{Marl}$. An analysis of available information suggest 1.0 m as a conservative estimate for the depth to $GWT$ for all areas.

One of the liquefaction potential maps is based on $M_w = 7.3$ and $GWT = 1.0$ m for all areas. The other liquefaction potential map is based on $M_w = 6.9$, $GWT = 2.0$ m for areas covered by the Wando Formation, and $GWT = 1.0$ m for areas covered by younger materials. Values of liquefaction potential for the $M_w = 7.3$ map were high when comparing to previous liquefaction potential studies conducted on the Charleston peninsula (Hayati and Andrus 2008) and in Mount Pleasant (Heidari and Andrus 2010). The values of liquefaction potential for the $M_w = 6.9$ map match more closely with observed field behavior and previous maps created for Charleston peninsula and Mount Pleasant. Areas with the deepest depth to the Marl and areas with artificial fill and younger natural sediments were predicted to be at highest risk in both maps.
5.3 Application of Liquefaction Potential Maps for Improving Resilience

The potential uses for the liquefaction potential hazard maps created in this study were discussed in Chapter 4 with respect to Charleston roadways. A map of SCDOT roads was added to the $M_W = 6.9$ liquefaction potential map. Roads at the most risk of damage, crossing areas with $P_{LPI>5} = 80 - 100\%$, were located in areas at the southern end of Charleston peninsula, James Island, and western Mount Pleasant. All bridges in the quadrangles were located in areas with $P_{LPI>5} = 60 - 100\%$.

Resiliency of Charleston’s bridge infrastructure against liquefaction was briefly discussed. To improve the resilience of bridges against liquefaction, it is recommended that preventative measures be taken for future construction and existing structures, and that inspection and repair efforts begin immediately after an earthquake.

5.4 Limitations

Liquefaction potential maps were expressed in terms of $LPI$. $LPI$ is an index, not an engineering property. As noted by Heidari (2010), it is important to remember that probability values shown in the liquefaction potential maps are relative values that are best compared within the context of the specific map. For instance, if a specific area demonstrates a probability of $40\%$ $LPI \geq 5$, another area with a probability of $80\%$ will be twice as likely to liquefy.

Fills in Charleston can range from liquifiable granular soils to non-liquifiable high plasticity fine-grained soils. Several areas within the quadrangle, particularly the west tip of Mount Pleasant, the south end of Daniel Island, Clouter Island (between Daniel Island and North Charleston), and the old Naval Base on the Cooper River (west of Daniel Island) are the locations of former dredge spoil basins consisting of high plasticity fine-
grained soils. Additionally, these dredge spoil materials were placed on top of marsh deposits that also consisted of high plasticity fine-grained soils. These areas likely have lower liquefaction potential than predicted. Fills on the Charleston peninsula are known to contain mostly granular soils deposited on top of natural deposits with granular soils, and likely predict liquefaction potential more accurately. As vertical profiles for fill locations were not available, the assumption of af type III curves allowed for conservative estimates of liquefaction potential. Site specific investigation can be conducted to determine if non-liquifiable material soils lie beneath af.

Map layers used for calculations in this study consisted of grid rasters of 30 m x 30 m pixels. Therefore, the accuracy of liquefaction potential values is limited to the nearest 900 m². The maps are useful for viewing regional variations. Testing should always be performed for actual construction projects to confirm site conditions.

5.5 **Recommendations for Future Work**

Based on the results of this research, the following tasks are recommended for future work:

1. Additional study and testing are needed to map groundwater table depths for the Charleston area and quantify seasonal fluctuations.
2. Additional testing is needed to create liquefaction probability curves for individual geologic units and thereby improve the accuracy of the maps.
3. Additional work can be done to consider other hazard levels for the area, such as a 2500-year return period hazard.
4. Additional study is needed to determine the sensitivity of $P_{LPI>5}$ to the input variables.
5. The liquefaction potential maps developed in this thesis can be used in loss estimation programs and identify infrastructure vulnerable to liquefaction.

6. The methodology for creating the liquefaction maps presented in this study can be used to map other areas of the greater Charleston area.
REFERENCES


the U. S. Geological Survey, Award No. 03HQGR0046, Civil Engineering Dept., Clemson Univ., Clemson, SC.


