Gravel Liquefaction Assessment with the Dynamic Penetration Test at Non-Liquefaction Sites in Valdez, Alaska and L'Aquila, Italy

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Gravel Liquefaction Assessment with the Dynamic Penetration Test at Non-Liquefaction Sites in Valdez, Alaska and L’Aquila, Italy

Nicholas James Linton

A thesis submitted to the faculty of Brigham Young University in partial fulfillment of the requirements for the degree of Master of Science

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ABSTRACT

Gravel Liquefaction Assessment with the Dynamic Penetration Test at Non-Liquefaction Sites in Valdez, Alaska and L’Aquila, Italy

Nicholas James Linton
Department of Civil and Environmental Engineering, BYU
Master of Science

The development of a reliable, and cost-effective in-situ method for characterizing the liquefaction potential of gravelly soils is a considerable challenge for engineers and researchers. The ability to accurately characterize the liquefaction potential of gravelly soils is an important consideration at port facilities and dams for example. The Dynamic Penetration Test (DPT) provides a reliable and cost-effective method for evaluating the liquefaction resistance of gravelly soils. Probabilistic liquefaction triggering curves based on DPT field data have been developed from data collected at 47 sites in China. However, using the DPT-based liquefaction curves for locations outside of the Chengdu plain in China where the data for the triggering curves were gathered may yield unreliable results. To improve the reliability of the DPT-based liquefaction triggering curves additional DPT field data form outside of the Chengdu plain is required. In total seven new non-liquefaction DPT case histories are presented in this report. Two of the case histories are based on DPT field data from Valdez, Alaska. The remaining five case histories were developed from DPT field data from L’Aquila, Italy. When plotted on the liquefaction triggering curves based only on the DPT data obtained in the Chengdu plain three of the seven data points plot in a position that indicates a considerable possibility of liquefaction despite these case histories being from locations where liquefaction did not occur. Roy (2021) developed new DPT-based liquefaction triggering curves with these seven new non-liquefaction case histories, DPT filed data from other sites around the world, and the DPT field data from the Chengdu plain. The three data points from the new case histories presented in this report that had a considerable probability of liquefaction when plotted on the curve developed only with the data from the Chengdu plain had a significantly lower probability of liquefaction when plotted on the new DPT-based liquefaction triggering curves. One of the data points from Valdez, Alaska decreased from a probability of liquefaction of around 50% to a probability of liquefaction of less than 30% when plotted on the new DPT-based liquefaction triggering curves. The reliability of DPT-based liquefaction triggering curves will continue to increase as the amount of available DPT data increases.

Keywords: liquefaction, gravel, probability, DPT, Valdez, L’Aquila, non-liquefaction
ACKNOWLEDGEMENTS

I would like to acknowledge everyone who had played a role in my academic success. First of all, my wife, who has helped and supported me throughout the process. Second of all, my parents, who have supported me and helped me to become who I am today. Thirdly, my committee members, who spent so much time assisting me through the research process.

Financial support for this study was provided by grants CMMI-1663546 and CMMI-1663288 from the National Science Foundation. This funding is gratefully acknowledged. However, the opinions, conclusions and recommendations in this paper do not necessarily represent those of the sponsors. We also express appreciation to Tim Weiss of the Alaska Department of Transportation and Prof. Sara Amoroso for arranging drilling access to sites in Valdez, Alaska and L’Aquila, Italy, respectively.
# Table of Contents

List of Figures ...................................................................................................................................... vii  
List of Tables ....................................................................................................................................... xi  
1 Introduction ....................................................................................................................................... 1  
   1.1 Background .......................................................................................................................... 1  
   1.2 Objectives ............................................................................................................................ 3  
   1.3 Scope .................................................................................................................................... 4  
2 Methods for Evaluating Gravel Liquefaction .............................................................................. 6  
   2.1 Standard Penetration Test (SPT) & the Cone Penetration Test (CPT) .............................. 6  
   2.2 Becker Penetration Test (BPT) ......................................................................................... 7  
   2.3 Dynamic Penetration Test ................................................................................................. 8  
      2.3.1 Test Background ...................................................................................................... 8  
   2.4 Development of the DPT-Based Liquefaction Triggering Curve .................................... 10  
   2.5 DPT-Based Liquefaction Triggering Curve Developed by Roy (2021) ......................... 15  
   2.6 Factor of Safety Against Liquefaction Calculations ....................................................... 16  
   2.7 Shear Wave Velocity ($V_s$) ............................................................................................. 17  
   2.8 CPT Based Liquefaction Triggering Curve Comparison ............................................... 20  
   2.9 Excess Pore Water Generation in Gravelly Soils ............................................................ 21  
3 Soil Parameters Calculated ............................................................................................................. 23  
   3.1 Relative Density .................................................................................................................. 23  
   3.2 Shear Wave Velocity .......................................................................................................... 24  
4 1964 Alaska Earthquake Effects in Valdez, Alaska ................................................................. 26
List of Figures

Figure 2.1. Component sketch of tripod and drop hammer setup for dynamic penetration tests (DPT) along with DPT cone tip. (Cao et al., 2013) ................................................................. 9

Figure 2.2. Probabilistic Liquefaction Triggering Curves for Gravelly Soils. (Cao et al., 2013) 14

Figure 2.3. Shear wave velocity based probabilistic liquefaction triggering curves for gravels. (Cao et al., 2011) ........................................................................................................................................... 19

Figure 2.4. CPT-based probabilistic liquefaction triggering curves for sand (Boulanger et al., 2014) ............................................................................................................................................. 21

Figure 4.1. Map of Alaska showing the epicenter of the March 27, 1964 Alaska earthquake and the location of New and Old Valdez. (Google Maps 2020) ................................................................. 29

Figure 4.2. ShakeMap for 9.2 Mw 1964 Alaskan Earthquake event (ShakeMap by USGS https://earthquake.usgs.gov/earthquakes/eventpage/official19640328033616_30). ...................... 29

Figure 4.3. Map showing the regions of uplift and subsidence during the 1964 Alaska earthquake. (USGS https://earthquake.usgs.gov/earthquakes/events/alaska1964/img/map-2x.jpg) ........................................................................................................................................... 30

Figure 4.4. Distribution and nature of material in Valdez and vicinity. (Coulter et al., 1966)..... 31

Figure 4.5. Photos of the Port of Old Valdez. The photo on the right was taken on June 13, 1964. The photo on the left was taken on September 23, 1963 (Coulter et al., 1966) ................. 33

Figure 5.1. Map of Italy with the Epicenter of the April 6, 2009 earthquake and the city of L’Aquila labeled (Google Maps 2020). ......................................................................................... 37

Figure 5.2. Geologic map and profile of the L’Aquila basin (Amoroso et al., 2008). ............... 38

Figure 6.1. Aerial image showing DPT sounding locations (Site 1 and Site 2) relative to Shannon and Wilson test borings B-1 and B-2 in Valdez, Alaska (Googlemaps, 2016) ............ 41
Figure 6.2. Photograph of the truck-mounted CME 75 drill rig used in Valdez, Alaska. .......... 42

Figure 6.3. Grain size distribution curves for Site 1 in Valdez 9.1 – 9.8 meters. (Shannon and Wilson)......................................................................................................................................................................................... 43

Figure 7.1. Aerial image of DPT sounding locations in L’Aquila Italy. ................................. 46

Figure 7.2. Map of Site 1 in L’Aquila with location and type of all available data labelled........ 47

Figure 7.3. Map of Site 2 in L’Aquila with location and type of all available data labelled.......... 48

Figure 7.4. Map of Site 3 in L’Aquila with location and type of all available data labelled........ 49

Figure 7.5. Map of Site 4 in L’Aquila with location and type of all available data labelled.......... 50

Figure 7.6. Map of Site 5 in L’Aquila with location and type of all available data labelled......... 51

Figure 7.7. Photograph of the truck-mounted Fraste drill rig used in L’Aquila Italy.................. 52

Figure 7.8. Grain size distribution curves in the critical layer for Sites 1 through 4 in L’Aquila. 55

Figure 7.9. Grain size distribution curves for Site 1 in L’Aquila 2.6 to 6.4 meters. ................. 56

Figure 7.10. Grain size distribution curves for Site 1 in L’Aquila 7.3 to 11.5 meters. ............... 56

Figure 7.11. Grain size distribution curves for Site 2 in L’Aquila 1.5 to 5 meters. ................. 57

Figure 7.12. Grain size distribution curves for Site 2 in L’Aquila 5.8 to 9.5 meters. ............... 57

Figure 7.13. Grain size distribution curves for Site 3 in L’Aquila 0.6 to 4.8 meters. ............... 58

Figure 7.14. Grain size distribution curves for Site 4 in L’Aquila 1 to 5.7 meters. ................... 58

Figure 7.15. Grain size distribution curves for Site 5 in L’Aquila 3.7 to 12.2 meters. (i.m.o.s Ponte Rosarolo Soil Investigation) .................................................................................................................. 59

Figure 8.1. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 1 (Sports Field) in L’Aquila......................................................................................................................................................................................... 64
Figure 8.2. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 2 (Japan) in L’Aquila ..... 65
Figure 8.3. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 3 (AQA3) in L’Aquila ... 66
Figure 8.4. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 4 (AQA4) in L’Aquila ... 67
Figure 8.5. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 5 (Water Plant) in L’Aquila (i.m.o.s Ponte Rosarolo Soil Investigation) ................................................................... 68
Figure 8.6. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 1 in Valdez ..................... 69
Figure 8.7. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 2 in Valdez ..................... 70
Figure 8.8. DPT liquefaction triggering curve developed by Cao et al. (2013) showing Chengdu, L’Aquila, and Valdez Data points. .................................................................................................... 75
Figure 8.9. DPT liquefaction triggering curve developed by Roy (2021) showing Chengdu, L’Aquila, and Valdez data points. ......................................................................................................... 76
Figure 8.10. Probability of liquefaction based on shear wave velocity for Site 4 (AQA4) in L’Aquila ........................................................................................................................................ 80
Figure 8.11. Probability of liquefaction based on shear wave velocity for Site 5 (Water Treatment Plant) in L’Aquila ....................................................................................................... 81
Figure 8.12. Shear wave velocity triggering curve with including three data points from L’Aquila, Italy ........................................................................................................................................ 82
Figure 8.13. DPT and SPT Data for Site 1 in L’Aquila ................................................................. 84
Figure 8.14. DPT, SPT, and CPT Data for Site 5 in L’Aquila ..................................................... 85
Figure 8.15. DPT and SPT Data for Site 1 in Valdez ................................................................. 86
Figure 8.16. DPT and SPT Data for Site 2 in Valdez ................................................................. 87
Figure 8.17. $N'_{120}$ blow counts for the heavy hammer and the light hammer at the same
depth plotted together. .............................................................................................................. 90
Figure 8.18. Light & heavy hammer $N'_{120}$ values and cumulative $N'_{120}$ plot for L’Aquila
site 1 ............................................................................................................................................ 92
Figure 8.19. Light & heavy hammer $N'_{120}$ values and cumulative $N'_{120}$ plot for L’Aquila
site 4 ............................................................................................................................................ 93
Figure 8.20. Light & heavy hammer $N'_{120}$ values and cumulative $N'_{120}$ plot for L’Aquila
site 5 ............................................................................................................................................ 94
List of Tables

Table 1.1. List of earthquakes with reports of gravel liquefaction ................................................. 4
Table 5.1. PGA Values Used for Sites in L’Aquila ...................................................................... 37
Table 6.1. Coordinates of DPT Soundings in Valdez, Alaska ...................................................... 40
Table 6.2. Grain Size Distribution at 9.1-m in Borehole B-3 ....................................................... 43
Table 7.1. Coordinates of DPT Soundings in L’Aquila, Italy ...................................................... 45
Table 7.2. Grain Size Distribution at Each Site in L’Aquila in the Critical Layer ....................... 54
Table 7.3. Table Showing all Soil Data Available at Sites 1 through 4 in L’Aquila ................. 60
Table 8.1. Critical Layer Depth Range (m) for Sites in Valdez and L’Aquila ............................. 63
Table 8.2. Relative Density Range in the Critical Layer for Each Site ........................................ 72
Table 8.3. Shear Wave Velocity Range in the Critical Layer for Each Site ................................. 78
1 Introduction

1.1 Background

Liquefaction is an occurrence in which granular soil behaves like a heavy fluid. The transition of granular soil from a stable to a liquefied state is caused by increased pore water pressure and decreased effective stress induced by ground motion (Youd et al., 2001). The increased pore pressure is caused by the tendency of granular material, like sand and gravel, to compact when subjected to cyclic shear deformations produced by ground motions (Youd et al., 2001). Sand is especially vulnerable to compaction and the resulting increased pore water pressure. Although, less common, gravelly soils can also be affected by increased pore water pressure when subjected to cyclic shear deformations. If the pore water pressure becomes greater than or equal to the vertical total stress, liquefaction will occur. Over the past century gravel liquefaction has been observed at multiple sites in at least 20 earthquakes as summarized in Table 1. Gravel liquefaction is a particular concern for many port facilities and dams constructed on gravel foundations before the liquefaction hazard was recognized. The identification of soils that may experience liquefaction in the event of an earthquake is an important consideration in geotechnical engineering. While there are reliable and cost-effective methods for evaluating
Evaluating gravelly soil for liquefaction potential is more difficult than evaluating the liquefaction potential of sands due to the larger grain size of gravels. The Cone Penetration Test (CPT) and the Standard Penetration Test (SPT) are often used to determine liquefaction potential in sandy soils (Rollins, 2018). However, the CPT and SPT tests are not suitable methods of evaluating gravelly soils because they often report higher than realistic resistance in gravelly soil (Cao et al., 2013). When the SPT split spoon sampler or the CPT cone tip contact the large particles found in gravelly soils, they are unable to push the particles out of the way (Rollins, 2018). This increases the reported resistance of the soil to a level that is much higher than the actual resistance. The inability of the SPT and CPT to accurately characterize gravelly soils creates the need for alternative methods that can be used to characterize gravelly soils.

One method for characterizing gravelly soil is the Becker Penetration Test (BPT) where a 6-inch pipe is driven into the ground with a diesel hammer. The BPT is commonly used in North America, requires expensive specialized equipment, and relies on indirect correlations, making it cost prohibitive for smaller projects and potentially unreliable (Cao et al. 2013). Another method involves freezing the soil, then extracting it to be tested in a laboratory setting (Kokusho, 1994). Freezing the soil before removal is expensive and time-consuming (Rollins, 2018). In addition, specialized lab testing equipment is required for the large diameter cyclic shear tests. The development of a more cost-effective and reliable method of characterizing gravelly soils is an important consideration in geotechnical engineering.

The Dynamic Penetration Test (DPT) has the potential to be more cost-effective, reliable, and accessible than other procedures currently in use for evaluating the liquefaction potential of
gravelly soils (Cao et al., 2013). The DPT test uses a similar procedure to that of the SPT test. The two primary differences between the DPT and SPT are that the DPT employees a larger cone tip and a heavier hammer. The larger cone tip allows it to move around or break up large gravel particles as it is driven into the ground.

Liquefaction triggering curves based on DPT data have been developed by Cao et al. (2013). These curves provide a boundary between liquefiable and non-liquefiable soils based on the penetration resistance; however, they are based on a limited data set from one earthquake and one geologic environment. Additional data for different earthquake magnitudes, tectonic settings, and depositional environments would improve the existing DPT-based liquefaction triggering curves. The improvement in gravel liquefaction prediction depends on accurate and reliable data from case histories where gravels did and did not liquefy. While the case histories reported for gravel liquefaction in Table 1 provide gravel liquefaction data points, case histories from non-liquefied gravel sites are also critical to constrain the boundary. To this end, the focus of this study is to collect and analyze data at sites that did not liquefy during the 1964 Mw 9.2 earthquake in Alaska and the 2009 Mw 6.3 earthquake in L’Aquila, Italy. These new non-liquefaction case histories will help to improve DPT-based liquefaction triggering curves.

1.2 Objectives

1. Develop seven new non-liquefaction case histories for the DPT based on field test data collected at two sites in Valdez, Alaska and five sites in L’Aquila, Italy.

2. Compare the probability of liquefaction equation developed by Cao et al. (2012) with the probability of liquefaction equation developed by Roy (2021).
3. Compare the shear wave velocity from DPT correlations developed by Roy (2021) with the MASW shear wave velocity at locations where MASW data is available.

Table 1.1. List of earthquakes with reports of gravel liquefaction

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Year</th>
<th>$M_w$</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mino-Owari, Japan</td>
<td>1891</td>
<td>7.9</td>
<td>Tokimatsu and Yoshimi (1983)</td>
</tr>
<tr>
<td>Fukui, Japan</td>
<td>1948</td>
<td>7.3</td>
<td>Ishihara (1985)</td>
</tr>
<tr>
<td>Alaska</td>
<td>1964</td>
<td>9.2</td>
<td>Coulter and Migliaccio (1966)</td>
</tr>
<tr>
<td>Friuli, Italy</td>
<td>1976</td>
<td>6.4</td>
<td>Sirovich (1996a, b), Rollins et al. (2020)</td>
</tr>
<tr>
<td>Miyagiken-Oki, Japan</td>
<td>1978</td>
<td>7.4</td>
<td>Tokimatsu and Yoshimi (1983)</td>
</tr>
<tr>
<td>Armenia</td>
<td>1988</td>
<td>6.8</td>
<td>Yegian et al. (1994)</td>
</tr>
<tr>
<td>Roermond, Netherlands</td>
<td>1992</td>
<td>5.8</td>
<td>Maurenbrecher et al. (1995)</td>
</tr>
<tr>
<td>Hokkaido, Japan</td>
<td>1993</td>
<td>7.8</td>
<td>Kokusho et al. (1995)</td>
</tr>
<tr>
<td>Kobe, Japan</td>
<td>1995</td>
<td>7.2</td>
<td>Kokusho and Yoshida (1997)</td>
</tr>
<tr>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>7.8</td>
<td>Chu et al. (2000)</td>
</tr>
<tr>
<td>Muisne, Ecuador</td>
<td>2016</td>
<td>7.8</td>
<td>Lopez et al. (2016)</td>
</tr>
<tr>
<td>Kaikoura, New Zealand</td>
<td>2016</td>
<td>7.8</td>
<td>Cubrinovski et al. (2017)</td>
</tr>
</tbody>
</table>

1.3 Scope

To accomplish the objectives, I performed two DPT soundings in Valdez, Alaska. Five additional soundings were performed by Prof. Rollins, Prof. Sara Amoroso, and Anthony Rosas in L’Aquila, Italy. I then used the data collected at each of the seven sites to calculate various parameters pertinent to soil liquefaction. For comparison, I calculated the probability of liquefaction and factor of safety against liquefaction based on the probability of liquefaction equation developed by Cao et al. (2013), and the probability of liquefaction equation developed...
by Roy (2021). When shear wave velocity data was available, I compared it against the shear wave velocity obtained from a correlation equation with DPT penetration resistance developed by Roy (2021). The data collected as part of this study can be added to existing liquefaction triggering curve datasets to improve their reliability and accuracy.
2 Methods for Evaluating Gravel Liquefaction

2.1 Standard Penetration Test (SPT) & the Cone Penetration Test (CPT)

The Standard Penetration test and the Cone Penetration test are commonly used to determine the liquefaction resistance of sands and fine-grained soils (Cao et al., 2013). As mentioned, the SPT and CPT tests do not provide reliable results when used to determine liquefaction resistance in gravelly soils. Although liquefaction may be correctly predicted when the gravel is loose, liquefaction resistance may be overestimated as the density increases (Cubrinovski et al., 2018; Dhakal et al., 2019; Dhakal et al., 2020). When the SPT split spoon sampler and the CPT cone tip reach a soil layer containing gravelly soil, they will likely report a resistance that is artificially high (Cao et al., 2013; Dhakal et al., 2020). This occurs because the split spoon sampler of the SPT and the cone tip of the CPT cannot easily break up or move aside the larger particles found in gravelly soils. In this case it becomes difficult to determine if the increased resistance is a result of increased density or interference due to gravel-sized particles. This increased resistance may not be representative of the resistance of the soil (Cao et al., 2013; Kulhawy et al., 1990).
2.2 Becker Penetration Test (BPT)

The Becker Penetration Test (BPT) was developed in the 1950s, and has become the primary test used to determine penetration resistance of gravelly soil in North America (Rollins, 2018). The testing apparatus is a 168-mm diameter, 3-m long, double walled casing. The penetration resistance is defined as the number of blows required to drive the casing 30 cm into the soil being tested. A correlation between BPT blow counts and SPT blow counts in sand was developed by Harder and Seed to facilitate the use of the BPT test in liquefaction evaluation (Harder Jr, 1997; Harder et al., 1986). The equivalent SPT blow count in sand must be corrected for Becker bounce chamber pressure and atmospheric pressure at the elevation where the test is performed. The corrected SPT blow count in sand is referred to as the $N_{BC}$ value. The $N_{BC}$ blow count is then used to determine the $N_{60}$ value using a correlation developed by Harder and Seed (Harder Jr, 1997; Harder et al., 1986). The $N_{60}$ blow count can then be corrected for energy and overburden pressure to produce the commonly used $(N_1)_{60}$ blow count. The SPT $(N_1)_{60}$ value can be used in the procedure suggested by Youd et al. (2001) to determine the cyclic resistance ratio (CRR) which in turn can be used to evaluate the liquefaction resistance of the soil. In order to apply the BPT in the evaluation of liquefaction resistance of gravelly soils, the BPT blow count must be converted to an equivalent SPT $(N_1)_{60}$ value through a series of corrections and a correlation. The correlation used between the BPT and SPT introduces uncertainty into the results of the liquefaction assessment (Rollins, 2018).

In addition to the uncertainties introduced from correlating BPT values with SPT values, correcting for friction resistance against the casing adds another level of complexity to the BPT. The increased complexity and specialized equipment required to conduct the BPT results in a high cost of mobilization (Rollins, 2018). The uncertainties and high cost of mobilization of the
BPT test create the need to develop a cheaper, more reliable test for liquefaction assessment in gravelly soils.

2.3 Dynamic Penetration Test

2.3.1 Test Background

The Dynamic Penetration test was developed in China in the 1950s. The DPT consists of driving a 74-mm cone with a 120-kg weight dropped from a height of 100-cm. The hammer falls on an anvil that is attached to a 60-mm rod. The 74-mm cone attaches to the end of the 60-mm rod. Advantages of the DPT are that it has a relatively low cost of mobilization, it provides a direct correlation with liquefaction in gravel from field tests, and it can accurately characterize gravelly soils (Cao, 2019; Cao et al., 2013; Rollins et al., 2019). The resistance of the soil is measured as the number of blows required to drive the cone 10-cm into the soil. Chinese code also specifies another resistance value called N_{120}. The N_{120} value is the number of blows to drive the cone 30-cm into the soil (Code, 2001). This value is obtained simply by multiplying the number of blows to drive the cone 10-cm by three. At sites where a drill rig other than the drill rig used in China is used an energy correction must be applied to produce an equivalent N_{120} value. Figure 2.1 shows a diagram of the DPT including dimensions (Cao et al., 2013).

The DPT testing procedures have been standardized by Chinese design code (Chinese Design Code, 2001). This is advantageous because it allows engineers to apply a codified procedure when performing the DPT lending to consistent results.
The low cost of mobilization is due to the fact that many drill rigs in North America are equipped with most of the hardware necessary to perform a DPT test. In many cases, the only additional equipment needed to perform a DPT test is the DPT cone and an adapter to connect the cone to the 60-mm rod of the drill rig. In addition, the procedure for performing a DPT test is relatively simple, reducing the amount of training required.
2.4 Development of the DPT-Based Liquefaction Triggering Curve

In May of 2008 a 7.9 $M_w$ earthquake struck the Chengdu plain in southwestern China. The epicenter of the earthquake was located in Wenchuan County and devasted much of the Sichuan Province. The earthquake killed more than 100,000 people. The Chengdu plain in particular was devasted by liquefaction during the earthquake. A team of geotechnical specialists immediately began the work of investigating occurrences of liquefaction and the resulting damage. In total 118 sites that exhibited surface manifestations of liquefaction were identified and documented. Of the sites that were identified and documented 47 were drilled and tested. Nearly all of the sites where liquefaction was identified were underlain by loose gravels at shallow depths. The majority of these sites were located in the Chengdu plain (Cao et al., 2013).

The determination of liquefaction potential in gravelly soils can be difficult as discussed. The DPT was the preferred method for the characterization of the gravelly soils found in the Chengdu plain. The DPT is widely used in China and had already been in regular use in the Chengdu region prior to the 2008 earthquake due to the prevalence of gravelly soil found there. The DPT soundings that were conducted in the Chengdu plain following the 2008 earthquake were the first DPT soundings conducted in gravelly soils that are known to have liquefied (Cao et al., 2013). The 47 sites where DPT soundings were performed were split between sites that did exhibit surface manifestations and those that did not. Of the 47 sites tested, 19 exhibited surface manifestations of liquefaction and 28 did not. The determination of liquefiable layers in the soil profile is dependent on the grain size of the material. The DPT test does not produce soil samples making it impossible to determine which layers in the soil profile liquefied. This problem was overcome by the nearly continuous collection of soil samples in core holes within two meters of
the majority of the DPT soundings. The samples were obtained using 90-100-mm core barrels (Cao et al., 2013).

An important consideration when performing the DPT is the energy that is transferred to the drill rod. Cao et al. (2013) used a pile driving analyzer to determine the energy transfer ratio. The energy transfer ratio expressed as a percentage, is the percentage of the theoretical energy that is transferred to the drill rod. Cao et al. (2013) found that the drill rig used for the soil investigation performed after the 2008 Wenchuan earthquake had an energy transfer ratio of 85% when the hammer was dropped at a height of 100-cm. The energy transfer ratio is different for different drilling rigs. For this reason, it is important to quantify the energy transfer ratio of the drill rig being used when performing the DPT.

The critical layer was then determined using the DPT blow counts and the collected soil data. Three rules were applied to each set of data to determine the critical layer for liquefaction.

1. “Fine-grained clayey soils and soils above the water table were classed as nonliquefiable.” (Cao et al., 2013)

2. “The penetrated sediments were divided into layers based on soil type, saturation, (above or below the water table), and uniformity of $N_{120}$ values.” (Cao et al., 2013)

3. “For sites with surface-liquefaction effects, the layer below the water table with the lowest general $N_{120}$ values was identified as the layer that liquefied. For sites without surface-liquefaction effects, the layer below the water table with the lowest average $N_{120}$ was identified as the most liquefiable layer, but was assumed not to have liquefied” (Cao et al., 2013)
To account for the differences between drilling rigs and each site, two corrections must be made to the DPT blow counts recorded during each sounding. The two corrections that need to be applied to the raw blow counts are 1) for the energy delivered, and 2) for the effects of overburden pressure. The energy correction accounts for differences in the hammer energy delivered to the rod in the soundings performed as part of this study ($E_{\text{Delivered}}$) and the energy delivered using the standard Chinese DPT hammer and drop height ($E_{\text{Chinese DPT}}$). A correction for this difference in hammer energy was developed by Seed (1985) and is given by:

$$N_{120} = N_{\text{Measured}} \frac{E_{\text{Delivered}}}{E_{\text{Chinese DPT}}}$$  (1)

Where $N_{\text{Measured}}$ is the number of hammer blows required to drive the DPT cone 10-cm multiplied by three. $E_{\text{Delivered}}$ is the energy delivered to the drill rod during the sounding, while $E_{\text{Chinese}}$ represents the energy that the standard Chinese DPT hammer delivers to the drill rod. The $N_{120}$ value represents the equivalent Chinese DPT blow count for each interval of the subsurface tested. The $E_{\text{Delivered}}$ and $E_{\text{Chinese DPT}}$ terms can be calculated using the equation shown below.

$$E = Efficiency \times Drop \ Weight \times Drop \ Height$$  (2)

The Efficiency term in the equations above is the measured hammer energy transferred to the rod divided by the theoretical free-fall hammer energy. A study conducted by Cao et al. (2013) concluded that the Chinese DPT transferred approximately 89% of the energy to the drill rod. The standard weight and drop height for the Chinese DPT test are 120-kg and 100-cm respectively.

Like the SPT, the DPT blow counts also must be corrected for the effects of overburden pressure. Liao et al. (1986a) developed an overburden pressure correction factor $C_n$, which is given by the equation:
\[ C_n = \left( \frac{100}{\sigma'_{o}} \right)^{0.5} \leq 1.7 \]  \hspace{1cm} (3)

where \( \sigma'_{o} \) is the initial vertical effective stress in kPa and 100 is atmospheric pressure in kPa. The limit of 1.7 was suggested by Rollins et al. (2018) to provide consistency with corrections using SPT and CPT testing.

The \( N_{120} \) blow count is corrected for overburden pressure by multiplying the \( N_{120} \) value by the \( C_n \) correction factor. The corrected blow count is referred to as \( N'_{120} \). The equation to correct for overburden pressure is given by:

\[ N'_{120} = N_{120} \times C_n \]  \hspace{1cm} (4)

Seed et al. (1971) developed the simplified method for determining the cyclic strain ratio (CSR). The equation used to determine the CSR is shown below.

\[ CSR = 0.65 \left( \frac{a_{max}}{g} \right) \left( \frac{\sigma'_v}{\sigma_v} \right) r_d \]  \hspace{1cm} (5)

Where the \( a_{max} \) is the peak ground acceleration, \( g \) is the gravitational acceleration constant, \( \sigma_v \) is the initial vertical total stress, \( \sigma'_v \) is the initial vertical effective stress, and \( r_d \) is a reduction factor for depth. The \( r_d \) reduction factor was developed by Youd et al. (2001).

The CSR and \( N'_{120} \) data gathered in the Chengdu plain were used to develop a liquefaction triggering curve (Cao et al., 2013). Figure 2.2 shows the liquefaction triggering curve as well as the data points that were collected during the post-earthquake soil investigations (Cao et al., 2013).
The curved lines shown in Figure 2.2 represent the probabilistic liquefaction curves derived by Cao et al. (2013). Equation 6 provides the probability of liquefaction ($P_L$) based on a correlation with $N'_{120}$ and CSR values developed.

$$PL = \frac{1}{1 + \exp\left\{-\left(8.51 - 0.36N'_{120} + 2.21\ln\text{CSR}(M_w = 7.9)\right)\right\}}$$

(6)

The $M_w = 7.9$ shown in equation 6 indicates that the equation is only valid for a 7.9 moment magnitude earthquake. The original equation developed by Cao et al. (2013) has been modified to a 7.5 moment magnitude earthquake using the magnitude scaling factor developed by Youd et al. (2001) given by:

$$MSF = \frac{10^{2.24}}{M_w^{2.56}}$$

(7)
The probability of liquefaction equation that was developed by Cao et al. (2013) only includes data points from the soil investigations that were performed in the Chengdu plain after the 2008 Wenchuan, China earthquake. The limited data are problematic when attempting to use the triggering curves for locations outside of the Chengdu plain. Additional data points are needed in order to further constrain the liquefaction triggering curve. Data points with a high CSR that help to constrain the upper portion of the triggering curve are especially needed.

2.5 DPT-Based Liquefaction Triggering Curve Developed by Roy (2021)

The second equation that was used to determine the probability of liquefaction, as part of this study, was developed by Roy (2021) based on a regression of a larger set of DPT test results from around the world and is given by the equation:

$$ PL = \frac{1}{1 + \exp(0.0013N^{1.3} - 1.9M_w - 6.35\ln CSR)} $$

(8)

where \( M_w \) is the moment magnitude and CSR is the cyclic stress ratio. The CSR is obtained from the simplified equation originally developed by Seed and Idriss (1971). However, because the magnitude of the earthquake is a variable in the function, it is not necessary to use a magnitude scaling factor. The CSR can be scaled to a 7.5 \( M_w \) event if desired using the MSF equation developed by Roy (2021) as given by:

$$ MSF = 9.43e^{-0.3M_w} $$

(9)

If the CSR is scaled to a 7.5 \( M_w \) earthquake, then 7.5 must be used in the probability of liquefaction equation as the moment magnitude. There are two primary differences between the equation developed by Cao et al. (2013) and the equation developed by Roy (2021). The first
major difference is that the moment magnitude of the earthquake is a variable in the equation developed by Roy (2021). The second major difference is that the equation developed by Roy (2021) incorporates DPT data from a variety of sites around the world with where gravel liquefaction has been observed.

2.6 Factor of Safety Against Liquefaction Calculations

Like the probability of liquefaction, the factor of safety against liquefaction is a measure of the liquefaction resistance of the subsurface. The factor of safety against liquefaction (FS) is provided by the simplified procedure originally developed by Seed et al. (1971) and is defined by the equation:

\[
FS = \left( \frac{CRR_{7.5}}{CSR} \right) MSF
\]  

(10)

where CRR\textsubscript{7.5} is the cyclic resistance ratio for an equivalent 7.5 moment magnitude event, CSR is the cyclic stress ratio, and MSF is the magnitude scaling factor. The Cyclic Stress ratio is equal to the cyclic shear stress developed by the earthquake (\(\tau_c\)) over the initial vertical effective stress (\(\sigma'_o\)) prior to the earthquake. The equations for CSR as proposed by Youd et al. (2001) is given by:

\[
CSR = 0.65 \left( \frac{a_{max}}{g} \right) \left( \frac{\sigma_v}{\sigma'_v} \right) \frac{r_d}{MSF}
\]  

(11)

where \(r_d\) is a stress reduction factor, \(M_w\) is the moment magnitude of the earthquake, \(a_{max}\) is the peak ground acceleration, and MSF is the magnitude scaling factor. The \(r_d\) term accounts for flexibility in the soil profile as proposed by Seed et al. (1971). For non-critical sites, \(r_d\) versus depth (\(z\)) in meters can be calculated with the equations developed by Liao et al. (1986b):
In this case, the equation for CRR was developed by assuming that a 30% probability of liquefaction correlates with a factor of safety against liquefaction of one (Liao et al., 1988). The CRR equation was determined by setting the probability of liquefaction $P_L$ equal to 0.3, then solving for CSR. This produced an equation for the CSR value that would produce a 30% probability of liquefaction. The resulting CSR equation was then used in the factor of safety against liquefaction equation. Since the CSR in this case represents a factor of safety of 1, CRR is equal to CSR. By this methodology the CRR equation was developed based on the CSR, probability of Liquefaction, MSF, and FS equations described previously. Equation 13 shows the equation for CRR based on the probability of liquefaction equation developed by Cao et al. (2013) using the MSF as suggested by Rollins et al. (2018).

$$CRR = \frac{0.0128}{e^{-0.35*N_{120}}^{0.473}}$$ (13)

The same process was followed to develop a second CRR equation based on the probability of liquefaction equation developed by Roy (2021). The CRR based on the probability of liquefaction equation developed by Roy (2021) is shown in equation 14.

$$CRR = \frac{0.875}{e^{(1.9*M_w-0.00113*N_{120}^{3})^{0.157}}}$$ (14)

2.7 Shear Wave Velocity ($V_s$)

There are various methods available to obtain shear wave velocity data including the multichannel analysis of surface waves (MASW) procedure, the cross-hole procedure, the down-
hole procedure, and the seismic dilatometer test (SDMT) procedure. As part of the study performed by Cao et al. (2013) described in section 2.4 shear wave velocity data were gathered at 47 sites in the Chengdu plain utilizing the MASW procedure (Cao et al., 2011). Of the 47 sites where $V_s$ data were gathered 30 exhibited surface manifestations of liquefaction and the remaining 17 did not. A probability of liquefaction equation based on shear wave velocity developed by Zhaoji (Shi et al., 1993; Zhaoji, 1986) for use in sandy soils was evaluated by Cao et al. (2011) to determine its effectiveness at predicting liquefaction resistance in gravelly soils at the 47 sites in the Chengdu plain. Cao et al. (2011) found that the probability of liquefaction equation developed by Zhaoji (Shi et al., 1993; Zhaoji, 1986) predicted liquefaction at 25% of the sites where surfaces manifestations of liquefaction were present and it predicted no liquefaction at 88% of the sites where no surface manifestations were observed (Cao et al., 2011). This shows that the probability of liquefaction equation developed by Zhaoji (Shi et al., 1993; Zhaoji, 1986) did not accurately predict liquefaction at the 47 sites in the Chengdu plain. This is likely due to the fact that the probability of liquefaction equation developed by Zhaoji (Shi et al., 1993; Zhaoji, 1986) was developed for use in sandy soils. These findings demonstrate the need for a shear wave velocity-based liquefaction triggering curve specifically for gravelly soils.

Cao et al. (2011) used the shear wave velocity data collected after the 2008 Wenchuan earthquake to create a liquefaction triggering curve. The triggering curve based on shear wave velocity data at the 47 sites in the Chengdu plain is shown in Figure 2.3. As can be seen in Figure 2.3 the shear wave velocity-based triggering curves developed by Cao et al. (2011) are based on a rather sparse data set. In addition, all of the data points for the curves came from the same geologic setting and the same magnitude earthquake. These two facts limit the reliability of the
shear wave velocity-based triggering curve developed by Cao et al. (2011). The addition of data points from sites of varied geologic composition and different earthquake events would serve to improve the accuracy of the triggering curve.

The probability of liquefaction equation developed by Cao et al. (2011) is given by:

\[
PL = \frac{1}{1 + \exp[-(\theta_0 + \theta_1 V_{S1} + \theta_2 \ln(CSR))]} \tag{15}
\]

where \(\theta_0\), \(\theta_1\), and \(\theta_2\) are parameters obtained by regressing the 47 \(V_s\) data points developed by Cao et al. (2011) using the logistics method. The regression performed by Cao et al. (2011) produced the values of 11.97, -0.039, and 1.77 for \(\theta_0\), \(\theta_1\), and \(\theta_2\), respectively.

Figure 2.3. Shear wave velocity based probabilistic liquefaction triggering curves for gravels. (Cao et al., 2011)
2.8 CPT Based Liquefaction Triggering Curve Comparison

To illustrate the need for additional DPT data at gravel liquefaction sites, a CPT-based liquefaction triggering curve for sandy soils developed by Boulanger et al. (2014) was included in this report and is shown in Figure 2.4. The CPT-based curves are based on a much larger data set than the DPT-based curves. When compared against the DPT-based triggering curve developed by Cao et al. (2013) shown in Figure 2.2, there is a clear difference in the quality of the triggering curves. The gap between the 85% and 50% probability of liquefaction curves shown in the CPT-based triggering curve is relatively small. While the gap between the 85% and 50% probability of liquefaction curves shown in the DPT-based triggering curve is much larger when compared with the CPT-based triggering curve. Because the CPT-based triggering curve contains more data points, errors associated with an individual data point have less impact on the overall curve shape than when a smaller dataset is used. This is likely the reason for the higher quality seen in the CPT-based triggering curve.

As noted previously, the DPT-based triggering curve developed by Cao et al. (2013) has various limitations, namely the lack of data points and the lack of variety in the geologic setting and earthquake magnitude of the data points. The CPT-based triggering curve developed by Boulanger et al. (2014) triggering curve, with far more data points, is an example of what future DPT triggering curves may look like. Increased DPT data from varied geologic settings and earthquake events should allow future DPT-based triggering curves to have a smaller gap between the 50% probability and the 85% probability similar to what is seen in the CPT-based triggering curve shown in Figure 2.4.
2.9 Excess Pore Water Generation in Gravelly Soils

The primary cause of liquefaction is the generation of excess pore water pressure when a granular material is subject to cyclic shear stresses. The grain size of a soil has a significant effect on its ability to disperse excess pore water pressure. Based on numerical analyses, Seed et al. (1976) determined that soils with a hydraulic conductivity greater than 0.3 cm/sec are not likely to liquefy because of their ability to quickly disperse excess pore water pressure.

Figure 2.4. CPT-based probabilistic liquefaction triggering curves for sand (Boulanger et al., 2014)
However, in some cases a soil with a hydraulic conductivity greater than 0.3 cm/sec may still liquefy (Chen et al., 2018; She et al., 2006).

If the soil with a high hydraulic conductivity is covered by an impermeable cap of clay or concrete, the excess pore water pressure may not be able to dissipate quickly enough to prevent liquefaction (Chen et al., 2018). A few of the sites included in this study meet this criteria.

The presence of sand in a gravelly soil has also been shown to decrease the hydraulic conductivity enough for liquefaction to occur. According to the Unified Soil Classification system a soil that is classified as gravel may be comprised of up to 49% sand. According to She et al. (2006), a gravel-sand material with a sand content of 30% or more may have a lower hydraulic conductivity, similar to that of the sand fraction despite being classified as a gravel because the sand fills the void space in the gravel and reduces hydraulic conductivity.
3 Soil Parameters Calculated

3.1 Relative Density

Relative density is normally determined using the SPT or the CPT. In the case of gravelly soils, relative density can be hard to determine with the SPT or CPT because of the large grain size. A correlation between \( N'_{120} \) and relative density provides a reliable method for determining relative density in gravelly soils. The equation for relative density as proposed by Rollins et al. (2020) is given by:

\[
D_r = \left( \frac{N'_{120}}{70} \right)^{0.5}
\]

At some of the sites there are SPT and CPT data available from previously performed soil investigations. When available, I calculated the relative density based on SPT and CPT and plotted them with the \( N'_{120} \) relative density correlation for comparison. The CPT based relative density was prepared by i.m.o.s. a geotechnical firm that preformed a soil investigation at the Ponte Rosarolo water treatment plant in L’Aquila, Italy. The team at i.m.o.s. calculated the relative density based on the CPT data using a correlation developed by Schmertmann (1976). As suggested by Kulhawy et al. (1990) a correction factor for grain size was used to determine
relative density where SPT data were available. The equation used to calculate the relative density fraction based on SPT blow counts is given by:

\[ D_r = \left[ \frac{(N_1)_{60}}{60 + 25 \log(D_{50})} \right]^{0.5} \]  

(17)

where \( D_{50} \) is particle size where 50% is finer. I also calculated the relative density based on SPT \((N_1)_{60}\) blow counts without a correction for grain size as proposed by Kulhawy et al. (1990). Equation 18 does not include a correction for grain size and will likely predict a higher relative density in gravelly soils than the actual relative density of the soil.

\[ D_r = 100 \sqrt[60]{\frac{(N_1)_{60}}{60}} \]  

(18)

3.2 Shear Wave Velocity

Correlations between SPT \((N_1)_{60}\) blow counts and CPT cone tip resistance and shear wave velocity exist. However, it is likely that these correlations would predict higher shear wave velocity than the actually shear wave velocity at sites with gravelly soils. For this reason, the development of a DPT based shear wave velocity correlation is an important consideration.

The DPT-based shear wave velocity correlation we used was developed by Roy (2021) using DPT, MASW, down borehole, and cross borehole data at 75 sites. The DPT-\(V_s\) correlation has an \(R^2\) value of 0.53. The equation relating shear wave velocity to the DPT \(N'_{120}\) value is given by:

\[ V_{S1} = 4.94N'_{120} + 154.5 \]  

(19)
The low $R^2$ value of the DPT shear wave velocity correlation developed by Roy (2021) suggests that the correlation only accounts for 53% of the variation in shear wave velocity. The addition of more data to the data set that the correlation is based on may increase its accuracy.
4 1964 Alaska Earthquake Effects in Valdez, Alaska

4.1 Earthquake Magnitude and Peak Ground Acceleration in Valdez, Alaska

On March 27, 1964, the second largest instrumented earthquake in history occurred in Prince William Sound near Valdez, Alaska (Mavroeidis et al., 2008). The location of the epicenter, Valdez, and Old Valdez are shown in Figure 4.1. Old Valdez refers to the location of the town of Valdez prior to the 1964 earthquake and Valdez refers to the current location of the town of Valdez.

The epicenter shown in Figure 4.1 is the location of initial subsurface rupture that caused the 1964 Alaska earthquake. The map shown in Figure 4.3 more accurately shows the scale of the earthquake. The fault that caused the 1964 Alaska earthquake is a subduction fault, with the Pacific Plate subducting underneath the North American Plate. As the Pacific Plate moves Northwest and subducts under the North American Plate, friction causes the leading edge of the North American plate to compress. The rupture of the fault line during the 1964 earthquake released the built-up strain energy and caused an area of subsidence and an area of uplift. The earthquake caused vertical uplift of an area of about 520,000 square kilometers. The vertical
displacements ranged from 11.5 meters of uplift to 2.3 meters of subsidence relative to sea level. (Stover et al., 1993)

Unfortunately, at the time of the 1964 earthquake, there were no seismographs located in the vicinity of Valdez. The PGAs for the sites in Valdez were obtained from the USGS Shakemap for the event. USGS Shakemaps incorporate a weighted-average approach for combining different types of data (e.g., recordings, intensities, ground motion prediction equations) to arrive at best estimates of peak ground motion parameters.

Boulanger and Idriss (2014) used Shakemaps to check PGA estimates for a number of sites in their liquefaction case history database with no nearby recordings. The Shakemap showing contours of PGA for the 1964 Alaska earthquake in the vicinity of Valdez is shown in Figure 4.2. The two sites in Valdez are relatively close, for this reason, the same PGA was used for both sites. The PGA that was obtained for the USGS Shakemaps is 0.44g where g is the gravitational acceleration constant. The PGA of 0.44g was used in the development of the CSR for Sites 1 and 2 in Valdez.

The USGS Shakemaps that were used to determine the PGA at the two sites in Valdez use empirical attenuation relationships to predict ground motion at what are referred to as phantom grid points. Phantom grid points are points on a uniformly spaced grid that overlays the
area of interest where recorded ground motion data is not available. The empirical attenuation relationship used depends on the region the Shakemap is being created for. The attenuation relationship used for sites in Alaska was developed by Youngs et al. (1997).

The ground motion predicted by the attenuation relationship is for rock. In order to interpolate between grid points where ground motion data is available and the phantom grid points where data is predicted by attenuation relationships the measured ground motions must be converted to equivalent ground motions in rock. After the measured ground motions have been converted to equivalent ground motions in rock, ground motion values are interpolated at 1.5-km spacing. The new interpolated predicted ground motions for rock are then amplified at each point for local site amplification. (Wald et al., 2005) The USGS Shakemaps have various limitations and should be used with caution. Two limitations of the Shakemaps produced by the USGS that are pertinent to this study are:

- “ShakeMaps are automatic computer generated maps that have not necessarily been checked by human oversight. Because the input data is raw and unchecked, the maps may contain errors. The maps are preliminary in nature and will be updated as data arrives from distributed sources.’’(Wald et al., 2005)

- “Interpolation, contouring, and color-coding can be misleading because data gaps may exist. Caution should be used in deciding which features in the contour patterns are required by the data. Ground-motions and intensities can vary greatly over small distances, so these maps are only approximate; at small scales and away from data points, they may be unreliable.” (Wald et al., 2005)

The shape of the Shakemap contours shown in Figure 4.2 and the shape of the uplift and subsurface rupture area shown in Figure 4.3 are relatively similar in size and shape. The
Shakemaps produced by the USGS are predicting the highest ground accelerations in the areas where uplift and subsurface rupture occurred which is what would be expected.

Figure 4.1. Map of Alaska showing the epicenter of the March 27, 1964 Alaska earthquake and the location of New and Old Valdez. (Google Maps 2020)

Figure 4.2. ShakeMap for 9.2 Mw 1964 Alaskan Earthquake event (ShakeMap by USGS https://earthquake.usgs.gov/earthquakes/eventpage/official19640328033616_30).
4.2 Geological Setting

Valdez is set on the outcrop belt of the Valdez Group of Late Cretaceous age (Coulter et al., 1966). The rocks within Valdez, and in its vicinity, have been characterized as Interbedded
slate and graywacke, with small amounts of argillite, arkosic sandstone, and conglomerate (Moffit, 1954).

The types of natural material located in the vicinity of Valdez are shown in Figure 4.4 (Coulter et al., 1966). The predominant material found in Old Valdez, labeled Valdez in Figure 4.4, is outwash delta, containing silty sand and gravel. The outwash delta that Old Valdez was built on was deposited by the Lowe river and streams originating from Valdez Glacier, located 4.5 miles to the Northeast.

![Figure 4.4. Distribution and nature of material in Valdez and vicinity. (Coulter et al., 1966)](image-url)
The town of Valdez, labeled proposed town site in Figure 4.4, is predominantly an alluvial fan, containing coarse gravel and sand. The alluvial fan that makes up the majority of the subsurface of Valdez was deposited by Mineral Creek. Two other predominant subdivisions of material are found in and around Valdez, bedrock and tidal flats. As shown in Figure 4.4, the tidal flats are predominantly composed of silty sand and organic mud (Coulter et al., 1966).

4.3 Description of Damage and Lateral Spreading in Old Valdez

The town of Old Valdez, Alaska, before the 1964 earthquake, was located on the eastern end of the Valdez arm of Prince William Sound. The town of Old Valdez was located in the glacial outwash delta of Valdez glacier (Coulter et al., 1966). As explained, the town was built on soil that was primarily silty outwash sand and gravel.

The silty sand and gravel, that the town of Old Valdez was built on, is particularly susceptible to liquefaction. The intense ground motion produced by the 1964 earthquake triggered a submarine landslide that is estimated to have involved 100 million cubic yards of material (Coulter et al., 1966). The photos shown in Figure 4.5 are before and after photos of the Port facility located in Old Valdez. The photo on the right was taken on June 23, 1964, three months after the earthquake. The photo on the left was taken on September 23, 1963, six months before the earthquake. As can be seen by comparing the photos, the docks at the Port of Old Valdez were almost entirely destroyed by the submarine landslide.
The landslide was caused by cyclic shear stresses resulting in a contraction of the sandy gravelly material that underlaid the port facility. As the soil contracted the pore water pressure increased until the vertical effective stress essentially equaled zero, causing the material to behave like a heavy fluid. The liquefied soil did not have enough residual shear strength to resist the gravitational shear stress trying to pull the mass of soil into the Sound. The result was a massive 1500 m long slide with a volume of $4 \times 10^8$ cubic meters (Lee et al., 2007; Parsons et al., 2014). The mass slid into Prince William Sound displacing a significant amount of water.

Coulter et al. (1966) cite three factors that contributed to the massive landslide. The first factor is liquefaction, which resulted in a loss of shear strength in the soil. The liquefaction induced lateral spreading is likely the primary cause of the submarine landslide. The second factor was the reported sudden withdrawal of the water from the shoreline reducing the
hydrostatic head on the soil and increasing the weight of the exposed soil layers. The third factor cited was that the earthquake happened at low tide (Coulter et al., 1966).

A sequence of four major waves hit the town of Old Valdez. The first two waves were likely caused by the massive marine landslide. The first of which was reportedly 30 to 40-ft tall (Coulter et al., 1966). The sequence of waves, and particularly the first two waves, caused extensive damage throughout the town of Old Valdez.

In addition to the landslide and resulting waves, widespread liquefaction occurred across the town of Old Valdez. Fissures and sand boils destroyed foundations and broke water and utility lines In the case of the high school, the foundation was split in half (Coulter et al., 1966).

4.4 Relocation of Old Valdez to Current Location

Due to the widespread damage described above, the decision was made to relocate the town of Old Valdez. The area labelled “proposed town site” in Figure 4.4, was determined to be the best area to which the town could be relocated. The proposed town site area was chosen for relocation for three reasons. The proposed town site is underlain by coarse alluvial gravel, a material that is generally more stable than material found in an outwash delta, where the original town was located (Coulter et al., 1966). Outcroppings of bedrock along the waterfront of the proposed town site also provide protection from sea waves. The outcroppings also act as a buttress and protect the toe of the alluvial fan from sliding or slumping (Coulter et al., 1966). The last reason that the proposed town site was selected was the absence of evidence of ground breakage. The lack of ground breakage at the proposed town site indicates that the underlying soil reacts favorably to strong ground motions (Coulter et al., 1966). The town of Valdez is now located in the area labeled “proposed town site” in Figure 4.4.
4.5 Post-Earthquake Soil Investigations in New Valdez

Following the 1964 earthquake, a soil investigation was performed in Valdez. The goal of the investigation was to determine if the area was a suitable location to relocate the town of Old Valdez to. The soil investigation was performed by Shannon and Wilson.

This soil investigation is important to this study because it provides data that are essential in the determination of the critical layer for the sites in Valdez. The soundings in Valdez were purposely performed near two borings that were performed by Shannon and Wilson in 1964. The borings provide the lithology of the subsurface in the areas where the DPT soundings were performed. Without knowing the composition of the subsurface, it would be impossible to determine whether any given layer is susceptible to liquefaction.

The locations of the borings performed by Shannon and Wilson are shown in Figure 6.1, later in the document. Boring B-3 and B-1 correspond with the DPT soundings performed at Site 1 and Site 2 in Valdez, respectively. The soil data presented later in this report for Site 1 and 2 in Valdez originate from borings B-3 and B-1. In addition to providing soil data, the investigation performed by Shannon and Wilson provides valuable post-earthquake reconnaissance information.

The report created by Shannon and Wilson helps researchers understand the conditions present at the site of Valdez after the 1964 earthquake. The report does not mention any surface manifestations of liquefaction near boreholes B-3 and B-1. This provides evidence that liquefaction did not occur in the vicinity of the soundings performed in Valdez.
5  2009 L’Aquila, Italy Earthquake

5.1 Earthquake Magnitude and Peak Ground Acceleration in L’Aquila, Italy

On April 6, 2009, a 6.3 Mw earthquake struck the L’Aquila basin. A map of Italy with the epicenter and the city of L’Aquila labeled is shown is Figure 5.1. As can be seen on the map, the epicenter was close in proximity to the city of L’Aquila.

The earthquake was caused by a NW-SE trending normal fault. The peak ground acceleration for Sites 1, 2, and 5 were determined using a shake map grid that was created by the Italian National Seismic Network and the U.S. Geological Survey. The PGA at the grid location closest to the sites mentioned above was used to avoid uncertainties in interpolating between grid points. Sites three and four are located near a seismograph station. The PGA for sites three and four is the PGA that was recorded at the nearby seismograph (largest of the two horizontal components). The PGAs used for each of the sites in L’Aquila are shown in Table 5.1.

5.2 Geological Setting

The L’Aquila basin is primarily composed of carbonic and sandstone rocks of the Mesozoic-Cenozoic period (Amoroso et al., 2008). The city of L’Aquila is surrounded by mountains, with
two major waterways, the Aterno river and Raio stream. The lower lying areas of the basin have been filled in with Holocene alluvial fill. The Aterno river and Raio stream are responsible for the alluvial deposits found in the L’Aquila basin. The thickness of the alluvial deposits reach 10-12m and contain gravel mixed with varied amounts of sand, sandy clay, sandy silt, and silty clay of dark color (Amoroso et al., 2008). Figure 5.2 shows a geological map and profile of the L’Aquila basin.

Table 5.1. PGA Values Used for Sites in L’Aquila

<table>
<thead>
<tr>
<th>City</th>
<th>Site</th>
<th>Alias</th>
<th>PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td>L'Aquila</td>
<td>Site 1</td>
<td>Sports Field</td>
<td>0.42</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 2</td>
<td>Japan</td>
<td>0.42</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 3</td>
<td>AQA3</td>
<td>0.44</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 4</td>
<td>AQA4</td>
<td>0.44</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 5</td>
<td>Water Plant</td>
<td>0.41</td>
</tr>
</tbody>
</table>

Figure 5.1. Map of Italy with the Epicenter of the April 6, 2009 earthquake and the city of L’Aquila labeled (Google Maps 2020).
5.3 Description of Damage

The earthquake that struck the L’Aquila basin on April 6, 2009 caused extensive damage to the city of L’Aquila. The earthquake caused 309 fatalities, 1,600 injuries, and displaced an estimated 40,000 people (Monaco et al., 2015). The severity of the damage caused by the 2009 earthquake sparked a significant amount of research. Damage was primarily owing to structural failure and building collapse rather than the ground failure.
Although liquefaction was rare, one case of liquefaction that was caused by the April 6, 2009 earthquake has been studied in the L’Aquila basin. At the site of Ponte Rosarolo adjacent to the Aterno River, sand boils of up to 15-cm thick measured from the existing ground surface to the top of the sand boil were discovered. In addition to sand boils, cracks running parallel to the riverbank of the Aterno river were observed and attributed to liquefaction (Monaco et al., 2015).

The DPT sounding performed at site 5 in L’Aquila is located near the previously performed soil investigation at Ponte Rosarolo. Nevertheless, no surface manifestations of liquefaction were exhibited at site 5 or any of the other sites in L’Aquila.
6 Dynamic Cone Penetration Soundings in Valdez

6.1 Test Locations in Valdez

Two DPT sounding were performed in Valdez, Alaska. The coordinates of the two soundings are presented in Table 6.1. The location of each DPT sounding was selected to be in close proximity to previously performed borings B-3 and B-1, performed by Shannon and Wilson, as shown in Figure 6.1. The positions of boring B-3 and B-1 are approximate, because they were determined from a site map that was created by Shannon and Wilson as part of a report that was published in 1964 and transferred to a modern map. Borings B-3 and B-1 show that the subsurface is primarily composed of sandy gravel. In addition to showing the location of borehole B-3 and B-1, Figure 6.1 shows the locations of the two DPT soundings performed in Valdez.

<table>
<thead>
<tr>
<th>City</th>
<th>Site</th>
<th>Alias</th>
<th>Latitude</th>
<th>Longitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Valdez</td>
<td>Site 1</td>
<td>Motel</td>
<td>61.1286389</td>
<td>-146.3600278</td>
</tr>
<tr>
<td>Valdez</td>
<td>Site 2</td>
<td>Church</td>
<td>61.1310556</td>
<td>-146.3600556</td>
</tr>
</tbody>
</table>
6.2 Test Procedure Valdez Sites

The DPT soundings, located in Valdez, were performed by an Alaska Dept. of Transportation drill crew using a CME 75 drill rig with an automatic hammer. A photograph of the drill rig used in Valdez is shown in Figure 6.2. The drill rig utilizes an automatic hammer that provided more consistent energy delivery. The CME 75 drill rig used in Valdez did not have a
120-kg weight hammer as specified in the Chinese code; however, the drill rig did have a 154.2-kg hammer. To compensate for the heavier hammer, the hammer was dropped from a height of 76.2-cm, as opposed to the 100-cm height that is specified in the Chinese design code (2001). This produced a hammer energy that is similar to that of the hammer energy specified in the Chinese design code (2001). Previously performed energy transfer tests by the Alaska DOT using a pile driving analyzer, found that the CME 75 drill rig delivered around 94% of the theoretical energy to the drill rod. A lighter SPT hammer (63.5 kg) was also available but was not used for the sites in Valdez because of time limitations for performing the tests.

The blow counts were measured for each 10-cm increment of penetration. A grease pencil was used to mark the drill rod at 10-cm increments to ensure blow counts for each increment were accurately counted. The CME 75 drill rig that was used in Valdez did have a lighter SPT hammer, however soundings were not performed with the lighter hammer in Valdez due to time constraints.

Figure 6.2. Photograph of the truck-mounted CME 75 drill rig used in Valdez, Alaska.
6.3 Subsurface Description of Sites in Valdez

The subsurface data for the two sites in Valdez comes from the Shannon and Wilson geotechnical report mentioned above. The Shannon and Wilson report includes a grain size distribution curve for borehole B-3, which is located near Site 1. However, it does not include a grain size distribution for borehole B-1, located near Site 2. The grain size distribution at a depth of 9.1-m in borehole B-3 is shown in Table 6.2. The grain size distribution clearly shows that the subsurface material is primarily gravel at a depth of 9.1-m. Although the grain size distribution for borehole B-1 is not provided, the soil lithology provided shows that the material from 1.8-m to 28-m is primarily medium dense to very dense sand and gravel.

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth Range (m)</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site 1</td>
<td>9.1 - 9.8</td>
<td>85</td>
<td>8.3</td>
<td>6.7</td>
</tr>
</tbody>
</table>

Figure 6.3. Grain size distribution curves for Site 1 in Valdez 9.1 – 9.8 meters. (Shannon and Wilson)
7 Dynamic Cone Penetration Soundings in L’Aquila

7.1 Test Locations in L’Aquila

Five DPT soundings were performed in the L’Aquila basin. The five test locations are shown in Figure 7.1. In addition to DPT soundings, various other tests were performed at each site. Data from previously performed soil investigations are also available at each site. The previously performed investigations showed that the subsurface profile at each of the sites has significant layers of gravelly material. Thus, each of the five sites was chosen for the gravelly soil found there, as well as the lack of surface manifestations of liquefaction following the 2009 earthquake. In addition, these sites were chosen because the previous SPT blowcounts (N\text{60}) and shear wave velocity (Dhakal et al.) values indicated that the critical layers might be close to the liquefaction triggering boundary. Table 7.1 presents the coordinates and aliases of each site. Each site was given an alias based on nearby landmarks to make it easier to differentiate between the test locations.

Figure 7.2 through Figure 7.6 show detailed maps of Sites 1 through 5, respectively in L’Aquila, Italy. Each figure shows the locations of all of the tests performed as part of this study as well as previously performed soil investigations. Generally, there are two DPT tests at each site, labelled
DPT1H and DPT1L. The number refers to the site number, the H refers to heavy hammer, and the L refers to light hammer. The difference between the light and heavy hammer will be explained later.

<table>
<thead>
<tr>
<th>City</th>
<th>Site</th>
<th>Alias</th>
<th>Latitude</th>
<th>Longitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>L'Aquila</td>
<td>Site 1</td>
<td>Sports Field</td>
<td>42.3562029</td>
<td>13.3595175</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 2</td>
<td>Japan</td>
<td>42.3587081</td>
<td>13.3593095</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 3</td>
<td>AQA3</td>
<td>42.3757718</td>
<td>13.338638</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 4</td>
<td>AQA4</td>
<td>42.3755446</td>
<td>13.3392986</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 5</td>
<td>Water Plant</td>
<td>42.3388457</td>
<td>13.3940489</td>
</tr>
</tbody>
</table>

7.2 Test Procedure L’Aquila Sites

The drill rig used in L’Aquila is significantly different than CME 75 drill rig that was used in Valdez. The three primary differences between the two drill rigs are the drop height of the hammer, the weight of the hammer, and the hammer’s lifting mechanism. The weight of the hammer in the Italian drill rig was 120-kg, the drop height of the hammer was 100-cm, and the lifting mechanism was semi-automatic. A previously performed study found that when using the heavy hammer, the Fraste drill rig that was used in L’Aquila, delivers 75% of the theoretical
energy to the drill rod using a PDA. However, when the lighter SPT hammer was used 65% of the theoretical energy was delivered to the drill rod. The lifting mechanism was semi-automatic in that the operator had to push a lever to raise the hammer for each blow. The Fraste drill rig that was used in L’Aquila matches the hammer weight and drop height that is specified in the Chinese design code. A photograph of the Fraste drill rig used in L’Aquila is shown in Figure 7.7.

Figure 7.1. Aerial image of DPT sounding locations in L’Aquila Italy.
Figure 7.2. Map of Site 1 in L’Aquila with location and type of all available data labelled.
Figure 7.3. Map of Site 2 in L’Aquila with location and type of all available data labelled.
Figure 7.4. Map of Site 3 in L’Aquila with location and type of all available data labelled.
Figure 7.5. Map of Site 4 in L’Aquila with location and type of all available data labelled.
Figure 7.6. Map of Site 5 in L’Aquila with location and type of all available data labelled.
The same process of marking the drill rod at 10-cm intervals was employed for the soundings that were performed in L’Aquila. The blow counts required to drive the DPT cone 10-cm into the subsurface were recorded and subsequently multiplied by three to determine the $N_{120}$ value. The $N_{120}$ was then used to calculate various parameters related to soil liquefaction. A lighter SPT hammer was also used to perform DPT soundings at three of the sites in L’Aquila. At two of the sites the light hammer DPT soundings had to be terminated prematurely due to high penetration resistance and fear that the DPT cone tip would break.

Figure 7.7. Photograph of the truck-mounted Fraste drill rig used in L’Aquila Italy.

7.3 Subsurface Description of Sites in L’Aquila

Borings were performed at each of the sites in L’Aquila except for Site 5. Samples taken from the borings were used to determine the composition of the subsurface. The soil data for Site
5 was provided by a previously performed soil investigation. The borings were performed about 2.5 to 3 m to the side of the DPT soundings. Soil boring logs are provided for each site in Figures 17 through 21. The soil profiles are typical of fluvial deposition with alternating layers of gravelly sand and sandy gravel with silt. Many of the profiles indicate a surface fill consisting of silt and or clay that could serve as an impermeable cap on the underlying gravel. Despite close proximity to the Aterno River, the water table was between 3 and 5 m at the time of field investigations.

A gradation was performed on all the samples to determine the grain size distribution and the resulting grain size distribution curves are presented in Figure 7.9 through Figure 7.15. In addition, hydrometer tests were performed in a number of cases with higher fines contents. Based on the soil gradations and visual descriptions, each sample was classified according to the Unified Soil Classification System (USCS). Symbols associated with the soil types are noted on the boring logs.

Table 7.2 shows the results of the gradations at the depth closest to the layer determined to be the critical layer. There is no grain size distribution available in the previously performed soil investigation at site 5, but the lithology provided in the soil investigation shows that the material in the critical layer is primarily comprised of gravelly material. The gradations for Site 1 through 4 show that gravel was the major constituent at, or near, the layer determined to be the critical layer.

Gravel contents ranged from 40 to 68%, while sand contents varied from 11% to 37% (Table 7.2). Site 3, with the lowest percentage of sand (11%) also contained 21% fines. Although gravel was the major component at all sites, the combined percentage of sand and fines for the critical layer at each site varied from a low of 32% at site 3 to a high of 60% at site 2.
Typically, gravel size particles lose grain-to-grain contact when the percentage of sand and fines exceeds about 30%. These results suggest that the gravel particles were likely floating within a silty sand matrix in most of these critical layers. As noted by She et al. (2010), gravel-sand mixtures with sand contents higher than about 30% typically have hydraulic conductivities that are similar to that of the sand and would be susceptible to pore pressure generation during earthquake shaking.

Table 7.2. Grain Size Distribution at Each Site in L’Aquila in the Critical Layer

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth Range (m)</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Fines</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site 1</td>
<td>5.4 - 5.6</td>
<td>53.4</td>
<td>37.1</td>
<td>9.5</td>
</tr>
<tr>
<td>Site 2</td>
<td>2.5 - 2.7</td>
<td>40.3</td>
<td>31.3</td>
<td>28.4</td>
</tr>
<tr>
<td>Site 3</td>
<td>4.6 – 4.8</td>
<td>31.8</td>
<td>22.9</td>
<td>45.3</td>
</tr>
<tr>
<td>Site 4</td>
<td>3.7 - 3.9</td>
<td>59.1</td>
<td>26.4</td>
<td>14.5</td>
</tr>
</tbody>
</table>

The grain size distribution shown in Table 7.2 were determined from samples that were collected at each of the respective sites. Grain size distribution curves were also created as part of this study to allow for a more detailed analysis of each soil layer. The grain size distribution curves are shown in Figure 7.9 through Figure 7.15. The grain size distribution data in the curves shown in Figure 7.9 through Figure 7.14 were analyzed by Prof. Sara Amoroso et al (2020). Table 7.3 shows all of the soil data collected for Sites 1 through 4 in L’Aquila. The grain size distribution data for the curve shown in Figure 7.15 were developed as part of a soil investigation performed by i.m.o.s. a geotechnical engineering company based in Italy. Grain size distribution
data is not available for Site 5 in L’Aquila for the depth that has been determined to be the most likely to experience liquefaction.

The grain size distribution in the critical layer is especially important when considering the liquefaction potential of a soil. Figure 7.8 shows the grain size distribution curves in the critical layer for Sites 1 through 4 in L’Aquila. Figure 7.8 shows the material in the critical layer for Sites 1 through 4 in L’Aquila are primarily comprised of gravel.

![Figure 7.8. Grain size distribution curves in the critical layer for Sites 1 through 4 in L’Aquila.](image-url)
Figure 7.9. Grain size distribution curves for Site 1 in L’Aquila 2.6 to 6.4 meters.

Figure 7.10. Grain size distribution curves for Site 1 in L’Aquila 7.3 to 11.5 meters.
Figure 7.11. Grain size distribution curves for Site 2 in L’Aquila 1.5 to 5 meters.

Figure 7.12. Grain size distribution curves for Site 2 in L’Aquila 5.8 to 9.5 meters.
Figure 7.13. Grain size distribution curves for Site 3 in L’Aquila 0.6 to 4.8 meters.

Figure 7.14. Grain size distribution curves for Site 4 in L’Aquila 1 to 5.7 meters.
Figure 7.15. Grain size distribution curves for Site 5 in L’Aquila 3.7 to 12.2 meters.
Table 7.3. Table Showing all Soil Data Available at Sites 1 through 4 in L’Aquila

| Location | Sounding | Alias | Depth (m) | ($) (%) | (%) (%) (%) (%) (%) (%) (%) (%) (-) (-) (%) Symbol Nome |
|----------|----------|-------|-----------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|
| L’Aquila | 1 Sports Center | S1 2,60 - 2,80 | 12.9 50.4 36.7 ND NP 0 | 0.01 <0.001 ND <3.5 91.5 CL LEAN CLAY |
| L’Aquila | 1 Sports Center | S1 3,50 - 3,70 | 25.0 9.5 2.5 ND NP 0 | 7.1 0.039 0.896 <3.5 2.899 13.04 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 1 Sports Center | S1 4,00 - 4,20 | 33.8 15.0 6.5 ND NP 0 | 2.76 0.006 0.179 <3.5 1.933 23.4 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 1 Sports Center | S1 4,20 - 4,40 | 2.4 0.2 ND NP 0 | 21.379 5.388 12.033 >3.5 1.257 0.38 GW WELL GRADED GRAVEL |
| L’Aquila | 1 Sports Center | S1 4,80 - 5,00 | 21.2 4.4 0.2 ND NP 0 | 7.89 0.088 1.805 >3.5 4.692 9.46 GW-GM WELL GRADED GRAVEL WITH SILT WITH SAND |
| L’Aquila | 1 Sports Center | S1 5,40 - 5,60 | 7.7 2.5 ND NP 0 | 2.18 0.077 0.178 >3.5 0.896 1.933 GM WELL GRADED GRAVEL |
| L’Aquila | 1 Sports Center | S1 6,20 - 6,40 | 1.4 0 ND NP 0 | 21.379 5.388 12.033 >3.5 1.257 0.38 GW WELL GRADED GRAVEL |
| L’Aquila | 1 Sports Center | S1 7,30 - 7,60 | 2.4 0.2 ND NP 0 | 21.379 5.388 12.033 >3.5 1.257 0.38 GW WELL GRADED GRAVEL |
| L’Aquila | 1 Sports Center | S1 8,30 - 8,50 | 2.4 0.2 ND NP 0 | 21.379 5.388 12.033 >3.5 1.257 0.38 GW WELL GRADED GRAVEL |
| L’Aquila | 1 Sports Center | S1 9,00 - 9,20 | 2.4 0.2 ND NP 0 | 21.379 5.388 12.033 >3.5 1.257 0.38 GW WELL GRADED GRAVEL |
| L’Aquila | 1 Sports Center | S1 9,80 - 10,00 | 2.4 0.2 ND NP 0 | 21.379 5.388 12.033 >3.5 1.257 0.38 GW WELL GRADED GRAVEL |
| L’Aquila | 2 Japan | S2 1,50 - 1,70 | 29.4 34.5 35.4 ND NP 0 | 0.032 <0.001 ND <3.5 74.8 CL LEAN CLAY |
| L’Aquila | 2 Japan | S2 2,50 - 2,70 | 26.0 18.2 7.4 ND NP 0 | 4.84 0.004 0.088 <3.5 0.400 28.4 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 2 Japan | S2 3,20 - 3,40 | 34.9 13.1 3.0 ND NP 0 | 4.35 0.022 0.297 <3.5 0.925 17.8 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 2 Japan | S2 4,80 - 5,00 | 64.9 21.2 12.5 ND NP 0 | 9.315 0.030 1.049 >3.5 3.938 15.4 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 2 Japan | S2 5,80 - 6,00 | 54.8 29.9 13.7 ND NP 0 | 3.963 0.030 0.598 >3.5 3.008 17.1 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 3 AQA3 | S3 0,60 - 0,80 | 2.7 0.1 ND NP 0 | 0.118 0.002 0.038 >3.5 4.614 48.5 SM SILTY SAND |
| L’Aquila | 3 AQA3 | S3 1,60 - 1,80 | 24.0 18.2 7.4 ND NP 0 | 8.40 0.004 0.088 <3.5 0.400 28.4 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 3 AQA3 | S3 3,50 - 3,70 | 31.8 22.9 20.6 ND NP 0 | 3.52 0.010 0.350 >3.5 1.693 31.2 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 4 AQA4 | S4 1,00 - 1,20 | 2.7 0.8 ND NP 0 | 0.032 <0.001 ND <3.5 74.8 CL LEAN CLAY |
| L’Aquila | 4 AQA4 | S4 2,50 - 2,70 | 22.8 5.8 0.5 ND NP 0 | 8.01 0.020 0.152 <3.5 2.751 6.7 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 4 AQA4 | S4 3,70 - 3,90 | 17.6 10.0 3.3 ND NP 0 | 10.92 0.030 1.575 >3.5 9.126 14.5 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 4 AQA4 | S4 4,70 - 5,00 | 81.9 9.7 1.9 ND NP 0 | 12.15 0.181 4.989 >3.5 11.351 8.51 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 4 AQA4 | S4 5,40 - 5,70 | 67.2 27.3 5.5 ND NP 0 | 11.56 0.107 1.526 >3.5 1.881 6.64 GW WELL GRADED GRAVEL WITH SILT WITH SAND |
| L’Aquila | 4 AQA4 | S4 5,80 - 6,00 | 19.8 9.0 2.5 ND NP 0 | 7.84 0.039 1.688 >3.5 9.427 12.7 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 4 AQA4 | S4 6,70 - 6,90 | 22.7 11.3 1.6 ND NP 0 | 8.31 0.020 1.217 <3.5 6.360 14.2 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 4 AQA4 | S4 7,80 - 8,10 | 22.8 9.4 2.8 ND NP 0 | 9.52 0.030 1.221 <3.5 4.525 13.5 GM SILTY GRAVEL WITH SAND |
| L’Aquila | 4 AQA4 | S4 8,80 - 10,00 | 2.4 0.2 ND NP 0 | 21.379 5.388 12.033 >3.5 1.257 0.38 GW WELL GRADED GRAVEL |

Grain Size Distribution

- Limits
- Other Grain Size Information

Soil Classification

USCS

Other Grain Size Information

Location

Gravel Sand Silt Clay

Grain Size Distribution Limits

- other information
8 Results

8.1 Development of DPT Profile

Each of the seven locations where DPT soundings were performed were unique. In addition, a different drill rig was used in Valdez and L’Aquila. The hammer weight available on the CME 75 drill rig used in Valdez, Alaska was 154.2-kg. To compensate for the heavier hammer the weight on the CME 75 drill rig was dropped from a height of 76.2-cm. The ratio of hammer energy actually delivered divided by the energy delivered by the Chinese DPT hammer as calculated by equation 1 for the sites in L’Aquila was 0.84 and 0.29 for the 120 kg and 63.6 kg hammers, respectively. The ratio of the hammer energy actually delivered dived by the energy delivered by the Chinese DPT hammer for the two sites in Valdez was 1.047 for the 154.2 kg hammer that was used.

The $N'_{120}$ values for the five sites in L’Aquila and the two sites in Valdez are shown in Figure 8.1 through Figure 8.7. The $N'_{120}$ value for the sites included as part of this study range from 6 to 140 blows per foot while most of the data lies in the 10 to 70 blows per foot range. The upper limit of the ranges discussed are well into the territory of material that would be considered dense to very dense.
8.2 Critical Layer Determination

The critical liquefiable layer was determined using the method proposed by Cao et al. (2013) and is dependent on three factors. The three factors are listed below:

1. “Fine-grained clayey soils and soils above the water table were classed as non-liquefiable” (Cao et al., 2013).

2. “The penetrated sediments were divided into layers based on soil type, saturation, (above or below the water table), and uniformity of \( N_{120} \) values” (Cao et al., 2013). The critical layer was selected for a thickness of about one meter or more to provide a representative average \( N'_{120} \) value that is less affected by thin peaks or troughs (Boulanger and Idriss, 2014).

3. “For sites with surface-liquefaction effects, the layer below the water table with the lowest general \( N_{120} \) values was identified as the layer that liquefied. For sites without surface-liquefaction effects, the layer below the water table with the lowest average \( N'_{120} \) was identified as the most liquefiable layer, but was assumed not to have liquefied” (Cao et al., 2013).

For each of the seven tests performed as part of this study the three criteria listed above were used to determine the layer most likely to be the critical layer. The critical layer was typically selected over an interval of one meter or more to provide a more representative \( N'_{120} \) that was less affected by thin peaks or troughs (Boulanger and Idriss, 2014). The critical layer depth range for each site is shown as a transparent green rectangle over each chart in Figure 8.1 through Figure 8.7. In addition, the critical depth range for each site is summarized in Table 8.1.
Table 8.1. Critical Layer Depth Range (m) for Sites in Valdez and L'Aquila

<table>
<thead>
<tr>
<th>City</th>
<th>Site</th>
<th>Alias</th>
<th>Critical Layer Range (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L'Aquila</td>
<td>Site 1</td>
<td>Sports Field</td>
<td>5.2-6.2</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 2</td>
<td>Japan</td>
<td>2.3-3.3</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 3</td>
<td>AQA3</td>
<td>3.7-4.7</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 4</td>
<td>AQA4</td>
<td>3.5-4.6</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 5</td>
<td>Water Plant</td>
<td>6.7-8.1</td>
</tr>
<tr>
<td>Valdez</td>
<td>Site 1</td>
<td>Motel</td>
<td>8.0-9.2</td>
</tr>
<tr>
<td>Valdez</td>
<td>Site 2</td>
<td>Church</td>
<td>7.0-8.0</td>
</tr>
</tbody>
</table>

The probability of liquefaction, the factor of safety against liquefaction, the relative density of the soil, and the shear wave velocity were also determined for each of the sites. The plots were placed side by side to make a comparison easier between the different parameters at each depth. Figure 8.1 through Figure 8.7 show the DPT profile, soil profile, relative density, shear wave velocity, factor of safety against liquefaction, and the probability of liquefaction for each of the seven sites. The probability of liquefaction was also used to help determine the critical layer at sites where there were multiple layers with a low average $N'_{120}$ value. At Site 1 and 2 in Valdez the predicted probability of liquefaction was used to help determine the critical layer because multiple layers at these sites have similar average $N'_{120}$ values. At these sites the layer with a low average $N'_{120}$ value and a consistently high probability of liquefaction were chosen as the critical layer. The plots also include relevant data obtained from previously performed soil investigations near each site. The locations of the previously performed soil investigations can be seen in Figure 6.1 for the sites in Valdez, and Figure 7.2 through Figure 7.6 for the sites in L’Aquila.
Figure 8.1. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 1 (Sports Field) in L’Aquila
Figure 8.2. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 2 (Japan) in L’Aquila
Figure 8.3. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 3 (AQA3) in L’Aquila
Figure 8.4. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 4 (AQA4) in L’Aquila
Figure 8.5. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 5 (Water Plant) in L’Aquila (i.m.o.s Ponte Rosarolo Soil Investigation)
Figure 8.6. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 1 in Valdez
Figure 8.7. DPT, Soil Profile, Relative Density, Shear Wave Velocity, Probability of Liquefaction, and Factor of Safety Against Liquefaction data for Site 2 in Valdez
8.2.1 Relative Density

I plotted the relative density based on the relative density DPT correlation developed by Rollins et al. (2020) for each of the sites using equation 16 and the results are shown in Figure 8.1 through Figure 8.7. The range of relative density values for the site in L’Aquila is between 25% and 100%. The range of relative density values for the two sites in Valdez is between 35% and 140%. The relative density value of 140% at Site 1 in Valdez indicates that the soil there at the depth of 0 to 2 meters is very dense.

The relative density based on the SPT data, after correction for grain size (Equation 17), follow the same trend as the relative density determined from the DPT data. The relative density based on SPT data after correction for grain size are labelled SPT Dr Corr. In Figure 8.1 and Figure 8.3. This shows that the DPT provides a reasonably reliable relative density estimate without any correction for grain size. The relative density based on the SPT blow counts without the correction from Kulhawy and Mayne (1990) is also included in Figure 8.1 and Figure 8.3. The uncorrected relative density fractions based on the available SPT data are labelled SPT Dr on the relative density plots and were calculated with equation 18.

As can be seen in Figure 8.1 and Figure 8.3, the correction from Kulhawy and Mayne (1990) shifts the relative density left, almost on top of the relative density determined using the DPT correlation developed by Rollins et al. (2020). The similarity in values of relative density based on the DPT correlation and the Kulhawy and Mayne (1990) SPT grain size correction suggests that the uncorrected relative density fractions, based on the SPT blow counts, are artificially high. The similarity in values of relative density between the DPT correlation and the Kulhawy and Mayne (1990) SPT correction also lend credibility to the DPT relative density correlation. The average relative density in the critical layer for each of the sites is shown in
The relative density of the soil in the critical layer ranges from 45% to 58% in the critical layer for the seven sites included as part of this study. The relative density in the critical layer for each of the sites are relatively similar to one another. This shows continuity between the determination of the critical layer using the three rules proposed by Cao et al. (2013) and the DPT relative density correlation proposed by Rollins et al. (2020).

8.2.2 Probability of Liquefaction

The probability of liquefaction was evaluated using two equations. The first equation (Equation 6 in section 2.4) was developed by Cao et al. (2013). The equation developed by Cao...
et al. (2013) is based on the CSR and $N'_{120}$ values and was developed using DPT data gathered after the 2008 earthquake in Wenchuan, China. The second probability of liquefaction equation (Equation 8 in section 2.5) was developed by Roy (2021) using DPT data from sites around the world.

The probability of liquefaction calculated using each of the two equations is presented on the same plot for comparison. The plots show that the two equations follow the same general trend in most cases. Because the triggering curve developed by Roy (2021) includes more data points from sites with varied geological composition from around the world it is likely more accurate than the curve developed by Cao et al. (2013). As the available data increases the DPT based triggering curve will continue to improve and become more accurate.

The layer with the highest probability of liquefaction generally corresponds with the critical layer that was determined using the three rules proposed by Cao et al. (2013) as shown in Figure 8.1 through Figure 8.7. This lends credibility to the accuracy of the DPT based triggering curves.

### 8.3 Comparison of DPT Liquefaction Triggering Curves

The purpose of developing the seven new non-liquefaction case histories for the DPT is to increase the available data for DPT-based liquefaction triggering curves. The liquefaction triggering curve developed by Cao et al. (2013) only includes data from the 2008 Wenchuan, China Earthquake.

Figure 8.8 shows the DPT based liquefaction triggering curve using the new case history data points. Each point is plotted as the average DPT $N'_{120}$ and average CSR in the critical layer at that site. As shown in Figure 8.8, four of the five points in L’Aquila plot below the 15% probability of liquefaction curve, indicating a low potential for liquefaction. This result is
consistent with the fact that no surface manifestations of liquefaction (sand boils, settlement, fissures, or lateral spreading) were observed at these sites. These results indicate that the Cao et al. (2013) curves are accurately predicting the observed behavior.

In contrast, both sites in Valdez, and one site in L’Aquila, produced points that are close to the 50% probability of liquefaction curve even though no liquefaction was observed at these sites. This probability of liquefaction is higher than would have been expected for a non-liquefiable site and suggests that some adjustment to the Cao et al. (2013) might be desirable to produce better prediction of observed behavior. These non-liquefaction data points are important because they lie right on the statistical boundary between not likely-to-liquefy and likely-to-liquefy. Therefore, these points are important in defining and bounding the liquefaction triggering curve. The points from Valdez are especially important because they have a high CSR value. The high CSR value helps to bound the upper portion of the triggering curve where the data from the 2008 Wenchuan earthquake is limited.

Figure 8.9 shows the data points from Chengdu, L’Aquila, and New Valdez plotted on the DPT triggering curve developed by Roy (2021). The triggering curve developed by Roy (2021) was developed with more data points than are shown in Figure 8.9 from various locations around the world. Only points from Chengdu, L’Aquila, and New Valdez are shown in Figure 8.9 to make it easier to see the data points pertinent to this study. As can be seen in Figure 8.9, the two points from Valdez have a lower probability of liquefaction than is predicted by the triggering curve developed by Cao et al. (2013). Since the sites in Valdez did not liquefy during the 1964 earthquake the lower probability of liquefaction predicted by the triggering curve developed by Roy (2021) suggests that it is more accurate than the curve developed by Cao et al. (2013).
Figure 8.8. DPT liquefaction triggering curve developed by Cao et al. (2013) showing Chengdu, L’Aquila, and Valdez Data points.
Figure 8.9. DPT liquefaction triggering curve developed by Roy (2021) showing Chengdu, L’Aquila, and Valdez data points.
8.4 Comparison of Measured Shear Wave Velocity and DPT Shear Wave Velocity Correlation

One of the goals of this study is to compare measured shear wave velocity and a shear wave velocity-DPT correlation developed by Roy (2021). The sites in L’Aquila have shear wave velocity data from MASW tests performed by Prof. Amoroso. Three of the sites in L’Aquila also have shear wave velocity data from previously performed soil investigations. The type of test that was performed to develop the shear wave velocity profile is important because the type of test performed may influence the results. The shear velocity profile at Site 1 and 2 were developed from MASW data using an array of sensors at the ground surface. The shear wave velocity profile at Sites 3 and 4 was developed from down borehole seismic data. The shear wave velocity profile at site 5 was developed by seismic dilatometer test (SDMT) data. While the SDMT and downhole tests provide a direct measurement of the shear wave velocity, MASW provides an interpretation of the velocity profile that is non-unique.

The DPT shear wave velocity follows the same general trend as the measured shear wave velocity for each of the three sites that have shear wave velocity. The shear wave velocity as predicted by the correlation shown equation 19 varied by an average of 34%, 21%, 70%, 23%, and 27% from the measured shear wave velocity at Site 1, 2, 3, 4, and 5 in L’Aquila, respectively. The largest deviations between the predicted and measured values tended to happen in the shallow portions of the profile where the material was generally looser. This suggests that the shear wave velocity-DPT correlation may perform better in dense soil. These preliminary results show that the DPT shear wave correlation that was developed by Roy (2021) has potential and merits further investigation. The average shear wave velocity in the critical layer for each of the seven sites is shown in Table 8.3. The shear wave velocity predicted by the shear wave
velocity-DPT correlation is significantly lower than the measured shear wave velocity for sites 1 through 4 in L’Aquila.

Table 8.3. Shear Wave Velocity Range in the Critical Layer for Each Site

<table>
<thead>
<tr>
<th>City</th>
<th>Site</th>
<th>Alias</th>
<th>Average Shear Wave Velocity in the Critical Layer (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>DPT Correlation</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 1</td>
<td>Sports Field</td>
<td>248</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 2</td>
<td>Japan</td>
<td>273</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 3</td>
<td>AQA3</td>
<td>226</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 4</td>
<td>AQA4</td>
<td>241</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>Site 5</td>
<td>Water Plant</td>
<td>253</td>
</tr>
<tr>
<td>Valdez</td>
<td>Site 1</td>
<td>Motel</td>
<td>242</td>
</tr>
<tr>
<td>Valdez</td>
<td>Site 2</td>
<td>Church</td>
<td>259</td>
</tr>
</tbody>
</table>

8.5 Probability of Liquefaction Based on Shear Wave Velocity

The measured shear wave velocity at each of the sites can be used to predict the probability of liquefaction using the shear wave velocity based liquefaction triggering curve developed by Cao et al. (2011). As discussed above the triggering curve developed by Cao et al. (2011) is limited by the data that it contains. The data utilized in the shear wave velocity-based triggering curve all came from the same geological area and the same earthquake event. The lack of diversity of the data makes the triggering curve less reliable when used to predict liquefaction potential outside of the Chengdu plain where the original data was collected. The addition of data points from sites outside of the Chengdu plain will serve to increase the reliability of the shear wave velocity-based liquefaction triggering curve.
Equation 15 was used to determine the probability of liquefaction based on shear wave velocity for all of the sites in L’Aquila. Sites 1 through 4 produced a small probability of liquefaction because the shear wave velocities at these sites are high. The probability of liquefaction based on $V_s$ data for Sites 4 and 5 are shown in Figure 7.8 and Figure 7.9 respectively. The probability of liquefaction based on $V_s$ is only shown for these two sites because the probability of liquefaction is so low for the other sites that plotting them is unnecessary. The probability of liquefaction based on DPT, shear wave velocity profile, and soil profile are also shown in Figure 7.8 and Figure 7.9, for comparison.

The probability of liquefaction based on $V_s$ shown in Figure 7.9 was calculated with the $V_s$ data labelled “Prova SDMT”. This was done because the seismic dilatometer test likely produces more accurate results than the data produced by the MASW procedure. The probability of liquefaction predicted at Site 5 in L’Aquila by the shear wave velocity based liquefaction triggering curve developed by Cao et al. (2011) matches relatively well with the DPT based probability of liquefaction developed by Roy (2021) and Cao et al. (2013). The probability of liquefaction based on $V_s$ for Site 4 in L’Aquila shown in Figure 8.10 is significantly lower than the probability of liquefaction predicted by the DPT-based triggering curves.

One of the primary differences between the probabilities of liquefaction based on the DPT and $V_s$ is that the probability of liquefaction based on the DPT data has significantly more detail. The increase in detail is due to the fact that the DPT is able to collect data at 10-cm increments. The procedures available for collecting shear wave velocity are not able to produce data at a resolution of 10-cm. This is a significant drawback to utilizing shear wave velocity data in liquefaction resistance analysis. While shear wave velocity is not as detailed as DPT data, it is
significantly less expensive to obtain making it an attractive alternative to the DPT in some cases.

**Figure 8.10. Probability of liquefaction based on shear wave velocity for Site 4 in L’Aquila**

Figure 8.12 shows the liquefaction triggering curve based on $V_s$ developed by Cao et al. (2011) with the five data points from L’Aquila presented above added to it. As can be seen in Figure 8.12 one of the data points from L’Aquilia falls slightly to the left of the 50% curve while the remaining 4 sites plot to the right of the 15% probability of liquefaction curve. When compared with the probability of liquefaction predicted by the DPT based triggering curve developed by Cao et al. (2013) the $V_s$ based triggering curve predicts less liquefaction. This suggests that the $V_s$ based curve more accurately predicts liquefaction potential for the sites in
L’Aquila. The five data points form L’Aquila are helpful in bounding and defining the probability of liquefaction curve. The addition of data points to the $V_s$ based liquefaction triggering curve will serve to further improve the accuracy of the triggering curve.

Figure 8.11. Probability of liquefaction based on shear wave velocity for Site 5 in L’Aquila.
8.6 Comparison of DPT, SPT, and CPT Profile

Some of the sites have SPT or CPT data from previously performed soil investigations. Comparing the DPT profile with the SPT and CPT profiles illustrates the problem of using the SPT or CPT to characterize gravelly soils. The quantity and detail of SPT and CPT data available at the sites in this study are not sufficient to make any definitive conclusions. However, comparing the general trends is informative.
The plots shown in Figure 8.13 through Figure 8.16 show DPT data side by side with SPT and CPT data where available. SPT and CPT data was only available for four of the seven sites analyzed as part of this study.

Site 1 in L’Aquila, Italy had data from two SPT borings that were performed as part of previous soil investigation. The locations of SPT S1 and SPT S2 can be seen in Figure 7.2. The most obvious difference between the DPT and the SPT plot is the detail of the data. The DPT plot has data at 10-cm increments, while the SPT plot has data at approximately 1-m increments. The SPT data seems to follow the same general trend as the DPT data. However, the SPT seems to report a higher resistance than the DPT at a depth of 10 to 20 ft. In fact, the SPT blow count for SPT 2 reaches refusal \([(N_1)_{60} \geq 50]\) at a depth of about 10 ft.

The second site in L’Aquila that had data from previously performed soil investigations is Site 5. Site 5 has data from two SPT borings, and one CPT sounding. In both the SPT and CPT data, there is a clear spike at a depth of around 7 to 8 m. For example, SPT S2 reached refusal at a depth of 8 m. The spike is especially noticeable in the CPT data where refusal was reached at a depth of 6.7 m and the cone could not penetrate any deeper. The spike in the CPT data may be the result of the cone tip of the CPT meeting a large piece of gravel or cobble or simply a denser gravel layer. In contrast, the DPT penetration resistance increases somewhat at this depth but the increase in not dramatic.
Figure 8.13. DPT and SPT Data for Site 1 in L’Aquila
Both of the sites in Valdez have SPT data from a 1964 soil investigation performed by Shannon and Wilson. The SPT data from both borings match relatively well with the DPT data. This suggests that the SPT split-spoon sampler did not hit any large pieces of gravel as it was driven. If a large gravel particle had been encountered during the boring, a larger spike in the data would have been expected.
Figure 8.15. DPT and SPT Data for Site 1 in Valdez
The discussion above is purely observational. The spikes in the data may not have been caused by the SPT split-spoon sampler and the CPT cone tip meeting a large piece of gravel. However, it is a likely explanation. This illustrates the difficulty of determining whether a higher penetration resistance from the SPT or CPT indicates that the soil is denser or if it is being
artificially increased owing to the larger particle size. This also illustrates the need for the DPT in situations where gravelly soils must be characterized.

8.7 Cumulative N’\textsubscript{120} Comparison Light Hammer and Heavy Hammer

Most geotechnical drill rigs have a conventional 63.5-kg SPT hammer while some do not have a heavier 120-kg hammer. Using the lighter hammer will increase the DPT penetration resistance but could also provide greater resolution for looser soils. As noted, the $N'_{120}$ value for the SPT hammer can be obtained using Eq. 4.

Prof. Rollins made hammer energy measurements for the Italian SPT hammer using an instrumented rod section and a Pile Driving Analyzer device from PDI, Inc. These energy measurements indicate that the SPT hammer delivered an average of 65% the theoretical free-fall energy. The ratio of the energy delivered by the light hammer dived by the energy delivered by the Chinese DPT hammer was found to be 0.29 for the drill rig used in L’Aquila when using the SPT hammer (Rollins et al., 2019).

DPT soundings utilizing the light and heavy hammer were performed at three of the sites in L’Aquila. The heavy hammer DPT soundings and the light hammer DPT soundings were performed about one meter apart. This spacing was a compromise between having the holes close enough that they would encounter similar soil profiles but far enough apart that the subsequent hole would not be affected by the presence of the previous DPT hole. Figure 8.18, Figure 8.19, and Figure 8.20 show the $N'_{120}$ plots with the heavy and light hammer alongside the cumulative N’\textsubscript{120} plots for site 1 in L’Aquila, site 4 in L’Aquila, and site 5 in L’Aquila, respectively.
8.8 Development of Cumulative $N'_{120}$ Plots

As noted previously, direct comparison between the $N'_{120}$ for lighter (63.5 kg) SPT hammer and the heavier DPT hammer after correction for energy effects shows a relatively low correlation coefficient. Figure 8.17 shows the light and heavy hammer $N'_{120}$ values for the same depth plotted against a straight line at a 45-degree angle. Theoretically the relationship between the light and heavy hammer data points should be one-to-one, lying along the straight line plotted in Figure 8.17. The data points in Figure 8.17 for Site 1 and 5 in L’Aquila are scattered relatively close to the straight line with a few outliers. However, the data points for Site 4 exhibit significant scatter. Of course, scatter would be expected even if the same hammer energy was used at two adjacent soundings as a result of local variations in stratigraphy and gradation. It is possible that the significant scatter seen in the data collected at Site 4 in L’Aquila could be the result of small vertical offsets or particle size. The data available as part of this study are not detailed enough to determine whether small vertical offsets or particle size are the cause of the scatter observed at Site 4. Future studies are needed to further explore possible causes of the excessive scatter seen at Site 4. To minimize the effects of these local variations, plots of cumulative energy-corrected $N'_{120}$ were produced for each pair of soundings as shown in Figure 8.18 through Figure 8.20. The cumulative $N'_{120}$ plots help to minimize the effects of outlier data points by adding the outlier data points to the sum of all the data before reducing their impact.
Figure 8.17. $N'_{120}$ blow counts for the heavy hammer and the light hammer at the same depth plotted together.

The difference between the two curves at L’Aquila site 1 is less than 15%; however, the error for the L’Aquila sites 4 and 5 reached 30% and 35%, respectively. This result stands in contrast with similar comparisons involving loose to medium dense gravels where the error in the cumulative $N'_{120}$ curves was often less than 10% (Rollins et al. 2020, Rollins and Harper 2018). One possibility that may account for the discrepancy between the error in previous studies and the error found in this study is the difference in the density of the material tested. The blow counts
recorded in this study are much higher than those from the study performed by Rollins and Harper (2018). The prevailing theory is that as the density of the material increases the error between the light and heavy hammer \( N'_{120} \) values also increases.

When the DPT soundings were performed Dr. Rollins and Anthony Rosas, were attentive to stop the test if there seemed to be a risk of breaking the DPT cone tip. For this reason, gaps in the light hammer DPT data can be seen in Figure 8.18 and Figure 8.19. The gaps in the data made it difficult to compare the cumulative DPT blow counts from the heavy and light hammer. To overcome this problem, I assumed the cumulative light hammer and heavy hammer are the same where the gap in the light hammer data ends. While this assumption may not be realistic, it does allow for a general comparison between the light and heavy hammer, despite the gaps in the light hammer data.

Despite the limitations in the light hammer data, there are a couple of useful observations that can be made. When the resistance of the soil increases, the cumulative \( N'_{120} \) from the light hammer tends to decrease. This trend can be seen in Figure 8.18 through Figure 8.20. In Figure 8.19 at a depth of 7-m to 12-m, the resistance of the soil as measured by the heavy hammer DPT is high. The resistance of the soil, as measured by the light hammer at the same depth, is significantly lower. This trend can be seen in Figure 8.18, at a depth of 10-m to 13-m and in Figure 8.20 at a depth of 2-m to 8-m. The data available are too limited to make a conclusion on the trend described above. Further research is necessary to better understand the trend.
Figure 8.18. Light & heavy hammer $N'_{120}$ values and cumulative $N'_{120}$ plot for L’Aquila site 1.
Figure 8.19. Light & heavy hammer $N'_{120}$ values and cumulative $N'_{120}$ plot for L’Aquila site 4.
Figure 8.20. Light & heavy hammer $N'_{120}$ values and cumulative $N'_{120}$ plot for L’Aquila site 5.
9 Summary and Conclusions

1. The five sites in L’Aquila, Italy and the two sites in Valdez, Alaska analyzed as part of this study provide valuable data for DPT liquefaction triggering curves. These seven sites help to bound the current DPT liquefaction triggering curve by adding to the total number of data points available for use in the triggering curve. The two sites in Valdez are especially useful as they have a high CSR value helping to bound the upper portions of the DPT based triggering curve.

2. Three of the Sites in L’Aquila plotted to the right of the 15% PL curve with one falling on top of the 30% PL curve and one just to the left of the 15% PL curve when plotted on the triggering curve developed by Cao et al. (2013). All five of the sites in L’Aquila plot to the right of the 15% PL curve when plotted on the triggering curve developed by Roy (2021). Similarly, both sites in Valdez moved to the right when plotted on the curve developed by Roy (2021) one of them moving to the right of the 30% PL curve. This suggested that the triggering curve developed by Roy (2021) is more accurately predicting liquefaction potential since these sites are non-liquefaction sites.
3. The DPT-shear wave velocity correlation performed poorly at estimating the measured shear wave velocity for the five sites in L’Aquila. The average error between the shear wave velocity-DPT correlation developed by Roy (2021) and the measured shear wave velocity data is 34%, 21%, 70%, 23%, and 27% at Site 1, 2, 3, 4, and 5 in L’Aquila respectively. The shear wave velocity-DPT correlation performed poorly at shallow depths at all three sites where blow counts were relatively low. This suggests that the shear wave velocity-DPT correlation performs best in soil layers with an \( N'_{120} \) blow count of above approximately 20.

4. The relative densities, as estimated by previously performed SPT and CPT soundings, were significantly higher than the relative densities estimated by the DPT correlation developed by Rollins et al. (2020). After correcting for grain size, the relative densities based on SPT were similar to the relative densities estimated by the DPT correlation. This suggests that in gravelly soils the uncorrected relative densities based on SPT and CPT soundings may be higher than the actual relative density of the soil.

5. The use of a lighter SPT hammer when performing DPT soundings has been found to yield similar results to soundings using the heavier DPT hammer after energy correction in studies performed on the subject (Harper and Rollins 2018). However, in this study the lighter hammer produced significantly different results than the heavier hammer. This may due to the higher than normal penetration resistance recorded at the sites included as part of this study. It appears that as the penetration resistance of the soil increases the light hammer tends to produce significantly different values than the heavy hammer.
References


98


